British Dam Society Conference 2004 – Canterbury, Kent

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

Chapter 4. Flood impact and alleviation

European research on dambreak and extreme flood processes

MW MORRIS, HR Wallingford Ltd, UK
MAAM HASSAN, HR Wallingford Ltd, UK

SYNOPSIS. Effective risk management for dams requires an understanding of potential hazards and an assessment of the various associated risks. This requires analysis of potential impacts which, in the case of dambreak, requires an ability to reasonably predict conditions that may result through failure or partial failure of a dam. The IMPACT Project focuses research in five areas related to dambreak, namely breach formation, flood propagation, sediment movement, geophysical investigation and assessment of modelling uncertainty. This paper provides an update on this 3-year programme of work with an overview of some initial findings, particularly in relation to work on breach formation.

THE IMPACT PROJECT

The IMPACT Project (Investigation of Extreme Flood Processes and Uncertainty) is a research project running for 3 years from 2001-2004, funded by the European Commission and supported in the UK by Defra and the Environment Agency. The focus of work is directed at four process areas (breach formation, flood propagation, sediment movement, geophysical investigation) and assessment of uncertainty within modelling tools. These research areas were identified during earlier research (Morris, 2000) as areas where predictive ability was relatively poor, and hence 'weak links' in any risk assessment or emergency planning studies.

Programme of work

Research into the various process areas is undertaken by groups within the overall project team. Some work areas interact, but all areas are drawn together through an assessment of modelling uncertainty and a demonstration of modelling capabilities through an overall case study application. The IMPACT project provides support for the dam industry in a number of ways, including:

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

- Provision of state of the art summaries for capabilities in breach formation modelling, dambreak prediction (flood routing, sediment movement etc)
- Clarification of the uncertainty within existing and new predictive modelling tools (along with implications for end user applications)
- Demonstration of capabilities for impact assessment (in support of risk management and emergency planning)
- Guidance on future and related research work supporting dambreak assessment, risk analysis and emergency planning

Each work area is briefly outlined below, followed by a focus upon work investigating breach formation. More detailed information on all areas of the project may be found via the project website at www.impact-project.net.

Breach Formation

Existing breach models have significant limitations (Morris & Hassan, 2002). A fundamental problem for improving breach models is a lack of reliable case study data through which failure processes may be understood and model performance assessed. The approach taken under IMPACT was to undertake a programme of field and lab work to collate reliable data. Five field tests were undertaken during 2002 and 2003 using embankments 4-6m high. A series of 22 laboratory tests were undertaken during the same period, the majority at a scale of 1:10 to the field tests. Data collected included detailed photographic records, breach growth rates, flow, water levels etc. In addition, soil parameters such as grading, cohesion, water content, density etc. were taken. Both field and lab data were then used within a programme of numerical modelling to assess existing model performance and to allow development of improved model performance.

Flood Propagation

Work on flood propagation focussed on two different aspects, namely, prediction of flood flow conditions through urban areas and prediction of flood conditions in real topography.

Whilst river modelling has become a routine part of design and analysis of river works, the way in which flooding of urban areas is predicted has not been ‘standardised’. A number of different approaches may be taken, such as simulation of streets as flow channels, simulation of key areas as storage reservoirs or simulation of general flow by increased roughness. The objective of this component of work is to compare various approaches and hence identify differences and perhaps the best approach. This work has been undertaken through analysis of both field and lab data. Physical modelling of flow through urban areas provided base data for model comparison.

Sediment Movement

Under dambreak or extreme flood conditions, significant volumes of sediment may move. In the near field, close to a breach or failed dam, sediment will be entrained and carried with the surging flow. In the far field, the nature of flow and sediment conditions may produce significant changes to the river such as lateral widening, braiding or major changes in course. With respect to dambreak assessment and emergency planning, sediment movement and deposition may significantly affect bed, and hence surface water, levels as well as provide an obstruction for access.

Research is underway through a combination of laboratory modelling and numerical simulation. Initial work is focussing upon developing new relationships for sediment entrainment under extreme and varying conditions. It is noticeable that current approaches for predicting breach growth or sediment movement during dambreak all utilise existing sediment transport equations that are typically based upon long term steady state conditions.

Geophysics & Data Collection

This 2-year module of work was added to the IMPACT project through a programme to encourage wider research participation with Eastern European countries. The work comprises two components; firstly review and field testing of different geophysical investigation techniques and secondly collation of historic records of breach formation.

The objective of the geophysical work is to develop an approach for the 'rapid' integrity assessment of linear flood defence embankments. This aims to address the need for techniques that offer more information than visual assessment, but are significantly quicker (and cheaper) than detailed site investigation work. Research is being undertaken through a series of field trial applications in the Czech Republic at sites where embankments have already been repaired and at sites where overtopping and potential breach is known to be a high risk.

The objective of collecting breach data is to create a database of events that includes as much information as possible relating to the failure mechanisms, local conditions, embankment material and local surface materials. Analysis may then be undertaken to identify any correlation between failure mode, location and embankment material, surface geology etc.

Uncertainty Analysis

The objective of work here is to establish the uncertainty that may be present in modelling predictions, and subsequently how this might influence use of the information by the end user. Uncertainty in component model predictions (i.e. breach model, flood propagation model, sediment model etc.) is being established, followed by a combined assessment on an overall case study to demonstrate techniques and conclusions.

Conclusions from the IMPACT Project Research
Conclusions from the IMPACT project research will be presented and discussed in full during a final project workshop, to be held in Zaragoza (Spain) on 27-29th October 2004.

This paper will now focus on work undertaken during Years 1 and 2 of the project within the breach formation theme area.

A FOCUS ON BREACH FORMATION

The objectives of this area of research work were to:

- Collate reliable field and laboratory data demonstrating failure processes for cohesive and non cohesive embankment failure (failure mainly by overtopping, but also through piping)
- Objectively assess existing breach model performance
- Allow further development and validation of breach models to improve performance
- Allow an assessment of the effect of scaling on breach data collection (i.e. field data versus laboratory data)

This was achieved by undertaking 5 field tests (up to 6m high), 22 laboratory tests and an extensive programme of numerical modelling with modellers participating from around the world, as well as within the EC.

Field Work

Five field tests (see Table 1) were undertaken as part of the IMPACT project, although additional tests were also undertaken as part of the Norwegian national research programme.

Table 1. Programme of field tests

Test	Nature	Height	Failure mode
Test #1	Homogeneous, cohesive	6m	Overtopping
Test #2	Homogeneous, non cohesive	5m	Overtopping
Test #3	Composite (Rock fill shoulders and moraine core)	6m	Overtopping
Test #4	Composite (Rock fill shoulders and moraine core)	6m	Piping
Test #5	Homogeneous (moraine)	4m	Piping

Figure 1 shows material gradings for each of the various test materials and Plate 1 shows Field Tests #1 and #5 at various stages of testing.

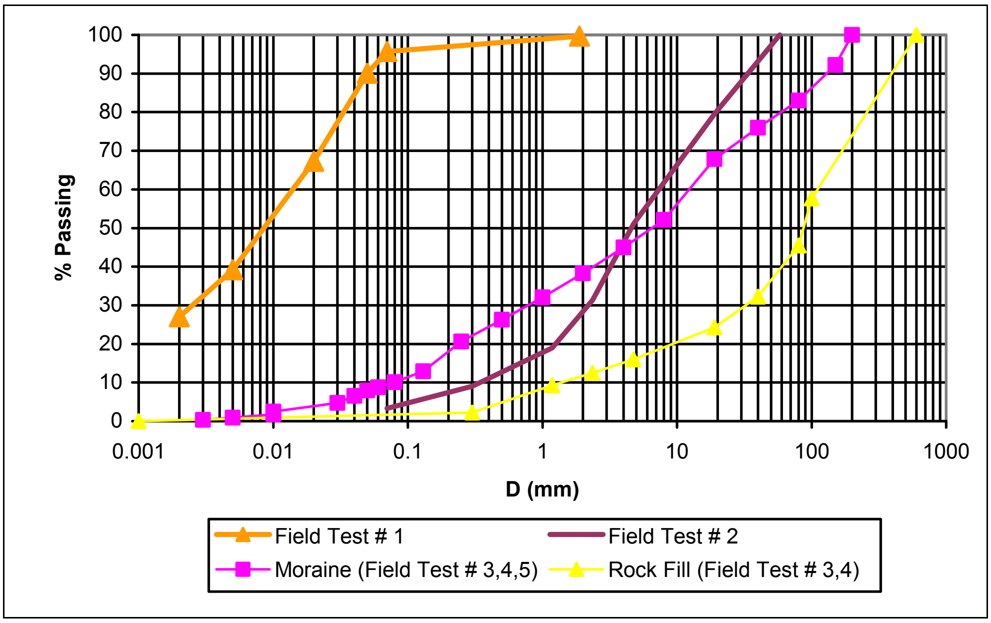


Figure 1 Materials used for the five field tests

Plate 1 Field Test #1 (left) and Field Test #5 (right)

In order to help understand the process of breach formation, a range of data was collected during the tests including water levels, flows and soil properties. Monitoring the rate of breach growth was assisted by the use of movement sensors that were buried within the body of the dam. These sensors recorded the time at which movement occurred, so by recording where the sensors were buried, it was possible to recreate a picture of the breach growth pattern after the failure had occurred.

Initial Findings of Field Work

Whilst data is still being analysed at the time of writing, some initial observations may be made:

Breach Growth & Discharge

Many existing breach models predict discharge by assuming that supercritical flow occurs within the main body of the breach. Flow can then be calculated using a weir equation and the width of the breach. It can be seen from Plate 2 (Field Test #1) that this is not always the case. In this photo it can be seen that the flow through the breach is controlled by a curved weir created by erosion of the upstream embankment face. This ‘control section’ gives weir flow over a length significantly greater than the breach width.

Plate 2: Weir control section

Lateral Erosion of Embankments

Many existing models assume a uniform and sometimes predefined distribution of erosion of material in order to predict breach growth (e.g. uniform growth of a trapezoidal section). It can be seen from Plate 3 below that lateral growth occurs through erosion of material at the base and sides of the breach with discreet failures of the side slopes leading to growth. Note also in this photo that whilst erosion is occurring at the sides, as indicated by coloured water, the flow through the centre is relatively clear, suggesting minimal sediment transport. Most existing models that calculate sediment transport assume a uniform load.

Plate 3: Lateral erosion of embankments (muddy water adjacent to eroding banks; clear water in centre)

Pipe Formation – Effects of Arching

Plate 1(right) shows pipe formation through a moraine embankment. Even after significant erosion has taken place, the crest of the embankment shows little sign of distress and no subsidence. Throughout growth of the pipe the load of the material above the hole has been distributed across the bank through an arching effect. Reliable prediction of breach growth through pipe formation requires a clear understanding and assessment of this process.

<u>Laboratory Work</u>

A series of tests were undertaken in parallel to field tests in the modelling laboratories at Wallingford. The majority of these tests were designed to reproduce and also extend the range of tests undertaken in Norway. This permitted an analysis of scale effect between field and laboratory experiments (1:10 scale factor), and created a wider range of data sets with which to analyse breach growth and assess model performance.

Two main series of overtopping tests were undertaken using the large 'flood channel facility' at Wallingford. The first series (2002) simulated breach growth through overtopping of non-cohesive material. This related to field test #2. Following an analysis of potential scaling mechanisms, the material used was also scaled at 1:10. In order to create a material with properties matching the material used in field test #2, but at a scale of 1:10, it was necessary to mix 4 different sands. The second series (2003) was undertaken to investigate beach formation through cohesive material. Failure was again by overtopping. The behaviour of cohesive material cannot be scaled exactly without also scaling other loads such as gravity. This option was not available to us (i.e. use of a large centrifuge) hence it was decided that tests would be undertaken using material similar to that used for field test #1 and any scale effects carefully considered. For example, analysis of material condition and hydraulic loading allowed an assessment of the scaling of critical shear stress and material erodibility.

Figure 2 shows the grading curves for both cohesive and non-cohesive tests and Plate 4 examples of each laboratory test.

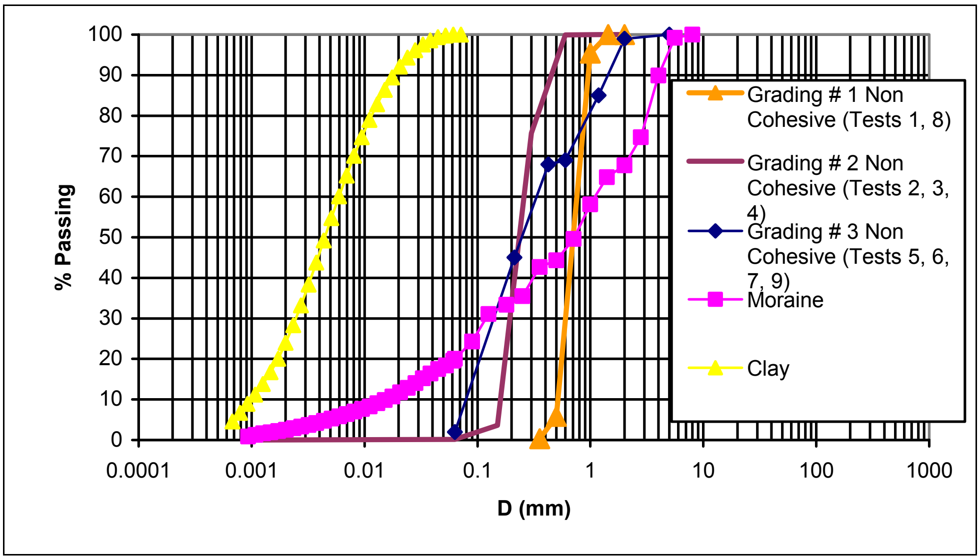


Figure 2 Materials used for cohesive and non-cohesive laboratory tests

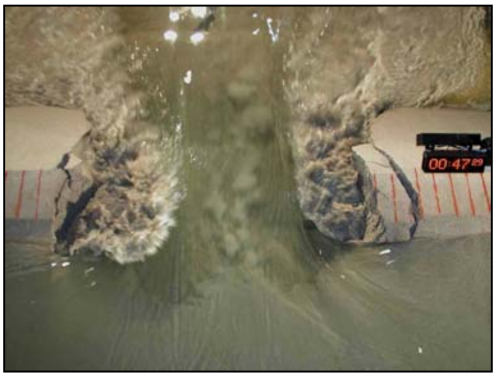


Plate 4 Laboratory test of breach formation through overtopping

Table 2 provides a summary of the tests undertaken. Parameters that were varied for the non-cohesive tests included geometry and material grading distribution around a constant D_{50} size. Parameters varied for the cohesive tests included geometry, compaction and moisture content.

Table 2 Objective of laboratory tests

Test	Nature / purpose of test
Non-Cohesive tests; 0.5m high; material gradings 1, 2 or 3:	
#1	Facility set-up / trial
#2	Scale of Field Test #2, but uniform material grading based upon D_{50}
#3	Repeatability of Test #2
#4	As Test #2, but breach initiation adjacent to side of flume
#5	Direct replication of Field Test #2
#6	As Test #5, but embankment face at 1:2 instead of 1:1.7
#7	As Test #5, but embankment crest width 0.3m instead of 0.2m
#8	As Test #2 but larger D_{50} for uniform grading of material
#9	As Test #5 but seepage allowed to develop prior to testing
Cohesive tests; 0.6m high; material clay or moraine:	
#10	Scale of Field Test#1
#11	Repeatability of Test #10
#12	As Test #10, but constructed with half compaction effort
#13	As Test #10, but constructed to optimum moisture content
#14	Continuation of Test #13
#15	As Test #10 but 1:1 gradient for downstream slope
#16	As Test #10 but 1:3 gradient for downstream slope
#17	As Test #10 but using moraine material

In addition to these 17 tests, a further 5 tests on pipe formation were undertaken. Two of these tests were to aid development of an appropriate failure mechanism to ensure that failure of the piping field tests occurred within a reasonable period of time. The remaining three were testing of pipe formation through 3 samples of real embankment (~1m^3) taken from the Thorngumbald Managed Retreat Site on the River Humber. This work was undertaken by Birmingham University and was also consistent with recommended R&D work under the EA / Defra *Reducing the Risk of Embankment Failure under Extreme Conditions* project.

<u>Initial Findings of Laboratory Work</u>

Whilst data is still being analysed at the time of writing, some initial observations may be made:

Headcut Erosion
Erosion of cohesive and non-cohesive embankments occurs in a different way. Cohesive material tends to erode via a series of steps – called head cutting. This was clearly seen in the field tests, but was also reproduced in the laboratory, suggesting that this process was not affected by scaling at 1:10 (see Plate 4 (left)).

Soil Properties, Condition and Seepage
The effect upon the rate of breach formation of variation in soil properties and / or condition was quite noticeable and in particular, the effects of variation in moisture content for cohesive materials. Changing the moisture content of the cohesive material from 20% to 30% (near optimum) changed the erodibility rate of the material by a factor of 12. This effect was significantly smaller when working with non-cohesive material, where allowing seepage through the bank to establish prior to testing appeared to have a minimal effect upon the eventual rate of breach growth.

Material Grading & Compaction
Many existing models represent embankment material by a single D_{50} value. Tests using different material grades, but each with the same D_{50} value, showed different behaviour, with, as might be expected, a wider grading material offering greater resistance to breaching. Also of significance is the degree of material compaction (or density). In one test, halving the compaction effort resulted in a significant change to the rate of breach formation. Specifically the rate of down cutting increased by x2.5, lateral widening increased by x5 and headcut erosion increased by x1.6.

Numerical Modelling and Analysis
A fundamental objective of the field and laboratory research work was to collect reliable data with which to validate and further develop numerical models for predicting breach formation. At the time of writing, model performance was being assessed through a controlled programme of testing such that field or lab data was only released after initial modelling predictions had been collated. This 'blind' and 'aware' approach to modelling ensured complete objectivity in the assessment of performance.

Whilst some initial results have been assessed, the extent of model performance assessment is not sufficient to allow reporting here. However, full results from this analysis work will be reported later during 2004 and posted via the project website (www.impact-project.net).

BREACH FORMATION: MIDTERM CONCLUSIONS AND OBSERVATIONS

The most striking observation (based upon the field and laboratory test data) is the clear relationship between the breach formation process and the embankment material properties and condition. Whilst this may seem obvious, it is a fact that many existing predictive breach models ignore such information and endeavour to predict the failure process based upon geometry, limited soil property information and hydraulic loading conditions. Whilst tests show that variations in material grading, compaction and moisture content (for example) can affect the rate of material erosion and hence breach growth by factors of more than x10. Where models fail to include even the most basic of soil properties or conditions, then the potential accuracy of their predictions will be significantly constrained.

Failure to account for the way in which breach growth develops will also limit modelling accuracy. For example, it is clear that rockfill embankments behave differently to non or low cohesive earthfill embankments which in turn behave differently to cohesive embankments. This difference applies particularly to the way in which the breach initiates. Most existing models make broad assumptions as to the way in which erosion occurs so as to provide an average rate of formation and hence discharge. Whilst this may be a valid approach for a specific material type and condition (against which the model has to be calibrated), this will lead to inaccuracies when routinely applied as a single solution or model applicable to all materials and conditions.

<u>Future Direction of research</u>

An extensive analysis of breach model performance is currently underway and should be completed by June 2004. This work will also link with an assessment of modeling uncertainty in order to provide the ‘end user’ with guidance on both the performance / accuracy of breach models, as well as the range of uncertainty that might be reasonable to expect within a model prediction.

In the longer term, it is clear that in order to improve our ability to predict breach growth we will require a much closer integration of soil mechanics and hydraulics analysis. Critical soil parameters that have the most influence upon the initiation and growth of a breach will need to be identified, along with methods for measuring or monitoring these parameters in the field.

REFERENCES

Morris M W (2000). *CADAM – A European Concerted Action Project on Dambreak*. Proceedings of the biennial conference of the British Dam Society, Bath.

Morris MW, Hassan MAAM (2002). *Breach Formation Through Embankment Dams and Flood Defence Embankments: A State of the Art Review.* Stability and breaching of rockfill dams workshop. Trondheim. April 2002.

IMPACT Project workshop proceedings:

Workshop #1	16-17 May 2002	Wallingford, UK
Workshop #2	12-13 September 2002	Mo-I-Rana, Norway
Workshop #3	6-7 November 2003	Louvain-la-Neuve, Belgium

ACKNOWLEDGEMENTS

IMPACT is a research project supported by the European Commission under the Fifth Framework Programme and contributing to the implementation of the Generic Activity on "Natural and Technological Hazards" within the Energy, Environment & Sustainable Development programme. EC Contract: EVG1-CT-2001-00037. The financial support offered by DEFRA and the Environment Agency in the UK is also acknowledged.

The IMPACT project team comprises Universität Der Bundeswehr München (Germany), Université Catholique de Louvain (Belgium), CEMAGREF (France), Università di Trento (Italy), Universidad de Zaragoza (Spain), Enel.Hydro (Italy), Sweco (formerly Statkraft Grøner AS) (Norway), Instituto Superior Technico (Portugal), Geo Group (Czech Republic), H-EURAqua (Hungary) and HR Wallingford Ltd (UK).

Particular recognition is given to Kjetil Vaskinn of Sweco (formerly Statkraft Grøner) for his role in managing the breach formation field tests in Norway and providing data for this paper.

A passive flow-control device for the Banbury flood storage reservoir

J ACKERS, Chief Hydraulic Engineer, Black & Veatch Consulting
P HOLLINRAKE, Senior Hydraulic Engineer, HR Wallingford Ltd
R HARDING, Project Manager, Environment Agency

SYNOPSIS. The Environment Agency is developing a scheme to protect the town of Banbury against flooding, principally by providing an 'on-line' flood storage reservoir on the River Cherwell upstream of the town. A flow control structure will be sited on each of the two branches of the river, incorporating a suitably designed throttle to limit discharges passed through the town in events up to a return period of about 200 years. Construction of the main works of the project is programmed to commence in 2005.

This paper describes the development of the design for the flow-control structures with the aid of a physical model at HR Wallingford. The design is based on a double-baffle orifice capable of maintaining discharges passed downriver within a target range of less than ±10% over a wide range of water levels in the flood storage reservoir.

INTRODUCTION

Banbury lies on the River Cherwell, a left tributary of the River Thames into which it flows in Oxford. The town has a long history of flooding, the most recent major flood being in 1998, with an estimated return period of about 100 years and total flood damage exceeding £12.5M. Flooding in Banbury is the result of the River Cherwell and associated local watercourses having insufficient capacity to convey the runoff from the upstream catchment, and has been exacerbated by development being allowed to take place on the floodplain.

The preferred solution, which was chosen taking account of technical, economic and environmental issues, is to provide an upstream 'on-line' flood storage reservoir, coupled with some local defences in the town. The flood storage reservoir will comprise the following main elements (see Figure 1):

- an embankment of maximum height about 5m and length 2.9km, running parallel to the northeastern side of the M40 and to the eastern side of the Oxford Canal;
- two similar flow control structures, one at the intersection of the embankment with each branch of the River Cherwell;
- service spillways incorporated into the control structures; and
- an emergency spillway incorporated in the embankment between the two control structures.

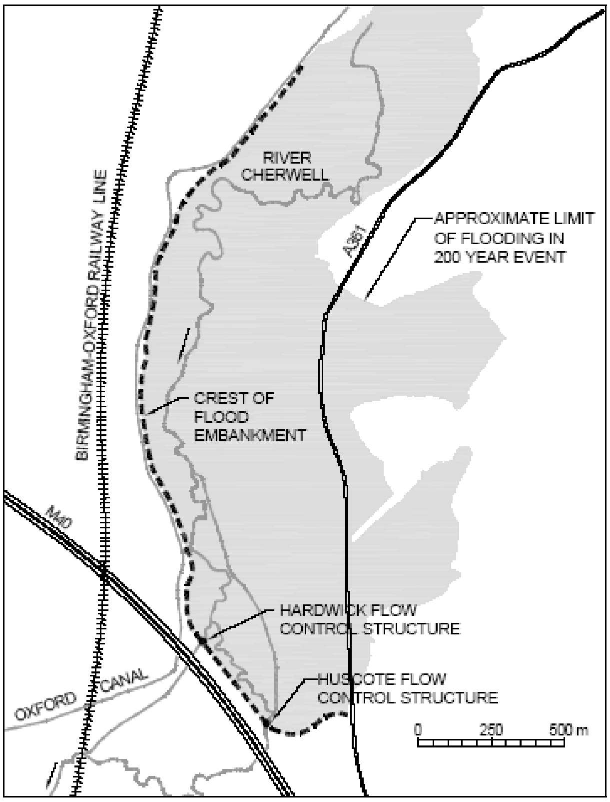


Figure 1 Location plan

The intention is that, in combination, the flow control structures should throttle the river flows to the maximum discharge which can be passed through the town, estimated as 38 m^3/s, impounding the additional flood discharge in the reservoir. The reservoir has been designed to accommodate the volume expected to be impounded in the design 200-year flood event.

When the reservoir is full, the spillways located alongside the control structure will overtop and provide a total discharge capacity approximately equal to that of the unattenuated peak of the 200-year flood. The emergency spillway will allow more extreme floods to be discharged without over-topping the rest of the embankments.

CONCEPT

The Environment Agency was keen that the control of discharges passed downriver from the flood storage reservoir should occur automatically, with no requirement for attendance by their operatives during floods. It was also considered desirable to place no reliance on power supplies or remote operation of the flow control structure. If practicable, a structure with no moving parts would also be preferred.

An ideal flow control device for an on-line flood storage reservoir would allow all discharges less than a target value to pass downstream without starting to impound. As the discharge continues to rise it would then allow the target discharge to pass downstream, impounding all of the excess. Such accurate control is difficult to achieve precisely, even in a fully automated gated system, but would have two advantages if it could be achieved:

- it would minimise the effect on the land within the impoundment area during minor floods (with a return period of up to about five years in the case of Banbury); and
- in the early part of larger floods, it would preserve as much as possible of the available storage volume for utilisation in attenuating the peak of the flood hydrograph, ultimately reducing the total flood storage needed and therefore lowering the peak water level in the flood storage reservoir.

A simple orifice meets the objective of having no moving parts, but results in the discharge rising as the square root of the net head. If a simple orifice is designed to limit the discharge to the target value when the reservoir is nearly full, this results in it starting to impound when the discharge is much less than that target value.

The above objectives led to consideration of the design concepts embodied in the baffle distributor devices which have been used for many years in irrigation systems, in particular the 'Neyrpic module'. Performance information on these devices is given by Neyrpic (1971), Alsthom Fluides (undated), UN/FAO (1975), Bos (1989) and a number of other standard references. The devices are designed to achieve a nearly fixed discharge out of a parent irrigation canal over a range of operating levels in the canal.

Two forms of the device are described in the references, one comprising a single baffle and the other a double baffle, of which the double baffle has the potential to provide a wider range of nearly fixed discharge, so was of particular interest. Although the performance information for double baffle distributors is apparently identical between the references consulted, at least three different variants on the shapes of the baffles are given.

Figure 2 shows the geometry of the device, based on the dimensions and shape quoted by Bos (1989), together with the quoted stage/discharge rating. As the upstream head rises, it impinges on the upstream baffle, which then acts as the control, with the jet clearing the underside of the downstream baffle. As the head rises further it overtops the upstream baffle and a transition to downstream baffle control occurs as the stage rises further. The discharge remains within a band of ±10% for heads between around 0.73 and 1.74 times a nominal design head.

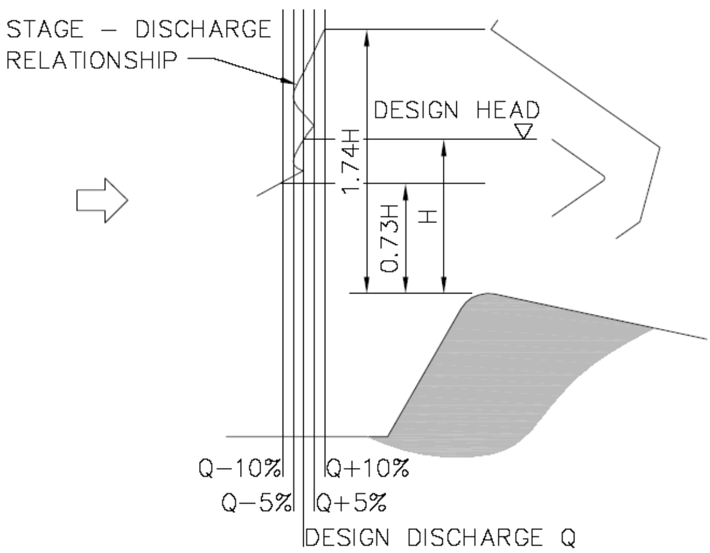


Figure 2 Double-baffle orifice layout and performance (after Bos, 1989)

PRELIMINARY DESIGN

Initial consideration of the outline design for the Banbury control structures indicated that the vertical and longitudinal dimensions should be exactly twice those of the largest standard irrigation distributor module illustrated by Alsthom Fluides (undated). The nominal design head is 2.58m, the nominal design unit discharge is 5.66 m^2/s and the target head range for ±10% is 1.88m to 4.50m. On this basis each bay would be 1500mm wide, giving a nominal discharge of 8.5 m^3/s per bay or 17 m^3/s per structure and therefore a total nominal downriver flow of 34 m^3/s, rising to about 38 m^3/s at the maximum positive deviation of 10%.

In a distinct departure from the designs in the references, it was decided that the invert profile should resemble a Crump weir, with upstream and downstream slopes of 1:2 and 1:5 respectively meeting at a sharp vertex. Factors in this decision were the simpler construction than the round crest of the original device and the possibility of predicting the lower part of the rating curve (before impingement on the baffles) by the use of the standard formula for a Crump weir.

Prior to model testing, a 'target' stage/discharge relationship was prepared, based on the formula for a Crump weir at low heads and on the relationship for baffle control given by Neyrpic (1971). Although precise compliance with that relationship was not considered an essential outcome of the testing, it was a useful aid for comparing the performance when various details were adjusted during the design development.

MODEL STUDY

The Banbury flow control device will be much larger than the largest of the standard irrigation distributor modules described in the references. Although the reported design and hydraulic behaviour would clearly be amenable to Froude scaling, several factors led to the decision to undertake a programme of project-specific model testing:

- the differences between the references regarding the appropriate configuration for the device;
- a concern that the baffle design shown in Figure 2 (and the other versions) would be vulnerable to debris accumulation;
- a recognition that metal fabrication might not be appropriate for the larger structure and that the use of thicker concrete structural members would have an impact on both the design and the resulting hydraulic behaviour;
- a suggestion in one of the references (Bos, 1989) that the hydraulic performance of the device would exhibit hysteresis, with different ratings for rising and falling stages; and
- the need for a verified rating relationship for use in design.

The model testing commenced with two versions of the double-baffle configuration, as illustrated in Figure 3. One is based on the simplest of the three variants which appear in the references, comprising angled baffles expected to be fabricated in robust steel plate; the other comprises simple vertical baffles, which are thicker than the angled baffles and intended to be suitable for construction in reinforced concrete.

The model design, construction and testing were undertaken at HR Wallingford, with a model geometric scale of 1:12 selected. The model was built mainly from PVC, to provide a suitable boundary roughness (comparable to concrete in the prototype) and the sidewalls in the vicinity of the baffle devices were built in Perspex to allow flow visualisation. Discharges were provided via a centrifugal pump and measured using an electromagnetic flowmeter, giving a basic accuracy of around 1%. Water levels were measured using manual micrometer point gauges reading to an accuracy of about 0.25mm (3mm in prototype terms).

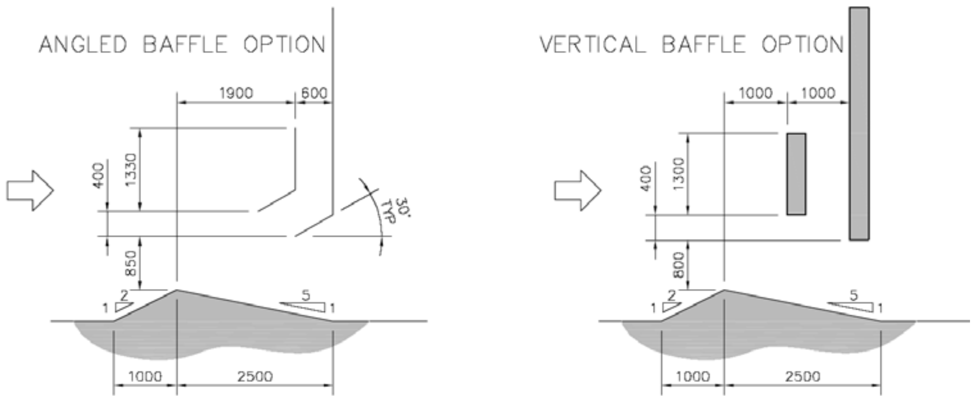


Figure 3 Double-baffle designs for preliminary testing

Each of the two flood control structures was expected to comprise a pair of the orifice devices side by side, separated by a central pier with a semi-circular nose. It was decided to reproduce a complete structure, including the two bays and central pier, in the model, although not the detail of the approach channel and exit channel. In the preliminary tests each bay contained one of the two different versions of the double baffle orifice device, with the approach channel to each bay closed off in turn in order to test one bay at a time. When the design development was complete, the chosen design was built into both bays and confirmation tests undertaken.

The model test programme thus comprised three stages:

- preliminary tests, using the preliminary designs illustrated in Figure 3;
- optimisation tests, in which a series of design adjustments, affecting the baffle positions, elevations and shapes were made and evaluated in a single bay; and
- final tests of the optimised structure in both bays of the model.

The preliminary and optimisation tests were carried out with the down-stream water level low enough to avoid any effect on the flow conditions in the structure. The final tests also used tailwater levels derived from flood simulations.

<u>Preliminary tests</u>

The preliminary tests on the configurations shown in Figure 3 showed a close agreement between the test results for the angled baffle and the target relationship (Figure 4), suggesting that the configuration chosen for the angled baffle testing was indeed a valid variant. The flow conditions during the transition from upstream baffle to downstream baffle control were unstable, with strong air-entraining vortices forming upstream of the upstream baffle, leading to oscillations in the approach channel. The instability was associated with water surface drawdown around the nose of the central pier and the formation of a standing wave immediately upstream of the baffles.

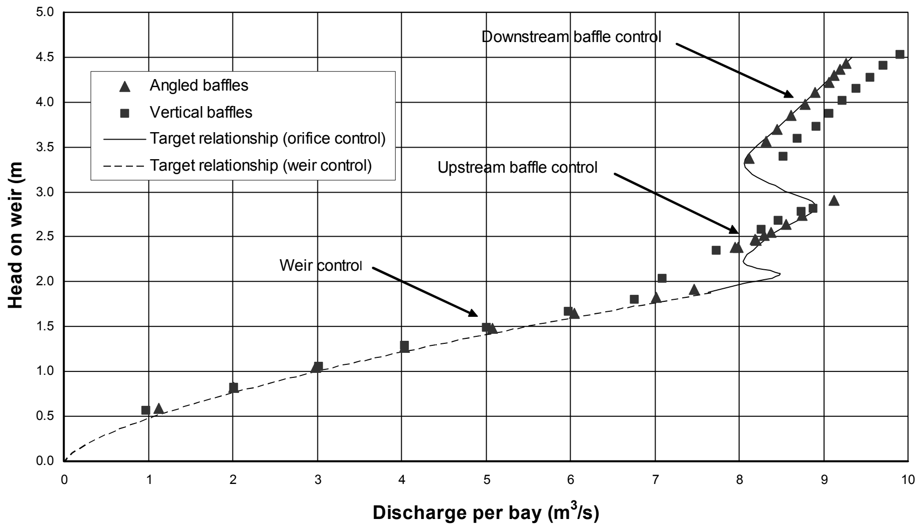


Figure 4 Stage-discharge relationships from preliminary tests

The preliminary vertical baffle arrangement gave a stage/discharge relationship (Figure 4) which diverged from the target relationship somewhat, with an earlier transition from weir to upstream baffle control. The flow conditions during the transition were again unstable, leading to oscillations in the approach channel, but without the strong vortex action found in the angled baffle device. The instability was again associated with the surface drawdown around the nose of the central pier.

Another notable feature was that the initial water surface contact on rising stages was with the downstream baffle, although the resulting effect on the approaching flow profile caused rapid contact with the upstream baffle, which then took over flow control, with the downstream flow surface clearing the underside of the downstream baffle.

Optimisation tests
As a result of the observations in the preliminary tests, it was decided to extend the central pier further upstream and to change its nose shape to a lens, with a 90° internal angle, in order to reduce the severity of the local drawdown and to allow substantial recovery upstream of the weir crest and

baffles. Because the angled version of the device had already closely met the target relationship, efforts were concentrated on optimising the performance of the vertical baffle option, as this was seen to offer a number of potential advantages, if a satisfactory rating relationship could be achieved.

A total of eight different versions of the vertical baffle device were investigated, adjusting the elevations of both baffles and the spacing between them, but in all cases with the upstream face of the upstream baffle 1000mm downstream of the vertex of the Crump weir. The various adjustments made were aimed at achieving two effects:

- a narrow range of discharges under baffle control, with the curve for downstream baffle control lying directly above the curve for upstream baffle control; and
- if possible, a direct transition from weir control to upstream baffle control, without the water surface first impinging on the downstream baffle.

The tests confirmed what was expected, that these two objectives are mutually exclusive – raising the downstream baffle to avoid it impinging first on the water surface inevitably shifts its control curve to the right and therefore increases the spread in the rating relationship.

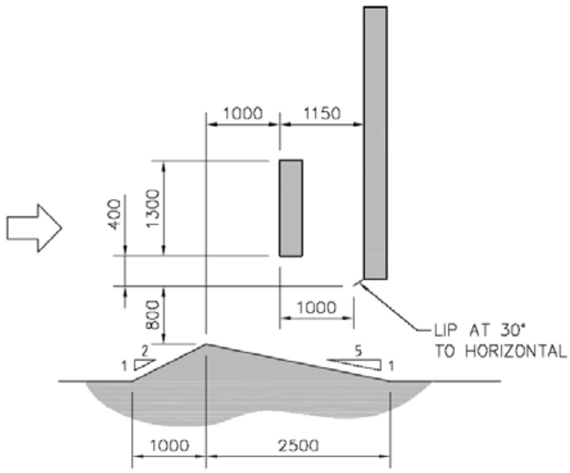


Figure 5 Optimised baffle arrangement

In order to reduce the spread between the curves for upstream and downstream baffle control, it was decided to introduce a small angled plate onto the front of the downstream baffle, resulting in the configuration shown in Figure 5. By making the contraction effect for the downstream baffle more severe than that for the upstream baffle, this had the desired effect, as illustrated in Figure 6. This figure also shows for comparison the relationships for the two designs shown in Figure 3, but now with the central pier extended.

Measurements throughout the preliminary and optimisation tests were made under virtually steady-state conditions, but in the course of rising or falling stages. In no case was any hysteresis effect detected.

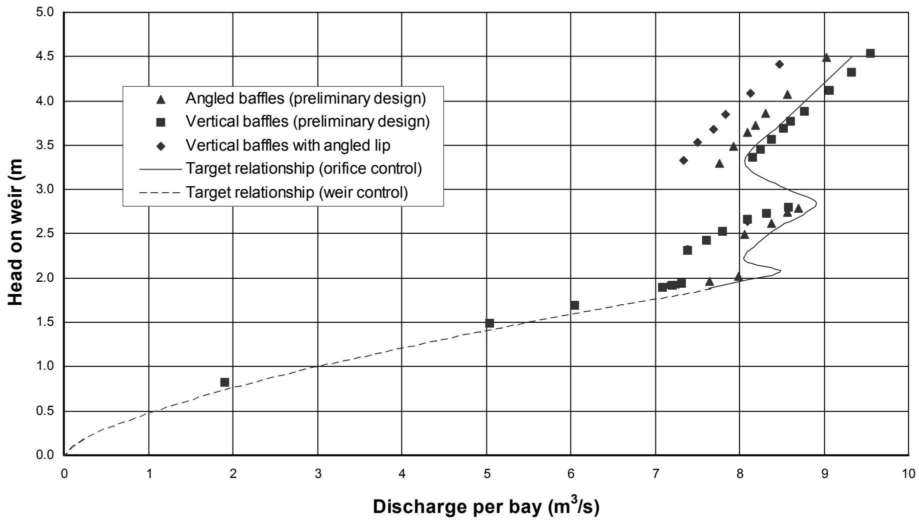


Figure 6 Stage-discharge relationship for optimised design

Although the results for the design shown in Figure 5 lie typically about 10% to the left of the target relationship, this was considered acceptable, because the requisite discharge capacity could be achieved by simply increasing the width of each bay to approximately 1.65m, which would have minimal cost and layout implications.

Final tests

The optimised baffle design shown in Figure 5 was built in both bays and tested under the following tailwater conditions:

- low, as used in the preliminary and optimisation tests;
- rising stage, as on the rising limb of a severe flood hydrograph; and
- falling stage, as on the recession of a severe flood.

The rising and falling stage tailwater levels were taken from mathematical model simulations of the 200-year return period flood, with the scheme in place and using the target rating relationship for the control structures. They are not necessarily representative of all flood conditions under which the flood storage reservoir will operate, but nevertheless give a realistic indication of the general effects of the natural range of tailwater levels on the performance of the control device.

Figure 7 shows the results for all the above cases, including a 'by-eye' best-fit line for the rising tailwater case. The results plotted with solid symbols relate to steady-state measurements, whilst those with open symbols are the results of measurements when steady conditions could not be maintained. (In the latter case, the discharge was measured taking account of the rate of change in the volume stored in the model between the device and the flowmeter.)

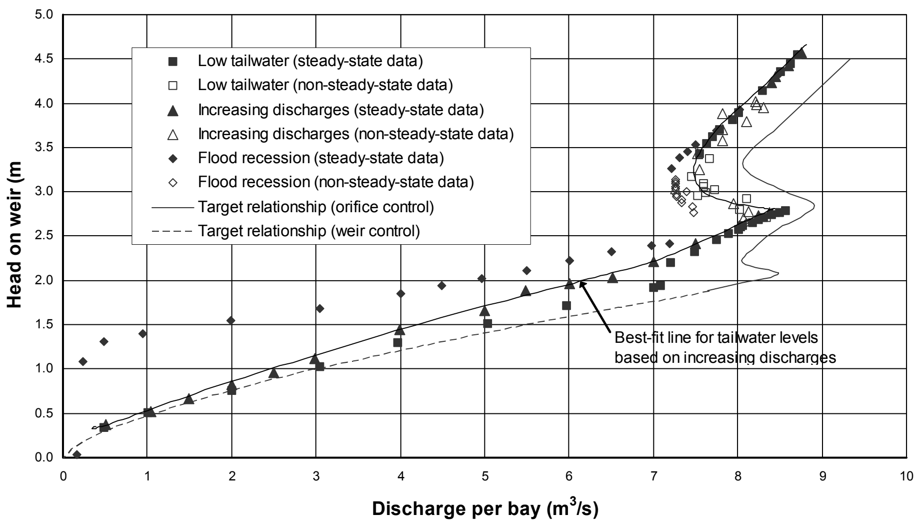


Figure 7 Final stage-discharge relationship for optimised design in two bays with various tailwater conditions

It may be noted (by comparing the results with those in Figure 6) that, for low tailwater levels, there is only a marginal difference in performance for the twin-bay version compared with the single-bay version. With the tailwater levels based on rising flood stages, the rating is affected by tailwater for heads between approximately 0.7m and 2.2m, but there is no significant difference in the performance of the device for heads between 2.2m and 4.5m. On falling stages, the higher tailwater levels have a modest effect on the behaviour of the device for stages between about 3.9m and 2.5m and a larger effect at lower stages. It should be noted that the hysteresis in this case is wholly driven by the applied tailwater levels and is not a fundamental characteristic of the device.

Plates 1 to 6 show various stages of flow behaviour for the optimised design with low tailwater levels and rising upstream heads.

Plate 1 First contact with downstream baffle

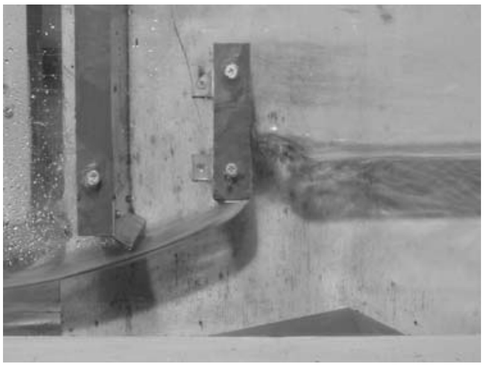


Plate 2 Control quickly transfers to upstream baffle

Plate 3 Upstream baffle starting to overtop

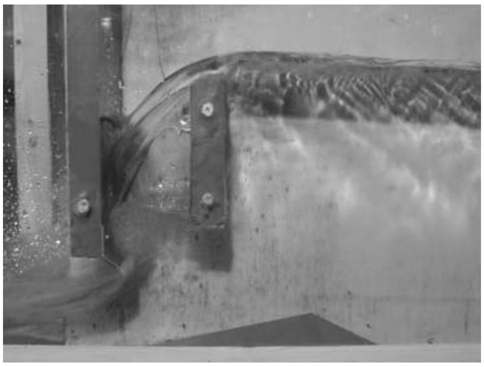


Plate 4 Downstream baffle starting to control flow

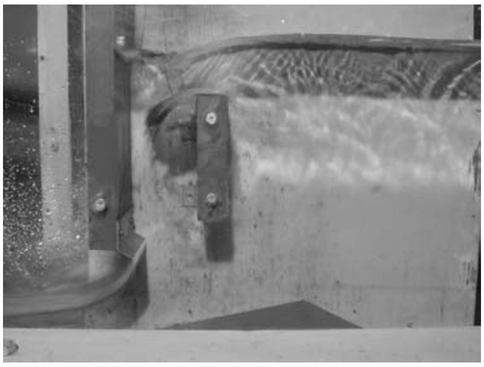


Plate 5 Upstream head still affected by weiring flow over baffle

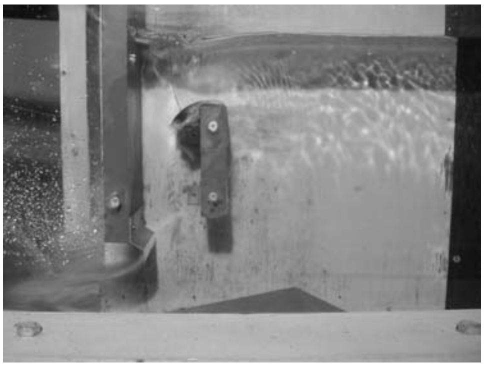


Plate 6 Upstream baffle virtually submerged

CONCLUSIONS

A passive flow-control device, based on a Crump weir profile and twin baffles, has been developed with the aid of a physical model, from the concepts embodied in the baffle distributor devices used in irrigation systems.

The device, which includes simple vertical baffles, with an angled lip on the downstream baffle only, is capable of controlling discharges within a band of ±10% for stages between 2.0m and 4.5m, provided that the downstream water level does not influence the flow conditions. No evidence of hysteresis was found.

On rising and falling stages in a simulated 200-year flood, the performance of the device is affected by the anticipated tailwater regime. On rising stages, which affect the utilisation of flood storage, the bottom of the ±10% discharge band is raised from about 2.0m to 2.2m.

ACKNOWLEDGEMENTS
The authors thank the Environment Agency for permission to publish this paper and gratefully acknowledge the assistance of their colleagues who provided assistance and comments on the draft paper. Whilst the model test results and other statements in this paper are believed by the authors to be an accurate reflection of the research work described, they should not be relied upon for any other purpose.

REFERENCES

Alsthom Fluides (undated). *Distributors.* Leaflet ref A6510A (similar to Neyrpic, 1971)

Bos, M.G. (1989). *Discharge measurement structures,* 3rd edition. International Institute for Land Reclamation and Improvement, Wageningen.

Neyrpic (1971). *Distributors*. Leaflet 65 10 71 A. Neyrpic Départment Fluides, Grenoble.

United Nations Food & Agriculture Organization (1975). Small hydraulic structures

Challenges on dam safety in a changed climate in Norway

GRETHE HOLM MIDTTØMME, The Norwegian Water Resources and Energy Directorate, Oslo, NORWAY

SYNOPSIS. Much research has been done recently on the possible future scenarios of climate change. The effects on runoff and extreme precipitation for Norway have been investigated, and the results indicate that there will be larger and more frequently occurring floods in the future. Some possible effects on dam safety in Norway are presented as well as some recent examples of dam incidents caused by unusual climatic conditions.

INTRODUCTION

Over the past few decades there has been an increased interest in climate change issues. Whether climate change is caused by natural variations or man-made emissions into the atmosphere (or a combination of these two mechanisms), we should be prepared to handle possible effects of the scenarios given by the researchers. A changed climate will evidently result in changes in the basis for safety evaluation of dams and other hydraulic structures, and updating of the design flood estimations may be necessary. Today the need for updating of the design flood estimations in Norway is evaluated as part of the regular dam safety reassessments, normally every 15 years according to the guideline on inspection and reassessment. That is, when there are considerable changes in the data series that has been used for the estimation of design floods, new design flood estimations must be performed. The guideline on inspection and reassessment is one of several new guidelines on dam safety that have been published over the last few years, and more are currently being prepared or are planned in the near future. The guidelines describe how the requirements in the regulations on dam safety can be fulfilled. Today the following regulations form the legal framework for dams, all of them with a legal basis in the Water Resources Act of 2001;

- Regulations on dam safety
- Regulations on classification of dams
- Regulations on qualification requirements
- Regulations on internal control

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

The development of the legal framework for dams in Norway is described in a recent article by the author (Midttømme, 2003). The Norwegian Water Resources and Energy Directorate (NVE), which is the regulatory authority on dam safety in Norway is responsible for the development of the new guidelines.

FLOOD ESTIMATION

Around 1980 major changes in the flood estimation methods were introduced in Norway. This resulted in a need for updating of the design flood and probable maximum flood (PMF) estimation for most Norwegian dams. Since then, new flood estimations have been carried out for more than 800 dams (the total number of classified dams is approximately 2300), sometimes followed by upgrading of the dam structure or spillway (Pettersson 1998). A new guideline on flood estimation was prepared as part of the recent revision of the legal framework on dam safety mentioned above. This new guideline requires that flood estimations be classified with respect to uncertainty based on an evaluation of available data. In addition, sensitivity analyses of the flood estimations are recommended. Otherwise, there are no significant changes with respect to methods for estimation of design floods and PMF in the revised regulations and guidelines. The new legislation will therefore not trigger a new general revision of design flood and PMF estimation for dams in Norway. The present method for flood estimation is described briefly in the following paragraphs.

In Norway two floods are defined; for spillway design and dam safety control, respectively:

- The "safety check flood" which must be bypassed safely without causing dam failure. Some damage to the dam may be accepted.
- The "design flood" which is a flood with a specific return period. This flood represents an inflow, which must be discharged under normal conditions with a safety margin provided by the freeboard. The design flood is the basis for the design of spillway and outlet works.

For high hazard dams the PMF is selected as the safety check flood and Q_{1000} (the flood with a return period of 1000 years) as the design flood, see Table 1 below.

The PMF is calculated by use of rainfall/runoff models on the basis of estimates of probable maximum precipitation (PMP). In most cases a snowmelt contribution should be added to the PMF. The design flood, on the other hand, has to be estimated with some kind of frequency analysis.

This is done, either by doing a single site analysis, or by doing a regional analysis. A regional analysis for Norway was prepared in 1978 and updated in 1997 (Sælthun 1997). The updated version introduces a new classification of regions and new estimates for the relationship between mean annual floods and the 1000-year floods. When flow records are insufficient or not available, the design flood can be calculated using rainfall/runoff models and estimates of precipitation events with a 1000-year return period. If possible, the results from this analysis are compared to calculated floods in similar catchments in the same area.

Table 1 Selection of floods in current Norwegian dam safety regulation

DAM HAZARD CLASSIFICATION (CONSEQUENCES)	DESIGN FLOOD	SAFETY CHECK FLOOD
HIGH	Q_{1000}	PMF
SIGNIFICANT/MEDIUM	Q_{1000}	PMF or 1,5 x Q_{1000}
LOW	Q_{500}	-

The 1000-year flood is defined in the guidelines as "*the inflow flood, with a return period of 1000 years, that results in the highest water level in the reservoir given particular conditions for operation of spillways and initial reservoir level"*. For routing through the reservoir in order to find the design outflow flood and the corresponding water level, one of the general requirements is that initial water stage in the reservoir be set to the highest regulated water level (HRWL). Transfer tunnels for water into the catchment are normally considered open, while transfer tunnels out of the catchment are considered closed. More details about requirements and methods for flood estimation can be found in the guidelines (NVE 2002) and in an article prepared for the ICOLD European Symposium in Barcelona in 1998 (Pettersson 1998).

SEASONAL AND REGIONAL FLOOD CHARACTERISTICS

Norway is a country with distinct seasonal and regional variations of climate and runoff. The variability in floods over the year and from region to region is exemplified by two catchments shown in the figure below. There are three main causes of natural floods: snowmelt; rain on snow; and rain. Autumn floods are caused by heavy rain and saturated soil, sometimes in combination with melting of newly fallen snow. Spring floods are a result of snowmelt and may be increased due to rain or melt water flowing over frozen ground. Spring floods tend to have longer duration than autumn floods, but there are exceptions. Typical areas dominated by spring floods are Southeast Norway and Finnmark, the northernmost county of Norway (on the mainland). In coastal areas there may be no seasonal distinction between spring floods and autumn floods. Floods may appear at any time of year, and summer is often a low runoff season. Some Norwegian river basins also contain glaciers. Glacial runoff can be dominant in catchments with glaciers covering only a small percentage of the area. Characteristic for these catchments is floods during summer (Sælthun 1997).

Many rivers are regulated and the purpose of most Norwegian reservoirs and dams is hydropower production. Consumption of water from the hydropower reservoirs is usually highest during winter, when there is normally very little inflow to the reservoirs due to precipitation falling as snow. Thus, the large reservoirs are mostly empty during late winter, which is a benefit when there are severe spring floods. On the other hand, the reservoirs are filled during summer and autumn, and offer very little storage capacity for autumn floods.

CLIMATE AND RUNOFF SCENARIOS FOR 2030-2049

As part of the research project RegClim, the global climate scenarios developed by IPCC (The Intergovernmental Panel on Climate Change) has been downscaled in order to prepare for impact studies of climate change in Norway. The most probable scenario, according to the RegClim-project, is that the annual temperatures in Norway will increase by 0.25 C/decade up to 2050 (Iversen, 2003). It may be worth noticing that the increase in temperature is estimated to be highest in the winter months and in the northern parts of the country. There is also an expected increase in annual precipitation for the whole country, and the highest increase is estimated to occur in the western part of Norway in the autumn. A minor increase in wind velocities and the number of storms is also expected, especially in Central Norway.

Based on the results from the RegClim-project, work has been done to estimate annual and seasonal runoff for the period 2030-2049 (Roald et.al. 2002). The runoff scenarios are based on two modelling strategies, i.e. modelling by the use of two different versions of the HBV-model for 42 catchments in Norway. Both modelling approaches are based on the same rainfall/runoff model. A comparison of the change in runoff simulated by the two models, show that the difference is generally quite small. The results differ by 2% or less for 30 of the 42 catchments. With a few exceptions, the results show a general increase in annual runoff for all the catchments for the period 2030-49 compared with the control period 1980-99. The highest increase in annual runoff is estimated to 20% in the western part of Norway. A study of the simulated changes of the seasonal runoff show that the winter runoff will increase significantly in the southern part of East Norway and in catchments along the coast of West Norway. The spring runoff will increase in the inland and the mountainous part of most of Norway, but the highest increase will be in the coastal catchments of Finnmark, the northernmost county of Norway. The summer runoff will decrease in most of the catchments, while the autumn runoff will increase in West Norway and Finnmark. More details are shown in Figure 1 below.

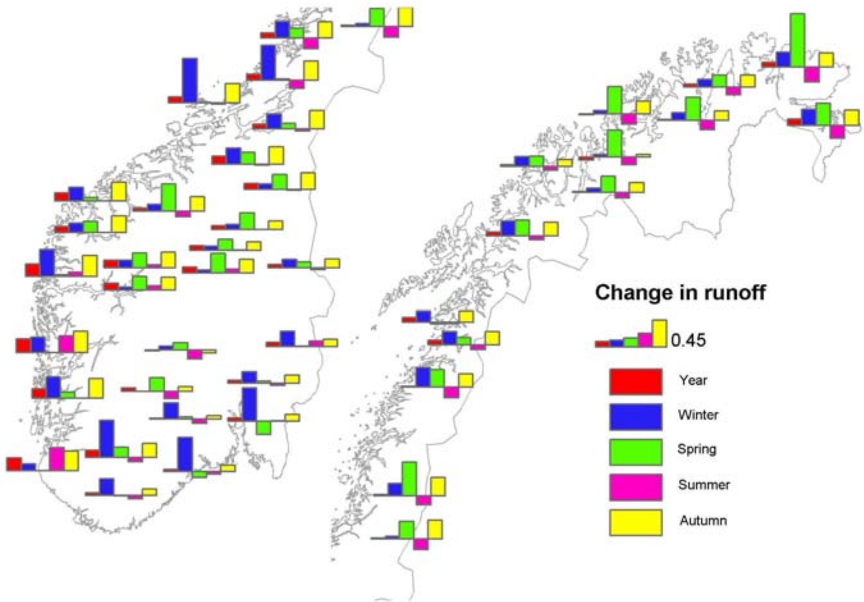


Figure 1: Seasonal changes in runoff for scenario-period 2030-49 compared to control-period 1980-99 (Roald 2003).

Scenarios of extreme precipitation of duration 1 and 5 days due to climate change have also been developed for Norway (Skaugen et.al. 2002). Time series of 1000 years have been generated on basis of downscaled precipitation values from a global climate model. The study indicates an increase in extreme values and seasonal shifts for the scenario period 2030-49, compared to the control period 1908-99. The study is based on simulations at 16 locations in Norway and the results show a significant regional variability.

POSSIBLE EFFECTS ON DAM SAFETY

The majority of dams in Norway were designed and constructed long before the first dam safety regulations were made valid in 1981. Even though many dams and spillways have been upgraded already to meet the present safety standards given in the new legal framework (see above), there is still a need for upgrading and rehabilitation of many dams, also as a consequence of damage due to deterioration and ageing processes. Floods and extreme weather will cause extra strain on dams and spillways, and the latest results from the studies of climate change and effects on runoff and extreme precipitation indicate that we should be prepared for more extreme weather and larger and more frequently occurring floods. For spillways that have rarely been in use so far, it should be noted that damage in the spillways might occur at discharges much lower than the design flood (Kjellesvig 2002). Recent studies on damages to concrete dams, which is the leading dam type with respect to number of dams in Norway, showed that 5 % of these dams had damages, which were considered to be a threat to the overall safety. It is also worth noticing that 44 % of the Norwegian concrete dams had been repaired already, and that 40% needed repair in the near future. There were also several reports about dams that had been repaired more than one time (Jensen 2001), indicating that the dam owners put too little emphasis in finding the actual cause of damage and/or the best repair method. In total, we may be facing a growing need for repair works in the future for all dam types. However, as there seems to be an increase in winter and/or spring runoff in larger parts of the country, it may be more difficult to find an appropriate time for doing necessary rehabilitation and upgrading works than what is the situation today. The present energy situation in Norway may also worsen the situation. There is an increasing gap between energy production capacity and energy consumption. As Norway is totally dependent on hydropower, it may be a problem to gain acceptance for closing down of hydropower facilities and/or lowering of reservoir levels in order to do necessary upgrading and rehabilitation of dams and spillways.

A significant increase in the design flood values for dams may be the result of climate change, and this will further lead to insufficient spillway capacity and/or freeboard at many dams. Even without climate change, floods larger than the design flood are likely to occur. Reasons for this may be for example short time series or incomplete time series used for design flood estimation. The challenge of floods exceeding the design flood is therefore relevant to discuss in any case, as well as the possibility of experiencing more extreme weather and more frequently occurring floods than what we have experienced so far. Typical problems related to operation of dams during floods and recommendations for handling large floods and more frequently occurring floods are given by Kjellesvig & Midttømme (2001) and Kjellesvig (2002). The main conclusion is simple; upgrade the dams and spillways in order to provide safe bypass of floods exceeding the design flood. Redundancy in spillway systems is also promoted as a safeguard against flood related damages and any following consequences, along with good monitoring systems for early detection of adverse conditions. Another possibility would be to lower the HRWL in order to increase the freeboard, and thereby be able to store more floodwater in the reservoir. A simple solution to the problem of increasing design flood values is always to add extra safety margins when a dam is upgraded in the future as recommended by Bergström (2003) in a recent seminar focussing on climate change within in the hydropower sector. The challenge is probably to persuade the dam owners to select a more expensive solution than required today, even though this may be a sensible and perhaps also economic solution in the long run.

As mentioned above, floods cause an extra strain on dams and spillways, and in some cases the utmost consequence of a flood is a dam failure. A recent example from Norway is the failure of Nervatn Dam in Norway in January 2002. Nervatn Dam was a small concrete dam classified in the low consequence class. The failure was caused by erosion of the right abutment, which resulted in the overturning of a 20m long section of the concrete dam (Pedersen 2002). The stop logs in the spillway had not been opened because the dam owner was too late in reacting to the increasing water level. The flood was caused by extraordinarily warm weather combined with rain (i.e. 250 mm in 4-5 days). The rainfall had an estimated return period of 200 years. The consequences were limited to damage to three downstream bridges including one main road bridge, and the loss of approximately 1 million m^3 of water (50 % of the reservoir capacity). The failure of Nervatn Dam can not be assigned to climate changes directly, but the incident is a reminder of the fact that dams may be more vulnerable to floods, and effects of floods, in the future.

The probability of experiencing more ice-related problems in the future is also of special interest to Norwegian dams. Changes in temperatures may influence on the probability of “ordinary” floods and the more rarely occurring glacier lake outburst floods (GLOFs; also denoted jøkulhlaups) in glacier dominated catchments, as well as on other ice-problems such as icings/aufeis and ice-jams. A recent glacier lake outburst flood at Blåmannsisen glacier in northern Norway is believed to be a result of climate changes (Engeset 2001). The flood was probably caused by a deficiency in ice-mass, that is, the accumulation of winter precipitation (as snow) could not compensate for snow and ice melt during summer. The flood at Blåmannsisen resulted in a 2.5 m increase in the water level in the Sisovatn reservoir, corresponding to a 40 millions m^3 increase in reservoir volume. The water level in the previously glacier-dammed lake, which released water into Sisovatn, decreased by 70-80 m. Due to the low reservoir level prior to the flood, the Sisovatn Dam and downstream areas were not affected (Josefsen 2001). An interesting point in the case of Sisovatn is that a glacier lake outburst flood from Blåmannsisen had not been considered a probable exceptional load on the Sisovatn Dam in the most recent safety reassessment. The consultant performing the safety reassessment of the dam had probably not been made aware of the possibility, whereas the hydrologists performing glaciological investigations had foreseen this possibility several years in advance (Pedersen 2003).

CONCLUSION

It is difficult to conclude on how climate change will affect the safety of our dams in the next few decades, even though some scenarios have been pointed out as more probable than other scenarios by the researchers. The key parameter with respect to dams is the design flood, but the frequency of smaller floods is also of interest. Given the most probable scenarios, we should be prepared for more extreme weather and larger and more frequently occurring floods. The result will probably be an increased need for upgrading and rehabilitation of dams. As a regulator on dam safety, NVE will continue to evaluate the results from the climate change research. Further studies on climate change effects in Norway will naturally be of special interest in this context, as well as other research related to dam safety. The current approach to climate change with respect to dam safety is to evaluate actual changes in the data series used for flood estimation, i.e. to trigger a reaction to any observed changes in the climate (belated wisdom). The guideline on flood estimation points out that flood estimations must be classified with respect to uncertainty based on an evaluation of available data. This classification of the flood estimations is meant to be a support for NVE in the further evaluation of any structural or non-structural measures for the dams. The guideline on flood estimation also recommends sensitivity analyses. The latter can be helpful in order to evaluate possible consequences to dams of specific climate scenarios.

REFERENCES

Bergström, Sten (2003). *Klimat och vattenresurser – senaste nytt från Sverige* (in Swedish). Presentation at EBL-seminar, Stjørdal, Oct.2003.

Engeset, R.V. (2001). *Klimaendringer gir jøkulhlaup ved Blåmannsisen* (in Norwegian), www.nve.no, Nov.2001.

Iversen, Trond (2003). *Sluttrapport RegClim 1997-2002* (in Norwegian), http:\\regclim.met.no, Nov.2003.

Jensen, V. (2001). Survey about Damage, Repair and Safety of Norwegian Concrete Dams. In Midttømme et.al. (eds) *Dams in a European Context*, Balkema/Swets & Zeitlinger, Lisse.

Josefsen, Aage (2001). Personal communication.

Kjellesvig, H.M. and G.H. Midttømme (2001). Managing dams and the safe passage of large floods. *Hydropower and Dams*, issue 2, vol 8.

Kjellesvig, H.M. (2002). *Dam Safety – The Passage of Floods that Exceed the Design Flood*. Dr.ing.thesis 2002:84. Norwegian University of Science and Technology, Trondheim.

Midttømme, Grethe Holm (2003). Changes in the legal framework for dam safety in Norway. *Hydropower and Dams*, issue 5, vol 10.

NVE (2002). *Retningslinje for flomberegninger til § 4-5 i forskrift om sikkerhet og tilsyn med vassdragsanlegg* (in Norwegian). NVE, Oslo.

Pedersen, V. (2002). Personal communication.
Pedersen, V. (2003). Personal communication.
Pettersson, L.-E. (1998). Flood estimations for dam safety in Norway. In Berga (ed*) Dam Safety*, vol 2, Balkema, Rotterdam.
Roald, L.A., T.E. Skaugen, S. Beldring, T. Væringstad, R. Engeset & E.J. Førland (2002). *Scenarios of annual and seasonal runoff for Norway based on climate scenarios for 2030-2049.* Consultancy report no10-2002, The Norwegian Water Resources and Energy Directorate, Oslo.
Roald, L.A. (2003). *Klimaendringer og store flommer* (in Norwegian). Presentation at NNCOLD-seminar, September 2003, Stjørdal, Norway.
Skaugen, T., M. Astrup, L.A. Roald and T.E.Skaugen (2002). *Scenarios of extreme precipitation of duration 1 and 5 days for Norway due to climate change.* Consultancy report no 7-2002, The Norwegian Water Resources and Energy Directorate, Oslo.
Sælthun, N.R. (1997). *Regional flomfrekvensanalyse for norske vassdrag* (in Norwegian). Report 14-1997, NVE, Oslo.

Weedon Flood Storage Scheme - the Biggest Hydro-Brake® in the World

G P BOAKES, Design Project Manager, Halcrow Group, UK.
A STEPHENSON, Proposals Manager, Hydro International, UK.
J B LOWES, Construction Project Manager, Halcrow Group, UK.
A C MORISON, All Reservoir Panel Engineer, Halcrow Group, UK,
A T USBORNE, Project Team Manager, Environment Agency, UK.

SYNOPSIS. The Northamptonshire villages around Weedon in the upper River Nene valley, suffered disastrous flooding in 1947, 1992 and 1998, with Weedon Bec being particularly badly affected. The channel through the village is constricted by historic developments and the opportunity to enlarge the channels was not available. Restricted culverts under the railway embankments downstream compounded the flood situation. To alleviate the problem the Environment Agency and Halcrow Group developed an upstream on-line storage reservoir scheme.

The project includes a 450m long, 6.8m high clay embankment across the valley, with a culvert on the line of the original river channel to carry the controlled outflow. A 150m long concrete-block spillway carries excess flood flows over the embankment. The embankment site has been landscaped to minimise visual impacts and the borrow area has been developed into a large wetland area as a habitat for aquatic flora and fauna.

The key component of the flow control system is a 6.5 tonne, stainless steel Hydro-Brake® Flow Control device located in the dam inlet structure. The Hydro-Brake® was designed by Hydro International to control the maximum outflow rate despite fluctuating head, and incorporates the facility to adjust the controlled outflow between 8 and $12m^3/s$. The use of the Hydro-Brake® helped reduce the upstream storage requirement and hence the land take and frequency of flooding involved.

This paper provides a description of the options considered during the design stage of the flood defence scheme, details of the actual design and construction of the dam, an explanation of how the Hydro-Brake® operates and the benefits it provides over other forms of flow control.

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

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

BACKGROUND TO THE PROJECT

The Problem

The village of Weedon Bec is situated west of Northampton and suffered serious flooding from the River Nene during Easter 1998. The village had no formal flood defences and there was a risk of flooding once every three years. 95 properties were at risk of flooding, 45 were flooded in the Easter 1998 event and many others were affected. The major cause of flooding was the restriction to flow at a road bridge within the village and at the culverts under the railway embankment downstream of the village.

A range of options was considered, but it soon became apparent that all options other than upstream flood storage were unacceptable. Channel improvement to pass flood flows required the existing river channel to be doubled in size, producing unacceptable loss of land and disruption. The cost of enlarging the road bridge and railway culvert would also have been very high. This option also produced an unacceptable increase in downstream flows through several villages and Northampton, which were already at risk of flooding. Containment of floodwater within the river channel would have required construction of flood walls through 30 private gardens. The cost would have been high, there would have been unacceptable disruption to residents and there would be access problems for future inspection and maintenance.

Conveniently, within one kilometre upstream of the village, the river flows through a well-defined valley with little habitation and this forms a suitable location for flood storage.

Scheme Selection

Having determined that flood storage was a viable and acceptable option, studies continued to determine the location of the dam and storage area and the most economic standard of flood protection.

The dam location was determined by consideration of:-

- Minimising the size and cost of the dam while achieving the required storage capacity.
- Avoiding flooding of property within the flood storage area.
- Minimising visual impact.

The location was largely dictated by the position of public roads and the Grade II Listed Dodford Mill, which is adjacent to the river approximately one kilometre upstream of the village. The dam is located approximately 100 metres upstream of the Mill, behind a belt of trees that obscures the view of the dam. Consideration was given to locating the dam 100 metres further upstream at the confluence of two branches of the river, but this would have resulted in a lower, longer, more expensive dam. A smaller dam could have been located further downstream but this would have resulted in the regular inundation of Dodford Mill, making it uninhabitable.

The standard of flood protection provided by the flood storage area was determined by economic evaluation. The project was grant aided by Defra. The economic evaluation, carried out using Defra procedures, determined that the project qualified for grant aid and that the most economic standard of protection would be 1 in 50 years.

DESCRIPTION OF THE PROJECT

The scheme was completed in autumn 2002 and comprises an earth fill dam with a maximum height of 6.8m and a crest length of about 450m. The storage area occupies the valleys of the Newnham and Everdon arms of the River Nene as shown on Figure 3. The capacity of the reservoir to spillway level is 810,000 m^3, providing a 1 in 50 year standard of protection to Weedon Bec. The flooded area at full capacity is 370,000m^2. The in-bank capacity of the river channel through the village of Weedon Bec is 10 m^3/s. The flood storage reservoir reduces the peak flow through Weedon Bec from 26 m^3/s to 10 m^3/s during a 50-year event. Figure 1 shows the dam under construction.

Figure 1: Weedon Dam under construction. The borrow pit is at the lower right. Note retained tree and hedge lines screening the embankment.

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

Embankment & Cut-off

The underlying geology at the dam site is blue Lias Clay. On the sides of the valley this was found directly beneath the topsoil. In the valley bottom it was covered to depths of up to 4m by mixed alluvial deposits ranging from soft silty clays to shallow sand and gravel beds. The gravel beds were considered to be potentially interconnected to the existing river channel and sufficiently permeable to provide seepage paths beneath the embankment.

The embankment was founded on the surface of the alluvium after topsoil and surface stripping. A cut-off trench was excavated to the Lias Clay beneath the centre of the embankment on either side of the outlet culvert, itself founded on the solid clay in the original channel bed, and backfilled with clay. The gravelly material excavated from the foundation was retained and used to improve the roadway on the dam crest.

The embankment was formed as a homogenous clay bank, generally with 1:3 side slopes, using some 32,000m^3 of firm Lias Clay. Initial plans were to excavate this from a borrow pit on the hillside, but this was rejected in favour of a borrow pit in the valley bottom, reinstated to form a wetland, even though this required the removal and stockpiling of between 1.5 and 4 m depth (12,000m^3) of alluvial overburden. Consideration was given to using the alluvial material in the upstream shoulder of the embankment, but it proved too soft to withstand tracking without drying.

The clay fill was placed and rolled at natural water content to form a hard fill material. Despite this, the clay material has the potential to crack on drying, always a concern on flood embankments normally kept empty. To help counter this, a horizontal geo-mat was incorporated in the non-spillway sections of the bank 0.5m below finished crest level, and the crest was topped with hoggin formed by mixing the clay fill and alluvial sands and gravels from the cut-off trench, stockpiled for the purpose.

The borrow area has now been landscaped to form a lake surrounded by tree and shrub planting. As much as possible of the original, established hedge and tree lines around the site have been preserved, and additional areas around the dam have been planted as woodland to break up the view of the dam from a distance.

Spillway

Located upstream of Weedon Bec, the reservoir is Category A in accordance with "Floods and Reservoir Safety" and was designed to safely pass the Probable Maximum Flood (PMF) which was assessed to be 195m^3/s. The spillway is formed by a 150m long lowered section of the dam crest. The crest, downstream slope and buried stilling basin are reinforced with tied cellular concrete blocks so that the spillway can safely pass the PMF with a depth of some 800mm over the crest and a maximum velocity on the downstream face of less than 8m/s. The downstream face of the spillway section was flattened to 1:4 to achieve this. The concrete blocks have been covered by a sacrificial layer of topsoil planted with grass so that the embankment blends in with the surrounding countryside when viewed from a distance. The non-spillway section of the embankment has a crest level 1.6m above the spillway crest to provide the recommended wave freeboard.

Downstream of the buried stilling basin, the water discharging from the spillway passes through an existing mature hawthorn hedge, on to fields forming a gently sloping flood plain, and from thence back to the river. As the spillway will only operate with the downstream channel already bank full, only minor erosion is expected downstream of the spillway, even in extreme floods, and out-of-bank flooding downstream is expected to be less frequent than at present.

Outlet Structure and Controls

The flow from the reservoir passes through a 2.4m wide by 2.1m high box culvert constructed on the line of the original river channel. Alternative options for controlling flows through the culvert were considered. An essential requirement of any option was that it had to be capable of permitting passage of both fish and small aquatic mammals through the culvert and control structure under normal operating conditions.

Alternative controls considered included:-

- A fixed orifice with an area of 1.2m^2, limiting the downstream discharge to 10 m^3/s at full head.
- A penstock located at the upstream end of the culvert. This would have initially been set to provide a fixed orifice with an area of 1.2m^2. Use of a penstock would permit manual adjustment should this prove necessary. However, because of the height of the dam, it would have been difficult to provide a visually acceptable arrangement to allow the penstock to be adjusted during a flood event with the reservoir full.
- A penstock housed in a chamber within the dam so that it could be adjusted manually from the crest during an event.
- An electrically or hydraulically actuated penstock to automatically adjust the penstock as the reservoir filled to maintain a constant downstream peak discharge of 10 m^3/s.
- A float operated radial gate to maintain a constant discharge of 10 m^3/s
- A Hydro-Brake® which provides a reasonably constant discharge up to a maximum of 10 m^3/s.

Controls at the downstream end of the culvert were avoided because this would have pressurised the culvert though the dam, which has the potential to lead to leakage and consequent hydrostatic pressures within the dam fill.

Various of the above options were rejected for the following reasons:-

- Arrangements to allow manual operation of penstocks during a flood event were not considered to be of practical benefit because it would be unrealistic to expect Agency staff to operate them safely during a flood event. The penstock would therefore, in effect, act as a fixed orifice.
- A penstock housed in a chamber within the dam would create a confined space, which was not acceptable to the Agency.
- A fixed orifice would cause unnecessary, frequent and significant flooding upstream of the dam which would limit use of the land for agriculture, which was unacceptable to the affected landowners. Early storage of water did not, however, have a great influence on the height of the dam.
- There is no power supply near to the site for automatic gate operation and to provide this added greatly to the scheme cost. There would also be a risk of power or equipment failure during a flood event.
- There is a risk of failure of operation of equipment only intermittently used or tested, which was unacceptable to the Agency.
- There would be a significant maintenance requirement, which the Agency wished to minimise.
- A float operated radial gate across the culvert exit controlled by downstream water level was rejected because it would have pressurised the culvert and required maintenance.

The Hydro-Brake® was chosen on the basis of its simplicity, low maintenance requirements and relatively low cost for this site. The final arrangement is shown in Figure 2. The Hydro-Brake® restricts the flow more at low head than an automatically controlled penstock, but it does allow a reasonably constant discharge to pass at both high and low heads. A comparison of the stored flood levels and storage areas for the control options is given in the following tables. Figure 3 shows the flooded areas.

The data in these tables show that the use of a fixed orifice rather than a Hydro-Brake® would have only increased the dam height by 300mm. However at low return periods (when there is no need for flood storage to prevent flooding in Weedon Bec) the flood level and area flooded are much lower with a Hydro-Brake® or automatic penstock. With a Hydro-Brake®, in a 1 in 3 year event, the flooded area is limited to the area immediately in front of the dam, which is largely occupied by the borrow pit. With a fixed orifice the flooded area would extend into the surrounding fields.

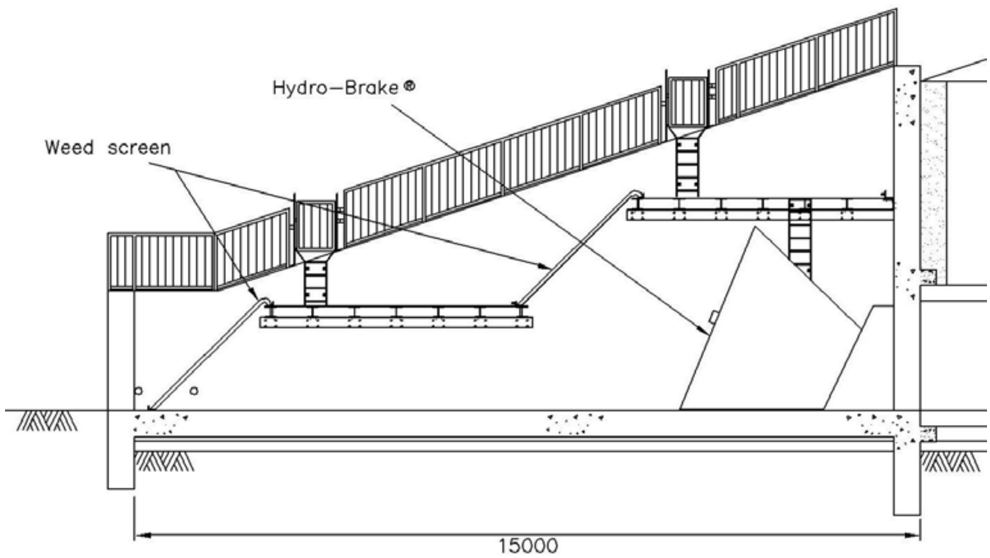


Figure 2: Section through the intake structure and Hydro-Brake®

Table 1: Storage Areas and Levels

Control	1 in 3 Years		1 in 50 Years	
	Level (mAOD)	Flooded Area (m^2)	Level (mAOD)	Flooded Area (m^2)
Fixed Orifice	89.6	146,145	91.7	417,890
Automatic Penstock	88.2	21,660	91.5	379,610
Hydro-Brake®	88.6	49,635	91.4	369,370

Table 2: Approximate Return Periods at which Storage would Commence

Control	Return Period when Storage Begins
Fixed Orifice	1 in 1 year
Automatic Penstock	1 in 3 years
Hydro-Brake®	1 in 1 year

As can be seen from the above figures an automatic penstock would have reduced the flooded area in low return periods further than a Hydro-Brake® but this was not possible for reasons explained previously.

Adjustment of the Hydro-Brake® is possible so that the peak discharge can be varied from the value determined by computer modelling should this prove to be necessary in practice. The peak discharge can be varied from 8 to 12 cumecs by the removal or addition of stop logs bolted across the Hydro-Brake® inlet.

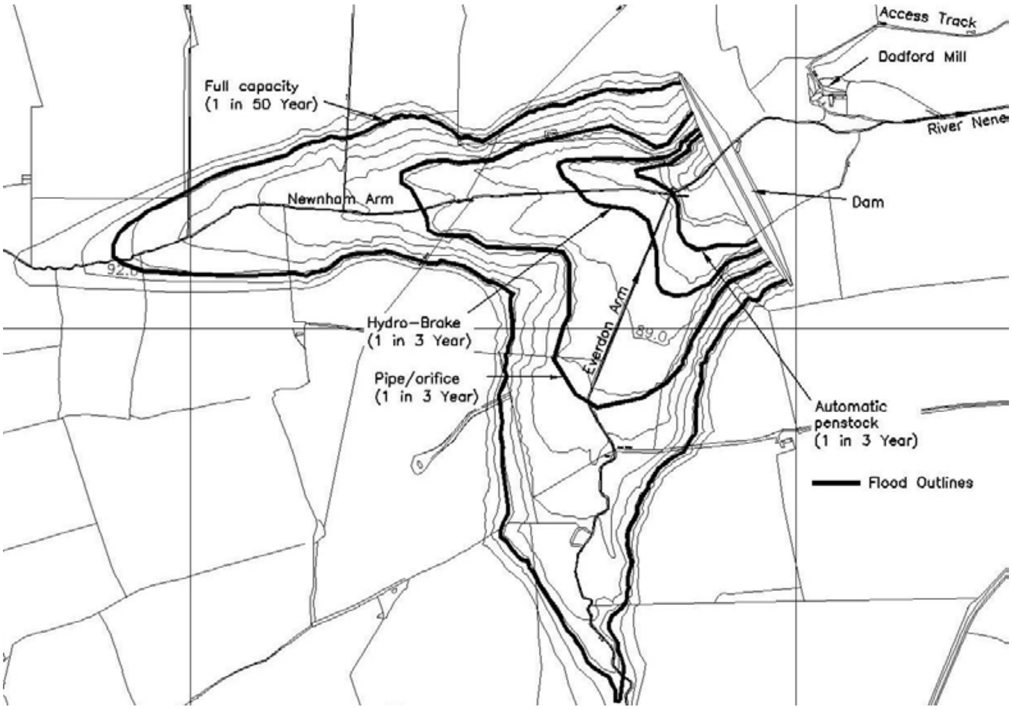


Figure 3: Comparison of flooded areas.

A trash screen has been provided upstream of the Hydro-Brake® and there is also a security screen at the downstream end of the culvert to prevent access by unauthorized people, particularly children.

Water level sensors are provided upstream and downstream of both the trash screen and the security screen, these are linked by telemetry to the Agency's control centres in Peterborough and Kettering. This allows monitoring of water level upstream and downstream of the dam and also shows if there is a difference in water level across the screens indicating that there may be a build up of trash.

USE OF THE HYDRO-BRAKE® FOR FLOW CONTROL

History of development and previous use of the Hydro-Brake®

The Hydro-Brake® is a proprietary gravity operated vortex flow control device designed by Hydro International plc. Outwardly having the appearance of a coil-shaped or conchoidal 'shell', units typically range from less than 1m to over 3m in length. The secret of their proven performance lies in the precise design of their shape, size, inclination and approach characteristics – not in expensive and complicated mechanical engineering.

In the United Kingdom, the first known major use of vortex flow control was to control and dissipate energy in drop shafts. The first commercial application in the UK as an integral part of drainage infrastructure to attenuate flows and alleviate flooding, was in 1980. Worldwide, more than 13,000 Hydro-Brake® Flow Controls are already in use, the majority having been installed on new developments to maintain flow rates equivalent to those of the greenfield site (pre-development run-off rates).

Prior to the Hydro-Brake® at Weedon becoming the 'Biggest Hydro-Brake® in The World', its predecessor had been installed as part of the Ashford Flood Alleviation Scheme over 12 years ago. This unit, which is basically the same shape and type as the Weedon Hydro-Brake® (without the in-built adjustability), has an outlet approximately 1.25m in diameter, whereas the Weedon Hydro-Brake® has an outlet diameter of 1.75m.

Experience to date with the Aldington installation has been very positive with the Hydro-Brake® performing exactly as expected. During the flooding experienced in that area in October and November 2000 the storage area at Aldington was actually overtopped, whilst the Hydro-Brake® discharged at precisely the correct levels. This reservoir was designed to retain floods of up to 1 in 100 year return period with a controlled discharge, illustrating the severity of the rainfall at that time. It has been well documented that had the Ashford Flood Alleviation Scheme not been in place at that time, Ashford would have suffered enormously. Older parts of town, close to the international railway station, would have flooded and about 100 houses would have been under water.

There has been much development of the Hydro-Brake® Flow Control since its original conception over 20 years ago, with constant ongoing testing and research to improve the hydraulic characteristics and develop more efficient units. Several new types have been introduced in recent years providing larger openings thus reducing the risk of blockages, as well as improved head / flow characteristics which reduce the amount of upstream storage required without exceeding the maximum required flow rate.

<u>Hydraulic characteristics</u>

The Hydro-Brake® is a self-activating passive flow control device with no moving parts and requiring no external sources of power to operate it. Instead, it uses the inherent energy in the flow field to control flows in sewerage systems, drainage channels and outlets from storage systems.

As flows build-up, a Hydro-Brake® typically exhibits two distinct modes of operation (see Figure 4 below). In the first mode, termed pre-initiation, the unit behaves like a large orifice, allowing relatively high flow volumes to be discharged at low operating heads. As the operating head increases, the volute of the Hydro-Brake® fills and the upstream water energy is converted into rotary motion within the device. This generates increasingly higher peripheral velocities, which eventually results in the creation of an air core, occupying most of the outlet of the device. In turn, this produces a back pressure that opposes the through flow of water. This second mode, is termed post-initiation, the 'throttling' effect causing the device to behave like a conventional orifice control or throttle pipe having a significantly smaller opening than the outlet size of the equivalent Hydro-Brake®.

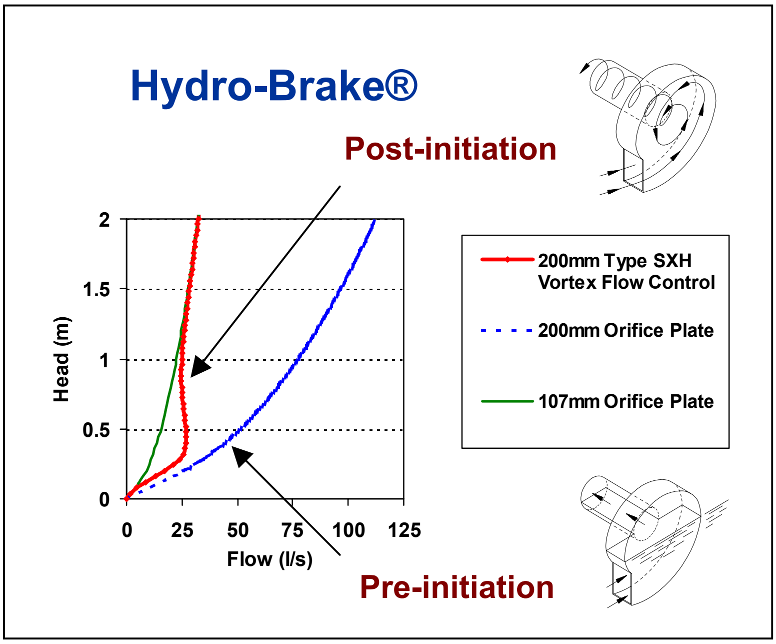


Figure 4: The Hydro-Brake® : Flow and Head Characteristics for the Pre and Post-initiation Phases

The two most obvious advantages in the use of a Hydro-Brake® at Weedon were reduced upstream storage requirements and comparatively larger openings. Other advantages include the lack of power required to operate as well as the absence of any moving parts. These factors coupled with the typical self-cleansing properties of a Hydro-Brake® result in a much reduced maintenance commitment.

Any drawbacks with the use of Hydro-Brake® Flow Controls tend to be either perceived or avoidable. The purchase costs are often quoted as a barrier, but can virtually always be outweighed by the savings in storage requirements and reduced maintenance costs. Another perception is that they are sometimes prone to blockage, especially when used in a foul / combined sewer application. Any flow control in a drainage or sewer system is, by its very nature, a restriction of some sort with an outlet size generally smaller than the system leading up to that point. The passage of objects larger than that opening is potentially a problem with any form of control and it is therefore important that consideration is given to preventing large masses from reaching it. It is true to say that the unique shape of a Hydro-Brake® generally prevents there being a straight path through the control, but with a correctly designed chamber or inlet structure including good benching etc., problems can always be avoided.

SCHEME CONSTRUCTION

An ECC Option C contract for the project was let to Edmund Nuttall Ltd in February 2001 with the flood storage dam comprising one section of a four-section contract for the Environment Agency. The agreed target price of £1.0 million was negotiated in April 2002 following completion of detailed design and having value engineered the project with the contractor.

Construction work commenced in April 2002 with a 34 week construction period. The planned sequence of operations is summarized below, although in practice there was some overlap of these activities.

- Establish site
- Install temporary bridge and divert the river into a temporary channel
- Construct culvert, headwalls and associated structures on the line of the existing channel
- Divert the river back through the culvert and reinstate the temporary river diversion
- Strip surface, excavate cut-off and place fill to the cut-off and dam
- Place erosion protection to the embankment and spillway
- Install Hydro-Brake®, trash rack and security screen
- Place topsoil, reinstate site and landscape

After some delays early in the contract, fine weather allowed the earthmoving to proceed quickly, so that it was essentially completed in October 2002. High river flows then caused delay in installing the Hydro-Brake®, which was completed in November 2002. Completion of topsoiling, seeding and finishing works was delayed until spring 2003.

Installation of the Hydro-Brake® was programmed to take place after placing of the spillway blocks, as this would effectively cause the reservoir to become operational. As it turned out, placing of the Hydro-Brake® under winter rather than summer flow conditions was difficult, and would have been easier if done earlier. This could have been possible by leaving a temporary opening in the upstream headwall to supplement the flow through the Hydro-Brake® and prevent impounding in the reservoir until the spillway was ready.

Environmental Aspects

A mineral extraction licence had to be obtained from Northamptonshire County Council who were the approval body for the borrow pit and its restoration, whereas the building of the dam was subject to planning permission from Daventry District Council who were the approval body for the dam landscaping. The two bodies had different landscape approaches.

The dam landscaping was relatively straightforward. Cellular concrete blocks used on the spillway were topsoiled and seeded. Elsewhere, grass seed was sown on prepared topsoil. An area of this on the upstream face was covered with fibre erosion protection matting over the topsoil. Hedges that had been removed were replaced with new planting, and some additional screening woodland was planted.

The borrow pit area was less straightforward because Northamptonshire C.C. did not want to have a significant body of water in the restoration, although this had been part of the scheme concept preferred by the Environment Agency. As a value engineering exercise, the original restoration plan was modified using input from the main contractor with a specialist landscaping sub-contractor and the Halcrow project environmental scientist. The accepted restoration incorporates areas of native tree planting and wild flower mix seeding which provides a diversity of habitat. This area has now been handed back to the farmer owner who continues to manage it as envisaged although there is no formal agreement relating to it.

CONCLUSION

A detailed study of the options to address the frequent flooding in Weedon Bec identified an upstream on-line flood storage reservoir on the River Nene as the only viable solution. Investigation and design, with due regard to flooding frequency and environmental factors has produced an economic scheme, with minimum adverse impacts on the surroundings, largely using materials available on-site.

The selection of a Hydro-Brake® as a flow control has significantly improved the scheme hydraulic performance, particularly in reducing the frequency of flooding of the storage area, which in this case is actively managed arable farmland. While use of a Hydro-Brake® at Weedon is not unique in flood control schemes, this installation has pushed the boundaries forward in the scale of what can be accomplished using these proprietary devices.

Integrating design with the environment to maximise benefits from a flood storage dam: successful implementation at Harbertonford

W T BRADLEY, Design Team Leader, Halcrow Group Ltd, Leeds, UK
M E JONES, Environmental Assessment & Design Ltd, Exeter, UK
A C MORISON, AR Panel Engineer, Halcrow Group Ltd, Swindon, UK

SYNOPSIS. Environmental enhancement is not just about changing the seed mix and planting a few trees to hide the structures we build. Designing using the environment rather than seeing it as a constraint can help to produce cost-effective designs that bring great benefit to the environment and raise the public perception of the benefit dam engineering can bring to their lives. So often, the potential of embankment dams to benefit the environment is not taken advantage of. This paper explores what can be done to fully realise the potential of flood storage dams.

The award winning Harbertonford Flood Defence Scheme, described as "the future of flood defence schemes" by Sir John Harman, Chairman of the Environment Agency, was a combination of in-village channel lowering and upstream flood storage (Palmers Dam). This paper focuses on the flood storage element of the scheme and demonstrates the multifunctional benefits that have been delivered through the integrated work of design professionals, driven forward by the flood defence aim.

BACKGROUND

The UK flood defence industry

The flood defence industry is at present the source for the greatest number of Reservoirs Act dams being constructed in the United Kingdom. There are currently over 30 dams either recently constructed or under development. These dams vary in height from under five metres to over 15 metres and are generally earthfill embankment dams providing the temporary flood storage element of flood alleviation schemes. Such structures can provide cost-effective protection and offer flood defence benefits to the rest of the river catchment downstream through slowing floodwater progress and reducing flood peaks.

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

The Harbertonford Flood Defence Scheme

The Harbertonford Flood Defence Scheme, costing £2.6 million, provides flood alleviation to the picturesque village of Harbertonford, near Totnes in South Devon (refer to figure 1).

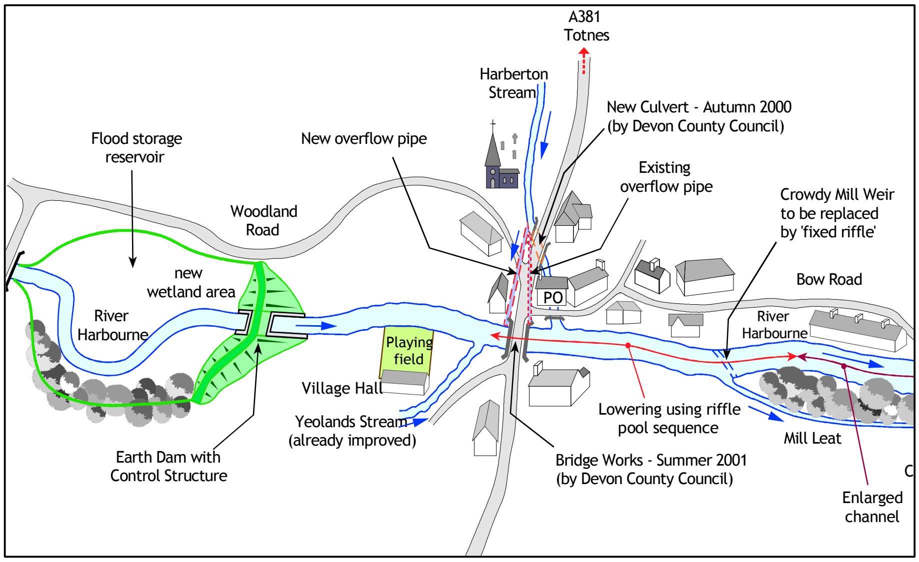


Figure 1: Schematic of overall scheme

Originating on Dartmoor, the River Harbourne flows through the village, where in the centre, by the historic bridge, it is joined by two tributaries. These watercourses have flooded the village 21 times in the past 60 years, including six times between 1998 and 2000. Flow in the River Harbourne varies from less than 1 cumec at low flows, to 28 cumecs at the 10 year flood flow, through to 300 cumecs for the PMF event (Probable Maximum Flood). The flashy nature of the catchment meant there was little warning for the residents of the village to prepare for the flooding and the misery it causes. One elderly resident of the village had resorted to living solely on the upper floor of her house.

This high frequency of flooding and associated damage resulted in considerable disruption and in January 2001, the scheme was accelerated by the Environment Agency (the client) to ensure that remedial works were designed and constructed in time to protect the village against possible flooding in winter 2002/03.

Solutions to the problem had been proposed during the previous thirty years, but all schemes previously put forward were deemed to cause too much damage to the environment and could not be justified. The village of

Harbertonford is designated as a Conservation Area and several listed structures, including the village bridge are also contained within it. Atlantic salmon, bullhead, sea trout and brown trout occur in the river and protected species are also present within the catchment, including otter and common dormouse.

The restrictions on the scheme development and the limited timescale focused the team to look at processes that could be used to achieve the scheme aim of providing a sustainable flood defence solution in place by winter 2002/03. This type of scheme would usually take four years to develop and implement. Only two years were available for this project.

Early studies showed that neither channel improvement nor upstream storage alone were capable of providing an appropriate level of flood mitigation. Storage sites available were too small to store the volumes of water required without unacceptable flooding of the upstream valley and channel works to carry full design flood flows required unacceptably large channels in the village centre or removal of downstream mill structures, which was also unacceptable. However an acceptable scheme was developed from a combination of both approaches.

THE DESIGN PHILOSOPHY

The benefits

Attention focused on five main objectives to ensure the design maximised the long-term benefits delivered. These were:

- maximise justifiable flood defence capability
- ensure reservoir safety
- minimise future operation and maintenance through working with the fluvial geomorphology of the river
- maintain and enhance biodiversity, amenity and landscape value
- minimise adverse effects on cultural heritage value

Through adding site-specific detail to these objectives, a clear framework was established to achieve the scheme aim of sustainable flood defence.

To deliver further long term benefits, the local school visited the dam under construction, and team visits to the school ensured the scheme construction and design were used as an educational resource, informing the children and teachers of the achievements of the scheme and the benefits it would bring. This promoted an understanding of their environment and allowed community ownership.

<u>The design process</u>

In describing the designs developed and techniques used at Harbertonford to deliver multifunctional benefits, it is worth summarising the approach used to generate these, which the authors believe represents a change from traditional practice.

The design philosophy adopted was to work with the environment, seeing it as an opportunity rather than a constraint. This meant using each other's skills most effectively; in particular, bringing the environmental scientist and other specialists to provide input directly into the design and not just comment on issues that needed to be considered, then periodically review designs.

Figure 2 below represents the values and perspectives held within the team. The process centred around the core values of Teamwork, Innovation and Consultation (TIC).

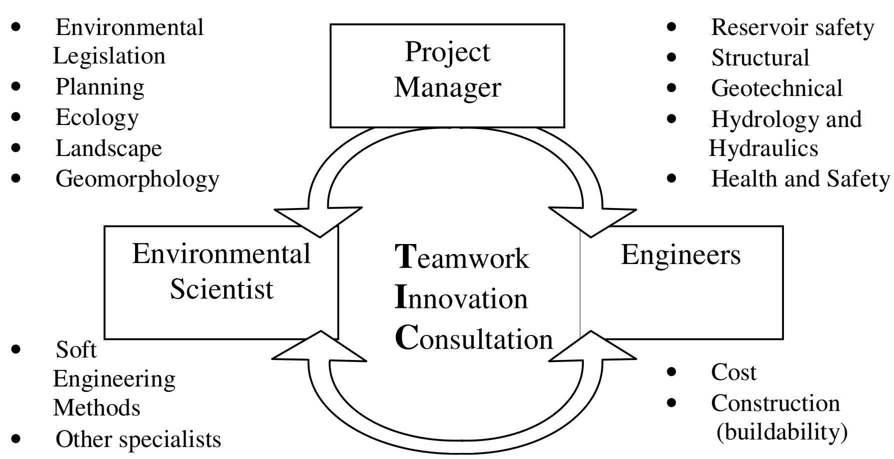


Figure 2: The TIC Process

Specialist advice was brought in at key stages throughout the overall scheme design, including notable inputs from the River Restoration Centre. Public consultation was also undertaken throughout the design process, the outputs of which further influenced the nature of the final scheme. This gave local 'ownership' of the scheme, which was considered to be very important for construction and operation phases.

All approvals were gained first time, on a fast-tracked schedule leading to a reduced design development time, earlier design certainty and more efficient designs.

THE DESIGN OF PALMERS DAM

Overview

The dam structure spans a 100 metre width of valley and is up to five metres high, constructed of 10,000m^3 of fill material. A concrete box culvert passes through the dam to carry the normal river flow. The outflow is controlled by two actuated penstocks with an automated control system and the structure can retain 150,000m^3 of floodwater, so is subject to the Reservoirs Act 1975. An overflow spillway occupies the majority of the dam crest.

Dam

The dam is located two kilometres upstream of Harbertonford in a quiet valley area with rural landscape that contains areas remaining from previous quarrying and milling industries. The area is designated as an Area of Great Landscape Value which was an important influence on the dam design. The structure was located away from public areas, but still provided good operational access and minimised tree loss.

The zoned, clay core embankment dam creates an online temporary impoundment of floodwater and by improving the channel capacity through the village from 15 cumecs (3 year flood) to 28 cumecs (ten year flood), the flood storage site available has been used to deliver the greatest flood defence benefit. Any further improvement of the channel capacity could not have been justified on cost and environmental grounds and the dam height was restricted to avoid flooding the public road and upstream properties and this determined the maximum justifiable standard of scheme.

The underlying geology at the dam site is slatey shale, weathered at the surface and overlain in the valley bottom by alluvium, typically to a depth of about 2m, consisting of a mixture of clays derived from weathered shale, and quartz sands. The valley sides consist of shale, weathered *in-situ*. The bedrock is exposed in the riverbed just downstream of the selected dam site.

The single-track, restricted-width public road to the site from the village was seen as a major constraint to construction, and it was decided early in the design process that as much of the material for the dam as possible should be sourced on site. This also minimized traffic impacts in the village.

Materials investigations showed that only a limited quantity of suitable alluvial clay for an impermeable core was available from a field upstream of the dam, typically as a 0.5m thick layer. To minimise the land area disturbed in excavating this, a zoned, clay core embankment dam with alluvial shoulders was adopted.

Spillway design

In view of the flood-prone village downstream, the spillway is designed to pass the Probable Maximum Flood (PMF) flood flow of 300 cumecs. The structural integrity of the dam is protected from the PMF overtopping flow by incorporating rock gabions in a stepped arrangement into the downstream face. These were placed at a gradient of 1 in 2 above the shoulder material surrounding the clay core. Integration of the landscape with the reservoir safety design is achieved by the addition of a zone of non-structural 'sacrificial' material above the rock gabions, creating a varying surface profile of between 1 in 4 and 1 in 8 (figure 3). This changed the visual appearance of the dam considerably.

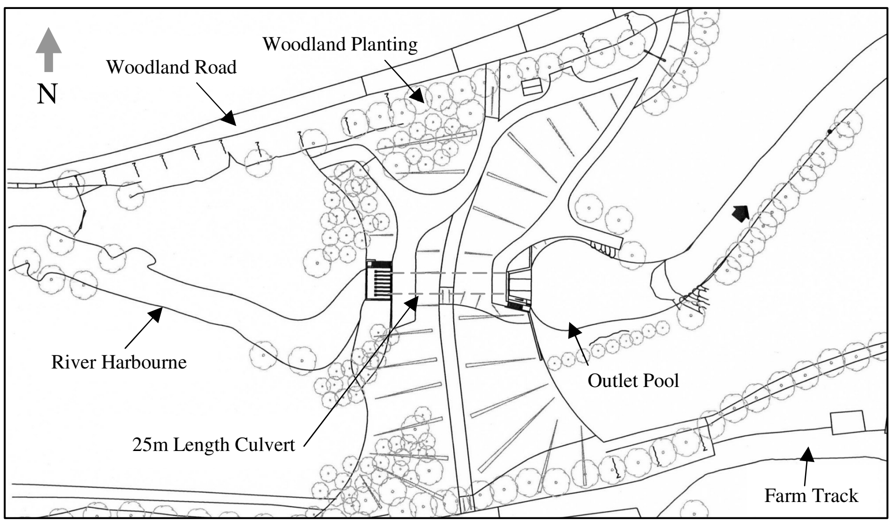


Figure 3: Layout plan of Palmers Dam.

The sacrificial material is covered with a mix of slower growing native grasses and wildflowers. The species chosen increase biodiversity whilst still retaining a good vegetative cover. Erosion resistance has been improved through the use of a 3-D geotextile in the root system. Importantly, the geotextile protection beneath the grass sward allows machine cutting of the grass when needed.

A Grasscrete crest locks the slope protection in place and the spillway occupies the full 100 metre length of dam crest to reduce overflow depth and the corresponding erosive force. Half of the spillway is set 0.5 metres lower and this is located on the southern section of dam, allowing continued operational access to the outlet structure (if needed) once overtopping flow has commenced.

Other design concepts

The upstream slope also incorporates a zone of non-structural material, again creating variable slopes, but also allowing the planting of broad-leaved woodland and scrub. This increases biodiversity and also provides a wildlife corridor linking the woodland on either sides of the valley. Badger protection mesh was placed against the 1 in 2 structural dam slope surface beneath the sacrificial material to protect from burrowing animals.

The sacrificial material represented an additional 5% on the cost of the dam, but since a minimal structural specification was needed, a wider range of materials could be used. This led to the project, including the channel enlargement works downstream, generating less than 10 percent waste to be taken away from site, most of this associated with contaminated material from the clean up of historic petroleum tanks during works through the village itself. The sacrificial material offers protection to the structural material, especially with regard to weather influences and retaining moisture contents.

Initial consideration was given to sourcing the gabion fill from waste in an abandoned slate quarry close to the dam site, so further reducing the amount of material brought to site. However enquiry revealed that the quarry and spoil tip posed difficult access issues and the area was an environmentally important area, so this idea had to be abandoned.

Culvert Design

The concrete box culvert which carries normal water flows through the embankment is designed to a four metre water width, similar to the natural river channel. This prevents throttling of the river flow and maintains the passage of the migratory fish, including salmon and sea trout, but also delays the point at which the culvert causes water to backup, reducing storage before the dam is designed to come into operation and associated sediment deposition.

The culvert is set with a lowered invert to allow a natural gravel bed formation and a minimum 300mm water depth. This follows best practice on migratory fish passage and has led to the creation of a varied bed profile, mimicking the natural river channel. The varying topography of the dam minimised the length of culvert needed to 25 metres, reducing build cost and length of river channel affected. With an internal height of over two metres, good airflow and access to both inlet and outlet structures, the lower risks for both authorised and unauthorised entry into the culvert ensures the culvert has not been classified as a confined space. This gives direct benefit related to future maintenance costs and safety liabilities.

Inlet arrangement

Screening of the culvert is achieved through bars at 600mm spacing. This allows most organic debris to pass through the culvert, fuelling the river system, reducing the risk of blockage and reducing the frequency of clearing operations. Screening to restrict unauthorised access would require a bar spacing of not more than 150mm, posing a higher risk of blockage, which could lead to earlier overtopping and consequential flooding of Harbertonford. This reasoning, together with the reduced safety liabilities of the culvert allowed agreement of the wider spacing. Additionally, the larger bar spacing allowed a smaller structure to be designed which has less impact on the landscape.

Constructing the culvert offline minimised the construction effects on the river and so consideration was therefore given to the best arrangement to guide the river to the new alignment. Previous dams have lined a new channel with hard bank protection such as gabions or walls, but the need for this was questioned. The main requirement is preventing scour of the embankment structure. This is most likely along the previous river path, so keying in a short length of blockstone into the bedrock in this location offers protection and a new channel excavated and lined with riverbed gravels guides the river to its new path. A bed check constructed upstream of the pool made from natural stone prevents long-term nickpoint erosion progressing upstream and contributes, with the natural curve of the river, to maintain a resting pool created at the inlet for fish passage (figure 4).

Figure 4: Photograph of inlet arrangement

Outlet arrangement

To dissipate energy and enable flow measurement, reinforced concrete stilling basins, as detailed by The United States Bureau of Reclamation (USBR) have regularly been used in south-west England. However to maintain accurate flow measurement, regular maintenance is needed to clear sediment deposition and ensure the structures dissipate energy as intended. At Palmers Dam, naturally outcropping rock was used to create a scour pool that improves fish habitat, is self maintaining and provides better landscape and visual amenity value. Water levels in the pool and culvert are maintained using a bed check created using natural slate. This was simple to construct and has performed exactly as intended from hydraulic perspective and visually mimics the existing rock outcrops.

Around the outlet (and throughout the scheme generally), sloping ground has been used rather than vertical drops wherever possible to reduce the need for fencing and allow members of the public who may get into this area a safe means of egress. Bird boxes were installed in the masonry faced walls as part of additional habitat creation.

Flow control system

Flood storage commences during a 1 in 10 year flood event. At a 1 in 40 year event the scheme standard will be reached and overtopping flow will cause the increased in-bank flow capacity through the village to be exceeded and progressive flooding to occur. The maximum storage efficiency making best use of the limited storage volume, the wide culvert, and the minimal screening were all possible due to the decision to adopt a variable gate control structure. This is located on the outlet structure which has minimised the impact on the landscape and allowed access to the gates even when the reservoir is impounding. A watertight culvert capable of withstanding up to a six metre head of water internally was achieved through using a combination of standard waterproofing seals between joints and a casing of mesh reinforced concrete around the box culvert structure.

The vertical penstock gates are normally held fully open, but are closed progressively to limit flows downstream to what can be accommodated within the river channel. The automated penstocks are controlled by a programmable control system which is operated in response to water levels downstream of the dam. Alternatively the gates can be closed remotely by a central flood control room in Exeter, however the gates can only be opened by manual control on site to ensure that the gates are not accidentally opened during a flood event.

Local power supplies were used and additional benefit delivered to the village through arranging the supply to be supported from two separate sources, thereby increasing the dependability of supply to the village as well as the dam. The programmable control system allows changes to be made to the operating regime in the future. Backup power systems have been put in place at the dam to minimise the risks of operational failure.

Sensors detecting the water level at the dam inlet now allow flood warnings well in advance of the level that was previously possible.

<u>Washland creation</u>

The benefits have been extended still further through taking advantage of the potential the borrow area provided. Twenty thousand tonnes of material was needed for the dam construction and this was substantially sourced from the field immediately upstream in such a way to create shallow scrapes. The remainder of the fill came from material excavated as part of the village works. The field was previously semi-improved grassland used for grazing and biodiversity has been increased through creating a wet woodland/grassland nature reserve. This design contributed to achievement of the Department of the Environment, Food and Rural Affairs (DEFRA) high-level biodiversity targets

Figure 5: Photograph of washland area one year after construction

Construction costs were saved through considering the final wetland profile at the outset, using the geotechnical information to target specific areas and depths and minimising earthmoving needed at the end of dam construction. The seeding and tree planting, which greatly surpassed the number of trees lost during construction has already started establishing within a year, creating an area that is regularly visited by the local population and a seating area has been provided to maximise this amenity value.

Wetland creation was only possible through the Environment Agency's decision to purchase the area involved. This aspect of the project has since been presented at the DEFRA annual conference as a case study project in best practice washland creation and showed the benefits of combined flood defence and biodiversity (Morris, 2003). Effective consultation from early stages with the landowners concerned has led to support for the solution and even an offer of assistance in the future management of the area.

CONCLUSION

There are a large number of embankment dams being constructed in the UK for flood defence purposes and consideration of the environment is a key factor associated with these. Palmers Dam was developed using an integrated team of professionals that brought the latest knowledge and experience into the process. This, together with proactive public and stakeholder consultation throughout, brought ideas and encouraged ownership of the scheme, all of which led to maximised benefits to the community, ecology and landscape. The designs also have reduced future operation and maintenance requirements and long-term health and safety liabilities through considered design and integration with the fluvial geomorphology of the river.

The reaction received from the residents at the opening of the scheme has shown the high regard with which they perceive the scheme and the integrated team that delivered it in time to save them from flooding that would have occurred on New Years Day 2003.

ACKNOWLEDGEMENTS

The Environment Agency and DEFRA funded this project. The authors gratefully acknowledge the key individuals within these organisations who shared the vision of the scheme and the benefits that it was to produce.

REFERENCES

Scottish Executive. (2000). *River Crossings and Migratory Fish: Design Guidance, A Consultation Paper.*

CIRIA. (1997). *Culvert Design Guide, Report 168.*

United States Department of the Interior, Bureau of Reclamation. (1987). *Design of Small Dams.* United States Government Printing Office.

Morris, J. (2003). *Managed Washlands for Flood Defence and Biodiversity*. 38th Defra Flood and Coastal Management Conference.

Raciborz Flood Reservoir

LAURENCE ATTEWILL, Jacobs Ltd, UK
EDOARDO FAGANELLO, Jacobs Ltd, UK

SYNOPSIS. The river Odra, which rises in the Czech Republic and disgorges into the Baltic, suffered an extreme flood in July 1997 which was responsible for the loss of 50 lives and over a billion dollars worth of damage in southern Poland. The return period of the flood is variously estimated between 250 and 1000 years.

The paper describes the studies for a flood reservoir to be constructed on the Odra just upstream of the ancient town of Raciborz. These studies include the hydrological studies, the hydrodynamic modelling of a 220km stretch of the river where most of the damage occurred, flood damage studies both with and without the proposed reservoir, environmental impact assessments and resettlement plans, in addition to the engineering studies of the dam itself.

INTRODUCTION

This paper describes the feasibility studies for a flood protection reservoir carried out by Jacobs GIBB in association with Hydroprojekt Warsaw for the Regional Water Management Board in 2002 and 2003. The study area is shown in Figure 1.

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

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

Figure 1: The study area

<u>Background</u>

The Odra river rises in the Czech Republic and flows north through Poland to the Baltic Sea. The river has been liable to flooding, 14 floods having been recorded in the last two centuries which have caused considerable damage to the cities, towns and villages of the upper Odra valley – Raciborz, Opole and Wroclaw. These floods have lead to a the construction of a complex system of flood defences including embankments, by pass channels and flood storage areas which are designed to be capable passing floods of up to a 1 in 100 year return period without serious damage.

Figure 2: The river Odra during the 1997 flood event

These flood defences were overwhelmed by the flood of 1997, known in Poland as the Great Flood, which exceeded all previous floods in flow rate and volume.

The possibility of the construction of a flood reservoir at Raciborz was first proposed after the 1880 flood and by 1906 a scheme with a storage capacity of 640Mm^3 had been designed. Over the course of the century the scheme characteristics evolved with a gradual reduction in the storage capacity due to the expansion of towns and villages. The present project, the conceptual design of which was prepared by Hydroprojekt, comprises three stages:

1. the Bukow Polder close the Czech border (constructed 2001)
2. the construction of a flood storage reservoir at Raciborz
3. channel improvements, and the construction of a new polder and bypass for the city of Wroclaw

RACIBORZ FLOOD RESERVOIR

Purpose

The primary role of Raciborz Reservoir is to reduce the frequency and severity of flooding in the Upper Odra River. This will be achieved in two ways:

1. Firstly, the reservoir will provide flood storage so that the flow rate downstream of the reservoir will be greatly reduced and the effectiveness of the existing flood defence system in containing the flows will be improved.
2. Secondly the reservoir will delay the timing of the flood peak at the confluence of the important left-bank tributary Nysa Klodzka with the Odra so that the adverse combination of the two floods that was so damaging in 1997 is very much less likely.

Two stages of development are envisaged: the first being the ‘dry reservoir’ the sole purpose of which is flood mitigation: in the second the reservoir will be partially impounded when important secondary benefits will be navigation, water supply and recreation. The reduction in flood storage resulting from partial impounding will be offset by gravel extraction from the reservoir in the ‘dry reservoir’ stage.

Layout

The location of the reservoir was selected to be upstream of the town of Raciborz and within Polish territory: the maximum flood level is constrained by a Czech/Polish protocol. The storage volume of the reservoir will be 185 Mm^3. In normal operation the reservoir will be dry with the river flowing through a main gated outlet structure into the bypass channel.

Releases will also be made through a subsidiary outlet into the old river that flows through the town. In times of flood the outflow through the outlet structure will be controlled by operating the gates so that excess water is stored within the reservoir. The outflow is varied according to the magnitude of the expected flood and therefore a flood warning system is essential. The strategy of the operation rules is that for any flood, irrespective of return period, the flood storage is used to its maximum extent and the reservoir outflow is selected to achieve this.
The layout of the reservoir is shown in Figure 3.

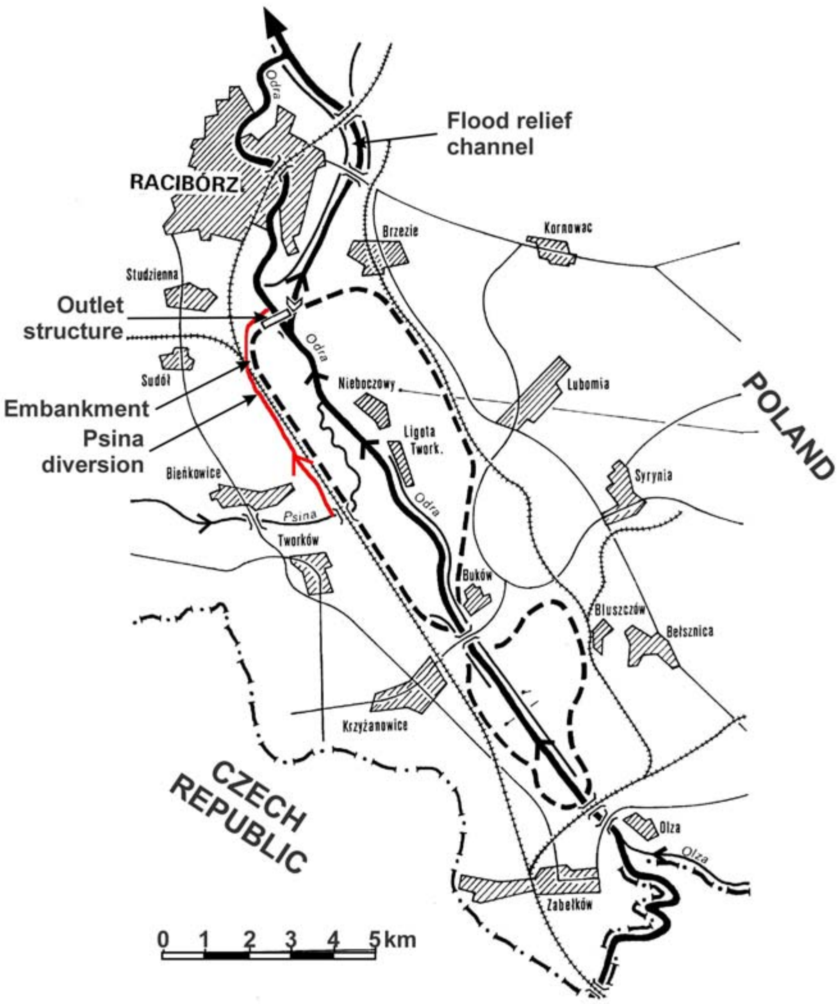


Figure 3: Layout of Raciborz reservoir

Design

Foundations

The reservoir is formed by an earth embankment dam, 22.5km long and with a maximum height of 10m. The dam foundations consist of alluvial deposits comprising alternate layers of silty clay and granular material, with interspersed peat lenses. The top layer will be stripped and the dam founded on the upper cohesive layer. Although foundation seepage is not an issue in the dry reservoir stage - the reservoir will not be full long enough for steady state seepage conditions to become established – it will become a consideration when the reservoir is permanently impounded and therefore a 5 km length of cut off wall through the main granular layer is proposed. In addition the excavation of embankment fill or gravel extraction will be prohibited within a 100 m strip of the upstream toe, the upper cohesive layer forming a natural blanket.

Embankment

The embankment will be constructed over a four year period, from April to October of each year. The embankment which utilizes both the cohesive overburden and the underlying sandy/gravel with the minimum of selection, is essentially a homogenous clay cross section with a substantial drainage bund at its downstream toe. Drainage of the underlying water bearing layer will be provided either by a trench drain or by wells, depending on the depth. A typical cross section of the embankment is shown in Figure 4.

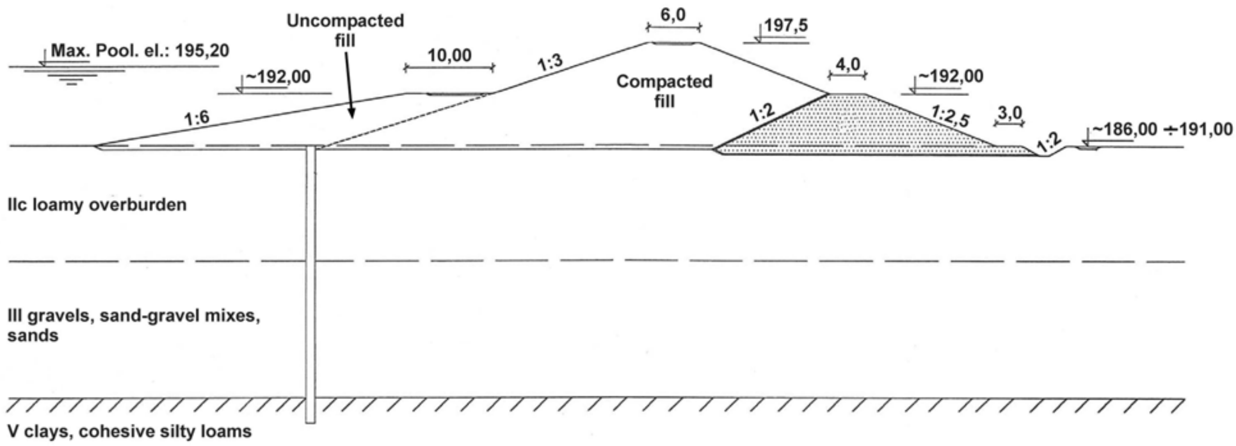


Figure 4: Typical cross section of the embankment

Outlet works and spillway

The outlet works comprise a gated structure consisting of

- a reinforced concrete forebay with a 115m wide apron
- 5 bays each 12m wide with one navigable flat sill at a slightly lower level. The bays are separated by 7.4 m wide piers which house the gate operating mechanism
- vertical lift gates, 1 per bay 12m wide x 8.5m high
- a reinforced concrete bridge over the bays
- a reinforced concrete stilling basin, 109m wide x 55 m long,
- an outlet channel, protected with gabions and riprap, discharging into the river downstream.

Operation

In normal operation the reservoir will be dry with the river flowing through a main gated outlet structure into the bypass channel. Releases will also be made through a subsidiary outlet into the old river that flows through the town. In times of flood the outflow through the outlet structure will be controlled by operating the gates so that excess water is stored within the reservoir. The outflow is varied according to the magnitude of the expected flood and therefore a flood warning system is essential. The strategy of the operation rules is that for any flood, irrespective of return period, the flood storage is used to its maximum extent and the reservoir outflow is selected to achieve this.

SOCIOLOGICAL AND ENVIRONMENTAL ASPECTS

Sociological and environmental issues are dominated by the need for 240 families living in the two villages of Nieboczowy and Ligota Tworkowska to be resettled. The study, which included the formation of an outline resettlement plan, was carried out shortly after the publishing of the World Commission on Dams reports and great effort was made to follow their precepts in this area. An alternative dam alignment excluding the village of Nieboczowy, proposed by the villagers was examined in detail but the reduction in storage volume made this option uneconomic. Despite two public meetings and door to door interviews public opinion remained adamantly hostile, largely due to fears of inadequate levels of compensation.

Compared with this the adverse environmental impacts are relatively minor and in any case are heavily outweighed by the environmental benefits of the scheme to the river valley downstream of the dam.

HYDROLOGY

Data

Flow data from the 20 gauging stations shown in Figure 5 were used in the hydrological analyses. The data record at most stations is at least 50 years.

1997 flood

The 1997 flood was caused by exceptionally intense and prolonged rainfall in the upper catchment: 200mm was recorded in the 5 days from 4th to 8^{th} July 1997 over a wide area with a peak intensity of 585mm over the same period at one station, Lysa Hora in the upper catchment. Peak flow rates are approximate because the river levels so far exceed the calibrated rating curves but were in the region of 3,120 m^3/s at Raciborz , and 3,640 m^3/s at Wroclaw, the upstream and downstream limits of our study area.

Probabilities

Synthetic input hydrographs for the model for a range of return periods were derived from an analysis of the historic flow data and a prediction of the peak flows for as range of return periods computed according to Polish standards by the Institute of Meteorological and Water Management. These predictions were based on the statistical analysis of historic data at single stations which led to the 1997 flood being assigned a return period of 1000 years.

Regional Analysis

The estimation of the frequency or the return period of rare floods is not easy, as extrapolation from the record at a single station involves uncertainty about the choice of statistical distribution to represent the extreme floods, and also the choice of method of fitting the curve to the records. It was therefore decided to test the sensitivity of the estimation of return period by considering the regional flood frequency approach to flood frequency analysis which has been found to give consistent estimates of the relation between flood magnitude and return period or frequency of occurrence when applied to areas of reasonable hydrological homogeneity.

The method depends on the collection of annual maximum flood series from the gauging stations within a region. The method derives from the approach developed during investigation of floods in the British Isles (Natural Environment Research Council, 1975) and its application to many different regions of the world has been described in a number of papers (Farquharson et al., 1987, 1992, 1993: Meigh et al.,1997; Sutcliffe & Farquharson, 1995).

After inspection of the results, the stations were grouped as follows:

- the 6 stations on the upper Odra in the Czech Republic

- the 6 stations on the main Odra below Raciborz
- the 4 stations on the Nysa Kłodzka
- the remaining 6 stations on the Odra.

The curves for these four regions are illustrated in Figure 5

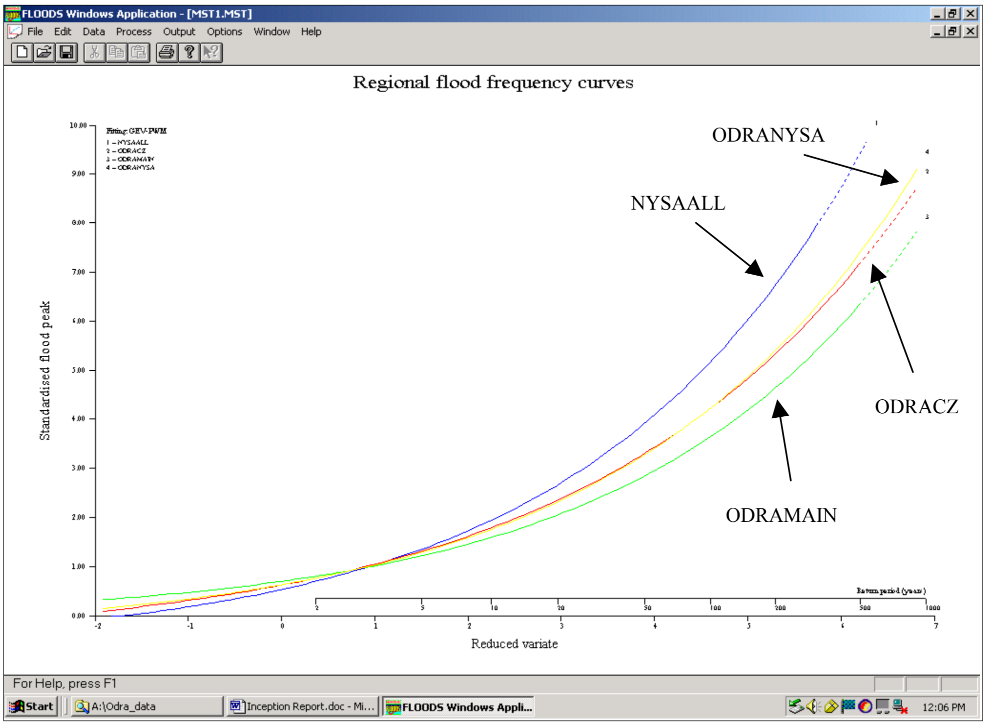


Figure 5: Regional Flood curves

It will be seen that the curves for the upper Odra groupings are very similar up to 50-100 years, but the Nysa curve is higher at longer return periods. The curve for the lower basins is different and is treated separately. The curve for the whole upper Odra is considered as reasonably representative of the whole area.

Comparison of the 1997 flood at individual stations, expressed as Q/MAF, with the regional curve derived from the 16 stations in the upper Odra basin, indicates a lower limit of the range of return periods which could be allocated to this event. This implies that the whole set of records is typical of the region, but it has been shown that the group curves are similar to each other and that the regional curve is similar to that of the upper Vistula. The lower limit for the return period of the 1997 event, on this basis, is about

200 years for the Odra stations from Bohumin down, but rather higher at about 300 years for the Nysa stations.

HYDRODYNAMIC MODELLING

Model description

The Upper Odra hydrodynamic model has been built using MIKE 11, a software package developed by the Danish Hydraulic Institute (DHI).

The model covers approximately 204 km of the main river channel (between the village of Olza, situated near the Czech border and the village of Trestno, immediately upstream of Wroclaw) and includes the floodplain on the right and left bank as well as the two main tributaries: the Nysa Klodzka and the Mala Panew (63.5 km and 18 km respectively). A total of 342 cross sections were included in the model. The cross sections used in the hydrodynamic model are the result of a recent survey carried out after the 1997 flood event.

The MIKE 11 model does not cover the six relevant reservoirs situated on these two tributaries. The effect of these reservoirs operating under the current rules has however been considered in separate flood routing calculations and the resulting outflows included in the model as input hydrographs.

The methodology adopted for modelling the floodplain of the Odra has involved three different techniques:

- extension of the in-bank river cross sections into the floodplain;
- introduction of flood cells so that when the bankful capacity of the main channel is exceeded water spills into them (this technique has been used in particular to model some of the polders);
- simulation of the adjacent floodplain as a 'separate river' using parallel river branches attached to the main channel by lateral spill units (link channels).

A global value of 0.05 of Manning's roughness has been considered in the computations although different values in the range of 0.03-0.100 have been used for many of the modelled reaches in order to reflect the land use and improve model calibration.

The Odra River is maintained as a navigable river. A large number of sluices and lock structures have been constructed along its watercourse as well as bypass and diversion channels in order to improve and increase the capacity of the system during a flood event. There are two main types of hydraulic structure which have been incorporated in the hydraulic model by using the recently surveyed river cross-sections (1997-2000) and the inventory of the existing hydraulic structures between the Polish-Czech boundary and Trestno compiled during the study:

1. weir complexes, which may consist of a variety of broad and sharp crested weirs, barrages, sluice and radial gates associated with culverts and/or bridges
2. polders

Only permanent structures have been taken into account and modelled as fixed (without operation rules). Removable gates have not been included in the model because they are usually removed from the watercourse during flood events. Navigation locks have been simulated with the gates open.

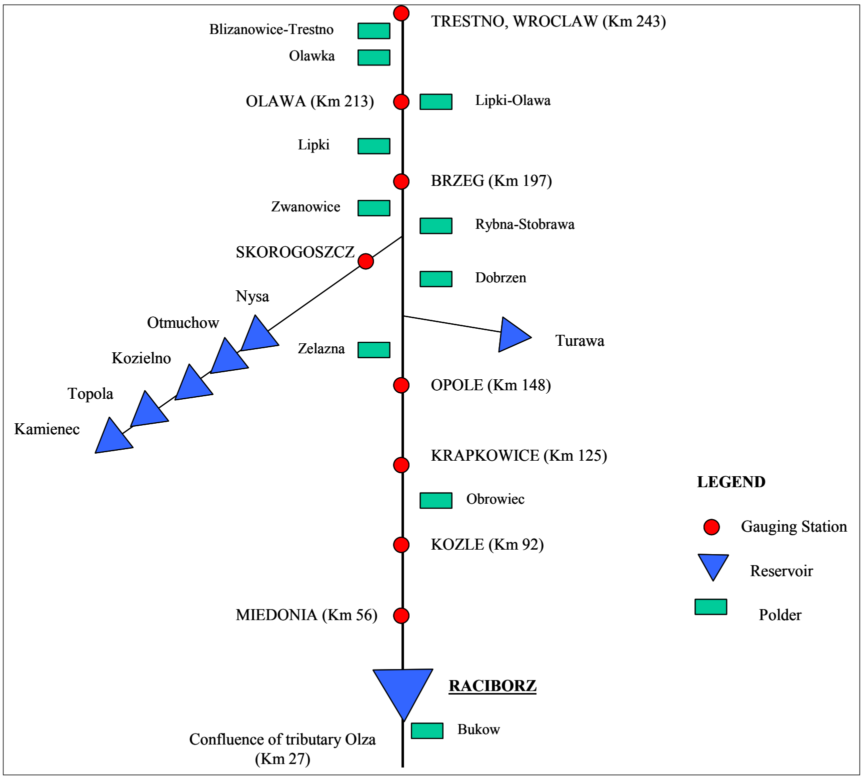


Figure 6: Model schematic

Model calibration

Once the hydrodynamic model was completed, it was calibrated progressively from upstream to downstream using the 1997 flood event by adjusting the roughness, the hydraulic structures and spill (out-of-bank) discharge coefficients so that modeled river levels match those measured as closely as possible. The results of the calibration are presented in Table 1

Table 1 Calibration results

Gauging Stations	Recorded peak level (m OD)	Modelled peak level (m OD)
Miedonia	186.73	186.67
Kozle	171.98	172.02
Krapkowice	165.83	165.88
Opole	154.89	154.70
Skorogoszcz	145.12	145.37
Brzeg	136.50	136.55
Olawa	129.64	129.66
Trestno	121.76	121.71

Model results

The model results, in terms of the reduction of the peak water level attributable to the Raciborz reservoir are shown in Figure 7.

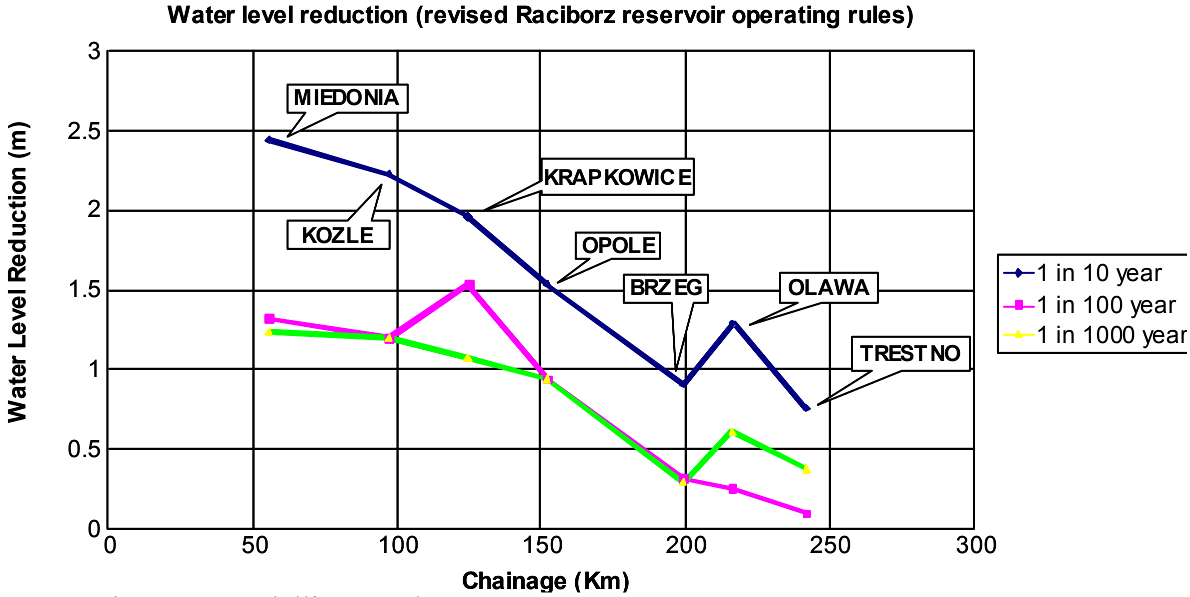


Figure 7: Modelling results.

Sensitivity

The sensitivity of the model results has been examined for the following changes:

- Reservoir volume
- Storm timing
- Operation rules for the reservoirs on the Nysa river

The model was run with a reservoir volume of 154Mm³ (16% reduction on the base case), representing an option which minimised resettlement, and with a reservoir volume of 290Mm³ (50% increase on base case) representing a possible future condition in which gravel deposits within the reservoir are extracted. The modelling showed that on average the effectiveness of the reservoir would decrease and increase by approximately 5% and 15% for the reduction and the increase in volume respectively.

The modelling shows that the river levels are not very sensitive to the relative timing of the main river and tributary flood peaks: a delay of 12 hours in the time to peak of the Nysa result in an increase in water level of 4cm below the confluence.

The sensitivity analysis shows that the flood levels on the Nysa river are sensitive to changes in the operating rules of these reservoirs and to the proposed construction of a new reservoir at Kamieniec Zabkowicki. However these changes will have little impact on flood levels in the main Odra river downstream of the confluence.

BENEFITS

Inundation mapping

The effect of the Raciborz reservoir on the area inundated by floods of the range of return periods considered is illustrated in Figure 8

Flood damages

Flood damages were estimated for the with and without reservoir cases by estimating the areas of each of 20 land use categories flooded in each case and applying damage init rates that were derived from 1997 flood damage data. The results are illustrated, for the range of flood probabilities, in Figure 9.

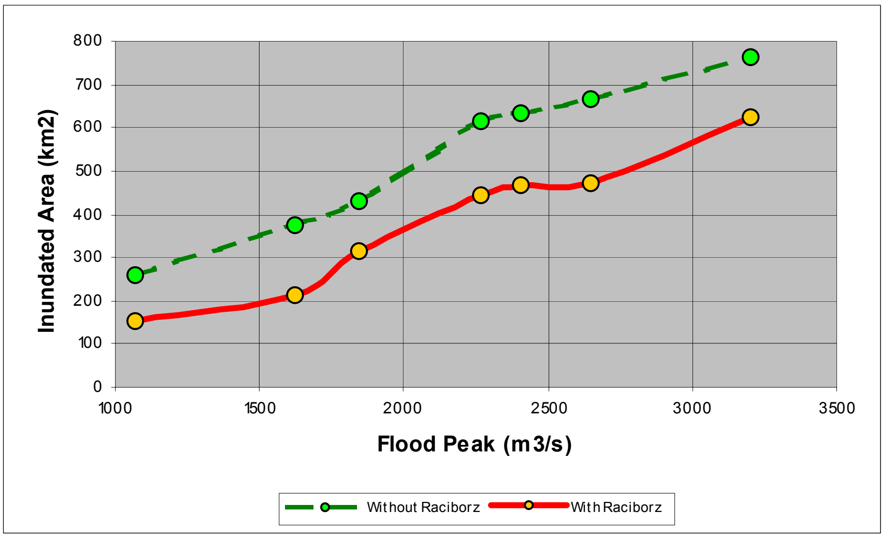


Figure 8: Inundation areas with and without Raciborz reservoir

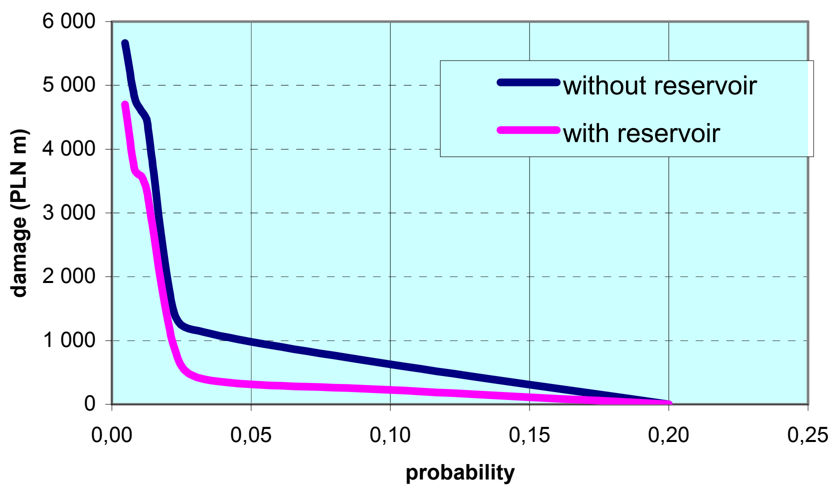


Figure 9: Reduction in flood damage

CONCLUSIONS

The conclusions of the study are that the Raciborz reservoir offers substantial but not complete protection against inundation of the Upper Odra against severe floods. The effectiveness of the reservoir is sensitive to reservoir volume and must be accepted that the proposed reservoir is at the small end of the range of useful volumes: a larger reservoir would be considerably more effective. The level of protection provided by the reservoir is naturally greatest immediately downstream and decreases, especially downstream of the Nysa confluence. However it is likely that Raciborz reservoir together with the implementation of the various channel improvements mitigation measures proposed for Wroclaw will provide adequate protection to that city. The effectiveness of the reservoir will depend on careful operation in which a reliable flood warning and conjunctive use with the Nysa reservoirs are vital.

ACKNOWLEDGEMENTS

The authors are indebted to RZGW Wroclaw for their permission for this paper to be published, and to their Polish and British colleagues for their help in its preparation. Particular thanks are due to Dr J V Sutcliffe (hydrology), Dr John Chatterton (flood damage assessment), Mr Paul Devitt (resettlement aspects) and to the support of Mr. Stanislaw Naprawa, Ms Alina Kledynska and Ms Karina Zachodni.

SESSION 5
FLOOD IMPACT AND ALLEVIATION

Chairman Chris Binnie
Technical Reporter Andrew Pepper

Papers presented

1. European research on dambreak and extreme flood processes
   M.W Morris & N Hassan

2. A passive flow-control device for the Banbury flood storage reservoir
   J Ackers, P Hollinrake & R Harding

3. Challenges on dam safety in a changed climate in Norway
   G.H Midttomme

4. Weedon flood storage scheme – the biggest hydro-brake in the world
   G.P Boakes, A Stephenson, J.B Lowes, A.C Morison & A.T Usborne

Papers not presented

5. Integrating design with the environment to maximise benefits from a flood storage dam: successful implementation at Harbertonford
   W.T Bradley, M.E Jones & A.C Morison

6. Raciborz flood reservoir
   L Attewill & E Faganello

7. Comments on failures of small dams in the Czech Republic during historical flood events
   J Riha

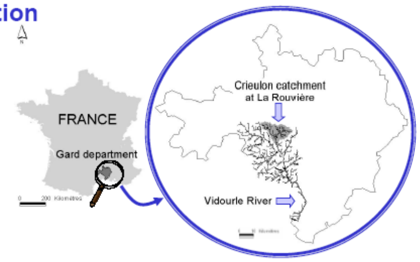
Paul Royet (Cemagref)

The event of September 8-9, 2002 on the Vidourle River catchment - Hydrological analysis

Thank you for the opportunity to present an analysis of a recent event in France that caused the overtopping of a large dam. You can see on this map Gard department in the south east of France.

Geographic and climatic information

- Location



- Average precipitation over the Vidourle catchment
  - annual average rainfall: 950 mm
  - decennial hourly rainfall: 65 mm
  - decennial 24 hour rainfall: 250 mm

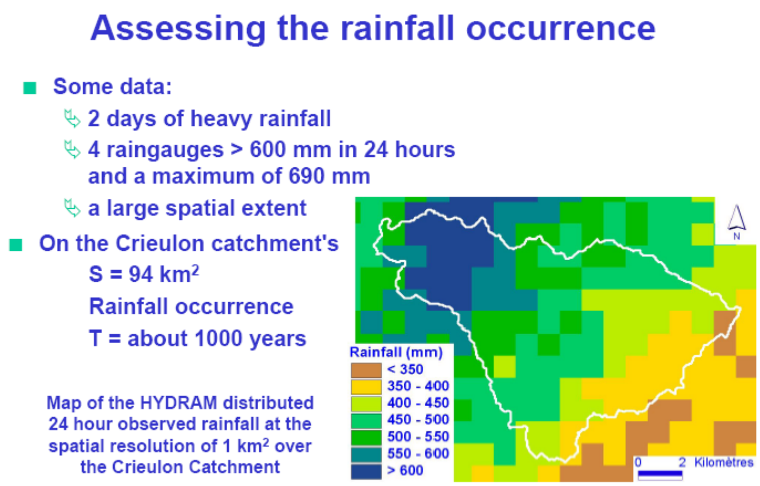
The Vidourle catchment is equipped with three flood mitigation dams; shown on this map is La Rouvière dam on the Crieulon Creek catchment. La Rouvière dam has a catchment of 94 sq km, and is a single purpose dam devoted only to flood mitigation. During the September 2002 event we had a 24 hour rainfall with a huge amount of rain - about 500mm. As you can perhaps see on the photograph the dam is a concrete gravity dam, retaining a 16 metre head and having bottom opening outlets and a crest spillway. The crest spillway was overtopped for about 20 hours and the remainder of the crest itself was overtopped for 6 hours. The storage capacity of the reservoir is $8Mm^3$ at the crest spillway level, but an additional $5Mm^3$ was available at the peak water level when overtopping the crest.

The pluviometric and hydrographic event at La Rouvière dam on the Crieulon Creek

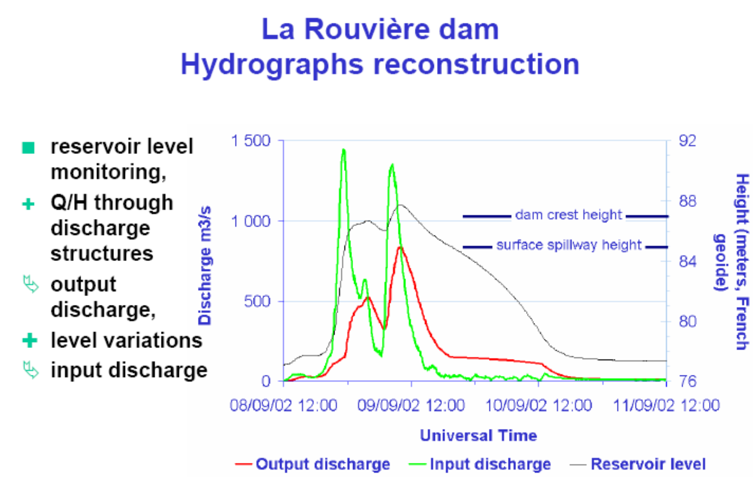
- 24 hours of rainfall with estimations of 492 mm to 576 mm
- overtopping of the dam crest (0.75m)
- 1.75m above MWL for design flood

The flood mitigation dam of La Rouvière on the Crieulon Creek on September 9th, 2002

For assessing the rainfall probability we had data from radar imaging calibrated with data from rain gauges. Using all that data in a regional hydrologic model we estimated that the probability of occurrence of the rainfall at the catchment scale was 10^{-3} or a 1 in 1000 year event.



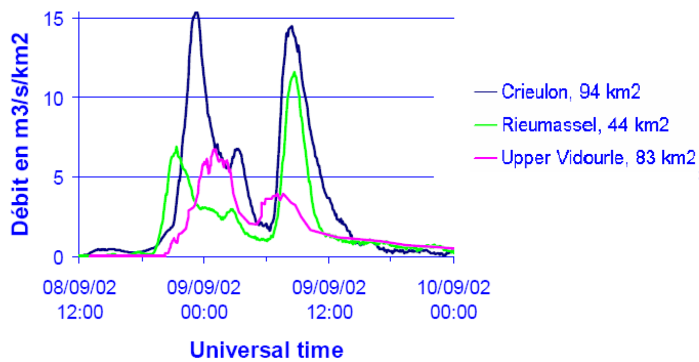
The event at the dam site was recorded at 5 minute steps by two different devices. One broke down at the first peak but we had a second one so we have a record of the reservoir level, the grey curve on the plot, and you can see that the reservoir level exceeded the spillway on the crest, as well as the crest itself. Knowing the relationship between reservoir level and outlet discharge it was possible to calculate the output hydrograph, you can see it the red curve, and knowing the relationship between the reservoir level and the dam volume, it was possible to calculate the input hydrograph, shown in green.



On this hydrograph you can see two peaks, with an input peak flow of about 1400 m³/s – remember that the surface of the catchment is less than 100 km². The first peak filled the reservoir and the flood attenuation was a very effective 64%, but the second peak arrived very soon after the first one resulting in a poor (only 38%) attenuation of this second flood.

The volume of flow was four times the reservoir capacity and the total runoff during that event was about 240mm. Comparing that to the design flow which had been calculated in the 1960's the peak flow was well estimated by our predecessors, but the effect of this one-off double-peak was completely unanticipated so the efficiency from the flood mitigation was overestimated.

Hydrographs reconstruction
comparison with the two other dams of the area



La Rouvière dam
Comparison with the design flood

	September 2002 flood 1st peak	2nd peak	Design flood (1963)
return period			5,000 years
input peak flow	1,445 m^3/s	1,360 m^3/s	1,535 m^3/s
output peak flow	525 m^3/s	840 m^3/s	600 m^3/s
attenuation	64%	38%	61%
runoff	$23\ 10^6\ m^3$ **245 mm**	$22\ 10^6\ m^3$ **235 mm**	$9{,}5\ 10^6\ m^3$ **101 mm**

The conclusion is that when we have a two-peak flood it is difficult to provide a high mitigation effect during the second peak. Fortunately there was no damage to the dam, which had a high safety factor due to the mass concrete in the foundation, but now it is necessary to look at the design flood of La Rouvière dam and all the neighbouring flood mitigation dams built in the same area.

François Lempérière (Hydrocoop)
Piano Keys Weir **-** Thank you for giving me the opportunity of presenting a new solution for free flow discharge. A Piano Keys Weir multiplies by up to four the flow capacity of a traditional weir for the same head.
Many existing dams need upgraded spillway capacity. Installing traditional gates is expensive. New solutions have been developed over the last 20 years such as earth fuseplugs, traditional labyrinth weirs, locally lining the downstream slope with roller compacted concrete, or increasing the dam crest level.
Traditional labyrinth weirs are vertical walls with a zig-zag layout shape which is much longer than the weir length. They double the flow of a traditional weir but their shape is such that they cannot be placed on top of a gravity dam section, that is to say they can be used only for a small proportion of dams.

A new solution for increasing capacity has been studied and optimised over the last five years. It is based upon vertical walls with a rectangular plan layout which has the shape of piano keys, which gives it the name of Piano Keys Weir or PK Weir. These have been examined in detail and optimised in four laboratories, one in France, another in the largest hydraulic institute in China, also in India and Algeria and the result is quite satisfactory because the flow capacity may be multiplied by 3 or 4 as compared with the traditional weir and they can be placed on the top of most existing free flow spillways.
It can be used equally as well for a small specific flow as for a large specific flow, and can be used for increasing the spillway capacity or the storage of a free flow discharge reservoir, or for making emergency spillways in addition to gated spillways. The quantities of reinforced concrete needed are reduced, giving a cost-effective solution. As the shape has been optimised it is possible within each country to design the details, carry out and standardize structural studies and to check the hydraulic efficiency.
As this solution has been developed and optimised by a Non Profit Association (Hydrocoop) there is no patent, although professional advice is available. For those of you who are interested, a paper on the subject has been published in Hydropower and Dams – 2003 Issue 5, or see : www.hydrocoop.org

Chris Binnie (Chairman)
Effects of Climate Change in Norway. You looked at the climate change for 2050 and you used predictions, I think, based on a generated time series of 1000 years. Does this mean that you generated 1000 years of data to get to the 2050 climate? Is that something you think we should be doing here?

Grethe Midttømme (Norwegian Water Resources and Energy Directorate)
I am not a hydrologist and those scenarios were developed by our Hydrology Department so I am not quite sure about the details, but the scenarios were made from models. The hydrologists were modelling 50 years ahead from today, not really using 1000 years of data. When I was talking about 1000 years of data I was talking about some new analyses they are going to do. They are going to see how the data series they are using today for the 1,000 year return period precipitation and the 1,000 year return period flood are affected and how this will result in any changes in the design flood.

George Hallows (Independent Consultant)
Mr Morris said that the HR BREACH model had not been derived by trying to reproduce the characteristics of actual failures such as Teton and Malpasset, but had been derived from physical principles, which he considered was an advantage. However I presume that he has compared what the model would have predicted for those sites with what actually happened. Could he please tell us how the output of the model compared with actuality for those two very different scenarios (that is an embankment dam developing a breach by erosion over time and the almost instantaneous loss of some two thirds of a concrete arch dam)?

Mark Morris (HR Wallingford)
What we wanted to avoid was producing a breach model which was calibrated to specific events (such as Teton, Malpasset and so on) but which does not actually perform well when it comes to the real challenge of a dam for which you have no information. The model we have developed applies to failure of earth embankments – not concrete structures – therefore we cannot compare predictions of the two.

Earlier I mentioned the issue of breach location in relation to the Teton failure. The failure of Teton was actually against a rock abutment and many breach modellers have used this data for calibration of their models without realising the significance of the breach location. When we applied the HR BREACH model to that dam failure we did not get a particularly good fit initially. After investigation we realised that this mistake had been made by many people. It was quite noticeable that when the growth of the breach in the model was constrained to account for the rock abutment, the model gave a much better fit to the actual observed data. So the approach we are going to continue to take is to assess the performance of the model against actual events, to develop the model based upon observed physical processes but not to calibrate the model to exactly match specific events.

Prof Peter Vaughan
A couple of comments also on breaching. This is very important work but I was involved with a small wadi dam in Israel a few years ago back which had potential piping problems. It was decided to fill it up temporarily for a few hours - it failed within nine. It failed like an arch, and we could actually see daylight between the downstream and the upstream sides before the crest fell in, in other words the crest was not affected. Now that must give serious difficulties in behaviour compared to the simplified scenario where I understood from the pictures that you simply flooded over the crest of your dams. If you do not have a notch or a starter to concentrate the flow (as you did) this is rather important.

The second comment I have, and which I would welcome your response on, is that the biggest risk we have in Britain is that dam cores settle suddenly due to the formation of hydraulic fracture and of course overtopping occurs. Some time was spent investigating the results of Dale Dyke failure and the work strongly indicated (although it is not possible to prove anything now) that this is what happened there, and a sudden flood wave occurred, inundating the valley. How different would the results of that be from the results of normal overtopping failure, and what allowances should we make for this effect?

Mark Morris (HR Wallingford)
Your first point was on piping. Yes, the processes are very different. The field tests that were done in Norway were three overtopping tests. You were quite right that they were initiated by having a notch set into the crest to focus the flows. For the two piping tests which we carried out in detail we placed a weakness through the dam. This was effectively a sand band and buried within it a notched pipe so we could jet the sand out to initiate the piping failure. In fact, we had exactly the situation you described where you could see daylight through the dam as the pipe grew. We could not control the levels upstream sufficiently so the reservoir drained.

And yes, you are quite right, the process of failure and the erosion processes are very different. One of the problems you will find with some of the earlier breach models is that they will just assume that, regardless of water level or process, that there is an average removal of sediment and material which is distributed around the shape of the pipe or the bridge - which is clearly not the case. You have erosion only where there is flow present, undercutting, and then you will have some sort of bank failure mechanism within the breach.

In terms of getting a wave effect, I said that you did not see settlement in that particular case and that is true but that was just one instance. In other situations you may see settlement. In some of the other breach cases where you had open breach you could see the sides of the test dam settling with cracking in the crest. It really comes down to the different mechanisms of erosion and internal support. If you are going to try and simulate it correctly you need a model which is going to have these different failure processes in place. Equally, failure processes for dealing with, say, composite structures and the interaction between erosion of the core and the bank are also needed.

You raised a point regarding the difference between normal overtopping and overtopping prompted through sudden subsidence of the crest, due to internal erosion and collapse. The difference between these events is likely to be in terms of the speed with which breach formation and failure takes place. The rate of failure for a normal overtopping event will be dictated by the rate of rise of the upstream water level, in conjunction with the embankment resistance to erosion. With a slow rise in water level, breach initiation (leading to breach formation and eventual failure) could be a relatively slow process (hours). With sudden subsidence, however, there is a step increase in water depth flowing over the embankment. If this step increase in flow exceeds conditions required for initiation, then erosion leading to breach formation and failure could be relatively quick (< hour). In this situation immediate action is needed to reduce or stop overtopping and prevent any further crest subsidence.

Russ Digby (Kenneth Grubb Associates Ltd)
Question for Alex Stephenson. Given that the Hydro-Brake® shown for the current project is so large and that the forces associated with such a large mass of rapidly rotating water (at 7m/s) will be commensurately large, does the design adequately cope with both the strength and the fatigue issues?

Alex Stephenson (Hydro International Ltd)
The Weedon dam Hydro-Brake® was quite heavily designed by structural engineering standards. We had to make sure that we got it right because it was larger than anything we had done before, although we do have 20 odd years of experience, albeit with smaller devices.

We carried out a great deal of work in-house just to get the software for analysing the structures to make sure that they are practical. The big concrete encasement does not show from the photographs, which in theory does not need to be there as the steel is sufficient. So we were 100% confident that this one would perform satisfactorily. Bearing in mind that the average Hydro-Brake® is 200mm to 300mm diameter this one is a bit different, so we did put a lot of work in and hopefully the years will show that we did get the calculations right.

Tony Morison (Halcrow Group)
I would add that the Hydro-Brake® is not actually supposed to initiate the vortex action until the 1 in 10 year flow is reached, so the total hours of operation under full vortex conditions are going to be relatively few in its normal design life.

Tim Jackson (Severn Trent Water plc)
The occupier of Dodford Mill, just downstream of the new dam, is a friend who, I know, had reservations about the proposals and of the position of the dam in relation to his dwelling in particular. How were these reservations dealt with, and what changes to the design were made to accommodate his concerns?

Tony Morison (Halcrow Group)
I was not personally involved in the discussion with the land owner at the mill, as this was done by our Peterborough office. I know that the dam was actually moved upstream about 100 metres and this was partly related to the screening and the location of the spillway relative to the house. On the other hand the spillway is not designed to come into operation until the 1 in 50 return period flood - and my understanding is that at present Dodford Mill actually floods at very little over the 1 in 10 year event.

Any house in the UK that describes itself as a mill and was once a water mill has this potential liability I am afraid. So in fact the extent of flooding at the house there should significantly decrease. The location of the spillway relative to the house was located with the house in mind and the flow through the culvert which comes through the Hydro-Brake® should, under normal circumstances, be in-bank flow. In fact the Hydro-Brake® has built into it the potential to restrict the orifice once its operation has been tested and experience built up, so that the downstream flow can be adjusted between about 8 and 12 m^3/s to ensure that it does not go out of bank immediately downstream.

Chris Owens (Atkins Water)
Weedon and Banbury Reservoirs**.** Both of these structures are presumably statutory reservoirs being designed to hold, or capable of holding, water. At what point do you consider it appropriate to issue the Preliminary Certificate?

Tony Morison (Halcrow Group)
The Weedon dam was constructed with a large orifice against which Hydro-Brake® was installed. The Preliminary Certificate was issued three days after the Hydro-Brake® was installed.

Alan Parsons (Halcrow Group)
On the Weedon Scheme I think you saw that we installed screens both upstream and downstream of the Hydro-Brake®, and there has subsequently been a problem with debris collection on the downstream screen, which is downstream of the outlet structure. We are having to make some modifications to that screen. The inlet screen and the Hydro-Brake® itself are passing debris with no problem. The downstream screen is just there to keep the public out and as its a smaller screen we are having problems with that one.

Andrew Pepper (ATPEC Ltd)
Weedon and Banbury Reservoirs**.** We have here two different devices performing the same function on different reservoirs. Could each have been used on the other reservoir, and was such an option, possibly involving multiple Hydro-Brake® units, ever considered?

John Ackers (Black & Veatch)
I have to confess that there had not been much technical discussion between the two projects until a couple of weeks ago when I emailed Tony Morison to ask how are we going to deal with any questions that tried to pit us against each other! But what I can say about the Banbury Scheme is that:

- firstly it would have required three or four Hydro-Brakes®, of a similar size to that at Weedon, installed in parallel; and
- secondly the embankment height and the maximum flood rise at Banbury are much less than at Weedon.

Weedon involves a culvert through or below the embankment. In the case of Banbury the two structures are full height structures and the embankment abuts the sides of the structures so there are some distinct differences. The device developed for Banbury can work at that range of heads, but it would not be so suitable for the greater heads which apply for the Weedon Scheme.

Tony Morison (Halcrow Group)
We really did not look beyond the Hydro-Brake®, having found it to be a suitable compromise between the hydraulic effect of an automatic gate with all its operational disadvantages and the simplicity of an orifice which has hydraulic disadvantages. We were pleased that the capacities that we were aiming for at Weedon could be accommodated by a single unit The unit is not complicated, in fact having walked through it I can tell you that inside it is extremely simple! It is the concept rather than the structure that is complicated and I can tell you that there are other schemes under design and consideration which do involve multiple Hydro-Brake® units. I've been working with at least one.

Ian Hope (Environment Agency)
Banbury Reservoir – Public Safety and Debris**.** What precautions are planned to exclude errant members of the public from the control structure? Also what account was made of debris build-up and exclusion from the flow control structure? If there are no screens proposed was this modelled?

John Ackers (Black & Veatch)
The scheme is still in the course of design, and also needs the approval of your colleagues at the Environment Agency. One of the key issues being taken into account is public safety, including handrailing and so on. Keeping members of the public away from danger has been an integral part of the design of the scheme.

Regarding debris, we felt that the design that has been evolved includes openings that are sufficiently large not to pose much of a risk of blockage by debris, so would not require debris traps upstream. However, there would be the option of installing traps if the consensus of opinion on that issue changes during the development of the design over the next year or so.

Alan Parsons (Halcrow Group)
Could we have some more information on the benefits of the baffle device compared with a normal orifice device in terms of how much storage that it has saved, how much it has reduced the heights of the embankments you would need, and the savings in costs that have accrued to the scheme?

John Ackers (Black & Veatch)
We carried out some comparative runs when the baffled orifice was first proposed which indicated that it will save about 0.1m in terms of the peak level reached in the reservoir during the design event. The value of the saved embankment fill is probably no more about £50,000, but there are also some effects on the costs of the structures and of the land which have not been quantified.

But, a stronger driver for the choice of the device was the effect on delaying the onset of impounding and reducing the peak level in the reservoir during more frequent floods, as they have more impact on the viability of continued use of the land for grazing. There is also a road that runs across the flood storage area, and a key issue was to minimise the incidence of that being flooded during those more frequent events. Comparative runs of the reservoir storage model indicate that the baffled orifice mitigates levels in the 10-year flood by about 0.3m.

The scheme has not yet been fully designed and costed, but my belief is that the cost of this form of control device is only going to be a few thousand pounds more than the cost of a simple orifice device.

Chris Binnie (Chairman)
Banbury Reservoir. The model testing seemed fairly extensive. What was the cost both of the testing and the engineers' input, and what was this as a percentage of the cost of the works? How generic are the results? As it is an Environment Agency project will the report on the model test be in the public domain?

John Ackers (Black & Veatch)
The model testing itself cost about £30,000, to which might be added our time of the order of £5000. This represents well under 0.5% of the capital cost of about £10M for the project.

I think that this solution, like the Hydro-Brake®, is one of a number of generic devices that can be considered as part of the portfolio of control devices for online flood storage reservoirs.

The results of the testing are already in the public domain in the form of this paper, although obviously there is more detailed coverage in the report from HR Wallingford. It is my understanding that the Environment Agency will be prepared to release additional information regarding the model testing to interested parties, upon request.