British Dam Society Conference 2004 – Canterbury, Kent

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

Chapter 6. Incidents and rehabilitation case histories

Challenges and limits - the feasibility of underwater rehabilitation work

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SYNOPSIS. The bottom outlet facilities of many dams all over the world will have to undergo rehabilitation within the near future. Not only the early dams, built in the 19th or in the beginning of the 20th century are affected. Even large hydraulic structures, designed and built in the second half of the 20th century are faced with this problem. Due to various reasons a complete draw down of a reservoir for inspection and rehabilitation purposes has to be considered as not feasible in most cases. That requires manned or unmanned underwater inspection and rehabilitation techniques at the submerged structures of a dam. The paper describes the experiences gained during the underwater rehabilitation activities of the Ruhr River Association and how these experiences can be applied to other projects in Europe at water depths between 20 and 120 meters.

INTRODUCTION

The layout of many hydraulic structures all over the world does not make provisions for repair works of the bottom outlet facilities. This is not only the case for the dams of the very early design periods around the beginning of the 20th century. Even many owners of large hydraulic structures, designed and built in the second half of the 20th century have to face this problem.

At the early design periods the complete draw down of a reservoir for repair purposes was usually considered possible. Therefore the original design of that time did not make provisions for underwater inspection and repair work. Nevertheless even some modern dams have design deficiencies related to inspection and rehabilitation as well. The lack of support structures for emergency gates for instance turns out to be a major problem. Safe working conditions for the diving crews can not always been taken for granted.

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

Nowadays, due to possible restrictions for water supply, hydropower generation, irrigation, leisure activities and due to the risk of severe ecological problems in and around the water body of a reservoir a complete draw down for inspection and rehabilitation purposes has to be considered as impossible in many cases. Therefore rehabilitation works have to be done during full or partial operation respectively maximum or reduced reservoir levels which allow an unrestricted supply of water, depending on the various purposes of the reservoir.
Extensive underwater work has been done at the reservoirs of the Ruhr River Association (in German: Ruhrverband). These reservoirs provide bulk water for the industry and about 5 million people in the Ruhr area and cover a design and construction period from the beginning of the 20th century until 1966. During the life cycles of these reservoirs of up to one century quite a number of structural and operational deficiencies became evident, not to mention the regular ageing processes. Therefore during the last 10-15 years extensive rehabilitation measures were carried out at the reservoirs of the Ruhr River Association, mainly in order to refurbish the bottom outlet facilities and to adapt the existing hydraulic structures to new operational needs at reservoir levels which allowed an unrestricted water supply. In the following the experiences gained during the underwater rehabilitation activities of the Ruhr River Association are described.

The rehabilitation strategies of the bottom outlet structures of every dam were based upon the following ideas:

- to move the new intakes upstream, away from critical and narrow cross-sections at the upstream foot of the dam
- to use the new intakes as support structures for emergency gates
- to replace the old gates
- to install new intake gates respectively guard valves

Every project led to new experiences in underwater rehabilitation technology, which is described in the following.

Some experiences have been shared with other dam owners in Europe, responsible for reservoirs with water depths between 20 and 120 meters.

UNDERWATER REHABILITATION CASE HISTORIES

The Moehne Dam

Figure 1. Aerial View of the Moehne Reservoir

Introduction

The Moehne Dam (Figure 1) was built from 1908 - 1912 as a curved masonry dam with a height of 40 m, a length of 650 m and a maximum storage capacity of 134.5 million m³. The Moehne Dam was considered as one of the largest dams in Europe of that time. It became known world-wide, when during World War II the dam was severely damaged by an allied bomb attack. The dam was destroyed to a height of 23 m and over a length of 77 m. The following flood wave of about 110 million m³ of water and a height of 6 – 7 m killed more than 1.200 people and devastated the Moehne Valley.

The concept for the rehabilitation of the Moehne Dam

The rehabilitation of the bottom outlets of Moehne Dam can be considered as the first milestone in underwater rehabilitation of the dams of the Ruhr River Association. The work started in 1992 and was finished in 2002. It has been described in (Heitefuss & Kny 2002) and (Klein, Harder & Klahn 2003). Nevertheless two important aspects of underwater work are worth to be mentioned.

Underwater concrete work

After the installation of the new pipework one of the final steps of the construction of new support structures for the emergency gates is usually the underwater concreting. It took various test pourings, until the optimal recipe for the underwater concrete was found. What has been an enormous problem during the Moehne project, turned out to be much easier at the next underwater rehabilitation projects, since there has been a remarkable advance in underwater concrete technology during the last ten years.

Use of a diving platform

When the rehabilitation work started at the Moehne Dam in 1992, this was also the start for the use of a special diving platform (Figure 2) which has been developed by the diving contractor in co-operation with the Ruhr River Association.

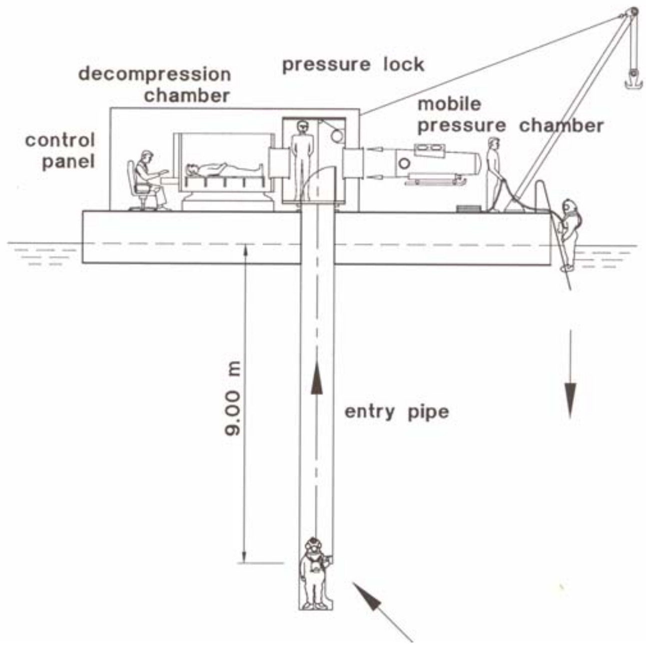

Figure 2. The diving platform with pressure chamber and entry pipe

This diving platform has proven to be a valuable tool for safe diving works at the construction sites of the Ruhr River Association for more than ten years. No diving accident worth mentioning occurred during this time.

The diving platform is equipped with two coupled decompression chambers, which are connected directly to a vertical entry pipe. This pipe reaches 9 m under the water surface and can be filled with compressed air. Thus, the diver can enter the pipe after his diving mission. He remains under pressure, disconnects the umbilical and can take off his diving mask or helmet. With an elevator he is lifted to a pressure lock, where he is undressed by an

assistant. Then he enters the decompression chamber for the reduction of pressure. This equipment allows the controlled decompression under dry and warm conditions, which both improves the decomposition of nitrogen in the divers body and prevents, that the diver catches a cold. This improves the safety for the diving crew and increases the effectiveness, since health problems of the divers are getting reduced.
Another technique to improve the effectiveness of the diving activities is the use of oxygen breathing masks in the decompression chamber. If the decompression has reached 0.6 atmospheres and less, the diver breathes pure oxygen in order to accelerate the nitrogen decomposition. According to the new German Safety Codes for Diving, the additional use of oxygen breathing gas extends the so-called ground time for instance at water depths between 30 and 40 meters up to 50 or even 60 % in comparison to the use of a regular breathing gas mixture inside the decompression chamber. Thus, at a water depth of 33 m the diver has a ground time of 80 minutes and a decompression time of 65 min.
The platform is also equipped with a mobile pressure chamber, which can be connected quickly to the pressure lock. In case of a diving accident there are two options. The diver is either placed in the mobile pressure chamber and brought to a hospital via truck or airlift within 30 minutes or medical assistance can be brought in via the second pressure chamber, which in this case is used as a pressure lock.

The Verse Dam

The Verse Dam is a multi-purpose reservoir with an earthfill dam with a concrete cut off and a height of 52 m.. Based upon the experiences from Moehne Dam the intake structures of Verse Dam underwent rehabilitation from 1995 - 1997. This work has been described in (Heitefuss & Kny 2002 / 1997).
At this project it proved, that a preliminary hydrologic study can be a valuable tool for an economical rehabilitation measure.
Before work began, a minimum reservoir level had to be found, which could guarantee the safety of water supply for the adjacent cities and allow reasonable and economical diving and decompression times for the diving crews. The calculations resulted in a minimum water depth of 38 m during underwater work, compared to a regular depth of more than 50 m. Thus the ground time of each dive was doubled.

The cost of the entire rehabilitation project amounted to nearly 2 million €. The cost of supply of steel (RSt 37-2) for the pipe-work and shield was about 8 € per kilogram. The additional cost for the installation of the pipe-work was about 14 € per kilogram of steel.

Figure 3. Aerial View of Ennepe Dam

The Ennepe Dam

The Ennepe Dam (Figure 3) is a curved masonry dam with a length of 350 m and a height of 50 m. It was built during the 4 years earlier than the Moehne Dam, based upon the same design principles. The concept for the rehabilitation of the entire bottom outlet structure was applying the techniques, which had already proven their feasibility at the Moehne and Verse Dam. In fact there was an additional challenge for the engineers of the Ruhr River Association, because it was necessary to install new intakes for bulk water at different water levels (Heitefuss & Kny 2002 / 2001).
Another major difference to the rehabilitation projects at Moehne and Verse Dam was use of stainless steel. This required special conditions during production and installation of the components in the plant and on the construction site.
The underwater rehabilitation work at Ennepe Dam took 5 years. The estimated cost of the entire bottom outlet rehabilitation project was 4 Mio. EURO. The cost of supply for the stainless steel was between 11 and 14 € per kilogram. The additional cost for the installation of the pipework was between 20 and 30 € per kilogram of stainless steel.

UNDERWATER REHABILITATION PROJECTS

Mornos Dam, Greece

Introduction

The engineers of the Ruhr River Association have been involved in a feasibility study for the rehabilitation of the bottom outlet of Mornos Dam, Greece. With a storage capacity of 780 million m³, a height of 126 m and a crest length of about 800 m Mornos Dam is one of the largest earth dams in Europe and very important for the water supply of Athens. Due to the dimensions of the upstream reach there is no access to the bottom outlet gate with conventional diving techniques. In the following some options for the inspection and rehabilitation, as well as some ROV–techniques, methods of saturated diving and pipe freezing are described.

Description of the Bottom Outlet and the Transfer Device

The bottom outlet of Mornos Dam has a capacity of 400 m³/s. The horizontal reach of the intake tunnel has a length of more than 450 m and a diameter of almost 10 m. It narrows to a square profile of about 3 x 3 m. The bottom outlet of Mornos Dam is equipped with two roller gates.

Right after the commissioning of the Mornos Dam it became evident, that the water losses of the upstream bottom outlet gate (considered as emergency gate) amounted up to about 0.5 m³/s. A so-called transfer device was built in order to make the upstream gate of Mornos Dam revisable. The basic idea of the use of the transfer device (Figure 4) was to pull the upstream valve under balanced pressure into the top of this pressure chamber, in order to be able to operate with the top of the transfer device separately from the water pressure inside the penstock.

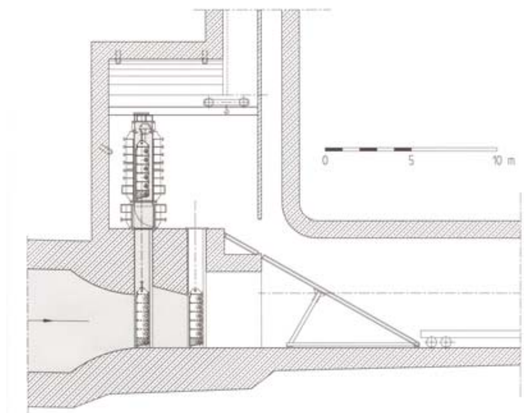

Figure 4. Use of transfer device inside the gate chamber of Mornos Dam

The use of the transfer device is a very complex and difficult operation. A failure has to be avoided. Therefore the transfer device has to go through a process of structural calculations and proofs.

The Use of Underwater Technology - Access with Divers from Upstream
At water depths of more than 60 m conventional diving reaches its technical and economical limits. The maximum water level of Mornos Reservoir requires the technique of saturated diving. The divers stay inside a system of chambers (so-called “habitat”) for several weeks and remain on “ground depth”. The divers are brought down to their working depth with a mobile diving bell. The supply of the diver out of the bell with breathing gas and heating water is maintained by means of an umbilical. According to the relevant safety regulations the length of the umbilical is limited to 30 m.
Therefore the use of this diving technique at Mornos Dam has to be ruled out due to the very long distance between intake and closure gate. For the same reason autonomous diving techniques is impossible too.

Access with an ROV (Remotely Operated Vehicle) from Upstream
During the last years the so-called ROV´s became widely used in underwater technology. The water depth of 120 m is no problem for a modern ROV. The main difficulty for an ROV at Mornos Dam is the enormous horizontal distance. A very powerful ROV has to be used, which is capable of driving 450 m from the intake tower to the upstream gate.

Access to the Bottom Outlet with an ROV from Downstream
It was examined, if the use of an ROV from downstream offers technical and economical advantages. By using the two gates as a lock it is possible to enter the bottom outlet with an ROV (Figure 5). This procedure requires the penetration of the umbilical through the downstream steel plate, which is no major problem. Then the upstream gate can be raised that much, that the ROV can pass safely underneath.

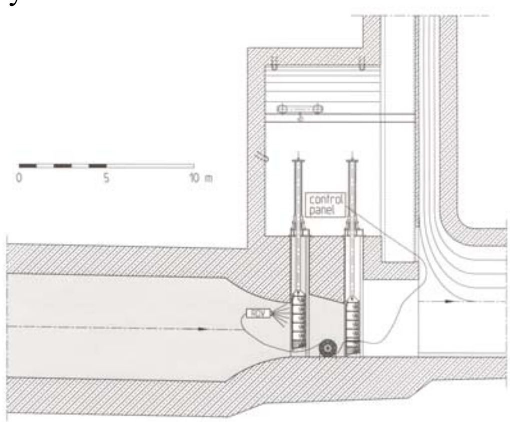

Figure 5. Inspection of Bottom Outlet with ROV from Downstream

Access with Divers from Downstream at full Storage Level

A technique is presented below, which allows diving activities at full reservoir level on the upstream side of the bottom outlet gate. This requires the use of the method of saturated diving. By installing a specific flange in the downstream gate a pressure chamber can be connected to the bottom outlet gate.

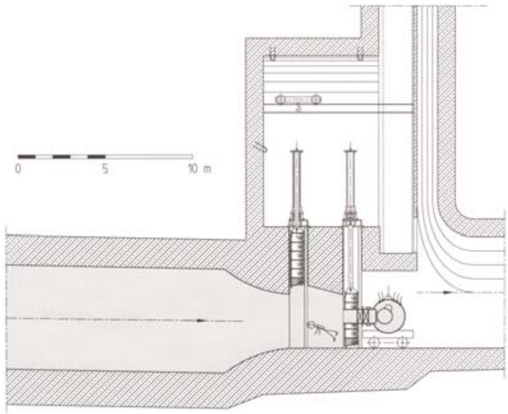

Figure 6. Access with Divers from Downstream

Access with Divers from Downstream at reduced Storage Level

A method of access with divers (Figure 6) at full storage level is described above. At a draw-down of the reservoir to about 80 - 90 m dives with partial saturation get applicable.

At these depths breathing gas mixtures like TRIMIX would be used, but there are still enormous decompression times from a dive with a ground time of 30 minutes:

water depth [m]	decompression time [min]
80 m	175 min
90 m	225 min
100 m	260 min

Use of non-conventional techniques

The basic requirement for the access to the bottom outlet under atmospheric conditions is an emergency gate, which has to be positioned in front of the intake trumpet upstream of the closure gate. Due to the conditions at Mornos Dam there is **no** way to install a conventional temporary emergency gate. Therefore in the following an approach is described, which possibly facilitates the access to the bottom outlet under atmospheric conditions with a non-conventional method.

Rehabilitation of the Bottom Outlet by means of Freezing
In the last 20 years the methods of freezing became widely used in the fields of geotechnical- and offshore-engineering. In the field of pipeline engineering the so-called pipe-freezing - the placing of ice plugs in pressure conduits for inspection and repair purposes - became a common method in the last years. By means of a so-called jacket the coolant is brought onto the pipe from outside. By freezing both the pipe wall and the medium inside an ice-plug is produced, which is connected tightly to the pipe wall.
The installation of an ice-plug of this size inside a bottom outlet has never been done before. Therefore material testing has to be conducted. The creep of ice-samples of this size has to be examined. For the production of an ice-plug of this size the problem of convection has to be solved

Rehabilitation of the Bottom Outlets of Early Embankment Dams
The experiences gained during rehabilitation of the Ruhr River Association's dams respectively their bottom outlets could be applied not only to a very large hydraulic structure like Mornos Dam. Also very early embankment dams with Puddle Clay Core could be refurbished, applying the same or adapted underwater rehabilitation techniques. A typical example is Lower Vartry Dam (Figure 7), near Roundwood, County Wicklow, Ireland.

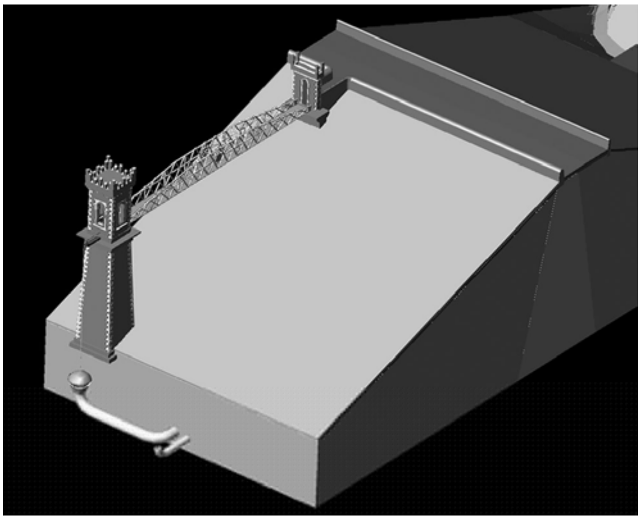

Figure 7. Schematic View of Upstream Side of Lower Vartry Dam

As a follow-up of the 12th Conference of the British Dam Society, *Reservoirs in a Changing World*, held at Trinity College, Dublin, September 2002 some aspects of the rehabilitation of the bottom outlet of Lower Vartry Dam have been discussed. It can be stated, that the basic rehabilitation techniques, which have already proven their feasibility at the Moehne, Verse and Ennepe Dam can be applied at Lower Vartry Dam as well, which might be (in brief):

- removal of sediment in the inner culvert and replacing it with concrete
- installation of guard valves inside the outer culvert behind stop-wall
- installation of new pipework and regulating valves in outer culvert
- use of the existing ball plug as a temporary emergency gate
- use of an additional valve at the intake as permanent additional guard valve respectively emergency gate.

Experiences of the Ruhr River Association show, that this work can be done at full reservoir level under full operation of Vartry Waterworks. Techniques like pipe-freezing to stop the flow during construction can not be used at Lower Vartry , since the pipes are made of cast iron, which has a tendency to fracture at extremely cold temperatures.

CONCLUSION

A number of successful underwater rehabilitation projects carried out by the Ruhr River Association has proven, that down to water depths of 50 – 60 m the rehabilitation techniques have become state of the art. Nevertheless, a number of fatal accidents in professional diving during the last years indicate, that safe working conditions should not be taken for granted. The dam owners have to insist and enforce, that the diving contractors provide the best possible safety features and working conditions. Otherwise fatal accidents are almost inescapable. Well equipped diving platforms with decompression chambers should be a must on every underwater rehabilitation site. It has also be stated, that the inspection and rehabilitation of very large hydraulic structures with water depths of more than 60 – 80 m and extreme dimensions especially at the upstream intake are still a challenge. The basic layout of many of these structures turns out to be rather disadvantageous for inspection and rehabilitation. Apparently minor design deficiencies can prove as extremely costly with regard to rehabilitation. The design of new structures should focus on this problem much more.

Some of the sophisticated inspection and rehabilitation techniques described in this paper (like ROV and saturated diving) can be considered as state of the art, but in combination with unusual features of the hydraulic structures (like extreme dimensions) there is still a lot of practical knowledge to be made. Techniques like pipe-freezing seem to be not ready for the practical use in large hydraulic structures yet.

Therefore the international exchange of experiences in the field of inspection and rehabilitation of large hydraulic structures is vital for the future of our water infrastructure.

ACKNOWLEDGEMENTS
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Ericht and Dalwhinnie Dam refurbishment and protection works

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SYNOPSIS. Following the statutory inspection of Loch Ericht reservoir both Ericht and Dalwhinnie Dams have been recategorised A from category B and assessed in relation to their capacity to safely pass a PMF event combined with wave surcharge allowances.

The paper describes the investigation, identification of the requirement for protection and subsequent design of works to primarily prevent wave surcharge levels overtopping the existing crest levels of both dams. Further refurbishment and protection works were also identified in relation to concerns over the ability of the spillway\corewall interface to resist erosion, poor spillway basin configuration and the potential vulnerability of the scour penstock during spill conditions at Ericht dam.

INTRODUCTION

The investigation and subsequent works carried out at Loch Ericht reservoir were required following the 10 yearly statutory inspection under the Reservoirs Act 1975 (1) which was carried out in June 2000 by Dr A K Hughes. The reservoir was recategorised A (general/minimum) from its previous category of B under the Floods and Reservoir Safety Guide (2). The various structures associated have therefore been assessed in relation to their capacity to safely pass a Probable Maximum Flood (PMF). Concerns were also raised over the vulnerability of the scour penstock and general spillway basin configuration.

DESCRIPTION OF RESERVOIR AND DAMS

Loch Ericht reservoir is situated approximately 75km northwest of Perth and was completed in stages over the period 1928 to 1954. The reservoir is one of the main storage reservoirs within Scottish and Southern Energy plc's (SSE) Tummel valley cascade hydro scheme system and provides long term seasonal storage from a catchment extending to 135.22km^2.

The reservoir is formed by the construction of Ericht and Dalwhinnie dams. Ericht dam at the southwest of the Loch comprises of sections of concrete gravity; concrete corewall with downstream grass covered embankment as support and is approximately 340m long and 14.3m maximum height above ground level. There is also a homogenous earth embankment section with grass covered upstream and downstream faces, approximately 65m long and 2.1m maximum height above ground level. Dalwhinnie dam at the northeast end is an embankment dam with a central concrete corewall supported by both upstream and downstream embankments, the upstream face is protected by concrete slabs and the downstream face is grass covered, approximately 350m long and 4.5m maximum height above ground level. The volume stored within the reservoir is 230 million m^3 with a surface area of 23.27km^2 at spillway level of 359.359mOD and water length of 24.4km. General sections of both dams are shown in Figures 1 and 2 respectively.

Scour from the reservoir is provided via a penstock that was added in 1957 as an extension to the original culvert through the concrete dam section. The penstock is a 2.13m diameter steel plate section extending 15.7m from the toe of the dam within the spillway basin. An anchor block with a 1.83m diameter disperser valve is located at the end of the penstock.

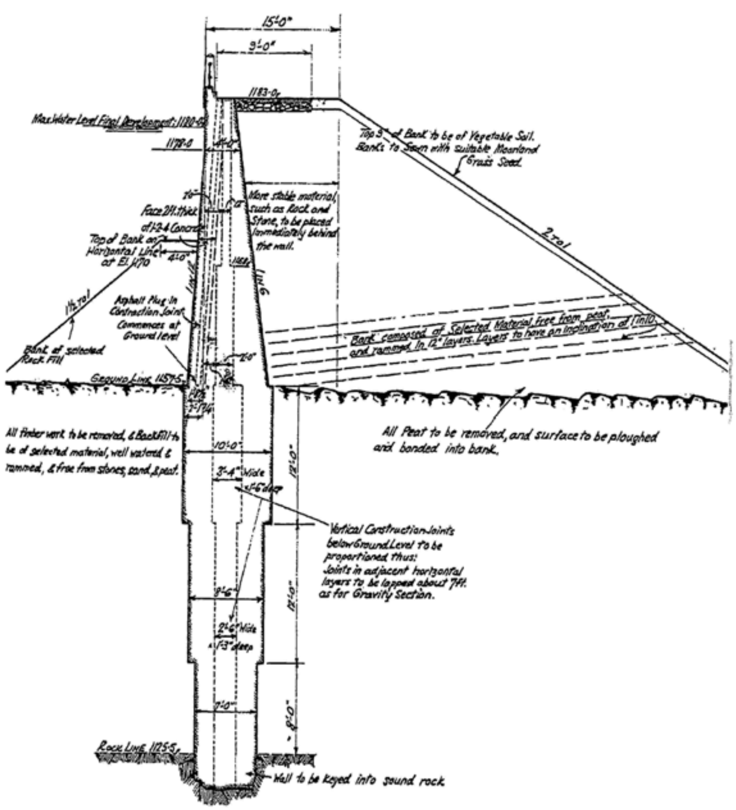

Figure 1 Typical cross section through Ericht corewall Dam

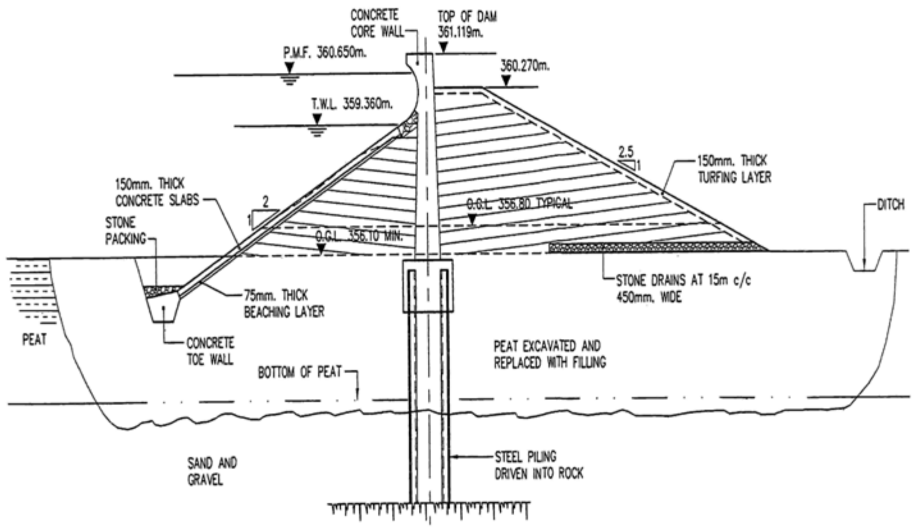

Figure 2 Typical cross section of Dalwhinnie Dam

RESERVOIR RECATEGORISATION

As a key element of the initial review the potential consequences, in particular the incremental consequences between PMF and dam breach were considered.

Ericht Dam failure

The inundation maps prepared and subsequent consequence study fully supported the recategorisation of the dam on the basis that should Ericht Dam breach other cascade failures would be likely in the Tummel valley reservoir system and hence a significant impact on communities beyond the next reservoir in cascade. The mapping also showed the cumulative effect on a number of isolated properties along the shoreline of Loch Rannoch.

Dalwhinnie Dam failure

Under normal conditions and flood events Dalwhinnie Dam prevents flow from the Loch Ericht catchment from passing into the River Truim. Should the dam breach the reservoir inundation mapping and subsequent consequence study clearly demonstrated that there would be an unacceptable level of inundation and significant impact on Dalwhinnie and the downstream communities. Application of Category A is therefore also appropriate for Dalwhinnie Dam.

INVESTIGATION PHASE

As part of the statutory inspection SSE led and implemented a detailed investigation to allow assessment of the impact of recategorisation and to prepare options for subsequent detailed design and implementation.

PMF and wave surcharge assessment

SSE completed flood studies for both PMF and 1 in 10,000 year return period events under worst case conditions of the syphons unprimed, due to some longstanding doubt over their operation and a snowmelt rate of 70mm/day. The following key flood results were obtained.

10,000 year (FSR) (3)	360.065mOD
10,000 year (FEH) (4)	360.348mOD
PMF (FSR)	360.575mOD

The wave surcharge levels using normal approaches were estimated for each of the component dams with a straight line fetch of 3.7km adopted for Dalwhinnie Dam rather than the bent fetch of 24.4km. The identified deficiencies are summarised in table 1.

Table 1 Wave surcharge assessment

Element	**Wave surcharge**	**PMF + wave**	**Crest**	**Deficiency**
Ericht gravity	0.64m	361.215m	360.58m	N/A
Ericht corewall	1.02m	361.595m	360.58m	1.015m
Ericht saddle	1.73m	362.305m	361.19m	1.115m
Dalwhinnie	2.93m	363.505m	361.19m	2.315m

As a result of the above analysis various parts of the dams were considered to be vulnerable and would be effected under extreme flows. Such sections required to be protected or modified in order that wave overtopping would not erode embankment sections, which if allowed too could ultimately lead to a breach of one or more of the dam sections.

Survey and Site Investigation

A full topographic survey was completed at both dams and the surrounding area in order that key dimensions and physical layouts could be confirmed. Site investigation works followed to confirm ground conditions and to provide information for the subsequent design of remedial works. Investigations comprised ten cable percussive boreholes, seven trial pits and 18 Macintosh probe penetration tests at Dalwhinnie Dam with two boreholes and seven trail pits at Ericht Dam. Disturbed samples were taken for subsequent grading analysis, seven falling head permeability tests were carried out and insitu standard penetration tests were made in granular material to assess the relative density. The sulphate contents and pH values of ground water and soil samples were determined. Piezometer standpipes were installed in four boreholes at Dalwhinnie Dam with pressure transducers attached to dataloggers and the water levels monitored and related to reservoir level.

Hydraulic model

In order to examine concerns raised over the vulnerability of the scour penstock and poor spillway basin configuration a 1 in 50 scale physical model of the spillway, scour penstock anchor block and adjacent river channel was constructed and tested by ABPMer. The model was built to provide an understanding of the flow mechanisms existing on the downstream side of the dam and in particular examine hydrodynamic loading and scour on the valve structure and the corewall embankment where it intersects with the spillway section of the dam.

The model confirmed that whilst the existing velocities and differential head across the penstock were not significant at $2ms^{-1}$ and 0.4m respectively the penstock would be submerged and the protruding body of the disperser valve may be vulnerable, in particular during flow build-up. Winter operation is to empty the penstock to avoid freezing, but the penstock was not designed to be submerged under this condition. With the high replacement cost of the valve if damaged by debris, it was considered prudent to encapsulate the penstock and provide a protection wall.

The water levels within the spillway basin were found to be at a level were erosion of the corewall embankment was possible, especially with an eddy between the penstock and the embankment toe. The optimum configuration and top wall level for a spillway basin training wall was developed using the model.

Localised infilling of the unlined spillway floor were also modeled to improve conditions during routine operation and avoid problems with water ponding around the penstock.

When comparing each of the configurations tested, the addition of a slab, penstock protection and a baffle wall did not significantly affect the hydrodynamic environment. The exception was an increase in eddy speed adjacent to the scour valve, however the scheme does provide substantial protection to the valve and penstock against impact of debris in flow from the dominant direction. The addition of the corewall embankment toe protection progressively reduced the strength of the eddy but with a corresponding detrimental effect on water levels and flow speeds in other areas.

Option Study

Following review of the various elements of the investigation measures were considered to prevent the wave overtopping. Comprising of reducing the reservoir operating level to create further freeboard; additional spillway capacity to reduce the flood lift; providing wavewalls to prevent overtopping; providing downstream protection and to create a rougher upstream face to absorb wave energy thus limit run-up.

Reducing the reservoir operating level would place restrictions on the generation output from Rannoch Power Station and would require large elements of the diverted catchment to be turned out during extended periods to maintain freeboard levels.

Limited potential exists for economically adding further spillway provision at Ericht and Dalwhinnie dams due to the nature of the embankment and corewall sections and the excessive overtopping levels that required to be mitigated. The main Inverness to Perth railway line traversing 100m downstream of the dam compounds this at Dalwhinnie.

Wave overtopping prevention by the addition of wavewalls is well proven and could be combined with additional upstream face rip-rap and or slope reprofiling to absorb energy and reduce wave heights. An optimum balance between upstream face rip-rap protection, wave wall and downstream erosion protection was considered the best solution at this stage.

The penstock protection and spillway basin improvement works required a compromise between the construction costs of implementing them and minimisation of the hydraulic forces and scour velocities.

At this stage SSE prepared an option study report to summarise the findings and to provide the basis for detailed design. A subsequent contract was awarded to Faber Maunsell Limited to carry out the detailed design.

DESIGN PHASE

PMF Reassessment

Following the interim guidance for owners and panel engineers issued by DEFRA (5) the PMF was reassessed by bench marking against the FEH 10,000 year rainfall depth for the critical storm duration. The all year PMP was 241 mm against an FEH 10,000 year depth of 285.65mm for an 18.5 hour storm. The PMF hydrograph was generated assuming the modified PMP storm depth (equal to the FEH 10,000 year rainfall) and routed through the reservoir. The view was also taken that the syphons would prime under such conditions and full account taken of this. This resulted in an inflow of $1684m^3s^{-1}$, outflow $459m^3s^{-1}$ and maximum water level 360.65mOD. An increase of $322m^3s^{-1}$ for inflow, $43m^3s^{-1}$ outflow and 150mm above an equivalent FSR estimate. Also 75mm over SSE's previous assessments which were considered to be conservative by assuming unprimed syphons.

Wave surcharge Reassessment

Wind-wave generation in most reservoirs is governed by fetch limited conditions for wave generation and deepwater conditions for wave propagation. However, these conditions were considered not to prevail for waves approaching Dalwhinnie. A detailed reassessment of wave conditions during the mean annual and the 1 in 200 year wind-wave event was carried out and is reported upon separately (6). The mean annual significant wave height is estimated at 2.12m for the PMF level of 360.65mOD. The 1 in 200 year significant wave height is estimated at 1.47m for the top water level of 359.37m. A significant reduction over previous estimates.

Fetch limited and deep water conditions apply at Ericht Dam, and the wave conditions approaching both the corewall and embankment dam sections was reassessed using the standard Donelon/JONSWAP method, as recommended in Floods and Reservoir Safety (2) and a bent fetch slightly longer than previously adopted. The mean annual significant wave height is estimated at 1.14m for the PMF level. The 1 in 200 year significant wave height is estimated at 1.55m.

The above estimated wave conditions for both Dalwhinnie and Ericht Dams were used together with the maximum flood levels to calculate wave overtopping discharge for freeboard assessment and wave loading for the structural design of wave walls. A methodology for deriving impact loading, occurring when waves break directly on the structure, was developed to provide an improved prediction of impact forces due to concerns over damage and instances of failures of wave walls, reported separately (6).

Value Engineering

A value engineering meeting was held to discuss preliminary design options for the works required. Formal value engineering techniques were used to evaluate options for overtopping protection at Dalwhinnie Dam, while the remaining items were discussed more informally.

The basic options considered for Dalwhinnie Dam were A) placement of open stone asphalt layer on upstream face with additional wave wall; B) placement of rip-rap on upstream face at existing 1:2 slope with additional wave wall and C) placement of rip-rap on upstream face at 1:4 slope with additional wave wall. All options assumed crest protection would be installed. Permutations included infilling the maximum depth section in the foreshore to limit the incident depth limited waves to the average depth condition, installation of downstream protection, use of grouted rip-rap to reduce the stone size, and inclusion of a tandem rock breakwater upstream to reduce incident wave height.

A value tree with importance weightings assigned to each criteria and the options were evaluated in more detail with the aim of identifying the best value alternative to be carried forward to detailed design. A decision matrix was developed from the weighted value criteria identified during the structuring of project objectives. The results of the decision matrix are shown in Table 2.

Table 2 Dalwhinnie Dam decision matrix results

Option	**Description**	**Total rating**
A iii)	Open stone asphalt layer	8.6
B iii)	Rip-rap at 1:2	6.9
BG iii)	Bituminous grouted rip-rap at 1:2	6.9
C iv)	Rip-rap 1:4	7.4
C v)+	Rip-rap at 1:4, tandem breakwater	6.5

The matrix analysis showed clearly in favour of option Aiii), placement of open stone asphalt on the upstream face, with wave wall, crest and downstream protection. This was partially due to the significant cost savings of this option; estimated to be approximately £200k cheaper than the next cheapest option considered.

Design Solutions

At Dalwhinnie Dam the upstream face will be overlaid with a 250mm thick layer of open stone asphalt and the low area in front of the dam infilled to the general level of 356.8mOD at the mitres of the dam. The wave wall will be raised by precast concrete unit's approximately 2m height, supported by an insitu concrete beam formed at the base of the existing wave wall on the upstream side, and anchored to the upper part. The crest and downstream face will both be armoured with concrete reinforced grass, to increase the tolerance to wave overtopping discharge. The works are generally shown in Figure 3. Mass concrete corewall extensions are also required at either abutment to prevent floodwater bypassing the dam and eroding the downstream embankment toe.

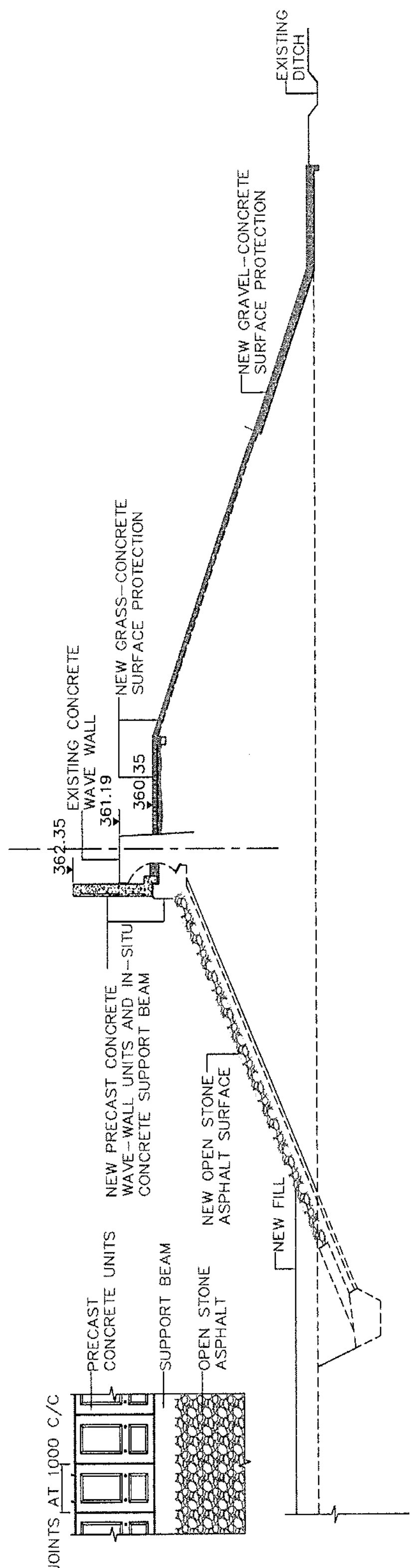

Figure 3 Dalwhinnie Dam

The corewall section of Ericht Dam requires a wave wall 1m in height to be added to limit overtopping to an amount acceptable for an unprotected downstream face. A reinforced concrete wall anchored onto the corewall section will provide this. Rip-rap is to be placed on the upstream face of the embankment dam at a slope of 1V: 2H to reduce wave run-up and overtopping discharge. A low berm will be formed above the crest level, negating the need for a wave wall. The crest of the embankment will be reinforced with grass-concrete blocks, to increase the tolerance to wave overtopping. Both sections are indicated in Figure 4.

In order to provide protection to the exposed penstock concrete encapsulation beyond the toe of the dam will be carried out and a baffle wall added to protect the protruding disperser valve. Permanent access to the interior of the penstock will be provided by 1m diameter flanged branch pipe. Improvement of the spillway training and invert protection will consist of placement of a reinforced concrete slab on the invert of the spillway channel to a maximum level of 347mOD, draining towards the river channel downstream. A reinforced concrete training wall along the interface with the corewall embankment be constructed to a nominal height appropriate for frequent spill events, with the remaining slope to be protected with grass-concrete blocks to prevent erosion during extreme events. The penstock and spillway works are shown in Figure 5.

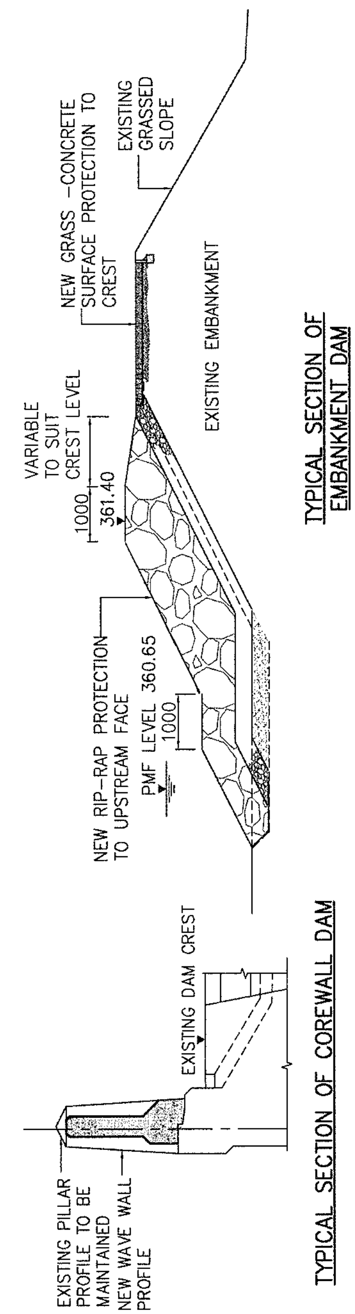

Figure 4 Ericht corewall and embankment dam works

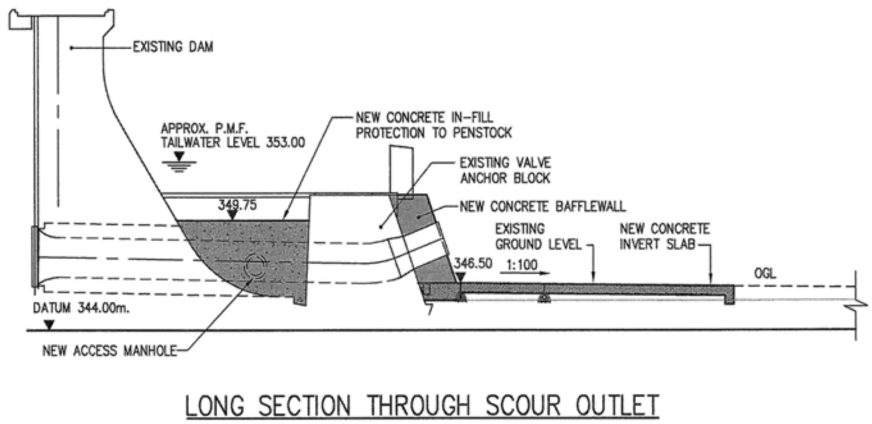

Figure 5 Ericht penstock and spillway works

IMPLEMENTATION

Consents

An application to implement the works was made under the Electricity Act 1989 (7) in December 2002. This Section 36 consent remains outstanding one year on for what should have been a minor consent application. No EIA was required and consultation processes were carried out with each local authority, Scottish Natural Heritage and local estates.

Contract Strategy

Tender documents were based on the NEC Engineering and Construction Contract (8) with an activity schedule, all for implementation of the works during 2003 with a reservoir draw down over 18 weeks. However due to consent delays the works have been deferred to 2004 with the subsequent increase in costs. Estimated costs are £700k and £300k at Dalwhinnie and at Ericht respectively.

Water management issues lead to the adoption of sectional completion on the spillway and scour penstock protection works in advance of the main works to allow compensation water to be released downstream of Ericht in the event of plant failure at Rannoch Power Station. This will also provide a further control on the reservoir level should it be required.

Valve and penstock Refurbishment

The original plan was to remove in advance and refurbish the disperser valve to coincide with the sectional completion of the penstock civil works. Shot blasting and repainting of the internal surfaces of the penstock and the addition of an access manhole was included within the civils scope to avoid interface issues during concrete works. Due to the consent delay SSE decided to mitigate this and carry out all of the penstock mechanical works in advance and awarded the works to Isleburn MacKay & MacLeod. The works were completed in December 2003 at a cost of £80,000.

CONCLUSIONS

Both Ericht and Dalwhinnie Dams have been recategorised A, which was fully supported on the basis of inundation mapping and consequence studies. Subsequent investigation demonstrated that certain elements of the structures would be vulnerable under PMF conditions and required to be modified.

PMF re-estimation increased inflow by 24%, outflow by 10% and the resultant flood lift indicated by 14% following the application of the FEH 10,000 year rainfall depth as an estimate of PMP. In this situation the difference in level adopted was relatively small in relation to the overall surcharges being considered, and the economic implications were generally acceptable. It may even provide some degree of insurance against subsequent changes to future methodologies.

Wave surcharge reassessment concluded that depth limited prediction methods reduced the significant wave heights compared to those estimated using the standard wave run-up method. A methodology for estimation of wave impact forces on the wavewall extensions at Ericht Dam has been established.

A value engineering exercise established an open stone asphalt system combined with a wave wall extension at Dalwhinnie Dam as the optimum solution, previously unconsidered in the investigation stage.

Delays to the consent process were partially mitigated by carrying out the penstock mechanical works in advance. Future reservoir projects will be considered closely and where appropriate not be subject to the section 36 consent processes.

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Wave Assessment on Loch Ericht

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SYNOPSIS. This paper uses the example of two dams to illustrate generic problems and solutions to the analysis of waves and wave forces on wave walls.

Dalwhinnie Dam and Ericht Dam impound Loch Ericht, which straddles Perthshire and Highland Regions in Scotland. Inadequate freeboard at both dams means that remedial works are required to increase the wave overtopping protection. In both cases, standard design methodologies had to be extended to achieve a credible design basis for the works.

INTRODUCTION

Loch Ericht is a natural loch drained by the River Ericht, flowing in a southerly direction into Loch Rannoch. The loch level was first raised in 1930/31 by the construction of Ericht Dam at the south western end, and further in 1937 by the raising of Ericht Dam and construction of Dalwhinnie Dam at the north eastern end. The reservoir is now over 24 km long. The reservoir is owned and operated by Scottish and Southern Energy plc, and supplies the Rannoch hydo power station as part of the Tummel Hydro Electric scheme.

Following a statutory inspection in June 2000 under the Reservoirs Act 1975 (1), Loch Ericht was recategorised from Category B to Category A. As such, both Ericht Dam and Dalwhinnie Dam were subject to more severe design flood standards, the result of which was that freeboard was inadequate for both dams. Concerns were also raised over the arrangement of the scour penstock and general spillway basin configuration at Ericht Dam.

FaberMaunsell (FM) was retained by Scottish & Southern Energy plc (SSE) for the detailed design of the works.

The works comprised wave overtopping protection and corewall extension at Dalwhinnie Dam, wave overtopping protection at Ericht Corewall Dam and Ericht Embankment Dam, and penstock protection, spillway channel training and invert protection at Ericht Dam.

This paper focuses on the assessment of wave conditions and wave forces for the design of wave overtopping protection works at Dalwhinnie Dam, and at Ericht Corewall Dam. A full description of the project is reported separately (2).

WAVE ASSESSMENT AT DALWHINNIE DAM

General

The wave surcharge on a dam is a function of the wave height and other wave characteristics. Initial estimates of the wave characteristics at Dalwhinnie Dam, and the corresponding remedial works required, were based on standard methods for waves developed in deep water conditions. It was appreciated that these methods were not necessarily fully appropriate for the conditions at Dalwhinnie Dam because of the extremely long, narrow nature of the reservoir, and shallowness in the approach region to the dam. A full review of the wave assessment methodologies was therefore carried out.

Wave prediction in reservoirs

Most wave prediction methods are based on measurements carried out in oceanic and coastal waters, with fetch lengths and fetch widths very different from those found in most UK reservoirs. The Saville/SMB method was the standard method in UK prior to the production of the 3rd Edition of Floods and Reservoir Safety Guide (3). A full review of available methods such as Saville/SMB, JONSEY and Donelan/JONSWAP can be found in HR Wallingford Report EX1527 (4).

Following concern that the Saville/SMB method did not provide good predictions of waves on long, narrow reservoirs, measurements of wind and waves were made notably at Megget reservoir and Loch Glascarnoch (5). It was concluded that while none of the methods gave particularly good agreement with measured wave heights for all wind speeds and directions, Donelan/JONSWAP gave fairly good agreement for a wide range of wind directions, and any errors in predicted wave heights were likely to be conservative. This simplified Donelan/JONSWAP method was subsequently recommended in the Floods and Reservoir Safety Guide, 3rd edition (3).

For the Loch Ericht study, it was felt that further investigation of the appropriate wave prediction method would yield little without site specific data, and that the simplified Donelan/JONSWAP method should be retained.

<u>Duration limited wave generation</u>

Wind-wave generation in most reservoirs is governed by fetch-limited conditions for wave generation. A duration factor is applied to the wind speed in the method described above to take account of the fact that usually fetch lengths in inland waters are small and waves fully develop within 15 minutes; waves are thus “fetch-limited”. However, Loch Ericht is unusually long, with a fetch approaching Dalwhinnie Dam of 24.4 km, shown in . Examination of the wave characteristics of the JONSWAP spectrum reveal that the minimum wind duration required to develop waves of this size is greater than 2.5 hours (refer to Figure 9 of BS 6349 Part 1 (6)). Waves on Loch Ericht are therefore “duration-limited”.

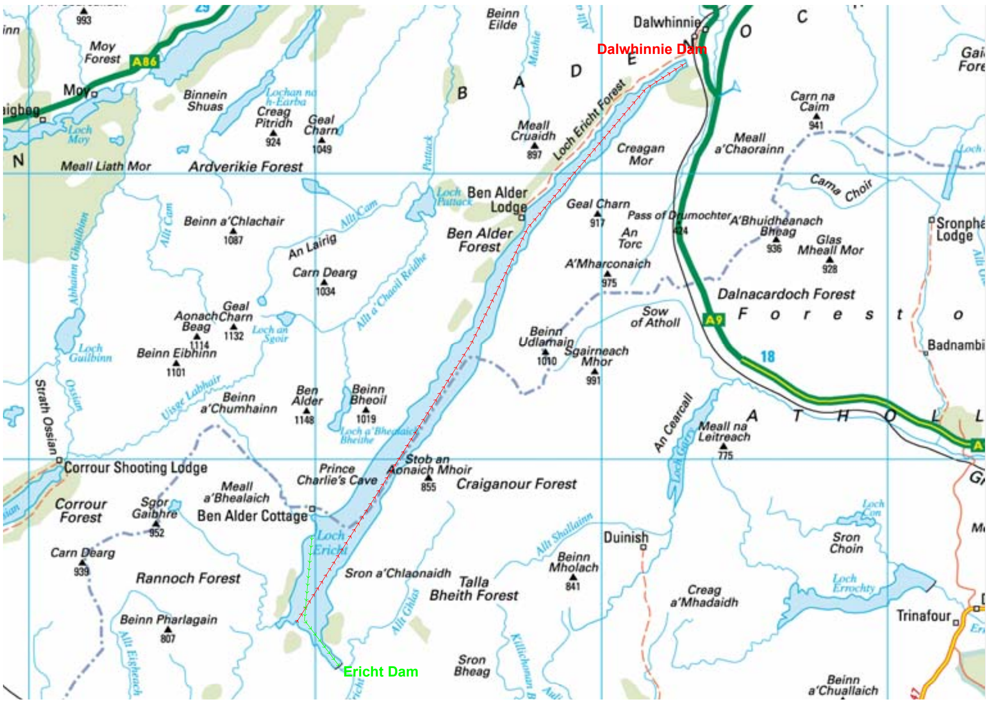

Figure 2: Loch Ericht showing fetch to Dalwhinnie and Ericht Dams

Various duration factors are given in CIRIA 83 Rock Manual (6), and these are applied to the mean annual maximum hourly wind speed to estimate mean annual maximum wind speeds for increased durations. For a duration of 2.5 hours, the appropriate factor is 0.97. Calculation of the design mean annual maximum wind speed and resulting significant wave height is shown in Table 1.

Table 1: Calculation of Dalwhinnie Dam significant wave height – deep water

fetch (m)	24400
fetch direction ^{O}N	239
50 year max hourly wind speed U_{50} (m/s)	23.50
return period adjustment f_T	0.79
altitude adjustment f_A	1.36
over water adjustment f_W	1.31
duration adjustment f_D (2.5 hour duration)	0.97
direction adjustment f_N	1.00
mean annual max wind U (m/s)	32.06
deep water significant wave height H_S (m)	2.84

<u>Wave conditions at Dalwhinnie Dam</u>

At Dalwhinnie Dam, shallow water depths extend approximately 1 km into the reservoir from the dam.

The onset of shallow water processes depends on the water depth (d) in relation to the deep water wavelength. The deep water wavelength was estimated assuming small amplitude wave theory:

$$L = \frac{gT^2}{2\pi} \qquad \text{giving } L_{op} = 54.33 \text{ m}$$

For deep water wave conditions, $d/L > 0.5$

For shallow water wave conditions, $d/L < 0.05$

The ground level at the toe of the dam is approximately 356.8 mAOD. The average depth of water at the toe for a Probable Maximum Flood (PMF) level of 360.65 mAOD is therefore 3.85 m. The lowest part of the toe towards the north end of the dam is 356.113 mAOD, giving a maximum depth of water of 4.535 m.

In this case, $d/L = 0.08$, in deepest water at the toe. Deep water wave conditions, where wave speed is determined solely by wavelength, therefore do not apply. Asymptotic shallow water wave conditions where wave celerity is determined solely by water depth are also not fulfilled. The speed of waves approaching Dalwhinnie Dam is therefore determined by both wavelength and water depth.

This result implies that shallow water processes will affect the waves approaching the dam.

Estimate of shallow water wave characteristics

Reducing water depths lead to the transformation of incoming deep water waves by refraction, shoaling and eventually breaking. On the assumption that the wave period is constant, these processes affect the wave height and wavelength.

Refraction

Wave refraction is a consequence of the wave moving out of deep water. The portion of the crest in shallower water has its celerity reduced and is progressively turned parallel to the bed contours.

Refraction due to the bathymetry of the foreshore upstream of Dalwhinnie Dam is difficult to assess without more detailed bathymetric data than was available. Budget and contract time constraints meant that numerical modeling could not be accommodated. It was therefore assumed that incident wave fronts are parallel to the loch bed contours which are themselves parallel to the dam face and no refraction takes place.

Shoaling

Shoaling is the increase in wave height and decrease in wavelength and wave celerity caused as waves propagate in reducing water depths. Using linear wave theory, this effect can be expressed as a shoaling coefficient K_S; the equation for is given in CIRIA 83 Rock Manual (6) as follows:

$$K_S = 1/\{[1+2kd/\sinh(2kd)]\tanh(kd)\}^{0.5}$$

Where k = wave number = $\frac{2\pi}{L}$ d = water depth

At the toe of the dam, the wavelength can be estimated using first order wave theory as follows:

$$L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)$$

For average water depth, L = 33.55 m

For maximum water depth, L = 35.89 m

This wavelength is used to estimate the shoaling coefficient to give a new estimate of significant wave height at the toe of the dam shown in Table 2:

Table 2: Shoaling wave heights

	Average d = 3.85 m	Max d = 4.535 m
Wavelength L (m)	33.55	35.89
Shoaling coeff K_S	1.128	1.063
Wave height H_S (m)	3.20	3.02

Wave breaking

As waves approach a shoreline, and the water becomes shallower, they may become unstable and break, either through steepness induced breaking, or depth induced breaking. In shoaling water, breaking is usually caused by the latter, but both should be considered.

Steepness induced breaking occurs when $H/L \leq (H/L)_{max} = 0.14\tanh(2\pi d/L)$
Depth induced breaking occurs when $H/d \leq (H/d)_{max} = \gamma_{br}$ (breaker index)

For regular waves, theoretical γ_{br} is 0.78, but in practice, for irregular waves, γ_{br} is found to be 0.5-0.6. A summary of the wave breaking calculations are shown in Table 3.

Table 3: Summary of wave breaking at Dalwhinnie

	Steepness induced breaking		Depth induced breaking		Comment
	H/L	H/L_{max}	H/d	H/d_{max}	
Average depth d = 3.85 m	0.096	0.086	0.83	0.6	steepness and depth induced breaking
Max depth d = 4.535 m	0.084	0.092	0.67	0.6	depth induced breaking

Depth induced breaking will be the most critical event in this situation. The theoretical breaker index of 0.78 occurs in 4.0 m water depth. The reported figure of 0.5 for irregular waves would cause waves to break in about 5.0 m of water, and a breaker index of 0.6 would cause breaking in 5.5 m of water. Depth induced breaking could therefore occur anywhere from 200 m from the dam, up to the dam face itself, assuming a foreshore slope of 1 in 120.

CIRIA 83 Rock Manual (6), figure 121 gives design graphs for shoaling on uniform slopes with breaking. For a foreshore slope of 0.01 or shallower (1 in 100), and relative water depth (d/L_{op}) of 0.07, the resulting significant wave height was found to be 1.86 m.

Owen (8) also suggested a simple method to provide an estimate of the upper limit to the significant wave height in any depth of water. The method describes simple empirical equations for varying foreshore slopes. For a slope of 1 in 100, the equation is as follows:

$$\frac{H_S}{d} = 0.58 - \frac{2d}{gT_m^2}$$

giving $H_S = 2.12$ m for average $d = 3.855$ m
$H_S = 2.47$ m for max $d = 4.535$ m

These two methods are the most up-to-date empirical methods available. They result in a fairly wide range of wave height and in the absence of any refining detail, we must adopt the more conservative figures from Owen's method.

The transformed wave characteristics at the toe of the dam for the mean annual wind wave therefore become:

Table 4: Shallow water wave characteristics – Mean annual wind

	Average depth d = 3.85 m	Max depth d = 4.535 m
significant wave height H_S (m)	2.12 m	2.47 m
peak wave period T_P (s)	5.90 s	5.90 s
mean wave period T_M (s)	5.13 s	5.13 s

These results indicate that waves incident on Dalwhinnie Dam are affected by the depth of water during a PMF event and break before or on the dam face. It should be noted that if wave breaking had not been indicated, the shoaling wave height is significantly higher than the deep water wave height, as shown in Table 2, and the resulting design wave height would have exceeded the deep water wave height.

<u>Change in wave height distribution</u>

Wind generated waves are irregular, having varying wave heights and wavelengths. The significant wave height used in many design methods represents the mean of the highest one third of the waves, or the wave height which is exceeded by 14% of waves. In deep water, wave heights tend to follow a Rayleigh distribution. However, as shown in Figure 109 in CIRIA 83 Rock Manual (6), the wave height distribution in shallow water is affected by wave breaking. This shows that the proportion of waves higher than H_S reduces due to shoaling and breaking.

Ideally, the entire wave spectrum should be transformed for shallow water effects and a new design wave selected from the transformed spectrum. However, this is not considered necessary as transforming deep water significant wave height through shallow water should produce a conservative result in terms of estimating wave overtopping discharge for wave wall design.

<u>200 year wave height</u>

The Floods and Reservoir Safety Guide, 3rd edition (3), recommends that if the calculated wave surcharge is greater than the flood surcharge, as is the case for Dalwhinnie Dam, then the total surcharge should be calculated again assuming the reservoir at initial reservoir condition plus the wave surcharge resulting from a 200-year wind speed. The higher surcharge should then be used.

For Dalwhinnie Dam, the water depth is less in this situation, and because the waves are depth limited, the 200 year significant wave height is less than the mean annual maximum significant wave height. The condition of PMF plus concurrent mean annual wind wave event is therefore more severe and this event was used as the design basis for the overtopping protection works. The final design wave height represented a significant reduction compared to previous estimates assuming deep water and fetch-limited conditions, and allowed a more economical solution to be designed, details of which are reported separately (2).

WAVE FORCES AT ERICHT DAM

At Ericht Dam a new wave wall to the corewall section was required to provide sufficient freeboard. The height of the new wave wall would be designed to ensure acceptable overtopping discharges during the more severe of the following conditions:

- Peak pool elevation during the PMF event, plus mean annual maximum wind-wave
- Top Water Level plus 1 in 200 year wind-wave

Initial investigation of the wave conditions approaching the dam indicated that the new parapet wall could be subject to impact wave loading, when waves break directly on the structure. Following evidence of damage to the breakwater at Amlwch on Anglesey, failure of a wave wall at Porthcawl and other instances, HR Wallingford has advised that wave impact forces be included in the analysis of all such walls.

For Ericht, a methodology for deriving impact forces and calculating effective forces for Ericht Corewall wave wall was developed to provide an improved prediction of impact forces, taking into consideration the duration of impact forces and 3-D spatial effects.

<u>Wave loading on structures</u>

Wave loading on vertical or composite structures can be either pulsating loads or impact loads:

Pulsating or quasi-static wave loading arises when a wave impinges directly against the structure, the wave surface rises up and applies a quasi-static pressure difference on the structure.

Impact or dynamic pressures occur when a wave breaks directly on the structure due to the particular combination of foreshore slope, water depth and wave characteristics. Wave impacts are generally of high magnitude, but of short duration, and may be too fast for massive structures, such as the corewall itself to respond to, but may be more critical for smaller structure components, such as the parapet wave wall. They are also spatially limited, so average loads will decrease with increasing section length.

Initial investigations indicated that for the mean annual wind/wave event at PMF level, wave loadings at Ericht Corewall Dam were pulsating. However, impact loadings might act on the proposed parapet wall during the 1 in 200 year wind/wave event occurring at top water level.

Development of wave impact loading methodology

Methods to estimate pulsating wave loads are reasonably well established. Goda's method (6) to estimate pulsating wave forces was used. However, methods to estimate impact loadings are far from comprehensive and, in particular, methods to estimate the wave pressure distribution in order to estimate the effective pressures on the wave wall alone had not been published.

Advice was sought from HR Wallingford, who has published the most recent design methodologies for estimating wave impact loadings. These methodologies were still incomplete with regard to a few aspects of wave impact loadings that were necessary to complete the calculations at Ericht.

The aim was to produce a methodology to provide an improved prediction of impact forces for Ericht Corewall wave wall, taking into consideration the duration of impact force and 3-D spatial effects.

The resulting methodology has been developed from published design methods (9), (9), (10), (11), supplemented with data held by HR Wallingford.

Procedure to derive effective wave forces on Ericht corewall parapet wall

Four sections of the west corewall dam were considered to investigate wave loading. Sections 1 and 2 were treated as composite structures, having a berm in front of the corewall, as shown in Figure 3; while sections 3 and 4 were treated as vertical walls. In the absence of survey data, a foreshore slope of 1 in 20 was assumed as this gave the highest design wave heights. The relevant structural and hydraulic data are given in Table 6 and Table 7.

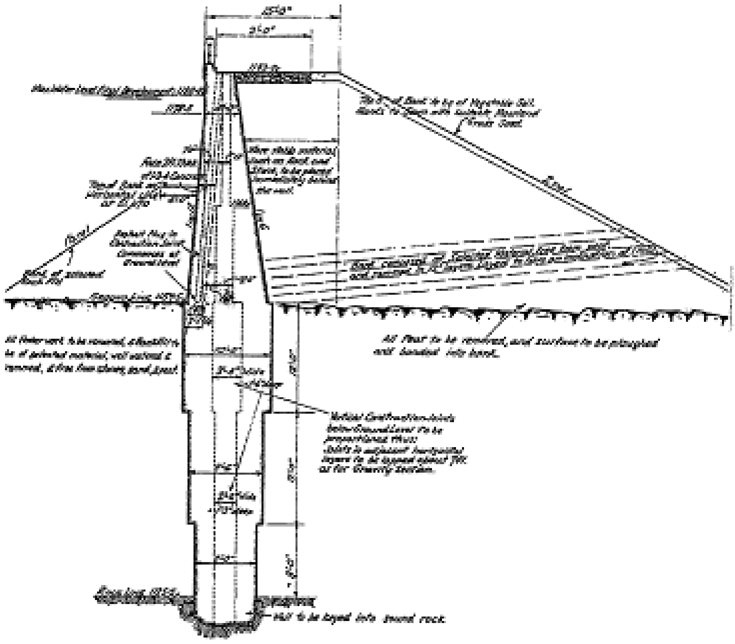

Figure 3: Typical section through Ericht Corewall (sections 1 and 2)

Table 6: Corewall structural data

Section	1	2	3	4
Top of wall (mAOD)	361.57	361.57	361.57	361.57
Wall base (mAOD)	360.58	360.58	360.58	360.58
Top of berm (mAOD)	356.62	356.62	N/A	N/A
Bed level (mAOD)	353.42	354.64	356.77	358.3
Foreshore slope	0.05	0.05	0.05	0.05

Table 7: Hydraulic data for Ericht Corewall Dam

Water level (TWL mAOD)	359.4
1 in 200 year wave height H_s (m)	1.55
1 in 200 year wave period T_s (s)	3.48
Deep water wavelength L_{op} (m)	24.9

The developed methodology is as follows:

1. Pulsating wave forces

As a first estimate of wave induced loads on the wall, Goda's method (6) was used to estimate pulsating wave forces, taking into consideration wave shoaling and shallow water depth-limiting effects. The results are shown in Table 8.

Table 8: Pulsating wave loads

Section	1	2	3	4
Total force (kN/m)	90	87	68	21
Total force on wave wall (kN/m)	14	13	10	3
Pressure at top of wall (kN/m^2)	10.84	10.43	12.16	0
Pressure at wall base (kN/m^2)	16.21	15.74	6.85	5.15

2. Identify likely loading conditions from parameter map

In order to assess whether wave impact forces are likely to occur, the geometric and wave parameters were checked against the parameter map given in (11). For the wave conditions considered, impacting waves are possible at all four sections.

3. Predict wave impact force

Impact forces were calculated using Allsop & Vincinanza's equation (12):

$$F_{imp,1/250} = 15(H_{si}/d)^{3.134}\ \rho_w\ g\ d^2$$

4. Impact force duration and dynamic effects

Impact force durations were then estimated from HR Wallingford test data. Using the outer envelope, the impact rise time was estimated at 0.4 seconds. Data within the average band suggested a rise time of 0.04 seconds. These impact durations were then compared to the natural frequency of the wall, estimated using a standard cantilever formula to lie between 0.014 and 0.028 seconds. In both cases, the impact rise time exceeds the natural frequency of the wave wall. It is therefore reasonable to assume that impact wave loads will not be significantly damped, and the wall will effectively experience this load as quasi-static.

5. Pressure distribution

The prediction methods above give total wave forces over the full active depth, not just the parapet wall. In order to establish the proportion of load acting on the parapet wall section, the pressure distribution over the height of dam must be known.

For pulsating wave forces, Goda's method assumes a trapezoidal pressure distribution. Although this was not intended by Goda to predict pressure distributions, the general level of wave forces predicted is well validated, and no better guidance is available. The proportion of wave force acting over the parapet wall must therefore be estimated using this distribution.

For impact forces, Hull (13) analysed measurements of wave impact pressures by McKenna (10) and others from which Hull developed a method for estimating pressure distributions as a function of the pressure at still water level (p_{max}) given in Table 9.

Table 9: Impact force pressure distribution

Pressure (kN/m^2)	Elevation (m relative to SWL)
$p_f = 0$	$1.2\ h_s^2/h_b$
$p_e = 0.08\ p_{max}$	$0.4\ h_s^2/h_b$
$p_d = 0.4\ p_{max}$	$0.17\ h_s^2/h_b$
$p_c = p_{max}$	0
$p_b = 0.4\ p_{max}$	$-0.25\ h_s^2/h_b$
$p_a = 0$	$-0.9\ h_s^2/h_b$

Where h_s = depth of water at toe of berm
h_b = height of top of berm above bed level

Note that this step could only be carried out for Sections 1 and 2 as the method is only applicable for composite structures. Figure 5 shows the pressure distribution plots for Section 1.

The effective pressures acting on the wave wall alone were estimated. Assuming a linear distribution between the pressure at the top of the wall and the base of the wall, the total force acting on the wall was then calculated.

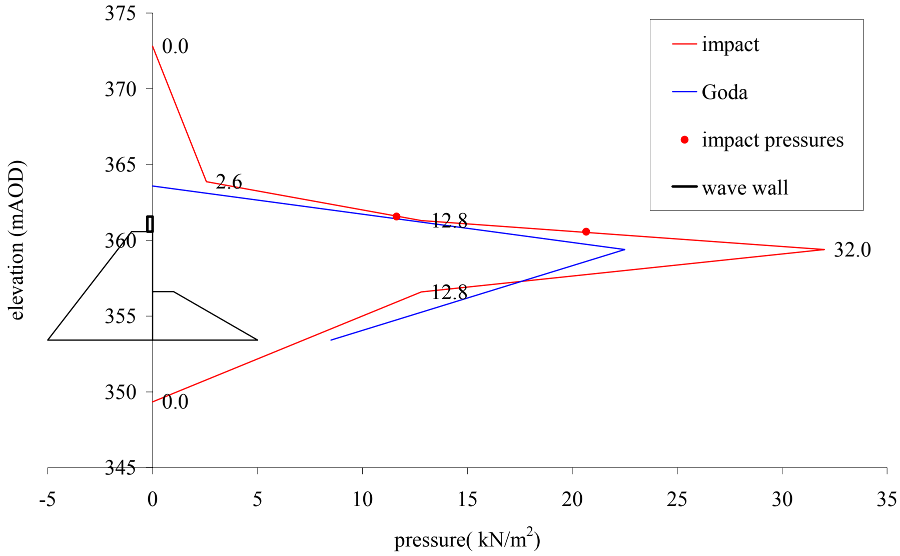

Figure 5: Section 1 pressure distribution

6. Three dimensional effects

Wave impact forces are spatially limited. Increasing unit length leads to a greater reduction in force, but unit length is more likely to be driven by construction methodology. For Ericht parapet wall, 1.5 m was considered to be the longest practicable pre-cast or in-situ cast unit. Guidance is given in (11) for methods to calculate the reduction in wave impact forces.

Assuming a wall unit length of 1.5 m resulted in a reduction in impact force of 9%.

7. Resulting design wave forces

Following the methodology set out above, Section 1 was found to give the highest force and pressure estimates for both pulsating and impacting loads. The results are given in Table 10.

Table 10: Wave loads on wave wall section

Loading	Impacting	Pulsating
Total force on wave wall (kN/m)	16	14
Pressure at top of wall (kN/m^2)	11.63	10.84
Pressure at wall base (kN/m^2)	20.66	16.21

The reductions in impact loading to account for 3-D spatial effects, and the pressure distribution of that loading have resulted in a total impact force on the critical Section 1 that is only marginally larger than the pulsating force. The impact on the design of the wave wall due to wave impact loading is therefore minimal. However, it is recommended that for cases where impact forces appear to be smaller than the Goda pulsating forces, the Goda force should always be used for design.

CONCLUSIONS

The Ericht project provided an exceptional opportunity to explore the limitations of standard methods to assess wave conditions and wave forces generated on reservoirs, and to extend those methods in a practical context.

The wave assessment at Dalwhinnie Dam has shown that not all reservoirs conform to the assumptions implicit in the standard method to estimate significant wave height using the simplified Donelan/JONSWAP, described in Floods and Reservoir Safety, 3rd edition (3), namely that:

- Fetch limited waves are generated on reservoirs
- Deep water conditions apply for waves approaching any dam

Reservoir engineers should be aware of these implicit assumptions and check whether they apply in any given location. Shallow water processes, in particular, can have a substantial effect on the wave height incident on the upstream face of a dam, and may offer significant savings. However, if approach characteristics are such wave breaking does not occur, design wave heights higher than deep water conditions may result.

Initial analysis of wave loads at Ericht Corewall Dam to be used in the design of a wave wall indicated that available methods to estimate impact loadings are far from comprehensive. In particular, methods to estimate the wave pressure distribution to allow an estimate of the effective pressures on the wave wall alone had not been published.

A methodology was developed to provide an improved prediction of impact forces for Ericht Corewall parapet wall, taking into consideration the duration of impact force and 3-D spatial effects. Impact forces can be substantially higher than pulsating forces. However, in this case, because of the position of the parapet wall relative to the still water level where the maximum impact pressure occurs, the impact force did not greatly exceed the estimated pulsating force.

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An Incident at Ogston Reservoir

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SYNOPSIS. In the autumn of 2001 an incident occurred at Ogston Reservoir which led to the catastrophic failure of the pipework in the draw-off shaft. An uncontrolled release of water commenced which was only prevented by the quick actions of two operatives in the shaft. This paper describes the investigations carried out to establish the cause of failure, the remedial works which were carried out and the lessons learnt.

INTRODUCTION

Ogston Reservoir is owned and operated by Severn Trent Water. Completed in late 1959, the reservoir is situated about 6 km north west of Alfreton, Derbyshire. The treatment works, situated immediately downstream of the reservoir, supplies water to areas in North East Derbyshire, Chesterfield and Sheffield.

The dam, which is an earthfill embankment with central puddle clay core, has a height of 19.8 metres and is 213 metres long. It impounds a maximum storage of 6,180,000 cubic metres of water.

The forebay tunnel, overflow shaft, valve tower, and combined overflow and draw-off tunnel are situated in the centre of the embankment and constructed of mass concrete. The complex arrangement of these structures is shown in Figures 1 and 2, with the draw-off tower forming a single structure with the overflow shaft. The overflow tunnel and draw-off tunnel are also formed as one structure.

There are three levels of draw-off comprising 24″ (600mm) diameter cast iron pipework and in-line guard and duty gate valves, feeding into a common 30″ (760mm) diameter draw-off stack. Water passes vertically downwards in the stack to join into a similar diameter cast iron draw-off

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

main which passes along the discharge tunnel under the embankment to Ogston Water Treatment Works.

The scour facility prior to the incident comprised 30″(760mm) diameter cast iron pipework with a 30″(760mm) guard gate valve known as G4, and a 700mm duty butterfly valve known as S1.

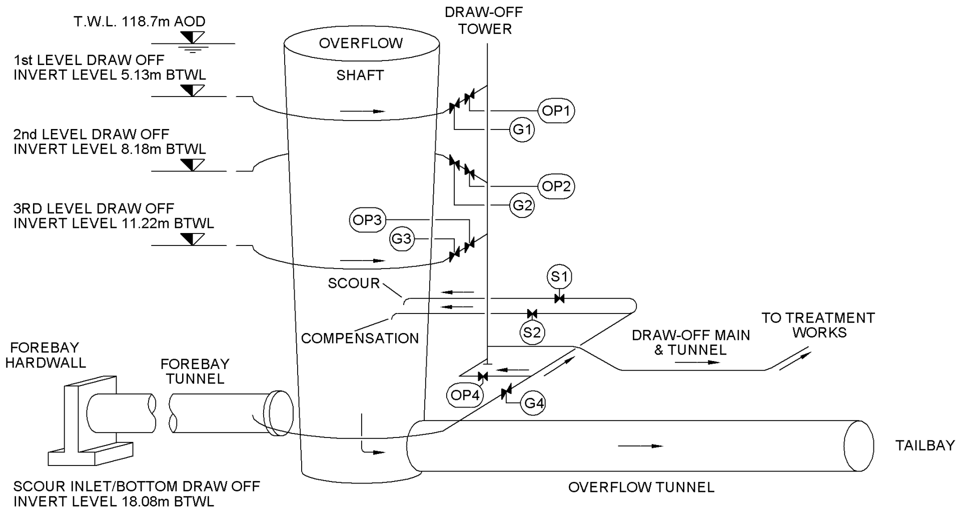

Figure 1: Schematic diagram of draw off and overflow arrangement

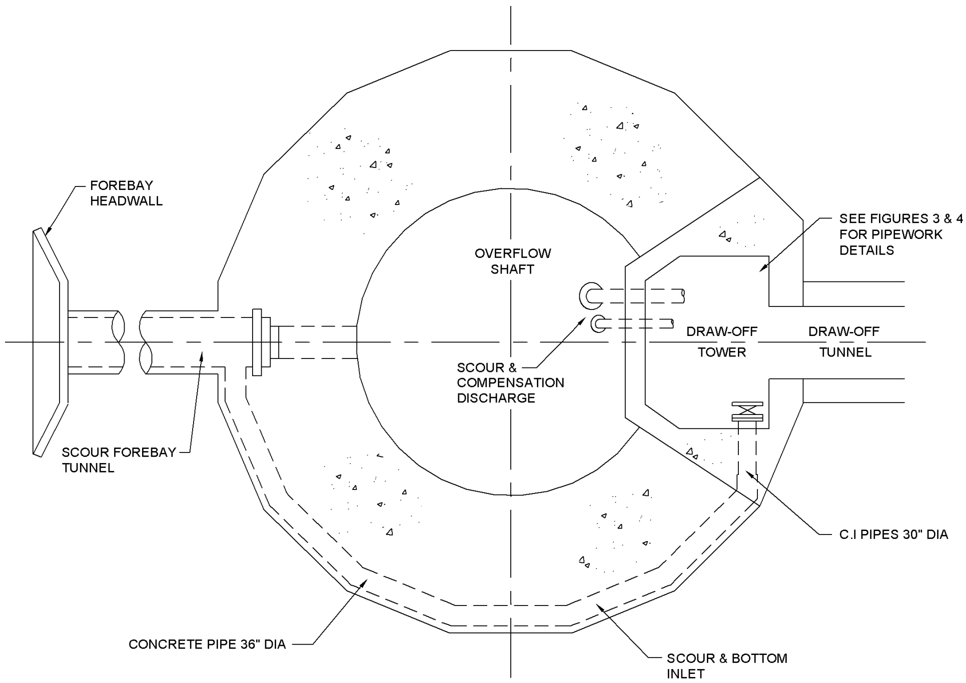

Figure 2: Plan showing scour arrangement

Discharge of scour water is via a pipe outlet through the wall of the draw-off tower into the base of the overflow shaft. A 9″ (225mm) branch connection from the scour pipework incorporated a Larner Johnson streamline valve, known as S2, for the release of compensation water. The butterfly valve (S1) was a recent replacement for the original 30″-24″-30″ (760-600-760mm) diameter Larner Johnson streamline valve which had been found to be in a poor condition, difficult to operate and requiring rehabilitation or replacement. The original and modified layouts of the scour valves in the draw-off tower are shown in figures 3 & 4.

INCIDENT

As part of the refurbishment process to return the Larner Johnson to a serviceable condition alternative valve options were considered due to the extent of the refurbishment work that would be required on the original valve. A value engineering exercise was carried out and a 700 mm diameter butterfly valve was chosen to replace the Larner Johnson. A Panel Engineer was not involved in the value engineering exercise, however subsequently one was consulted on the proposal to install a butterfly valve. The Panel Engineer, having carried out some calculations, commented that the velocities appeared to be high and recommended that confirmation be sought from the manufacturer as to the valve's fitness for purpose with respect to the maximum expected velocity and its location within the pipework arrangement. This confirmation was provided and the valve was obtained and installed.

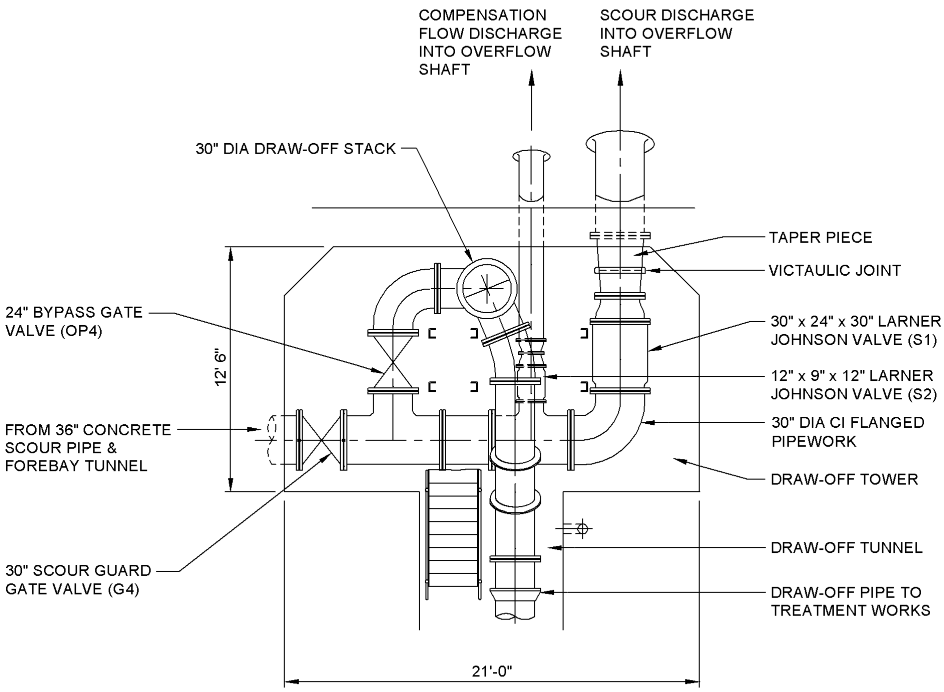

Figure 3: Original scour valve layout in draw-off tower

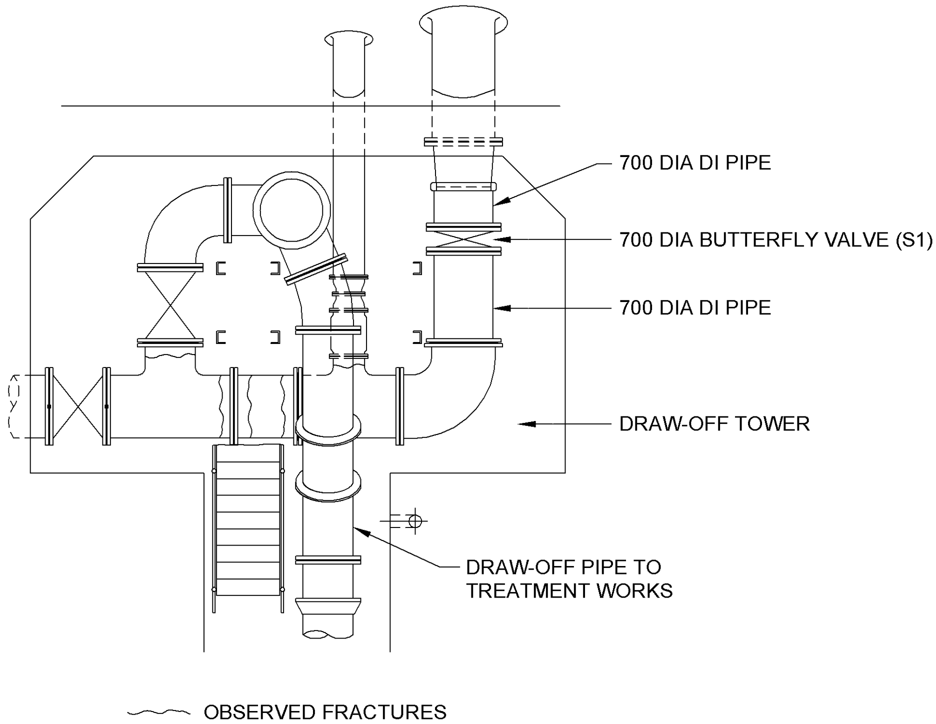

Figure 4: Modified scour valve layout in draw-off tower prior to incident

Following the installation of butterfly valve S1 some difficulties were experienced. Initially the valve was found to be very stiff to operate and a number of modifications were made including increasing the diameter of the operating hand wheel and increasing the capacity of the gearbox. During the commissioning tests on the butterfly valve, the pipework immediately upstream, including the compensation water branch, suffered catastrophic failure resulting in the sudden uncontrolled release of water from the scour pipe into the base of the draw-off tower where there were two men trying to operate the valve. This discharge quickly started to fill the draw-off tunnel until it blew the doors open at the downstream end allowing water to discharge back to the downstream tail-bay area.

Guard valve G4 was subsequently shut to isolate the discharge by the men going back through the discharging waters.

ADVICE

Dr Hughes, who was the appointed Inspecting Engineer at that time, having recently carried out a routine inspection of the reservoir in accordance with the Reservoirs Act 1975, was informed of the incident. Details and recommendations arising out of his subsequent site visit were included in his report.

Technical advice was provided to the owner throughout the project by Dr Hughes, and by the Review Panel, the owner's retained experts, headed by Mr R E Coxon.

Kellogg, Brown and Root (KBR) were appointed to design and supervise the construction and installation of all temporary, enabling and permanent works involved in the restoration of the scour and compensation facilities, taking due account of all personnel health and safety and reservoir safety considerations.

During the initial site inspections it became apparent that the draw-off tower and draw-off tunnel were not safe places to work, since catastrophic failure of the pipework had occurred in a number of places and had left the scour guard valve G4 unsupported and unrestrained in the base of the draw-off tower. Fortunately the flanges on the scour pipework either side of valve G4 appeared to have survived the surge pressures generated by the failure of butterfly valve S1. However, because of the uncertainty regarding the condition of the valve G4 and the adjacent flanged puddle pipe it was considered unsafe to operate the guard valve. Therefore there were now no means of effecting scour draw-off from the reservoir should the need arise and so 'emergency' remedial works were recommended by Dr Hughes 'in the interests of safety'.

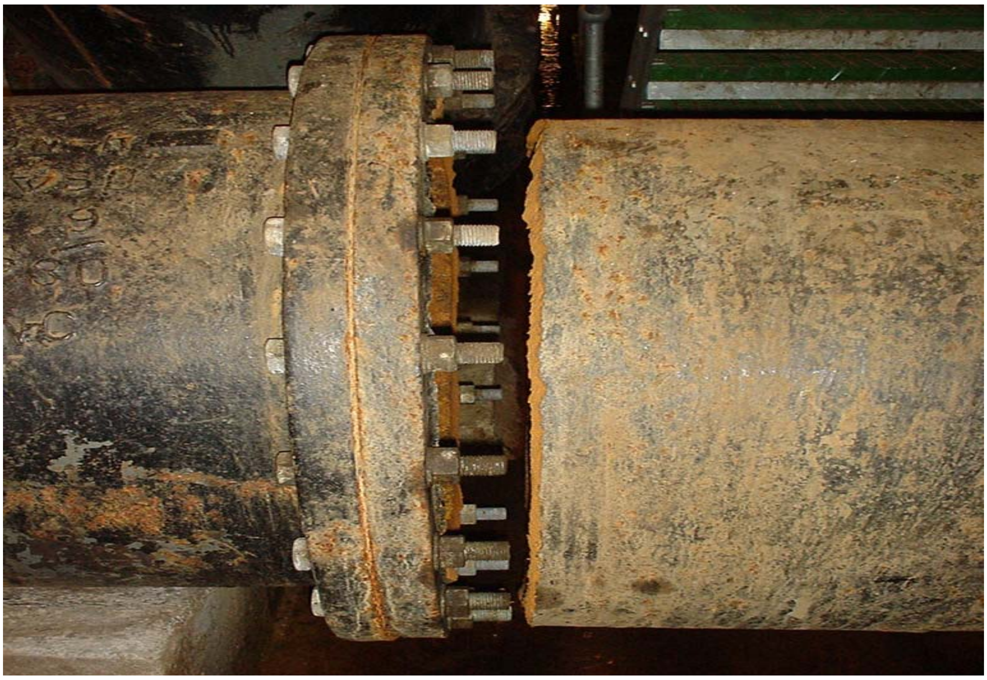

Plate 1: Fractured 30" scour pipe

Plate 2: Fractured compensation pipework

Plate 3: Damaged draw-off tunnel doors

The following permanent works were deemed to be necessary:-

- Reinstatement of the scour and compensation pipework and valves.
- Reinstatement of associated accesses and floor stagings where necessary.
- Reinstatement of tunnel access doors.

It was immediately apparent that:-

- The full extent of damage was unknown.
- That the working areas were very restricted.
- The reservoir would have to remain partially full during the works in order to protect the fisheries and the shoreline nesting bird population, as agreed with English Nature (the shoreline was a designated SSSI).
- The compensation flow of 6 Ml/d would have to be maintained.
- No record drawings existed – although construction drawings were available.

It was also recommended that:-

- An additional 600mm diameter washout facility be provided in the raw water supply pipeline to provide scour facilities and greater control of the reservoir water levels in the short term.
- A temporary bulkhead should be installed on the scour forebay tunnel headwall to enable safe access into the draw-off tower to facilitate the investigation and repair works. In order to assess the feasibility of the bulkhead installation an underwater survey of the scour forebay headwall would be required.

PROPOSED APPROACH

The approach proposed was to work closely with the owner to ensure the safety of the reservoir whilst undertaking the necessary investigations and surveys required to formulate a strategy for the method of repair.

Therefore, in order to achieve a successful outcome to the project, an ‘Operational Plan’ was written with the owner to:

- Provide the owner with sufficient information to operate the reservoir so as to meet water supply requirements and the needs of the scour valve repair project. In the case of the latter it was arranged to reduce water levels over an agreed timescale to meet the start date for the repair contract.

- Identify the steps necessary to ensure the safety of the reservoir, company project personnel and the public.
- Identify key contacts and responsible persons.
- Ensure that all statutory and legal requirements were met.
- Provide a framework for liaison with all interested parties.
- Ensure that all environmental issues were fully recognised and managed.

The Operational Plan was considered to be a ‘live’ document subject to continual review and update as the project proceeded. New operating control curves were drawn up and a number of draw-down and refill scenarios as well as ‘unusual events’ modelled to assist the operators and contractors engaged to undertake the surveys, investigations and permanent works.

It was not considered likely that the embankment would fail in the event of the scour pipework failing upstream of guard valve G4, however the uncontrolled release of water which would take place through the draw-off tunnel and the eventual draining of the reservoir had to be considered.

PROJECT PROGRAMME AND ENABLING WORKS

The Operational Plan included a very detailed programme and methodology covering all activities necessary to control the reservoir level over the winter period and achieve a managed draw-down to the lowest draw-off level in the spring of 2002 to facilitate the installation of the replacement scour pipework and valves in the draw-off tower.

An additional 600mm gate valve washout facility was provided, via a 600mm branch off the 24″ raw water supply main to the treatment works. This was installed and commissioned before the onset of winter.

An underwater survey was undertaken by divers. The objectives of the survey were to:-

- Determine the silt levels and accessibility of the scour forebay headwall.
- Undertake a survey of the forebay tunnel headwall.
- Assess the feasibility of installing a temporary watertight bulkhead. The bulkhead would be used to enable dewatering of the forebay tunnel and scour pipework around the overflow shaft, and allow examination of the embedded puddle pipe (immediately upstream of valve G4) in the draw-off tower wall. Information obtained would be used in the design of the subsequent valve replacement works.

PERMANENT WORKS

It was immediately evident from the inspection following the failure of the butterfly valve S1 that the surge pressures generated by the incident caused fracture and complete failure of the scour pipework. What was not known was whether the surge pressure had caused overstressing of the valve bodies and other fittings which did not show any visible signs of failure. It was possible that the surge pressures had caused damage to:-

- 30″ scour guard valve G4 and the associated puddle flanged pipe set into the wall of the draw-off tower.
- 36″ concrete scour pipe that ran around the base of the overflow shaft.
- 24″ bypass valve OP4, 90 degree bend and connection with the draw-off stack.
- 9″ Larner Johnson streamline valve S2.

In addition, the movement of the scour pipework may have caused damage to the 30″ and 9″ compression couplings downstream of the Larner Johnson streamline valves on the scour and the compensation pipework respectively.

The temporary and permanent works were designed, therefore, to replace the majority of the above pipework and valves by the construction of a temporary bulkhead on the scour forebay tunnel headwall, so as to provide a safe environment inside the draw-off tower for construction operatives and supervisory staff.

The temporary and permanent works involved:

- Contractor designed watertight bulkhead on scour forebay tunnel headwall with a facility to dewater the tunnel by pumping.
- Removal and replacement of all damaged and suspect pipework and valves.
- Construction of new thrust blocks.
- Carrying out in situ non-destructive testing of all built – in pipework, all couplings and any pipework likely to be left in position.
- Modifications to platforms, ladders and stairs as required.
- Replacement of damaged doors at entrance to access tunnel.

Following an assessment of the options for replacing the scour valves, KBR recommended that the butterfly valve should be replaced by a Larner Johnson valve. Fortunately it was possible to track down the original Larner Johnson valve which had been removed and have it refurbished for subsequent installation by the appointed contractor.

Because of the dangers associated with entering the draw-off tower and also the lack of detailed drawings there were a number of uncertainties and concerns at the time of tendering regarding the feasibility of using a bulkhead to facilitate the de-watering of the scour forebay tunnel and pipework. These uncertainties included:-

- achieving an adequate seal between the bulkhead and scour forebay tunnel headwall;
- the structural capacity of the headwall to support the bulkhead;
- the quantity of leakage into the forebay tunnel and scour pipework around the overflow shaft;
- the structural capacity of the scour forebay tunnel to withstand the proposed de-watering;
- the feasibility of manoeuvring the Larner Johnson valve along the draw-off tunnel;
- whether the Victaulic joints could be refurbished or replacements found.

CONSTRUCTION PHASE

Prior to awarding the contract detailed interviews with tenderers were held to ensure that their proposed methodology, risk assessments and strategies for dealing with the project uncertainties detailed earlier in this paper had been properly considered and evaluated. Following this process Norwest Holst Construction Ltd was appointed as Principal Contractor.

The contractor successfully completed the safe refurbishment of the scour facility in October 2002 by following the basic order of procedure detailed in the Operational Plan. The principal activities were:-

- Installation of a temporary bulkhead on the scour forebay headwall to allow dewatering of the forebay tunnel and the safe removal of the damaged scour pipework and butterfly valve.
- Carrying out a detailed survey of valve shaft pipework.
- Removal of scour guard valve G4 and testing of the embedded puddle pipework.
- Installation of anchor frame on puddle pipe flange and new 30″ scour guard valve. It is worth noting that outline pipework and valve designs were carried out by KBR at an early stage to facilitate the early procurement of the valves. The Contractor was given the detailed design of the pipework and valve arrangements following an accurate survey of the existing pipework in the tower. This survey could only be undertaken once the bulkhead had been fitted.
- Installation of new scour and compensation pipework and refurbished Larner Johnson valves. The Contractor elected not to

dismantle the Larner Johnson scour valve once it had been factory refurbished. Following delivery to the site a specially designed trolley enabled the valve to be moved along the tunnel and then positioned in the base of the shaft.

- Commissioning of valves and pipework.
- Removal of temporary bulkhead.

Plate 4: Draw-off main within the draw-off tunnel

Plate 5: Transporting the Larner Johnson Valve

INVESTIGATION OF VALVE FAILURE

Even though it was clear from the initial visit that the butterfly valve had been installed in a far from ideal position almost immediately downstream of a bend and discharging into almost free air with zero downstream pressure it was essential to find the reasons for the catastrophic failure witnessed at Ogston. Therefore an investigation was devised to determine the physical condition of the damaged pipework including:

- remaining wall thickness
- degree of corrosion
- evidence of welding
- flange rating
- strength

and to investigate the cause of the failure of the butterfly valve by:

- establishing that the valve had been constructed to the manufacturer's specification
- identifying the point and mode of failure
- performing strength tests on the failed components
- determining whether the valve had any locking device

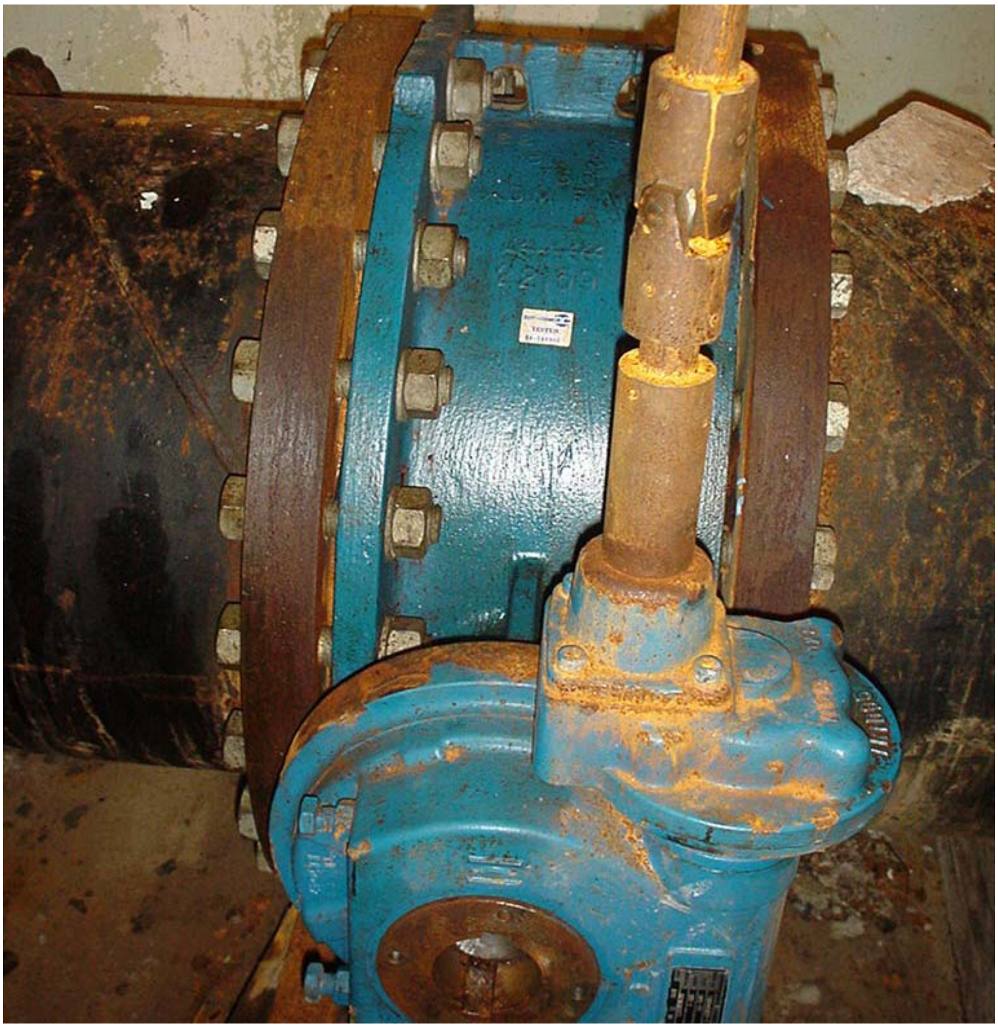

Plate 6: Failed butterfly valve in position and failed gearbox component

Following removal from the draw-off tower the butterfly valve and several pieces of pipework were taken to an independent testing laboratory for detailed examination and testing. A visual inspection of the valve identified no external damage, however, an internal investigation of the valve and gearbox made some interesting findings. The principal findings of the investigation were:

- The gearbox fitted to the valve was undersized for the application. The connection between the valve and the gearbox failed as a result of the excessive torque required to operate the butterfly valve beyond 50% open. This was due to the calculated presence of full cavitation and uneven flow profile, caused by the close proximity of the bend and the positioning of the valve.

- The capability of the gearbox was considerably reduced by one of four screws used for coupling the gearbox to the valve drive sleeve being missing.
- When the connection between the valve and gearbox failed there was nothing to prevent the valve slamming shut.
- There was little external corrosion of the pipes; however internal corrosion within the structure of the metal had reduced its tensile strength to some degree.
- The estimated surge pressures generated by the instantaneous closure of the butterfly valve would have resulted in the failure of new pipe to the same specification as that installed.

In summary, the failure of the pipework was due to the torque required to operate the butterfly valve being underestimated by its manufacturer and the gearbox being too small for purpose. This problem was exacerbated as one of the screws was not fitted into the gearbox drive sleeve and the remaining screws were not able to take the applied load. They subsequently failed allowing the valve disc to be rotated by the water flow, slamming closed and bringing the water flow to a sudden halt. The resulting change in momentum caused a pressure surge estimated to be in excess of 55 bars. This surge caused several sections of the pipework to fracture releasing a considerable quantity of water.

CONCLUSIONS

This paper describes an incident which put operatives at risk and resulted in the sudden uncontrolled release of water following the catastrophic failure of scour pipework. The failure of the pipework was caused by the fitting of an inappropriate valve and gearbox for the required duty and system configuration. The lessons to be learnt include:-

1. The specification for the design of a valve should take account of, inter alia, its purpose, location, fixings, hydraulic loading, adequacy and configuration of existing pipework and valves, thrust/tension resistance, intended operating procedures, frequency of use, accessibility and ease of operation, maintenance and facility for subsequent removal.
2. The valve manufacturer should design the valve, together with gearbox, actuator, etc, to meet the specification and should certify compliance by providing supporting calculations and details of works tests.
3. Due regard should be taken of such phenomena as cavitation and spiralling flow.

4. Engineers should understand how various valves work and consider their possible modes of failure. In the case of butterfly valves it needs to be recognised that a failure of the connection between the gate and the gearbox will result in the gate slamming shut instantaneously.
5. Expert advice both in terms of mechanical plant and reservoir safety should be sought when considering the replacement or refurbishment of valves in existing scour and draw-off arrangements.

ACKNOWLEDGEMENTS
The authors would like to express their thanks to Messrs Neil Williams (Principal Engineer Reservoirs) and Ian Elliott (Director of Engineering) of Severn Trent Water for permission to publish this paper.

Marmarik Dam Investigations and Remedial Works

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SYNOPSIS. Marmarik dam is a multipurpose embankment dam on the Sevan – Hrazdan cascade in Armenia. The dam is situated in one of the most seismically active regions in Armenia and in the vicinity of the reservoir numerous landslides could be seen. The dam was commissioned in January 1975 and twenty days later significant subsidence of the clay core occurred causing 14m settlement of the dam crest. The dam has never been rehabilitated and the reservoir has never been impounded.

The dam was investigated by JacobsGIBB ltd as part of the World Bank funded 'Technical Investigation of 60 Dams'. In addition to the slope failure, further issues include high regional seismicity, landslides adjacent to the dam and reservoir, inadequate spillway capacity, rehabilitation of the derelict outlet works and concerns regarding the foundation cut-off.

INTRODUCTION

Marmarik dam, situated in Kotayk Marz in Armenia, was originally designed to provide water for the future aluminium mining industry, a cement factory, two thermal power plants, irrigation of 2,000ha and flood water regulation. However, as the aluminium mining industry was never developed the dam changed ownership and the new owner became the Ministry of Water Resources. Marmarik dam is a part of the Sevan – Hrazdan cascade which significantly contributes in overall regional energy balance and provides water for irrigation systems and six power plants.

The dam was commissioned in January 1975 and twenty days after the commissioning significant instability of the embankment occurred, causing a 14m settlement of the dam crest over half of the dam crest length. Immediately after the subsidence a local company was commissioned in 1975 to investigate causes of the dam failure. It was found that the failure occurred as a result of high pore pressure in the clay core that was placed with a high moisture content. The dam has never been rehabilitated and therefore the reservoir has never been impounded. The river is diverted in a tunnel through the left abutment. This uncontrolled diversion has been left in operation since the construction period.

DESCRIPTION OF THE DAM

Embankment

Marmarik dam, Figure 1, was originally designed as a 64m high embankment with a clay core and compacted fill shoulders. The original dam crest was at 1914masl. The design cross section is shown in Figure 2. The shoulders were originally designed to be built of gravel from a borrow area some 5km downstream of the dam. However, only the bottom 5m of the embankment was constructed from gravel as the further use of the gravel borrow area was not permitted. Thereafter the embankment shoulders were constructed from a compacted sandy silt from borrow areas within the reservoir, but to the original design slopes and with no filters.

The central part of the dam is founded mainly on granular river alluvium and the abutments are founded on a thick layer of cohesive colluvial deposits. The designed foundation anti – seepage measures comprise a cut – off bored secant pile wall constructed up to 30m deep through the central part of the alluvial foundation and a grout curtain through the colluvial foundation at the abutments.

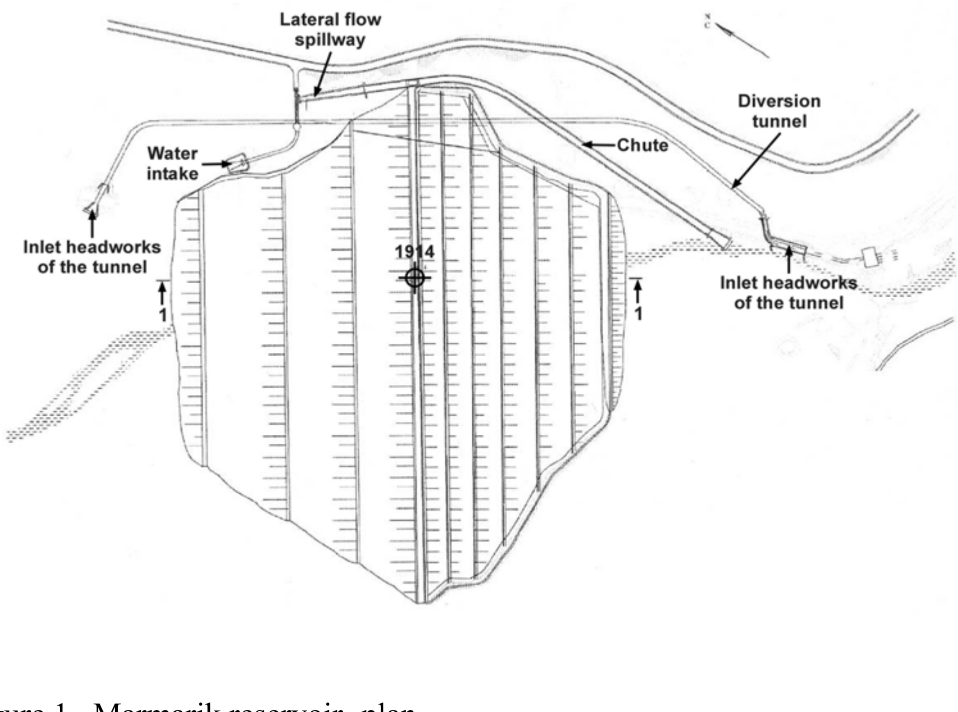

Figure 1. Marmarik reservoir- plan

Diversion tunnel

At its present state the river is diverted into a 3.2m diameter, D shaped diversion tunnel designed for a temporary condition, for a flood of 50m^3/s with a return period of 20 years. Since its completion, there have been three occasions on which the incoming flood exceeded the designed value but the flood was absorbed in the reservoir storage volume without risk of overtopping the dam. There is a side weir at the outlet end, which permanently maintains a minimum water level of 1.4-1.5m in the lower section of the tunnel. For that reason the complete inspection of the tunnel has never been carried out in the past.

Spillway

The spillway, situated on the left abutment comprises:

- A 60m long side-channel inlet weir
- A culvert under the dam crest
- A 6m wide discharge chute with a variable gradient

Outlet Works

The outlet works consist of a tunnel leading to a short connecting shaft to the diversion tunnel which is 11m below. The connecting shaft contains two 1.0m diameter outlet pipes which are cast into mass concrete which fills the shaft. A 50m deep, 6m diameter gate shaft is located at 5m offset from the connecting shaft. The gate shaft contains an emergency closure gate and a maintenance gate for each pipe.

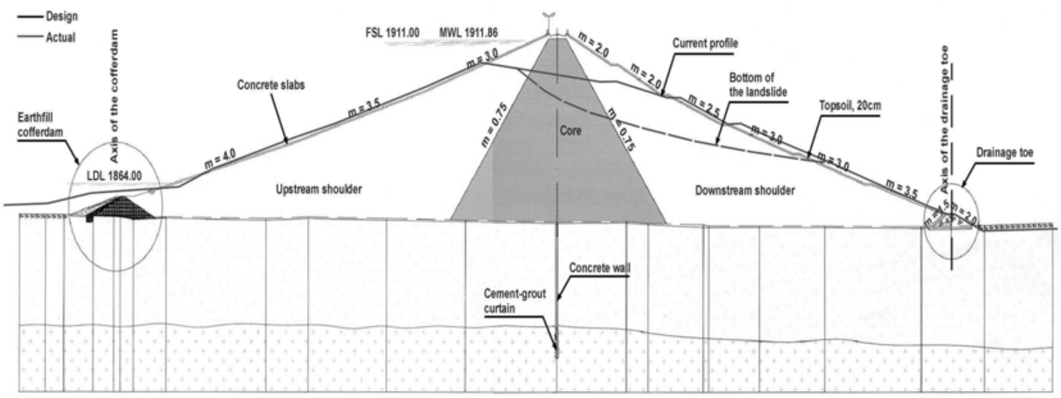

Figure 2. Cross Section 1-1

INVESTIGATIONS CARRIED OUT

Immediately after the subsidence of the dam, a local Armenian company was commissioned to investigate causes of the dam failure. The investigations were carried out between 1975 – 1978.

Under the 'Technical Investigation of 60 Dams' the following investigations and surveys were undertaken during 2002 - 2003:

- Topographic survey of the dam and the diversion tunnel
- supplementary ground investigations of the dam
- microseismic survey to establish site specific seismic parameters
- landslides hazard assessment and landslide ground investigation
- investigations of the diversion tunnel
- investigation of the efficiency of the foundation cut-off

The investigations undertaken during 2002-2003 are described below in more detail. Based on the results of the investigations and the findings of the investigations carried out during 1975-1978, geological, geomorphological and seismic conditions at the dam site were assessed as well as the status of the dam, foundation anti- seepage measures and the diversion tunnel.

Supplementary ground investigation

Supplementary ground investigation carried out to validate previous investigations included 480m of drilling through the dam, trial pitting, in – situ permeability testing and laboratory testing.

Microseismic survey

Microseismic survey comprised the following works:

- Seismic Refraction- carried out at 48 measuring points in the reservoir and 24 measuring points on the dam
- Measurements of ground micro - vibrations by using SMACH –SM and OMNILIGHT instruments - p-wave velocities were recorded in the surface deposits and in the bedrock, as well as the peak horizontal accelerations, vertical geomagnetic field and the distribution of predominant frequency spectra

Landslides hazard assessment and landslide ground investigation

Landslides of a seismogenic origin are widespread along the whole length of the southern (right) bank of the Marmarik River canyon. Four potentially hazardous seismogenic landslides were identified within the Marmarik reservoir area that may influence the dam safety, namely landslides N1 to N4. The landslides are shown in Figure 3. Landslide hazard assessment was carried out based on the analyses of satellite images and aerial photos that were taken in 1948, 1976 and 1986 as well as the field surveys carried out in 1975-78 and 2002.

Landslide N1 is located some 2 km to the S-SE of the Marmarik Dam, in the upper reaches of the Kiarkhana River, which is a lateral inflow of the Marmarik River and as such it poses a low hazard to the dam. The landslide was not investigated further.
Landslide N2 is located some 250m to the south of the dam. The landslide is situated close to the confluence of Kiarkhana with the Marmarik river. During 1969-1974, the material from the toe of landslide was excavated for construction of the Marmarik dam. The excavation destabilised the landslide leading to a development of presently active secondary landslides. The landslide was investigated during 1975-1978 site investigation. Thickness of the landslide varies from 30 to 80m, total volume is about $94x10^6m^3$.

Landslide N3 is 1.6km upstream of the dam and it was reactivated a number of times in the past, most recently during dam construction when the soil from the toe of landslide was excavated and used for the fill material. The landslide was investigated during the 1975-1978 site investigation. Thickness of the landslide varies from 40 to 60m, the total volume is about $16x10^6m^3$.

Landslide N4 is 5.2km upstream of the dam and at its toe, it branches into two landslides separated by some 700m. This landslide is the most distant from the dam, but it is the largest in volume. If it is triggered it could dam the Marmarik River and create a lake which, if the natural dam is breached, could induce a flood inflow into the reservoir. This landslide was investigated during 2003. It was found that landslide comprises a layer of rock debris with a soil matrix up to 50m thick, over a thin slip surface that overlays the in - situ rock.

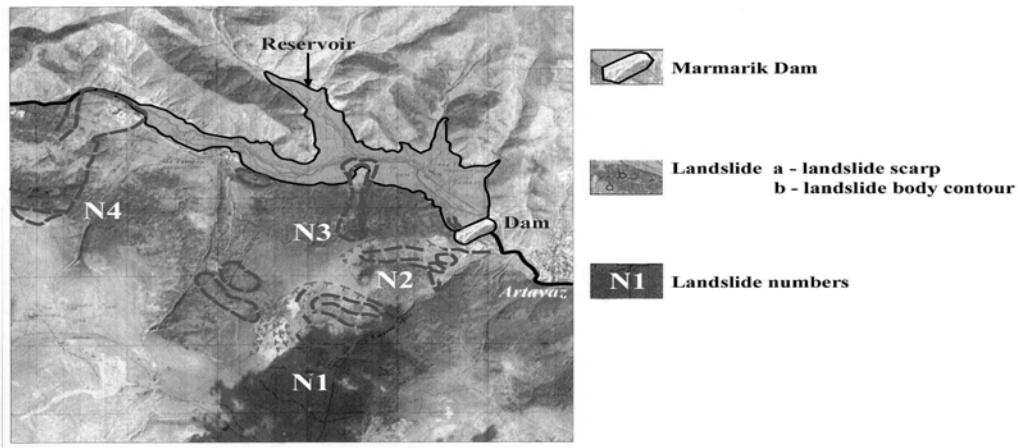

Figure 3. Landslide hazard map for the Marmarik reservoir

Investigations of the diversion tunnel

Investigation of the diversion tunnel comprised the following:

- Initial walk through and visual inspection
- Intrusive drilling through the tunnel lining
- Non destructive testing using a calibrated Schmidt hammer and a hand held ultrasonic meter to determine quality of the concrete.

Investigation of the efficiency of the foundation cut-off

As the reservoir has never been impounded, there is a significant uncertainty about the efficiency of the foundation anti-seepage measures, especially the ones through the alluvial foundation. The investigation of the effectiveness of the cut-off through the alluvial foundation was therefore accomplished by carrying out water pressure tests and indicator tests in the test section located in the centre of the river channel. The test section comprised three holes located 15m u/s of centreline (Hole1), 5m u/s of centreline (Hole 2) and 5m d/s of centreline and drilled down to the bedrock.
The water pressure tests were used to measure the difference in response of piezometers (placed in the foundation material upstream (Hole1) and downstream (Hole 3) of the cut – off) to water pressure applied in Hole2 drilled upstream of the cut – off. In Holes 1 and 3 piezometers were installed 5m below the fill/alluvium interface and 5m above the alluvium/bedrock interface. The difference of the response in Holes 1 and 3 is a measure of the permeability of the cut –off.

The water pressure test method was supplemented by the introduction of an indicator (salt solution) into the borehole and a comparison of the concentrations of the indicator throughout the borehole.

GEOLOGICAL CONDITIONS

Geology of the dam site comprises deeply weathered and fractured granodiorites and metamorphic complex of the Oligocene age. The dam site is located in fault-controlled river valley following the trend of a major northeast to southwest trending regional fault.

The central part of the embankment is founded on alluvial deposits (coarse sandy gravel) which fill the entire river valley and which are underlain by weathered granodiorite. The alluvial deposits vary in thickness between 10m and 30m and have a hydraulic conductivity of 10^{-4} to 10^{-5} m/s.

On the abutments the embankment is founded on colluvial materials, mostly silty clays. The colluvium covers the valley sides to a thickness of up to 20m and is derived from the weathering of granodiorites, with landslides in some areas. The hydraulic conductivity of the colluvium is 10^{-6} m/s.

Seismological conditions

Regional seismicity
Marmarik dam is located in a highly seismic area of Armenia. Some 13.7km north of the dam site runs the largest and most active Pambak – Sevan Fault. This is the main regional fault, 490km long, which in the past generated earthquakes of magnitudes up to 7.4. Also, very close to the site (5.2 km away), to the west of the dam, is the Garni fault, 198km long, which in the past generated earthquakes with magnitudes up to 7.0. The dam is directly situated on Marmarik fault, 30km long which joints the Garni fault. However, as no tectonic activity has been registered along the Marmarik fault in the Holocene, the fault is regarded to be seismically inactive.

Seismic design parameters
Seismic design parameters have been assessed based on the methodology given in Reference 1 as well as the site specific seismic hazard assessment.

The method in Reference 1gave the following design accelerations (return period of 475 year):
- Ground acceleration: a_{pk}= 0.144g
- Acceleration at the dam crest: a_{pk}= 0.555g

Site Specific Seismic Hazard Assessment was carried out using Deterministic Seismic Hazard Assessment (DSHA) and Probabilistic Seismic Hazard Assessment (PSHA). For site specific response the results of the microseismic survey were used.

The DSHA was used for assessing the maximum credible earthquake (MCE). Two past earthquakes were analysed; Mmax=7.5 along Pambak – Sevan fault and Mmax=7.1 along Garni fault. These earthquakes produced a peak horizontal acceleration of 0.44g and 0.82g at the dam's base and the crest respectively.

The PSHA produced the following peak horizontal acceleration:
a = 0.32g at the base and 0.6g at the crest (Return period of 100years)
a= 0.43g at the base and 0.81g at the crest (Return period of 250years- magnitude saturation occurs after 250years)

Based on the above analyses, the following design peak horizontal accelerations were recommended for checking stability of the dam:
- OBE= 0.32g at the base and 0.6g at the crest (Return period of 100years)
- MCE= 0.44g at the base and 0.82g at the crest (return period of large number of years)

Liquefaction analysis

Liquefaction assessment of the fill material was carried out based on the particle size distribution that did or did not liquefy during past earthquakes, Reference 2, and also the methodology given in Reference 3. According to the Japanese Seismic standard, the liquefaction potential is evaluated by calculating the liquefaction resistance factor, F_L. A soil layer having the liquefaction factor $F_L < 1.0$ is susceptible to liquefaction. An $F_L < 0.6$ was obtained for the fill in the top 10m of the dam (slipped mass) for an average seismic acceleration of 0.5g. Therefore some 60% of reduction in shear strength properties for that zone could be expected to occur during a strong earthquake.

STUDIES CARRIED OUT

Hydrological and Flood Routing

Two methods were used to analyse the flood inflows into the reservoir. The first, the SNIP method (Reference 4), is based on standard Russian techniques and is in general use in Armenia. The second is a statistical method that uses all annual maxima flow data recorded in the region and is derived from the approach developed during investigation of floods in the British Isles (Reference 5). The following results have been obtained for the 1:10,000 year peak flow:

- Regional Method: 147 m^3/s
- SNIP: 138 m^3/s

In addition to the 1:10,000 year flood, the flood that would result from breaching of the landslide dam due to reactivation of the Landslide N4 (see above) was also considered. The estimated peak inflow for this scenario was 1920m^3/s, with a volume of 2.4 million m^3.

The flood routing was carried out for the event of a 1:10,000 year flood as well as the event of a failure of a dam created by the N4 landslide. The flood routing was done for the existing condition (empty reservoir), for the design condition with the dam at its full height (FSL at 1911masl) and for an intermediate condition (partial impoundment).

Foundation seepage

Seepage through the dam foundation was analysed for two typical sections, namely for the deepest section with the piled cut-off and the abutment section with a grout curtain only. For a conservative assumption that the anti-seepage measures are ineffective, the total leakage through the dam foundation was assessed at about 100l/s.

<u>Stability analyses</u>

Existing condition of the embankment
Stability analysis of the upstream and downstream slope of the dam at its present condition was carried out using the parameters obtained from the investigations. The analyses demonstrated that the dam was stable with the reservoir empty. However, if impounded, the dam would be unsafe during rapid draw down (u/s slope) and steady seepage (d/s slope).

Stability after the remedial works to the embankment are implemented
Stability analyses was also carried out for three options for the remedial works. The embankment remedial works were developed so that minimum required factors of safety were satisfied for all loading conditions.

Stability of the landslides
The analyses carried out for the Landslide N2 showed that for sliding occurring along the predefined slip plane, factors of safety obtained were lower than unity even in the aseismic conditions. For possible new slip surfaces occurring within the landslide material, factors of safety obtained in aseismic conditions were higher than unity. However, in the case of an earthquake, slippage would occur. The slippage would most likely occur in a direction perpendicular to the ground contours, towards the Kiarkhana river and away from the dam and therefore would not directly affect the dam safety.

It was shown that the stability of the Landslide N3 is largely influenced by the reservoir water level. If the reservoir is filled the landslide would be re-triggered. The volume of the unstable mass was estimate to be 200,000m^3. It was shown that this mass would immediately raise the reservoir level by some 20cm. In addition a wave of 1.5m height would be induced. Such a wave, with its run up of some 2.8m would therefore need a minimum freeboard of 3m in order to prevent the dam from overtopping if the reservoir was full.

The volume of a potentially unstable mass for the Landslide N4 was assessed by stability calculations to be 2400m^3/m of the landslide length. That volume could create a 21m high natural dam which could impound a 2.4 million m^3 lake. As the river flow is some 3-4m^3/s the volume would be filled within a few days. In a major storm event this could take less than one day. The landslide ‘dam’ has been considered as an earth embankment and analysed for a dambreak. The analysis indicates a peak flood flow of 1920m^3/s and a flood volume of 2.4 10^6 m^3 (see above).

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

SUMMARY OF FINDINGS AND THE REMEDIAL WORKS

<u>Embankment</u>

Current crest elevation is approximately at 1900masl for a good part of the dam. Presently the upper part of the failed embankment material forms the top part of the core and the downstream shoulder. There is a very high perched water table within the dam body. Stability analyses demonstrate that the dam is stable in its present condition. However, if impounded to a level 3m below the current crest, the dam would be unsafe during rapid draw down (u/s slope) and steady state seepage (d/s slope). Furthermore, due to the high seismicity of the region, peak ground accelerations at the crest of about 0.6g could be generated. These accelerations are likely to cause liquefaction and strength reduction in the loose landslide material in the top part of the downstream slope and further contribute to the embankment's instability. It is therefore proposed to rehabilitate the embankment to improve its safety. Three options are developed as follows:

- Option 1 - Reinstate the dam to the full height with the crest at 1914 masl; Full storage level at 1911masl, total storage volume $36x10^6$ m^3
- Option 2 - Reinstate the dam to elevation of 1905masl; Full storage level at 1902masl, total storage volume $24x10^6$ m^3
- Option 3 - Reinstate the dam to elevation of 1889masl; Full storage level at 1886masl, total storage volume $10x10^6$ m^3

The earthworks proposed for the above options are shown on Figure 4.

<u>Foundation anti - seepage measures</u>

Foundation anti – seepage measures in the central part of the dam comprise a secant bored pile wall that was constructed through the granular alluvium into the bedrock. The field tests carried out in the deepest section indicated that the cut-off would reduce the overall foundation permeabily and the leakage would not exceed 100 l/s in the worst case scenario. Nevertheless, to reduce the uplift under the downstream shoulder, it is recommended that, for all three options of the embankment remedial works, 20 m deep, 200mm dia toe wells are installed along the downstream perimeter at 10m centres. The wells will comprise a perforated plastic tube wrapped in geotextile and placed inside a hole in a sand surround. Each well will discharge water into a collector trench which runs along the perimeter of the dam.

<u>Landslide hazard</u>

Four potentially hazardous landslides were identified in the vicinity of the dam. The landslides N1 poses a very low hazard to the dam. The landslide N2 is also likely to pose a low hazard, but because of its proximity to the dam it is recommended that monitoring instruments are installed in two monitoring profiles.

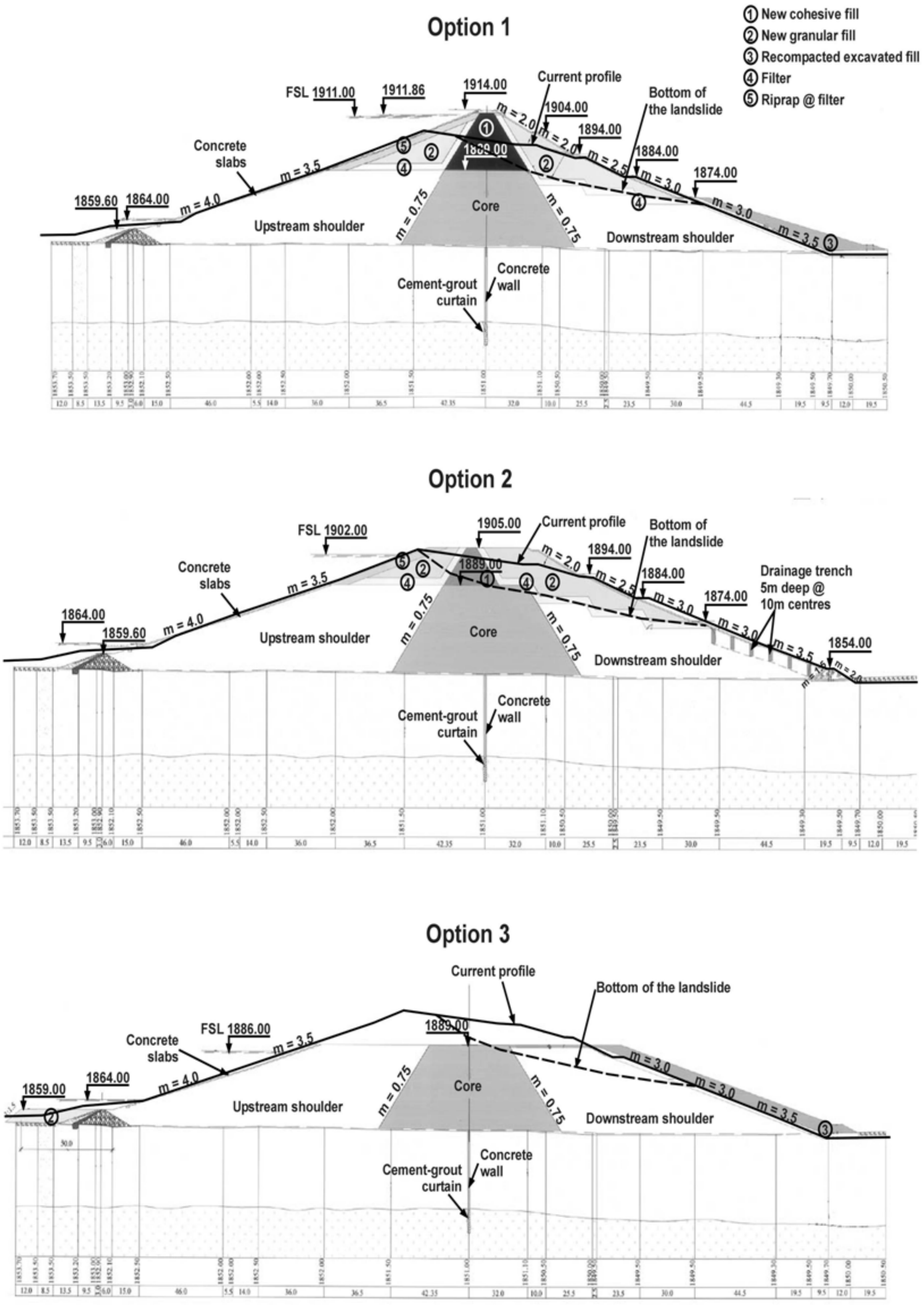

Figure 4. Options for dam rehabilitation

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

If the Landslide N3 slides into the reservoir it could create a 1.5m high wave and an allowance in the freeboard of 3m is made to accommodate the runup of such a wave. This landslide will also be monitored in two monitoring profiles.

If the landslide N4 collapses, it could block the Marmarik river and create a natural dam which if breached could create a 'dambreak' flood. It is recommended to install 1m high fuse gates (HydroPlus or similar) over the whole length of the spillway crest which could be activated should the landslide occur and the reservoir level needs to be lowered. Alternatively the spillway could remain conventional but the freeboard could be increased to 4m by provision of a 1m high concrete crest wall. It is also proposed to install monitoring instruments on the landslide and monitor the slope movements.

Diversion Tunnel

In its present state the river is diverted into a diversion tunnel designed for a temporary condition. The tunnel was inspected and investigated. The tunnel lining is of a satisfactory strength and the voids between the concrete and the rock are only of a limited extent. The tunnel is therefore considered to be stable in the short term. However the following remedial measures are recommended to enable its operation in the long term:

- Mass concrete plug upstream of the inlet pipes
- Consolidation grouting as a circumferential fan to a depth of 15m around tunnel over a 50m length downstream of the plug and backgrouting of tunnel lining in areas of voids.
- Replacement of tunnel invert downstream of plug
- Drainholes to be incorporated into invert to minimise hydrostatic loading.

Spillway

The existing spillway is in poor condition and requires substantial remedial works (Option 1). For Options 2 and 3 a new spillway is required at a lower level.

Outlet works

The outlet works require substantial refurbishment.

COST ESTIMATE FOR THE REHABILITATION OPTIONS

Costs for the three rehabilitation options are as follows:

- Option 1 - $10,5 M
- Option 2 - $7.5M
- Option 3 - $5.3M

A cost of decommissioning and breaching of the dam was estimated to be around $5M. It is likely that the client will go ahead with the rehabilitation Option 2.

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An update on perfect filters

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SYNOPSIS. Recent work shows that the probability of failure of dams resulting from internal erosion is often higher than that resulting from other threats. Filters to protect dams against erosion are therefore important. Most of our existing dams are not protected from internal erosion by filters. The 'perfect' filter equation links permeability of filters to the floc size of the soil they will retain. This permeability approach is useful in establishing the vulnerability or otherwise of existing dams to internal erosion because the permeability of fills can be determined by in-situ permeability measurements in boreholes. Floc sizes can also be simply determined using the principles of Stokes' Law in the laboratory. Some samples display murkiness which obscures the results. Examples of the use of perfect filters are given, including examples of retro-fitting of filters in dams in which they were not originally installed.

GUARDING AGAINST INTERNAL EROSION

It has long been suspected, and recent reservoir safety work for Defra (KBR & BRE, 2002) has demonstrated, that the probability of failure resulting from internal erosion of existing British dams is often greater than from the two other major threats, overtopping and earthquakes. Internal erosion is the process in which soil particles are eroded from the walls of cracks and discontinuities in earth dams by water flowing through them, often at high velocity because of the high hydraulic gradients through dams. Continued erosion leads to enlargement of the discontinuity, often as 'pipes' through the structure, which may erode back from the downstream end initiating a process of slope instability, crest lowering and overtopping that may ultimately cause failure. Internal erosion can be contained by 'filters', non-cohesive soils, usually medium silts to sands, which are sized to retain the soil particles eroded from the soil to be protected (the 'base soil') while allowing water to pass through. This prevents the development of erosion 'pipes' and thereby protects the structure.

How to design filters, particularly filters for existing dams, is likely to become an important dam safety issue in the coming years, and it is timely to update the information available.

CORES, COHESIVE SOILS, CRACKS AND EROSION

In dams, the element most vulnerable to erosion is the waterproofing element, the core, usually of clay. The protection of a dam core is probably the most critical function that a filter must perform. The consequences of failure can be severe damage and even catastrophe.

The vulnerability of cohesive clay cores to erosion arises because cohesive soils are able to sustain open cracks. Cracks or other leakage paths may form through cores during construction, during first filling, because of settlement, arching, hydraulic fracture or other causes. Filters should ensure that the presence of openings does not lead to loss of material from them.

VULNERABILITY OF EXISTING BRITISH DAMS TO INTERNAL EROSION

Most British dams are not equipped with filters and are therefore not equipped to resist internal erosion should it arise. Measures such as puddling, using very wet fill and wetting clay fills to make them softer, were all intended to make these vulnerable soils flexible and able to deform without cracking as the dam deformed in response to foundation settlement, water level variations, earthquakes and other loads.

To further reduce the vulnerability of narrow puddle clay cores in the older British dams, a zone of 'selected fill' was often placed on either side of the core. It was easier for early dam builders to use finer but non-cohesive soil as transition. It was easy to dig and compact and, following the exhaustive discussion at the inquest on the disaster at Dale Dyke dam, which failed in 1864 (Binnie, 1978), the desirability of well rammed fine-grained transition fills was understood and acted upon, more often than not. The 'selected fill' in the transitions may be of a grading that would provide filter protection to the core, as Vaughan (2000a) found at Ladybower, but it may often be cohesive and therefore able to sustain open cracks, making it too vulnerable to erosion and unable to act as a filter.

Fortunately, instances of internal erosion proceeding to serious damage are rare (Charles, 2001). Cohesive cores have considerable resistance to erosion unless they crack or develop concentrated leakage paths for other reasons. Thus satisfactory behaviour in operation may continue for ever. However, erosion may be occurring very slowly and not yet been revealed. Although the general experience is that dams grow safer with time, there is no justification for assuming that because they have not leaked or failed after a given time, they will never leak or fail.

In assessing the risks of internal erosion of dams, one of Vaughan's (2000a) conclusions was that 'usually there is considerable warning, allowing corrective action to be taken'. However, this is not always the case. Catastrophic wash out before remedial action can be taken is the big danger. The risks should be assessed by investigating the dam and, from a knowledge of its properties, evaluating the mechanisms by which internal erosion might develop and the speed at which it might occur. Appropriate defensive measures and surveillance routines can then be put in place.

FILTER DESIGN METHODS

Many methods have been put forward for filter design (e.g. CIRIA/CUR, 1991). Most apply to coarse materials, such as used in coastal protection, but the application of them results in the design of successively coarser layers, each of which is sized so that grains or particles from the adjoining layer will not pass through its neighbour. In an ideal filter, the pore spaces between particles should be just small enough to prevent the passage of the smallest of the protected grains. There is a wide range of sizes in the any granular material and a similar range of pore sizes. Consequently, most filters depend on some of the protected material moving into the filter to make it effective. This is called 'self-filtering'. Most filter rules for non-cohesive soils allow for this.

FILTER DESIGN FOR CLAY CORES

In dams, the element most vulnerable to erosion is the waterproofing element, the core, usually of clay. This poses special problems in filter design because using traditional rules to design filters to protect cohesive soils usually leads to filters of sizes which are themselves likely to be cohesive. These would be capable of keeping cracks open like the core they are intended to protect. Clearly, this offers no effective protection to vulnerable cohesive clay cores and it is generally accepted that different design principles should be applied.

These different principles address the issue of the actual size of the clay particles that filters must retain. Clay particles exist in nature in flocs, groups of individual particles. The floc size is related to the clay type and the pore water chemistry. In some circumstances, such as changes in pore water chemistry brought about by introducing water with differing chemistry, the flocs can be dispersed, partially to form smaller flocs, or completely to be dispersed into individual clay particles. In laboratory particle size distribution tests, the clay portion is artificially dispersed using a dispersant, and the sizes of individual clay particles are determined. Clays are defined as being 2 microns (0.002 mm) or smaller. Clay flocs are larger than this, often around 10 microns (0.01 mm), the medium silt size in the standard particle size distribution.

DISPERSION OF BASE SOILS

Chemical dispersion of the clay in dam cores has a history of causing erosion and washout in arid parts of Australia, Brazil, the U.S. and elsewhere. It is produced by a combination of the chemistry of the clay and the percolating water. Several chemical situations have been identified as causing it (Aitcheson & Wood, 1965; Emmerson, 1967; Stratton & Mitchell, 1976; Perry, 1987). Aicheson & Wood (1965) refer to a dam in Australia which washed out immediately when the water impounded was changed to relatively pure fresh water after several years of successful operation while holding water of a higher salt concentration. They also describe how arid conditions can lead to a ped structure with a much higher permeability than is expected in a clay fill. The large voids in such fill allow the dispersed particles which have been eroded to pass through them.

Dispersive soil is a special case. The authors know of no examples encountered in UK. However, as a precaution, all soils likely to be used in dams should be tested in prospective reservoir waters to demonstrate non-dispersion.

CRITICAL FILTER DESIGN

The most commonly used filter design method is the 'critical filter' approach developed by USDA Soil Conservation Service (1986) and Sherrard & Dunnigan (1989), also given in ICOLD (1994).

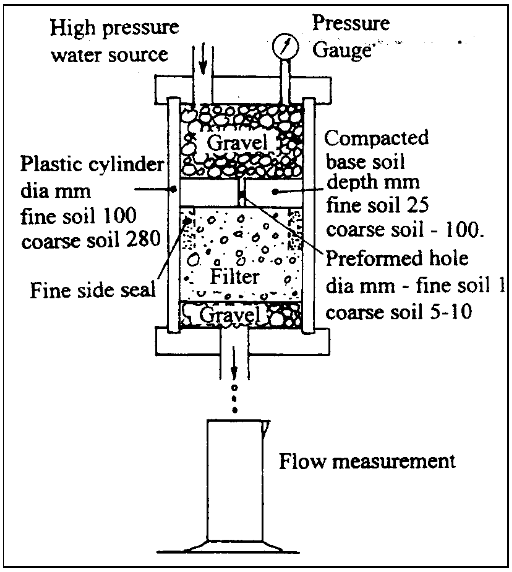

Figure 1 'No erosion' apparatus for the critical filter test

The method was based on an empirical laboratory 'no erosion' test, using the apparatus shown in Figure 1. Samples of base soil and prospective filter were tested by passing water under pressure through a small diameter hole in the base soil into the filter. If water discharged from the filter is clear, it is judged adequate; if water discharged from the filter is not clear, the filter is inadequate. From the results of many tests, the filter gradings that would protect the several groups of core materials were recommended. The groups of core materials are defined using the conventional (i.e. the dispersed, de-flocculated) particle size distribution.

'PERFECT' FILTER DESIGN

The alternative design method for filters for clay cores is the 'perfect filter' method. It was devised after sinkholes developed at Balderhead dam on first filling in 1967, as shown on Figure 2 (Vaughan et al 1970; Vaughan & Soares, 1982; Vaughan, 2000b). Segregation was identified in the erosion debris from the clay core found in the damage zone. The sand found in the eroded crack was the remains of the core fill, as the particle size distribution diagram on Figure 2 shows. The sand had been retained by the filter (designed to methods that precede both perfect and critical methods) but finer silt and clay-sized materials had passed through the filter because it was too coarse.

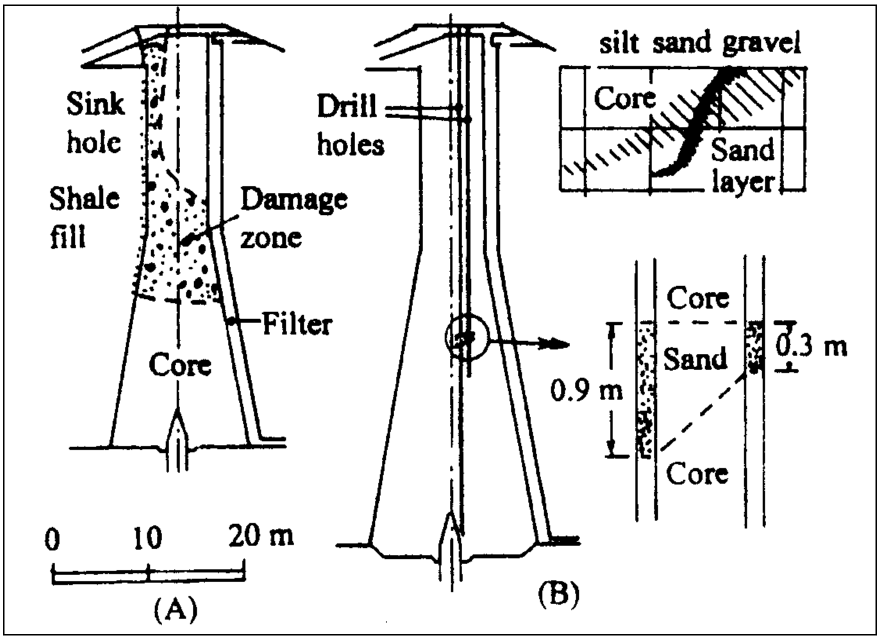

Figure 2 Balderhead dam, showing A - Damage zone where sink hole formed and B – Erosion hole filled with water washed sand

The perfect filter is required to retain the finest material which might be eroded from the walls of a crack in the core. This was taken to be the finest material obtained by mechanical dispersion of the clay in the appropriate water. This was usually clay flocs of around 10 μm (0.01 mm) particle size. Since this was a lower bound approach, no safety factor was required.

The design of a perfect filter involves two steps: first, the determination of the size of particle which must be retained and, second, the filter grading which is required to retain it. The filter grading required in design rules for non-cohesive soils is based on the finer sizes present, usually the 15% size. When the filter design was evolved it was found that the size of particle retained correlated well with filter permeability. The permeability of a filter is determined by the size of the continuous pores through it. Moreover the permeability is likely to vary with particle shape and it will vary with density of packing. While for uniform soils the permeability correlates with such an approach quite well, for well-graded soils the permeability depends on finer sizes and cannot be correlated with a particular percentage size.

The size of particle retained by a given filter was found experimentally by preparing different sizes of particle and passing them in dilute suspension through the filter (Vaughan and Soares, 1982). Either the sediment passed through the filter immediately or it sealed the surface, causing the flow rate to decrease rapidly. There was a small zone where the sediment clogged the surface more slowly. This was counted as retention. The test was more difficult to interpret when it was performed at a larger scale on filters containing gravel-sized particles. The results are summarised on Figure 3:

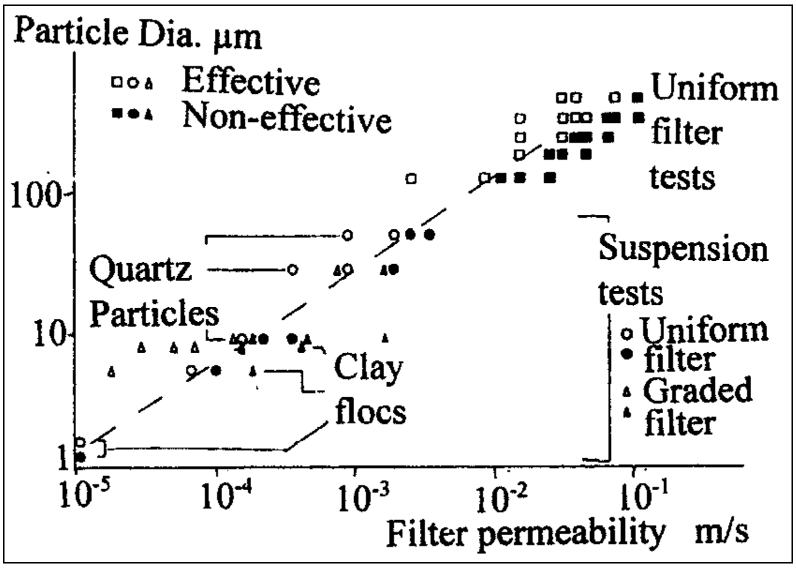

Figure 3 Summary of 'perfect' filter tests determining filter permeability required to retain base soils of various particle (and floc) sizes

Vaughan and Soares (1982) found that the relationship between filter permeability and the size of particles retained could be expressed as:

$$\delta_R = 1.49 * 10^3 (k)^{0.658}$$

where: δ_R = size of smallest particle retained in microns (10^{-6} mm)
k = permeability of filter (m/s)

The application of these findings to the erosion at Balderhead is illustrated on Figure 4. The grading of the core is shown, as is the grading of the portion of the core material retained by the 'actual' filter. This is the sand shown on Figure 2 above. The D_{15} range of the 'critical' filter that would have been provided to protect the core is also shown, as is the grading of the 'perfect filter'. It can be seen that the critical filter would have been too coarse to prevent the erosion that occurred through the cracks. Note also the modified core grading showing how it curtails at the minimum floc size, about 7 microns (0.007 mm), medium silt size.

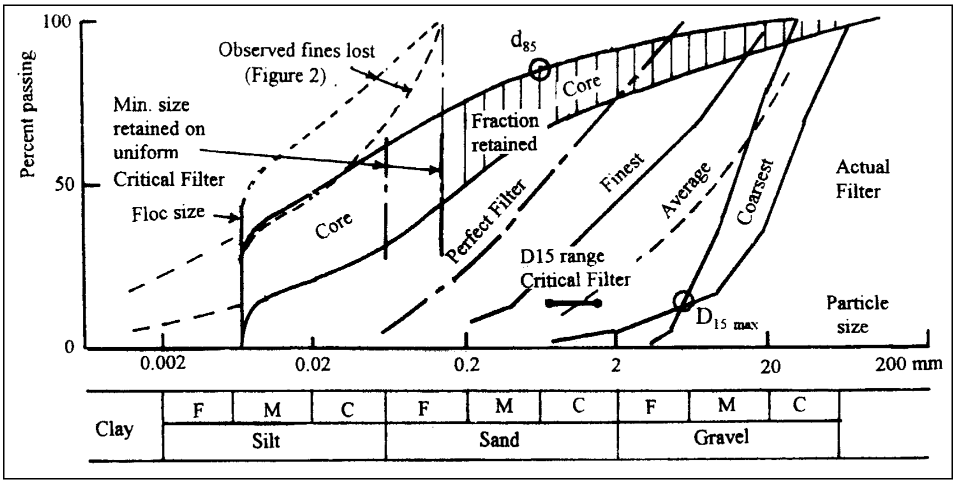

Figure 4 Filter base soil combination at Balderhead dam showing perfect filter, critical filter and observed segregation

COMPARISON OF PERFECT FILTERS WITH CRITICAL FILTERS

It is of interest to compare critical filters with perfect filters. This has been done by Vaughan (2000b) and the results are summarised on Table A below. The results are for filters of the appropriate critical filter base soil groups. No critical filter method 'no erosion' tests have been performed. The comparison has been made in terms of the minimum size of particle retained. This has been deduced from Sherrard & Dunnigan (1989) for the critical filters by first estimating the permeability of the critical filter from the relationship between permeability and the D_{15} size (Vaughan, 2000b).

The size of particle retained is then deduced from the perfect filter relationship.

Table A: Comparison of perfect and critical filters

Dam	Perfect Filter Design		Critical filter details deduced from Sherrard & Dunnigan (1989) Filter Provided			
	Floc Size δ_R (μm)	Permea-bility k (10^{-5} m/s)	Core Soil Group	D_{15} of filter (μm)	Permeability (10^{-5} m/s)	Size re-tained δ_R (μm)
Ardingly, UK	10	22	2	700-1500	319 - 1228	34-82
Carsington, UK	8	16	1	180	29	7
Cow Green, UK	6	10	2	700-1500	319-1228	34-82
Dhypotamus, Cyprus	6	10	2	700-1500	319-1228	34-82
Empingham, UK	10	22	1	90	9	3
Evinos, Greece	11	26	2	700-1500	319-1228	34-82
Kalavasos, Cyprus	5	8	2	700-1500	319-1228	34-82
Monasavu, Fiji	20	13	1	70	5	2
Balderhead, UK	7	13	2	700-1500	319-1500	34-82

The Critical Filters are more conservative than Perfect Filters for Group 1 cores (plastic clays) (e.g. 3 microns against 10 microns actual floc size at Empingham) and significantly less conservative for Group 2 cores (well graded sandy clays) (e.g. 34-82 microns against 7 microns at Balderhead). This is despite Group 2 cores giving poorer field performance. For the Group 1 cores the critical filters are more conservative than the perfect filters, despite the latter being able to arrest the smallest particle which may develop during erosion.

DETERMINATION OF FLOC SIZE

To use the perfect filter design method, the floc size of the core soil must be known. It is commonly determined using standard particle size analysis techniques (e.g. hydrometer) on samples slaked in reservoir water only, NOT subjected to the usual chemical dispersion process. Figure 2 above shows the results for the Balderhead core material. Often samples with and without dispersion, and sometimes without dispersion but in distilled, not reservoir, water, are also tested; these are the so-called 'double' and 'triple', respectively, dispersion tests. While the minimum floc size often shows up well in these tests, it is not always clear.

A simpler test (Head, 1992), which normally shows the floc size clearly, is based on Stokes Law, which relates the size of bodies falling through a liquid to their size. In our case, the smallest flocs sink slowest and can be seen as a falling front above which is clear water. The rate of fall of the front can then be used to determine the size of the smallest flocs present by using the version of the Stokes Law formula below:

$$D = 0.005\,531\ \{(\eta\ H) / (t\ (\rho_S - 1))\}^{0.5}$$

where: D = minimum floc size (mm)
H = distance floc front falls (mm) in time t (mins)
t = time (mins) to fall H (mm)
ρ_S = mass density of soil particles, should be measured, but is commonly in the range 2.6-2.7

The dynamic viscosity of water, η, varies with temperature, as follows:

Temperature (˚ C)	Dynamic Viscosity, η (mPa-s)
10	1.3037
15	1.1369
20	1.0019
25	0.8909

The rate of fall of flocs of the sizes normally encountered is quite rapid and Stokes law tests can be done quickly. The table below shows the time that flocs of various sizes take to fall 300 mm and gives information on the floc sizes and the filters required to retain them:

Mins to drop 300 mm	Terminal velocity mm/s	Floc size microns	Floc texture	Perme-ability perfect filter m/s	D_{15} uniform perfect filter mm	Texture D_{15} perfect filter
5	1.00	32.9	Coarse silt	3.04E-03	0.681	Coarse sand
15	0.33	19.0	Medium silt	1.32E-03	0.424	Medium sand
45	0.11	11.0	Medium silt	5.73E-04	0.265	Medium sand
90	0.0556	7.8	Medium silt	3.38E-04	0.196	Fine sand
180	0.0278	5.5	Fine silt	2.00E-04	0.146	Fine sand
360	0.0139	3.9	Fine silt	1.18E-04	0.108	Fine sand
1080	0.0046	2.2	Clay (defloc-culated)	5.12E-05	0.067	Fine sand

THE 'MURKINESS' PROBLEM

Sometimes in the Stokes Law test the falling front is not visible. The sediment can be seen to arrive at the base of the measuring cylinder, but the water above remains 'murky' and opaque, so that the falling front cannot be seen. The source of the murk is not known. It usually persists for extremely long periods, longer than even the smallest clay particles would take to settle, and it seems unlikely it comprises dispersed clay flocs. Its source may be the same as the source of 'colour' in treated water, although it is more severe, making the water opaque, not transparent as 'coloured' waters are. It seems prevalent in alluvial soils, perhaps because organic materials are present. It complicates a simple and useful test, easily done in the field, and research into its source and how to overcome the murkiness without affecting the validity of results would be valuable.

PERMEABILITY AND GRADING OF FILTERS

The use of a relationship that relates retained floc size to the permeability of the filter reflects the fact that permeability is related to pore sizes. However, measuring the permeability of a potential filter is less convenient than measuring its grading and the expression below (Vaughan 2000b), which is for uniform filters, is useful to give an early indication of the grading of potentially suitable filters:

$$k = 3 * 10^{-8} (D_{15})^{1.767}$$

where: D_{15} = D_{15} size of uniform filter (in µm, microns)
k = permeability of filter (in m/s)

Note that the actual filter, if not uniform (i.e. $D_{60}/D_{10} > 1$), will have a different permeability, and therefore a different filtering capability, and the permeability of candidate filters should be measured before they are used.

The permeabilities of filters retaining clays flocs are low and their drainage capacity is therefore limited. If filters are protecting fills that include permeable layers that may allow substantial quantities of seepage to pass, it may be necessary to provide a coarser drainage filter downstream of them to allow the seepage to escape freely. To pass the quantity, the hydraulic gradient across the low permeability, low capacity filter is high, and the gradient along the high permeability, high capacity drainage filter is low.

FILTER PROPERTIES

Filters should be non-cohesive, at least as placed. The 'sand-castle test' described by Vaughan & Soares (1982) is a convenient and quick means of proving non-cohesiveness at source. Granular soils may bond with age and develop cohesion, although so far as the authors know, no problems have been reported from this cause.

Filters must be internally stable and self-healing. Kenney & Lau (1985) and Lafleur et al (1989) give methods to check the internal stability of non-uniform filters.

A further check on the suitability of filters can be made by passing 'muddy' water made from (reservoir) water containing the base soil through a layer of the filter in a permeameter. Adequate filters retain the 'mud' and clear water passes through. Inadequate filters allow the muddy water to pass.

IN-SITU PERMEABILITY AS A GUIDE TO THE VULNERABILITY OF EXISTING DAMS TO INTERNAL EROSION

The filtering capacity of non-cohesive shoulder fills can be assessed from in-situ permeability measurements. For example, Vaughan (2000a) found non-cohesive silty sandy gravel transition fill at Ladybower to have a maximum permeability of $4*10^{-6}$ m/s. This provides perfect filter protection to the adjoining clay core in which the minimum floc size is about 10 microns. As the fill tested may not be uniform, use of a lower bound to the permeabilities measured may be appropriate. The filter relationship between the transition and the general shoulder fill should also be checked as transition fills may erode into coarse shoulder fills.

The perfect filter equation makes the connection between floc size retained and filter permeability, as follows:

$$\delta_R = 1.49 * 10^3 (k)^{0.658}$$

where: δ_R = size of smallest particle retained in microns (10^{-6} mm)
k = permeability of filter (m/s)

The equation was derived for non-cohesive filters with permeabilities ranging upwards from $1x10^{-5}$ m/s. Use of the equation to determine the floc size of soils that would be retained by soils with in-situ permeability less than $1x10^{-5}$ m/s should be cautious. If the soils are cohesive, improbable results emerge (Tedd et al, 1988). In practice, this means that in low permeability fills, the cohesiveness of the soil should be checked, and the floc size of cohesive materials should be determined in the laboratory.

Note that samples taken from boreholes in fills with substantial proportions of granular materials are likely to have lost fines and not be properly representative of the in-situ fill, consequently laboratory permeability tests do not give usable results. In-situ tests are needed, usually from piezometers installed in boreholes. These may also serve for measuring pore pressure in the investigation of old dams

However, as Charles et al (1996) point out, the sand in the sand-pockets in piezometers installed in fill will usually have a permeability up to about $2*10^{-5}$ m/s. If this is less than the fill in which the piezometer is sited, it will appear that the fill will retain smaller flocs than it is capable of retaining, an unsafe situation. A progressive approach to determining the permeability, and hence the filtering capacity, of fills that may be required to protect against internal erosion is therefore recommended, commencing with in-situ permeability tests in boreholes.

DAMS WITH PERFECT FILTERS

There is a growing body of dams with perfect filters, as listed on Table B below:

Table B: Dams with perfect filters

Dam	Perfect Filter Design		Filter Provided			
	Floc Size (μm)	Perm-eability (10^{-5} m/s)	Filter Soil Type*	Perm-eability (10^{-5} m/s)	Size re-tained δ_R (μm)	D_{15} of filter (μm)
Ardingly, UK	10	22	ns	9	3	230
Carsington, UK	8	16	psg	1 to 10	1 to 3	80-170
Cow Green, UK	6	10	ns	2	1	110
Dhypotamus, Cyprus	6	10	sng	1	1	1000
Empingham, UK	10	22	ng	8	3	100
Evinos, Greece	11	26	sng	10	3	220
Kalavasos, Cyprus	5	8	sng	4	2	600
Monasavu, Fiji	20	13	cr	4	2	210
Balderhead, UK	7	13				
Melton Mowbray, UK	4	12	ns	10	3.5	150
Audenshaw, UK	6	23	ns	10	3.5	

* ns = natural sand psg = processed sand and gravel ng = natural gravel cr = crushed rock sng = natural sand and gravel screened to remove coarse sizes

It has always proved possible to find or make perfect filter gradings for the cases listed above, although this was sometimes difficult. Dounias et al (2000) describe how river gravels were used as the core filter at Evinos Dam. Hughes et al (2001) and Bridle (2003) describe the filter investigations at Audenshaw and Melton Mowbray respectively.

It must be emphasised that the perfect filter is only required to protect against erosion by continuous reservoir flow through cracks or other flow paths which are in cohesive soils, and which can sustain such an opening without sealing by collapse. This is typically a core, but where the foundation is of erodible clay, a short length of perfect filter blanket is often added on the foundation downstream of the core, where significant hydraulic gradients exist. The principle of the Perfect Filter for cohesive soil is that erosion through concentrated reservoir flow is prevented. Intrinsically, no cause for such flow is presumed. A relatively thin layer of filter has been considered acceptable.

It is inevitable that the filter provided is less permeable than the Perfect Filter required. This gives a safety factor, although one is not required.

RETRO-FITTING FILTERS

In dams which are found to be unacceptably vulnerable to internal erosion, filters will be required. This presents some challenges. Although perfect filters will protect fills for which they are designed, fills in old dams may be variable. Also, if the filters are incomplete and do not cover the entire exposed fill, erosion may still occur. Protecting against foundation erosion is particularly difficult. Methods of retro-fitting filters to meet these challenges will have to be devised, probably derived from previous experiences, a few examples of which are described here.

At Lower Tamar, a filter layer was placed below a weighting berm on the downstream slope to collect and filter seepage passing through the core (Kennard, 1972). Care should be taken to make sure that arrangements such as this have a sufficient weight to secure against a concentrated leak (Vaughan, 2000a). Bailey (1986) describes the provision of a filter wall to prevent erosion through tension cracks near the top of the core and the installation of a geotextile filter behind a retaining wall at the toe of the downstream slope to filter seepage passing through Upper Litton dam. Talbot & Ralston (1985) give examples of retro-fitting of filters to deal with cracks and potential internal erosion in dams, including flood dams.

Jairaj & Wesley (1995) describe the construction of a filter wall drain using a bio-polymer slurry at Hays Creek dam. The wall drain was excavated using slurry support in the usual way, and the trench filled with filter sand placed by tremie pipe, displacing much of the slurry. Water and sodium hypochlorite was pumped through the sand/slurry in the trench to break down and remove the remaining slurry, leaving the sand as a filter at the required permeability in the trench.

Filter collars can be provided near the downstream ends of culverts and pipes through dams to limit risks of erosion along the interface between these structures and the dam fill. Talbot & Ralston (1985) advocate filter collars, and give information on suitable dimensions and positioning.

CONCLUSION

The perfect filter approach to providing filter protection against internal erosion in dams provides a rigorous means to design safe, effective filters. It can also be conveniently used to assess the vulnerability to internal erosion and the need for filters in existing dams. The aim of this paper is to make the perfect filter approach accessible to European, including British, dam engineers to assist them in keeping their dams safe from damage through internal erosion in the long term.

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Remedial drainage to Laggan and Blackwater gravity dams

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SYNOPSIS. Alcan's 48m high Laggan Dam and 26m high Blackwater Dam have both been reassessed for extreme floods and seismic loading, and stability at both was found to fall short of modern guidelines. In the case of Laggan Dam the critical load case was the PMF, which would overtop the substantial masonry walls of the spillway bridge. Blackwater Dam stability was found to be marginal under normal conditions and unsatisfactory under both extreme load cases because of its slender section and serious doubts that it could carry tensile stresses, particularly at the foundation contact. An assessment of alternatives found that remedial drainage provided the cheapest satisfactory solution at both dams. At Laggan Dam this involved drilling from the dam crest to intersect the gallery and from the gallery into the foundation. At Blackwater Dam, which has no gallery, the solution involves inclined holes from the downstream face to intersect the dam/foundation interface.

The paper sets out the studies, investigations and design of the remedial works, and implementation of the work at Laggan Dam. Implementation is still to take place at Blackwater.

ALUMINIUM IN THE SCOTTISH HIGHLANDS.
The British Aluminium Company was formed in 1894 with the aim of developing the new process of electrolytic reduction, which depends on the availability of cheaply produced electricity for the commercial production of aluminium. The company's first hydroelectric power station was built at Foyers on Loch Ness in 1896. It produced 3MW of power and was capable of satisfying one tenth of the world demand for aluminium, which at that time stood at two thousand tons per annum; this compares with over 18 million tons today.

The Company expanded their facilities in the Highlands with an aluminium smelter in Kinlochleven in 1907 and Lochaber Smelter in Fort William in 1929. Both of these are powered by their own hydroelectric schemes with Kinlochleven generating 20MW and the Lochaber Scheme 65MW. These schemes were each massive engineering undertakings in their day with a combined catchment area of 940sq km.

The Foyers Smelter was closed in 1967, although the hydropower scheme was taken over and redeveloped by Scottish Hydro Electric. In 1981 the British Aluminium Company was taken over by Alcan. The smelter at Kinlochleven was closed in 2000. However its hydropower scheme was retained and refurbished to produce supplementary power for the Lochaber Smelter. Lochaber smelter and hydropower scheme are still in operation. It is impressive that the original hydropower developments have stood the test of time, both being largely still in operation in their original form.

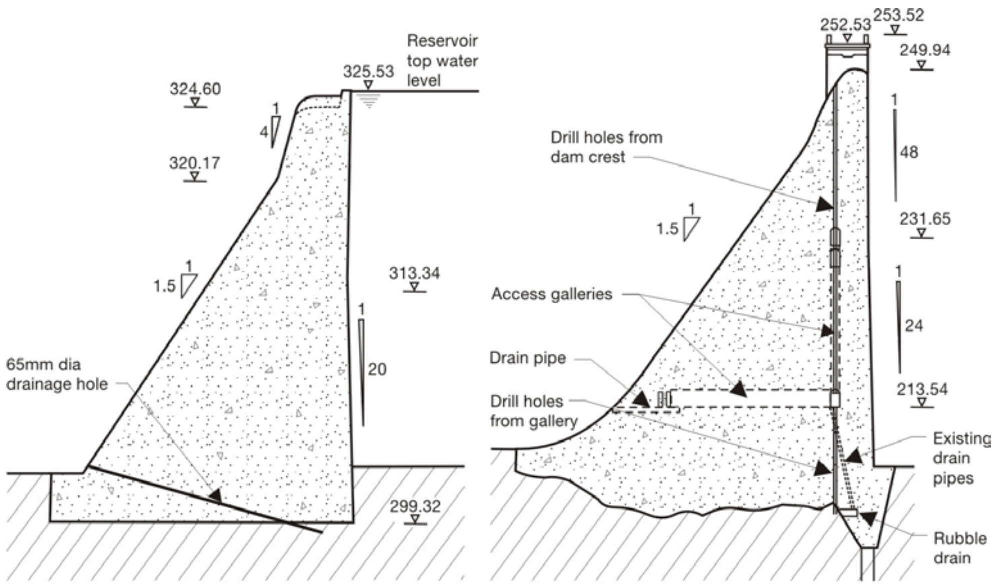

Figure 1 Sections of Blackwater and Laggan dams (not to scale)

BLACKWATER DAM

Background

Blackwater Dam impounds the main storage reservoir for the Kinlochleven Hydro Electric Scheme. The dam is situated some 70 miles north of Glasgow on the west side of Rannoch Moor. Access to the dam is via a 5 mile long rough track suitable only for small 4WD vehicles. The reservoir supplies water to the power station in Kinlochleven, 8km from the dam, via a covered free-flow channel and surface penstocks. Francis turbines and AC

generators were installed in the power station following the closure of the aluminium smelter in 2000, replacing the original 1908 DC Pelton units.
The 26 meters high concrete gravity dam was completed in 1909. It is 948 metres long and 503m of the crest, ie more than half the length of the dam, is overflow spillway at an elevation of 325m aOD. The dam section is shown on Figure 1. There is a central valve tower on the upstream face from which water is drawn-off through pipework in the base of the dam. The reservoir is 12 kilometres long and holds 111 million cubic metres at top Water Level. At the time of construction the reservoir was thought to be the largest in Europe. Reservoir levels often reach spill level during the winter months and are typically drawn down twelve to fifteen metres during the summer. The dam has been in continuous service since it's construction.

The dam is composed of a mass concrete matrix in which are embedded heavy granite displacers. Hearting concrete in the dam was a 1:5 mix with up to 50mm aggregate and a slightly richer 1:4 mix with 19mm aggregate used for the facings. Rock was quarried locally to the site but much of the sands and gravels were imported from a gravel bank in the tidal loch near Kinlochleven. The rock foundation was excellent with almost no fissures being found in the foundation area. The maximum depth of excavation to sound rock was only 4.6m. Cement mortar 50mm thick was laid on the rock foundation with a further 25mm thick layer placed prior to the placing of the concrete by derrick cranes. Large granite displacers weighing up to 10 tons were embedded in the concrete with many of them bridging the lift joints. Further details are given in the 1911 ICE construction paper [7].

The section of Blackwater dam is slender [Figure 1] and stability has always been recognized as marginal. The narrow section was commented on in the discussion following presentation of the construction paper in 1911 [7] in relation to other dams of the period. This appears, with hindsight, to be because uplift was not properly taken into account in the design. Moreover, at some time after this, the top water level was raised by about 0.9m by infilling the lower parts of the crest in the original stepped spillway.

The vertical cracks found in the dam shortly after completion were of great interest to the civil engineers of the time and may well have been influential in the inclusion of vertical contraction joints in subsequent dams. Interestingly a small water supply dam built shortly afterwards in the valley below has vertical joints. There were seven main cracks which opened up to 2.4mm wide. Water wept through the cracks and attempts to seal them using silicate solution and fine grout were largely unsuccessful. Subsequent treatment using peat introduced into the water on the upstream face of the dam were more successful. Practically no water was leaking through the dam by 1910, the cracks having apparently having been sealed either by sediments and peat in the water or by leached free lime from the cement. This built up as a hard white deposit on the downstream face dam and still builds up today, particularly below any small leaks and weeps.

The first inspections under the 1930 Act in 1933 and 1943 reported that the leakage through the contraction cracks was small, but stability was investigated in 1935, which included taking cores from the dam.

In 1963 the deterioration of the upstream side of the horizontal construction joints was considered to be a problem. Significant effort was put into sealing the upstream face of these joints and the vertical cracks by breaking out unsound mortar and concrete and refilling with mortar overlain with bitumen reinforced with a fiberglass mat. This significantly reduced the amount of leakage through the dam. Periodic maintenance of the upstream face sealing has been carried out since 1963.

In 1979 the stability of the dam was reassessed using information gathered from new core holes and piezometers, following which the frequency of inspections increased to five and then ten years. This work was recorded in an 1982 ICOLD paper [9].

Leakage readings are taken every two months from six fixed points downstream of the dam and also from the base of the draw off tower.
Foundation piezometers are read twice a year at high and low reservoir levels. In recent years some have tended to show a rising trend, and their condition has deteriorated, making readings less reliable. Tower plumb bobs are monitored annually. Movement stations are surveyed every four years.

Recent studies and investigations

Blackwater Reservoir lies above the town of Kinlochleven and is Category A to the standards in Floods and Reservoir Safety [2]. Following the 1993 periodic inspection, a hydrological assessment was carried out in 1995 [10] based on the Flood Studies Report [1]. This calculated a peak PMF outflow of 1043m^3/s, compared with the original design flood of about 377m^3/s. Calculated maximum flow depth over the 503m long spillway in its present configuration was 1.09m. This is within the height of the wave wall, although deficient in the recommended wave freeboard. A subsequent check has shown that the water level produced by a 1:10,000 year flood based on the Flood Estimation Handbook [5] is less than this.

A Flood Stability Assessment [11] of the dam was carried out in 1996 for the revised PMF design flood level. No provision for foundation or dam body drainage was included in the design, but uplift pressures are measured at the six piezometers installed in the 1970s. The gravity method stability analysis concluded that Blackwater Dam relies on tensile strength on the lift joints to provide the recommended factors of safety under both normal and PMF conditions. If tensile strength is ignored and cracks exist on lift joints in the upstream face, the factor of safety against overturning is about 1.15, as against a recommended value of 1.5 in the Engineering Guide to the Safety of Concrete and Masonry Dam Structures in the UK [4], and could drop to as low as 1.03 under PMF conditions. Both were considered unacceptably low. The main recommendations of the report were:

- that an investigation of the vertical tensile strength of the lift joints in the dam be carried out; and
- that the performance of the dam under seismic loading be included in the review.

These recommendations were reviewed and endorsed in 1997 in a Section 10 Inspection report, which also recommended that any unacceptable deficiency in stability should be remedied.

Historical research revealed that formation of the lift joints is described in the construction paper [7] as follows:

"Before concreting a new layer the surface of the old one below it was thoroughly cleansed, roughened, and covered with 1 inch of cement mortar, upon which, while still fresh and soft, the new concrete was deposited. This was done to ensure a sound and watertight seam between layers.

In this connection mention may be made of a particular characteristic of the rotary cement. A fine, brown scum formed on the surface of the concrete, and, if left, set hard with a skin like glass, this making it difficult to obtain a sound joint with the next layer. It became necessary, therefore, to destroy this skin by brushing the surface when partly set and thereby leaving it rough. Care was taken to have this done always."

The hearting of the dam was constructed incorporating large granite displacer blocks, weighing up to 10 tons. The extent of these is clear from the construction photographs [Figure 2]. Particular care was taken to include these across lift joints, and they have a significant influence on the tensile strength at the dam body. Inspection of the joints at the upstream and downstream faces, however, suggested that the concrete material in the joints is significantly weaker than the rest of the dam concrete [Figure 3].

Records of borehole investigations of the dam in 1935 and 1978 were examined, as were details of the 1978 investigation given in the 1979 Stability Report [8] and ICOLD paper [9]. However neither of these considered the tensile strength of the lift joints.

Figure 2 Displacer blocks on lift joints at Blackwater Dam

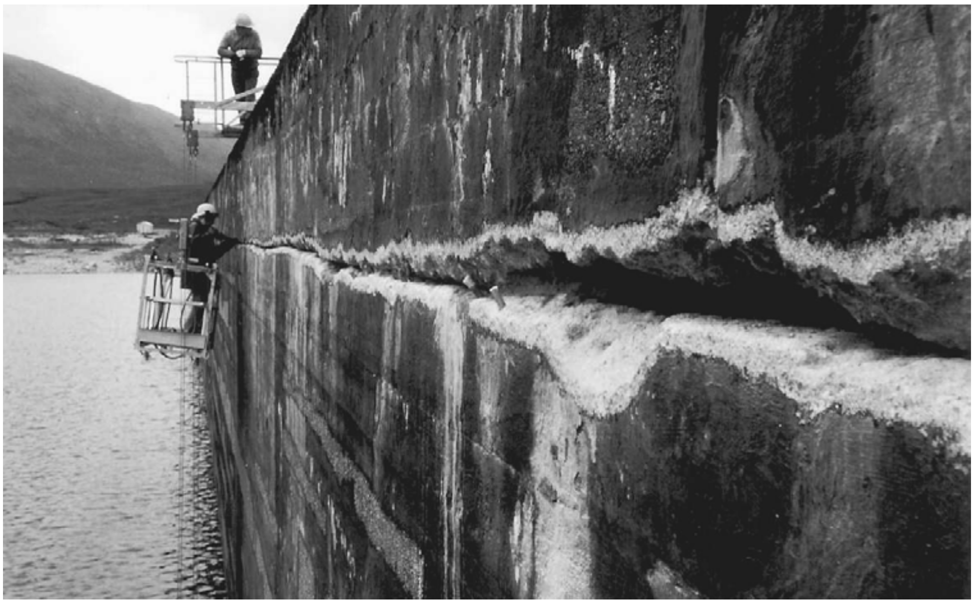

Figure 3 Raked-out joint in upstream face at Blackwater Dam

Alcan awarded a site investigation contract to Exploration Associates in July 1998 for drilling horizontal cores at joints and vertical cores through the dam and subsequent laboratory testing. Work commenced on site in August

1998 and was completed in October 1998. The factual report covering the drilling and testing was submitted in December 1998 [12]. Few of the lift joints were recovered intact and significant areas of honeycombing and poor joint bonding were found in the cores.

A further Stability Review [13] taking the collected data into account, concluded that, while concrete tensile strength could not be expected at lift joints, the rock displacers provided an acceptable degree of tensile strength across lift joints in the dam body. However unacceptable cracking could still occur at the dam foundation/rock interface. The cheapest option to remedy this was found to involve drilling inclined holes from the downstream face to intersect the foundation contact and so relieve uplift pressures.

An initial pseudostatic seismic analysis based on the UK Seismic Guide [3] had indicated tensile stresses at the dam heel likely to produce excessive cracking, leaving the post-seismic cracked section only marginally stable. However a lower 1:10,000 peak horizontal ground acceleration of 0.2g was derived from the Application Note to the Seismic Guide [6] and confirmed against site-specific seismic accelerations calculated for Scottish and Southern Energy dams in the area [15]. A more detailed 2-dimensional dynamic analysis using EAGD-SLIDE [14] subsequently demonstrated that, with the drainage works required for flood stability in place, direct seismic failure is most unlikely, although cracking may still occur, and that post-seismic stability is sufficient that any damage caused can be assessed and, if necessary, can safely be dealt with after the event.

Implementation

Design drawings and Tender documents for the recommended remedial drainage works at Blackwater Dam were prepared in 2003 and the works are planned for construction in 2004 and 2005.

LAGGAN DAM

Background

Laggan Dam impounds 40 million cubic meters for the Lochaber Hydro Electric Scheme. The dam is situated some 100 miles north of Glasgow at an elevation of 250m in Glen Spean, west of Spean Bridge. Access to the dam is via a short road immediately off the A86 trunk road. Now owned by British Alcan the reservoir supplies water via Loch Treig to the power station at the aluminium smelter in Fort William.

Completed in 1934, the dam is a conventional mass-concrete gravity dam some 48m high between general foundation and spillway crest level. It is slightly curved upstream in plan, but was designed as a purely gravity structure. The whole crest of the dam is a free-overflow spillway except for a central block housing siphons and gate control equipment. The spillway crest is broken into 29 bays by piers, supporting bridge arches. The upstream and downstream faces of the bridge consist of massive masonry wave walls. Six siphon pipes embedded in the dam concrete supplement the crest spillway discharge. The siphons make and break automatically at pre-set reservoir levels using a system of air valves.

The dam was built in 7 blocks with both copper strip and hot poured asphalt water stops in the joints. The bulk of the dam body is constructed of mass concrete with a nominal $15N/mm^2$ characteristic design strength. However recent testing of cores from the dam has shown average strengths of $28.5N/mm^2$, and a peak strength of $36N/mm^2$. Higher strength concrete was used in the external faces. The dam concrete contains about 5% of granite displacers. The dam foundation rock is fresh or slightly weathered granite. After the dam was completed, gunite was applied to the upstream face to reduce leakage through any contraction cracks that appeared. The gunite was applied in two layers onto a wire mesh fixed to the face of the concrete.

The dam is generally in excellent condition with practically no leakage or signs of movement. The original gunite facing has not been a success and can be seen lifting away from the face of the dam at low reservoir levels. Until the remedial drainage works, completed in 2001, no works of any significance had been required on the dam since construction.

The total leakage from the foundation drains into the gallery is monitored. Leakages are small and increase with increasing reservoir level. The foundation drainage system is tested annually by forcing water by means of a packer into each drain pipe in turn and noting connections to adjacent pipes. This monitors the integrity and porosity of the rubble drain system.

<u>Recent Studies and Investigations</u>

Laggan Dam is situated upstream of Roy Bridge and Spean Bridge and is considered as Category A by the standards of Floods and Reservoir Safety [2]. Following the 1993 periodic inspection, a Flood Hydrology report [10] was prepared in 1995 using the Flood Studies Report [1]. This calculated the routed PMF outflow through the siphons and over the free spillway on the dam crest as $2073m^3/s$ with a corresponding flood rise of at least 3.07m. This compares with the original design flood of $396m^3/s$ and corresponding flood rise of 0.87m above spillway crest given in the 1937 construction paper [16]. This assumes that the spillway discharge is unrestricted. Taking the effect of the arches into account, the peak PMF outflow is restricted to $1620\ m^3/s$ but the peak flood level rises to 4.09m above spillway crest level, or 0.54m above the top of the masonry wave wall on the dam crest bridge. A check has shown that the water level produced by a 1:10,000 year flood based on the Flood Estimation Handbook [5] is less than this.

Halcrow carried out a Flood Stability Assessment of the dam [17] in 1996 for the revised flood water level. The highest sections of Laggan Dam contain a foundation gallery and copper drain pipes are provided between the gallery and a rubble drain on the foundation at the back of the cut-off trench, but the dam body above the gallery is undrained, and drainage of the section below the gallery into the copper pipe drains is doubtful. The construction paper [16] reports that only 50% uplift pressure was allowed for in the original design of the dam. This was justified by the inclusion of the gunite layer.

The stability analysis concluded that the dam body stability relies on tensile strength at the lift joints to provide the recommended factors of safety under both normal and PMF conditions. Under normal conditions the dam structure requires a tensile strength of 0.3MPa at undrained lift joints to meet recommended factors of safety against tensile failure but would remain stable should cracking occur. Under the PMF the vertical strength at the lift joints to provide the recommended factor of safety of 2 against tensile cracking would need to be 0.7MPa. Should the upstream face fail in tension and a crack develop, the dam would become unstable against overturning under PMF conditions, although this assessment ignores the moderate arch shape of the dam. The main recommendations of the report were:

- that an investigation of the dam concrete and in particular of the vertical tensile strength of the lift joints in the dam be carried out; and
- that the performance of the dam under seismic loading also be reviewed.

These recommendations were endorsed in a Section 10 Reservoirs Act report in 1997, which also recommended that appropriate measures be taken to ensure adequate stability of the dam.

Alcan awarded a site investigation contract to Exploration Associates in July 1998 for drilling vertical cores from the dam and subsequent laboratory testing. The report was submitted in December 1998 [18]. Over 70% of the lift joints in the cores were found to be intact, and some of those broken were fresh, having been broken in the drilling process. Average tensile strength at intact joints tested was 0.7MPa.

A further Stability Review [19] was undertaken in 1999 using the data on the dam concrete from the drilling investigation. Higher than previously expected concrete density of 2450kg/m^3 improved the results, but not to an acceptable extent. Stability at PMF was acceptable in the upper third of the dam, at the drained foundation and where the section was drained by the gallery, but was unacceptable in the lower two thirds of the dam where the section was undrained. Pseudostatic seismic analysis showed that the seismic load case was less critical than the PMF, and that, while the dam section could be expected to crack under extreme seismic loads, post-seismic stability was acceptable, particularly when subjectively taking into account the curvature of the dam in plan.

Consideration of remedial options concentrated on providing adequate drainage to the dam section, and it was concluded that this was best done by drilling from the dam crest to intersect the gallery, and from the gallery into the foundation. This included both vertical and inclined holes. The layout of these was complicated by the curvature of the dam in plan, the need to drill holes down through bridge piers and across the spillway openings and the presence of siphon outlets, a bottom outlet and associated equipment embedded in the dam body. A new survey and 3-d AutoCAD model of the dam were prepared by RBJ Surveys Ltd of Glasgow at a cost of £15,000 to confirm the setting out of the holes.

Implementation

The drainage design required drilling: -

- 980m of minimum 65mm diameter vertical and inclined open drain holes from the dam crest up to 40m deep to intersect the gallery.
- 350m of minimum 50mm diameter vertical and inclined open drain holes from within the gallery to intersect the rock foundation.

In total there are 66 holes spaced at nominal 3m centres, but adjusted where necessary for reasons of access and to avoid built in parts such as the siphon pipes. All holes drain into the 0.9m wide by 1.8m high gallery. All of the holes were technically challenging due to the accuracy required to intersect the gallery or the very cramped conditions within the gallery.

The contract was put out to tender to six contractors and four competent tenders were submitted. The successful contractor was Ritchies Ltd of the Edmund Nuttall Group who successfully completed the works within the 16 week contract period in summer 2002. The contract value was £170,000.

The main Health and Safety hazard identified in the risk analysis was working in the gallery where access was via two small vertical shafts. Most importantly a rescue procedure was developed with the aid of the Mines Rescue Service. This was tried out in a full mock rescue. Rope access personnel were trained and retained on site in case a rescue was required. Other systems developed included those for communication, noise and dust control and movement of materials and drilling equipment.

Environmental control adjacent to a major watercourse was of prime importance. Great care was taken to reduce the risk of spills of oils and fuels and emergency procedures were developed. On the dam crest drill stems were shrouded and vented to a dust collection system with the cement and rock dust being disposed of off site. Dust suppression in the gallery was achieved using water mist injection into the compressed air.

All holes were drilled using down the hole percussive hammer techniques. A crawler mounted rig was used for vertical and inclined holes on the dam crest roadway. The contractor elected to drill at 95mm diameter in order to reduce drill string deflection with the top 3metres of each hole being cored and cased to aid directional control through the rubble filled piers and the "air gap" between the underside of the roadway bridge and the curved spillway below. The drilling system drilled well through light reinforcement and rock displacers in the concrete. At the final count only three out of 30 holes drilled from the dam crest just missed the gallery, this was likely caused by the drill head "glancing off" embedded steelwork. However all three were later located using a vibrating tool and intersected from the gallery. A special drill rig was fabricated to work in the confines of the gallery and used successfully to drill 50mm diameter holes inclined and vertically upwards and downwards. However progress was slow and a rock drill with an air leg was used to increase production rates on some of the shorter inclined holes.

Figure 4 Drilling equipment at Laggan Dam crest and gallery

CONCLUSIONS

Both Blackwater and Laggan Dams are historic concrete gravity dams of their period, constructed before modern design criteria were established. Both have served, and continue to serve, the original function for which they were designed. Despite the differences in their designs, remedial drainage to relieve uplift pressures has been adopted as the most appropriate and economic measure to improve stability at both dams to upgrade them to meet modern standards.

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Papan dam studies and remedies.

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SYNOPSIS. As part of a major irrigation refurbishment programme in Kyrgystan, seven major dams built in the Soviet era have been examined and rehabilitation measures designed and costed. Perhaps the most complex of these was Papan Dam near the city of Osh on the Silk Road. The dam is a 100m high gravel embankment with a grouted core, set in a very narrow limestone gorge. High regional seismicity and local fault alignments all increase the risk of failure of the works, and the provincial capital downstream is only part of the consequent hazard. This paper describes the inspection and investigation process, and the difficulties of identifying the seepage pattern in a three dimensional context. The justification of remedial measures currently in progress, and comprising a 70m deep diaphragm wall through the upper half of the dam core, is discussed. A short description of the bottom outlet and spillway and their current condition is given, together with comments on the parallel issue of reservoir operation and flood freeboard.

INTRODUCTION

Papan Dam has a height of 100m, above over 20m of alluvium, but is set in an extremely narrow gorge. The crest level is 1290 masl (metres above sea level) but the adjacent cliffs soar another 300m higher, with site access only by tunnel. The 90m crest length reduces to under 20m gorge width at foundation level, and this inner river channel winds within a doglegged and possibly faulted canyon. The embankment dam was constructed in three height stages, to different standards and concepts, and provided with a combined bottom outlet and spillway comprising an intake tower and tunnel.

The Papan water storage project was inspected under the World Bank funded Kyrgyz Irrigation Rehabilitation Project and a dam examination report (DER) issued in June 1999. This report was based on the historic drawings and records made available to the consultant TemelsuGIBB (Joint Venture). The technical and safety evaluation raised concerns in relation to:

- Earthquake resistance and possible fault break
- Flood routing and discharge reliability
- Seepages through the dam and high phreatic surface within the downstream shell
- Operation of hydromechanical equipment.

The situation of the dam as understood at this time is described by Jackson & Hinks in Dams 2000 (Reference 1). The recommendations of the Panel of Experts included restricting the maximum normal operating level of the reservoir to 1270 masl, until implementation of rehabilitation works permit safe operation to the full supply level of 1282 masl. The owner had already self-imposed such a reservoir level restriction since 1990, in response to seepages observed on the upper downstream slope at higher reservoir levels.

The site investigations were carried out in 2002 and were immediately followed by preliminary design of full depth core and curtain rehabilitation measures. Monitoring of the expanded piezometer layout over the following reservoir operating cycle gave a better indication of the internal seepage regime. Eventually the measured phreatic surface was used to calibrate a two dimensional seepage model, in which alternative cut-off works could be examined. This led to a contract being let for construction of a 70m deep plastic concrete diaphragm wall within the dam core from crest level, at a much reduced cost compared with treating the full 120m depth. However, the three dimensional reality of the seepage pattern is more difficult to define, and the adequacy of the remedial works will only be confirmed by piezometric monitoring, before and after the diaphragm wall construction expected in 2004.

The long investigation, monitoring and rehabilitation path for the embankment has given time for the other defective operation and safety aspects to be studied and resolved. In particular the bottom outlet capacity is limited as much by downstream interests as operational constraints. Rather than construct a second independent spillway, a compromise maximum reservoir operation level has been selected which will store and reduce the design flood peak while maintaining the irrigation benefit of the project. This resolves the apparent spillway discharge deficit in western eyes whilst improving on the typical Russian reliance on flood storage. Many local dam projects have no spillway at all, since they are oversized reservoirs for current yield and may be raised in future to suit water demand.

SITE INVESTIGATION

The layout and key sections for Papan Dam are shown in Figure 1, and construction took place from 1975 to 1985. The construction stages are indicated and the unusual downstream shell zoning that resulted. The few remaining piezometers had indicated a high phreatic surface across the downstream shell, suggesting a defect in the water tightness of the centrally located core/curtain and an internal hydraulic control under the downstream berm. The lower core was known to consist of a nine-line grouted zone constructed within a selected clean gravel fill, supported by sandy gravel shells. Details of the geometry of later stages relied on an obviously super-

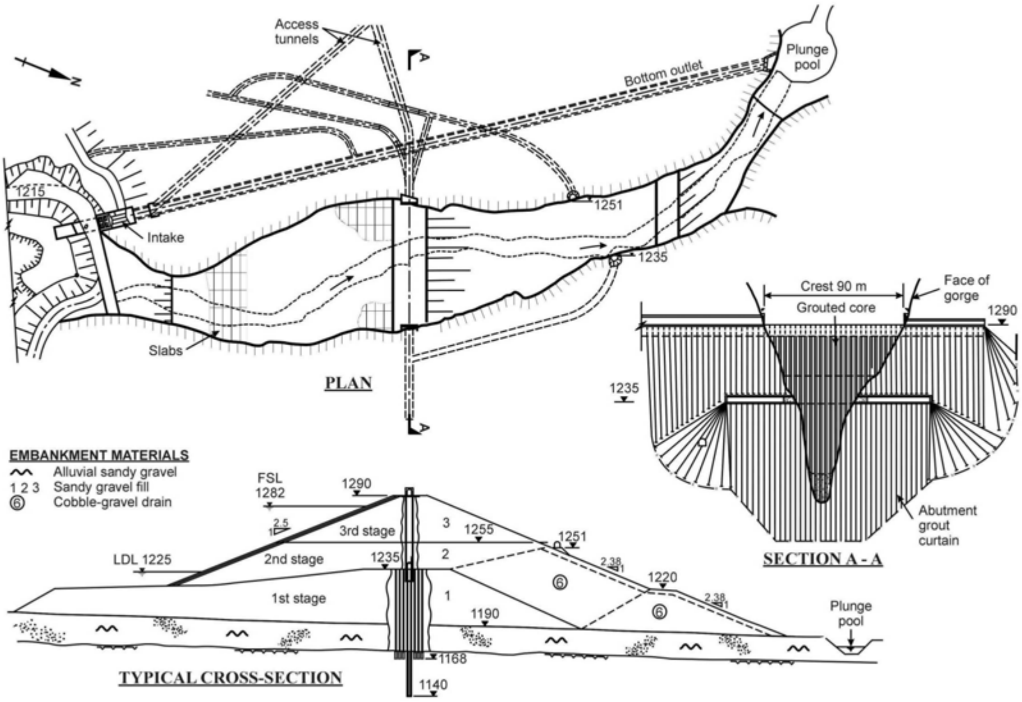

Figure 1. Papan Dam Plan and Sections

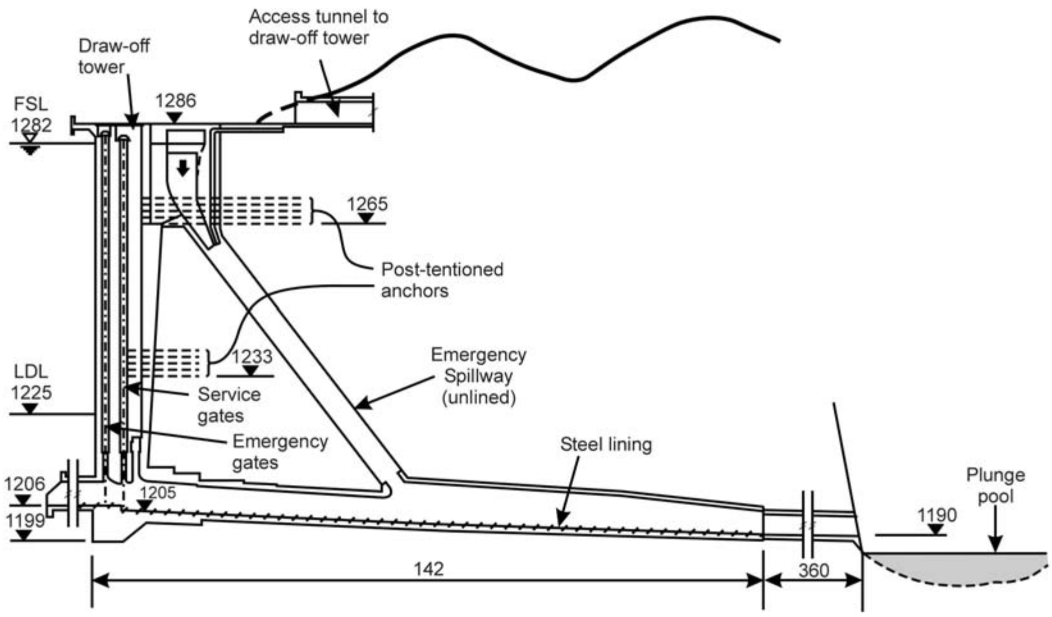

Figure 2. Combined Intake of Bottom Outlet and Spillway of Papan Dam

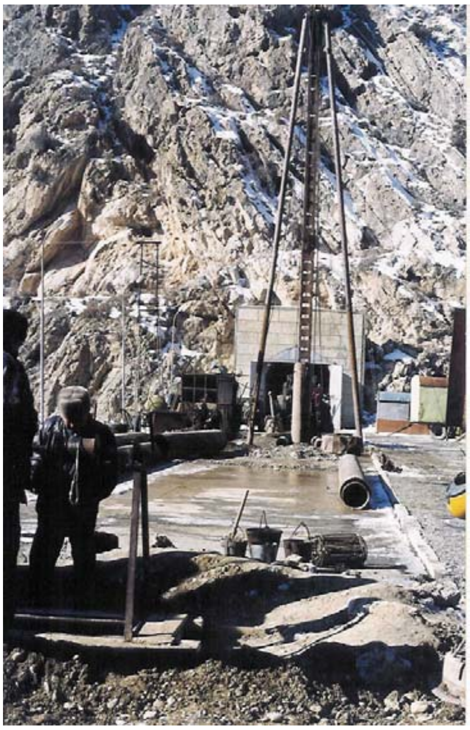

Fig.3. Investigation on Dam Crest

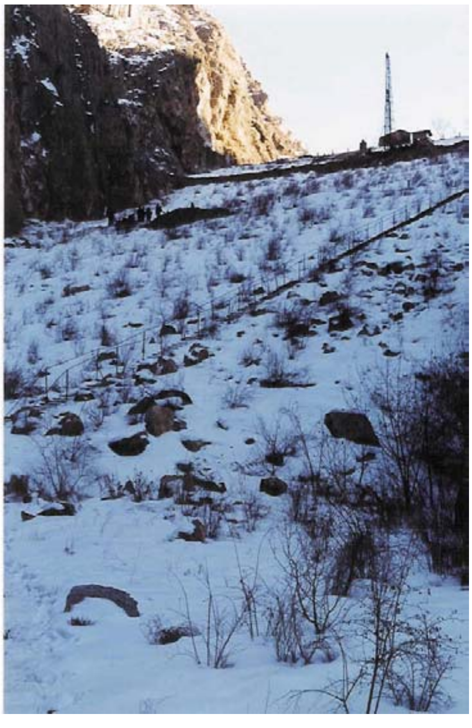

Fig.4. Downstream Slope from Berm

superceded planning drawing, but it was known that the upper core and shells were placed across the full dam width as a single sandy gravel zone followed by three lines of grouting near the dam centerline. The right abutment grout curtain at the upper level had been re-grouted in 1990 to reduce leakage into the gallery from 120 to 52 l/s.

A suitable layout of boreholes and piezometers was instructed with four percussion boreholes defining the cross-section including a 120m deep hole through the core zone. Fifteen rotary holes with piezometers were added in the downstream shell of the dam body, and four 20m deep trial pits - two from the crest into the core. From the tunnel and upper gallery four sets of two boreholes, inclined upstream and downstream of the curtain, were instructed and five minor holes at low levels within the abutments. The local water well drilling organization, called the Kyrgyz Geological Expedition, carried out the work. They provided a heavy-duty percussion rig (Figure 3) and lorry mounted rotary drills, the shafts being subcontracted to the Kyrgyz hydro institute (Kyrgyzhyprovodhoz) who also supervised day to day and carried out the soil testing (density and gradings).

Limitations & Results

Not surprisingly the $100,000 budget and one-month programme had to be doubled and trebled respectively. The recently independent state drilling enterprise had a learning curve on contractual obligations and did a magnificent job with the available equipment. The first deep hole alone took three months and the congestion on the narrow downstream slope prevented simultaneous drilling of more than two or three holes. Although accustomed to pumping out tests for wells, the drilling team found difficulty with constant head and falling head permeability test procedures within extremely deep boreholes. Providing formulae, instructions, occasional supervision and review of calculations is not enough to obtain accurate data from inexperienced testers, as we shall see below.

The steel casing type piezometers and acoustic (non-electrical) sounding equipment endemic to the region are difficult to reconcile with western practice –but the Kyrgyz in turn do not consider a narrow bore plastic pipe and electrical sounder subject to condensation on the pipe walls as reliable in cobbly-gravel fill, with calcareous deposition in all drains or pipework, and subject to frequent seismic shaking. Crucially the steel casing extends two metres below the response length and is capped at the bottom end. This provides a collector ‘bucket’ for washing out and removal of drilling mud during commissioning of the piezometer, since the rotary drilling depends on mud (and even lorry loads of loess) for stability of the holes within gravel fill. However, it also guarantees a piezometer reading even though the phreatic surface is far lower.

With hindsight there were also errors in the much discussed piezometer layout. Basically the arrangement of lines of piezometers yielding dam cross sections, and transverse cross-gorge sections was appropriate, but economies and misconceptions combined to frustrate this simple plan. The diamond shaped zone 6, placed in stage two as the downstream part of the downstream shell, proved to be such coarse cobble-gravel that permeability tests were meaningless and drilling was often curtailed by total loss of drilling fluid – all in the area thought to suffer from an exceptionally high phreatic surface. Setting out on a loose gravel surface (Figure 4), disfigured by a temporary zigzag access road, and confined by near-vertical, irregular canyon walls, was based on offsets from the crest and from a downslope steel access ladder. The narrowness and winding geometry of the inner gorge meant that many 'deep' holes simply hit rock prematurely. The layout had in any case been planned for monitoring seepage conditions for lake levels at or above 1270 masl, because of the high-level seepage reported. Eight out of 18 piezometers installed in the dam body are so shallow that they are never going to register a water level until after rehabilitation. Put simply, every deep piezometer counts for interpretation and there is no redundancy in the system.

The exploration in the dam core was particularly useful in indicating that the grouting had been only partially successful. The deep borehole appeared to indicate a large-scale window of high permeability in the lower core. Together with the 20m deep by one metre square, wood-braced, shafts a good impression of the upper core was also obtained. Eye witness accounts of construction had been useful in establishing that the gravel here was placed in 60 cm layers with non-vibrating roller compaction. The dumping and dozing placing sequence resulted in significant segregation, and only the top of each layer was compacted, resulting in effective horizontal stratification with permeable sub-layers and thin aquicludes. Only occasional boulders, wedges or sills of cemented gravel or plain cement grout takes were found in the upper core constructed by grouting this segregated, sandy gravel material. This scenario is consistent with the seepages observed at high levels on the downstream slope whenever the reservoir rises above about 1270 masl.

<u>Geology & Faults</u>

The strong crystalline Carboniferous/Devonian limestone forming the walls of the gorge is extensively jointed, with variable bedding and locally karst chimneys visible. No new rock cores were extracted as the whole area of the gorge had been extensively explored prior to construction, with addits, geophysical survey and deep coring. There appears to be a halo of stress-relieved, open jointed rock in the canyon walls, and a single line grout curtain on the dam core centerline. A major sub-horizontal, open plane of discontinuity is visible at a level around 1280 masl on the upstream right abutment. Prior to construction the groundwater flowed from right to left across the canyon, and the plateau above the gorge also imposes an intermittent flow towards the gorge. The South Katarsky fault runs along the side of the reservoir and across the upstream toe of the dam, and represents a major regional thrust fault. Overlying intact Middle Quaternary terrace deposits show that movement has not continued since that time. Conceivably the gorge location is determined by an associated tear fault, although its presence was discounted by the original (Tashkent) Design Institute, who opportunely searched for any evidence of breccia zones below the river or differential movement of the walls. The bends of the river gorge would require two suites of short en echelon faults to explain the erosion pattern. This subject was much discussed but not effectively proven as an active fault feature capable of past and future movement rather than a simple joint alignment. The definition of an active fault as defined for New Zealand in Reference 4 was found useful: repeated movements in the last 500,000 years or a single movement in the last 50,000 years. Detailed procedures to determine previous movements were proposed but not rigorously applied, due to the featureless massive limestone and lack of significant terrace remnants within the narrow gorge. This type of dam could in any case withstand minor fault movement without failure.

PRELIMINARY DESIGN

An interpretative report of the site investigation was prepared by GIBB just before the delayed end of the drilling contract and was sufficient to define the various material zones, sub-zones and their characteristics. This left the important piezometric monitoring during the reservoir operating cycle to a later date. The initial phreatic surface, determined on the dam cross-section for a 1248 masl reservoir level on 11/10/2000, confirmed that the inner downstream lower shell of first stage construction was saturated. However it also indicated a near horizontal water level just above the top of foundation alluvium running from the midpoint of the downstream shell to the dam toe. (30m lower than the high levels previously recorded on a single piezometer). Water was also appearing in the right bank lower gallery slightly in advance of and above the water level in the inner downstream shell. At this juncture all parts of the core curtain system were considered as possible culprits for the various leakage phenomena.

Accordingly, whilst awaiting more significant monitoring data, a design report was commissioned covering full depth rehabilitation of the dam core/curtain. This involved the feasibility of diaphragm walling or grouting to 120m depth in gravel, technical methodology and cost estimates. The assistance of Mr Gabriel Jorge, ex-S.American manager for Soletanche was obtained and four detailed projects drawn up:

- Full depth bentonite cement and silica gel grouting using tube-a-manchette on 9 lines. (Project 1)
- Full depth plastic concrete diaphragm wall construction with hydrofraise equipment. (Project 2)
- A hybrid project combining an 85m deep diaphragm wall with lower core grouting by angled holes from the ends of the lower gallery. (Project 3)
- Full scale re-grouting of the abutment grout curtain, or part thereof, to supplement the 1m embedment and 10m contact grouting halo included in the other three rehabilitation alternatives. (Project 4)

The estimated construction costs for the three alternatives core rehabilitation projects, with associated investigation and control monitoring, were $US 12.2 Million for core grouting Project 1, $US 7.5 Million for full depth diaphragm wall Project 2, $US 7.5 Million for the Hybrid Project 3. These prices were based on worldwide rates and included a 20% contingency for the isolated location and over half a million $US for mobilization. Each project was programmed to be completed in a single year in view of the snowbound winter conditions. The Hybrid Project 3 was capable of being subdivided into two phases with $US 4.8 Million allocated to the 85m deep diaphragm wall and $US 3.5 Million to lower core grouting from the ends of the existing lower galleries. Project 4 to extend the contact grouting halo to full abutment grout curtain rehabilitation was estimated at $US 3.6 Million, including the same 20% contingency and $US 0.25 Million for mobilization. The possibility of treating only part of the abutments on a pro-rata cost was mooted. The Interim Design Report A covering these matters was issued in March 2001, shortly followed in April 2001 by an initial Monitoring Review Report B from which a decision on the appropriate project to adopt or adapt was expected to emerge.

PIEZOMETRIC MONITORING

The gradual reservoir rise in the winter of 2000/2001 was from a base level of 1230 masl in June up to a peak of 1263 masl in mid-March as the snow-melt season progressed. Thereafter the irrigation releases exceeded inflow, but data up to August 2001 was subsequently analysed and added to the report graphs and figures. Although this reservoir range is rather limited compared with the full range of 1225 to 1282 masl, and has not been amplified in the subsequent years, it was sufficient to derive some surprising conclusions and to illustrate the limitations of the piezometer layout. The data was plotted against time in comparison with reservoir level and onto an idealized dam cross-section plus three gorge sections. These gorge sections were located downstream of the core, at midslope and through the downstream berm, conveniently breaking the data into manageable parcels and focusing on areas of interest. In addition the situation in the left and right abutments were separately analysed. The initial finding of a low-level, near horizontal water table across the downstream half of the downstream shell was confirmed for the range of reservoir levels as a sort of internal stable tailwater level, at around 1195 masl with a crossfall of under 1.5m (downstream gradient less than 0.02 below the berm). This prompted an enquiry into the historical record of high piezometric levels recorded at intervals over many years in the downstream slope. This was traced to a single standpipe piezometer (n8') that, although indicated on the drawing as extending down into the alluvium, in fact terminated at a high level –similar to the ghost readings it had been producing. Pouring ten metres of water into the piezometer resulted in a swift return to the residual level. Therefore all the initial stability checks for the 1999 DER, finding the dam just stable with some crest subsidence under earthquake MDE acceleration of 0.72g, were very conservative, since they were based on a false premise with the downstream phreatic surface 30m higher than reality.

The downstream berm feature is not some temporary cofferdam feature obstructing flow, but a drainage zone. A simple calculation of the flow through the narrow inner river channel alluvium, using measured insitu permeability at the measured low hydraulic gradient, indicates that the discharge is insufficient to maintain equilibrium so the rock walls of the inner gorge are also carrying seepage flows. Looking at the upstream gorge section the water levels in the inner downstream shell zone (first stage construction) are close to 1228 masl and conceivably fed from higher levels on the abutments. The left abutment plots are all based on dry readings, and this whole abutment is considered to be a groundwater sink. On the right abutment the readings are near the bottom of the inclined piezometers from the upper gallery down to 1250 masl (possibly in mud or trapped end sections), and water overflows from the lower gallery piezometers at 1235 masl level. The upstream piezometers faithfully reflect the reservoir level and appear to indicate a head drop of 3 to 8m across the grout curtain.

As part of the selected rehabilitation measures these raking piezometers from the high level will be extended downwards to overlap with the lower gallery and thus give a reliable reading in the critical range for intermediate reservoir levels. Additional rock drainage measures will probably convert them to permanently dry piezometers, unless right abutment seepage is a major source for maintaining the water level in the adjacent shell zone. The midslope gorge section shows a water level in the shell at 1201 masl, just 7m above the internal tailwater level and giving the overall downstream gradient as 0.05. There is thus an internal waterfall 27m high close to the downstream slope of the first stage construction. This was picked up during drilling as a high permeability (75m/d) sub-zone, and may simply represent the leading edge of the second-stage, diamond-shaped, internal drainage zone 6 of cobble-sized gravel or a rock boulder layer of rejects or riprap on the then downstream face. By accident or intention an inclined internal chimney drain has resulted downstream of which the internal phreatic surface is insensitive to reservoir level fluctuations.

Finally it was discovered that the line of existing piezometers immediately downstream of the 25m wide crest, again indicated on the drawing provided, was actually just one defunct piezometer. The reading on piezometer n2', thought to be on this line, actually corresponds to the gorge section 40m downstream of the dam core axis on which the line of new piezometers were installed (supposedly to fill the gap). There is thus still a gap in the critical area just downstream of the core where piezometric readings might indicate whether the seepage is passing through upper, middle or lower core. This may be indicated as a concave or convex phreatic surface joining reservoir level at the single upstream shell piezometer to the established upstream row of piezometer readings. Again this defect in the piezometer layout will be remedied with a new row of instruments installed to control the diaphragm wall performance. The seepage contribution of the much grouted right abutment and of the alluvium below the core (also grouted on just three lines) also remain unknown factors. Before taking a decision on rehabilitation measures and priority areas to be treated, a computer seepage model was commissioned in an attempt to resolve these issues.

SEEPAGE STUDIES

The two-dimensional seepage model was prepared using the program SEEP/W and the assistance of Clare Glenton. We started with the zone boundaries and permeabilities from the Site Investigation interpretation, and this created a phreatic surface as for a homogenous section, exiting onto the downstream slope near the top of berm level 1220 masl. This was plainly due to the supposed window in the lower core. Of course the water test results from one borehole towards the back of the core does not imply the window cuts through the whole 9-line grout curtain. Taking a probability view on the mass permeability of the lower core the window was closed down to 10% of its potential flow. Thereafter the model required 8 further iterations in order to adjust the phreatic surface to fit the monitored version. Each time the permeability of one or other key zone was modified, and in some cases the vertical/horizontal permeability ratios. This calibration process intrinsically accepted the overall seepage pattern of the piezometric data in preference to measured in situ point values of permeability. Even supposing that the insitu tests were accurate, they need not be representative of complex zones subjected to years of high gradients and particle migration (suffosion). Of course the zone interpretation in terms of geometry, permeability relative to adjacent zones, material type and placing method were respected as far as possible.

To avoid potential errors at the beginning, peak or end of the monitoring series, three calibration curves of phreatic surface were used corresponding to reservoir levels:

- 1262 masl – either side of the peak 1263 masl.
- 1250 masl – descending reservoir stage, and when it became available
- 1242 masl.-close to the August 2001 minimum of 1241.5 masl.

The model based on the first two of these phreatic surfaces, and incorporating significant modifications of initial permeability values, was then able to predict the third. The model replicates in two dimensions what is in truth a three-dimensional seepage pattern, with probable transfers from the right abutment and to the left abutment rock. It is thus superior to a simple zone/permeability model lacking calibration and a reasonable predictive tool, which was named the basic revised seepage model. However, it is not a unique model since the permeability of the ultimately controlling upper core zone is not affecting the calibration against the observed phreatic surfaces. This point was difficult to communicate since the original hypothesis of a downstream hydraulic control below the berm had permitted simple extrapolation of phreatic surface data to the highest flood levels. Nevertheless the model was able to simulate the observed seepages on the upper downstream slope at high reservoir levels, which was the primary concern of the owner. This was only possible because the particular program indicates flows above the zero pressure line. Hydraulic gradient contours were produced for the 1286 masl reservoir flood level, and values of 6.5 occur in the second stage shell adjacent to the top of the drainage zone, compromising the filter relationship between sandy gravel and rockfill. Even higher gradients are registered at the adjacent external slope around 1250 masl, indicating sloughing may be expected where seepage losses had indeed been observed at high reservoir levels. It also estimated unit width seepage flows at chosen sections, but care has to be used in deriving three-dimensional flows in the gorge situation.

By this stage the Panel of Experts including Professor Raymond Lafitte, Jonathan Hinks and Professor Bektur Chukin had co-opted a grouting specialist from Moscow, with records of the original lower core grouting. Based on his evidence they inclined to suspect that seepage loss flows were more likely to be crossing the core/curtain alignment through the upper core than elsewhere. The model was used to determine the effect of a diaphragm wall cutting off the upper core zone (first phase of the hybrid Project 3). Distinct depths of penetration of 85, 75, 70 and 60m were modeled and hydraulic gradients and seepage flows derived. All the plots were similar. Gradients behind the toe of the wall and within the downstream shell do not exceed 2.5, which should avoid suffosion effects, or at any rate avoid significantly increased particle migration over the current situation. Total leakage predictions are subjective, but unit width flows decrease only gradually with depth of cut-off wall and by up to 12% in comparison with the no cut-off version. The real benefit is thus only to cut off the horizontal seepage on privileged paths through the segregated sandy-gravel fill in the upper shoulder of the dam.

BOTTOM OUTLET & SPILLWAY

The combined spillway and bottom outlet system were described in Reference 1 and are shown here on Figure 2. TemelsuGIBB calculations for flood routing in the DER 1999 were based on western concepts such as passing the full PMF over a spillway, ignoring the bottom outlet. Dambreak would not only destroy the provincial capital Osh but also pass through the main agricultural zone of the Ferganah Valley, and cross several national borders as a flood wave on the Syrdarya River (Reference 2). There are numerous local precedents for designing dam projects with massive freeboard so as to absorb flood peaks as reservoir storage. In the unusual case of Orto Tokoi dam the spillway capacity had to be severely restricted to increase the live storage and water yield. In most other Kyrgyz dams additional spillway provision was recommended, and in the case of Papan this implied an expensive separate tunnel spillway with the possible addition of a mini-hydro station to augment the two 9m long weirs at the intake tower (Figures 2 and 5).

Understandably the client stalled on this issue until the necessary embankment rehabilitation was decided and permissible dam operating level was established. The possibility of glacial lake bursts within the far upstream catchment was mooted (Reference 1), but dismissed after examination of possible natural dams originating from landslides, fault movements or moraine deposits. Reservoir routing of PMF and 1:10,000 year floods had been carried out for the temporary maximum reservoir level of 1270 masl and for the normal maximum of 1282 masl (spillway crest level). The flood hydrographs comprise a narrow 2-day peak of 652 and 464 m^3/s respectively superimposed on snowmelt base flows of around 150 m^3/s. For the 1282 masl reservoir starting level, dam crest level of 1290 masl and the intake platform at 1286 masl were compromised by the extreme cases even with maximum gate discharge of 260 m^3/s. The two huge wagon gates would be difficult to adjust in a flood situation and only a 1:1,000 year flood could be accommodated at the irrigation setting of 20 m^3/s, although in reality the combined tunnel becomes the limiting factor with a 345 m^3/s maximum open channel flow. The inclined spillway shaft linking to the tunnel was said to be a textbook design, but no calculations or model tests were provided. The condition and presence of the shaft lining have not been confirmed (no access), the steel tunnel lining downstream of the gates has previously been repaired, the gates have vibration and aeration problems, and the tunnel outlet and plunge pool are sub-standard.

The reasons for this situation being tolerated only gradually became clear. First the maximum gate discharge is only 160 m^3/s as permitted by the gate manufacturer not 260 m^3/s as calculated for two fully opened gates. There is a continuous minimum discharge of 20 m^3/s compensation water for Osh city potable water. Fundamentally there is no benefit in filling the reservoir and even after rehabilitation a 1275 masl normal top water level will be applied. Flood routing then becomes much simplified with 45Mm3 extra flood storage, doubling that for the 1282-88 masl flood range. The gate vibrations occur at small openings and have now been measured and found to be within acceptable limits of the Soviet code (SNIP). The obstruction to the aeration shaft has been lifted and an aeration slot will be created, and minor welded insitu reinforcement of gates and linings added, rather than proposed major off-site gate modification. The tunnel outlet rock-support wall and plunge pool amplification at the inaccessible dam toe will be carried out, after diverting the compensation flow by a temporary adit linked to the tunnel.

DIAPHRAGM WALL CONSTRUCTION

TemelsuGIBB first proposed a no-action strategy based on maintaining a normal top water level of 1270 masl. The only example of diaphragm wall construction through a high dam core in a gorge of this magnitude appears to be Mud Mountain Dam in the USA, where problems due to arching of the core occurred (Reference 3). A separate Kyrgyzhyprovodhoz study had shown that the irrigation benefit of the extra 12m depth to 1282 masl (75Mm3 extra reservoir volume) was zero for the agricultural areas developed, due to unreliability of supply in most years. The Panel of Experts however did not accept no-action, since flood rises will temporarily affect the upper dam and long-term storage levels might not respect present agreements as the responsible personnel retire or change. The owner agreed that the opportunity to introduce a partial cut-off at limited expense from current funding should not be missed. The options of 60 or 70m depth were further explored, with consideration of later phases should lower core or right abutment grouting subsequently prove necessary. A key factor was that the Hybrid Project 3, with drillholes angled sidewards and downwards fom the confined and plugged ends of the lower galleries to achieve a nine-line curtain was simply impracticable. At this stage, after 20 years of consolidation of embankment gravel fill, the gallery ends could be simply joined by excavating across the partially grouted core – permitting sub-vertical groutholes and minimal overlap with the diaphragm wall. A diaphragm wall depth of 70m (down to 1220 masl) was eventually selected to give a generous 15m penetration into the lower core.

Contract documents were prepared based on the Engineer's scheme, but asking the Contractor to carry out final design to suit his capacity and plant for excavation and production, delivery and placing of plastic concrete mixes. The Iranian firm JTMA Co. was awarded the diaphragm wall construction contract early in 2003 at a sum close to the estimate. Competitors had priced the works at up to twice this sum, anticipating difficulties with the site location and inhospitability. The Contractor has reduced the typical panel length to 3.0m and volume to 210 m3, from the French based 8.8m and 616 m3. This gives a more manageable task in the congested area of the short dam crest with tunnel access only. The other advantage is that to achieve a nominal 1.0m depth key into rock, the hydrofraise has to cut a square panel base jutting into abutment rock as a triangle. Given the extremely high rock strength (estimated at 200 MN/m^2) the advantage of smaller triangles of the hard, siliceous, crystalline limestone to grind away is evident. The joint detail has also been simplified, cutting into the preceding panel and inserting a groutpipe rather than drilling separately.

Maintaining a one-metre wall thickness for the reduced wall depth assists the joint overlap being guaranteed against lateral deviation during excavation. The alignment of the wall has been moved a few metres downstream, to the back of the lower core, for fear of the hydrofraise being severely damaged by bumping into old steel grouthole casings left in the fill. Not surprisingly given the logistics of reaching Osh, of fabricating batching plant in Tehran and upgrading the hydrofraise excavation rigs in Italy, progress this year has been slow and limited to preparatory and auxillary works. It is now anticipated that the diaphragm wall construction will proceed in the spring of 2004. Contractual niceties come second to physical possibilities in remote locations and extreme winter weather.

CONCLUSIONS

The Final Dam Design Report was compiled to record this four-year study and issued in October 2002. While every effort was made to speed the process, the assumptions made to facilitate design packages were often proved unsatisfactory in the long term. The difficulty in obtaining and then verifying information and even drawings should not be underestimated. Piezometer layout sections were extremely misleading, and 10 years of hand plotted phreatic surfaces in full reservoir conditions simply erroneous. As for dinosaur research, one must first assemble an awful lot of fossil bones, and then ask the right questions in order to correctly interpret them functionally. In these days when safety inspections of 60 dams at a time are required, with two days input allotted for each and only limited access to and around the site, the inspector must explore the 'facts' and try to understand the mindset of the owner/operators. The painstaking investigation, monitoring and design review process applied at Papan dam were a rare opportunity to focus on the individuality and idiosyncrasies of a standard Soviet dam design applied to a narrow canyon. There were no short cuts, and only perseverance by all parties concerned will lead to satisfactory rehabilitation works. The very active role of the client, Project Implementation Unit for Kyrgyz Irrigation Rehabilitation (Ministry of Agriculture, Water Resources and Processing Industry) and invaluable assistance of Mr Fedotov (Kyrgyzhyprovodhoz) in pursuing these technical issues, providing information in difficult circumstances, and obtaining value for money solutions is acknowledged.

Figure 5. Papan dam – Reservoir and Intake Tower

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The Washburn Valley Reservoirs – spillway improvements

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SYNOPSIS. The Washburn Valley reservoirs comprise a cascade of four impounding reservoirs situated about 12 km to the west of Harrogate in North Yorkshire. The three lower reservoirs, Lindley Wood, Swinsty and Fewston, were formed between 1875 and 1879, by the construction of earth embankment dams with puddle clay cores. The upper reservoir, Thruscross, is a mass concrete gravity dam constructed in 1966. The upper three reservoirs supply water to Leeds while the lowest, Lindley Wood, provides compensation flows to the River Washburn.

The three lower dams would be overtopped during the Probable Maximum Flood (PMF). The situation was complicated by the publication of the Flood Estimation Handbook (FEH), which led to a review of the conceptual design and a lengthy delay, which was recovered by carrying out works at two dams in one season, instead of one per year as originally planned.

The rehabilitation works consisted of crest raising and spillway modifications at the three embankment dams.

- Lindley Wood: a 3 m high earth embankment was built downstream of the existing crest road, which will be inundated during extreme floods.
- Swinsty: the crest was raised 1.2 m and the multi-span masonry arched bridge replaced by a clear span.
- Fewston: the crest was raised 0.9 m and the multi span masonry arched bridge replaced by a clear span.

At £6.5 M this is one of the largest reservoir safety rehabilitation schemes undertaken by *Yorkshire Water* (YW). It was successfully completed in 2003 on time and within budget by team working.

BACKGROUND

The Washburn Valley reservoirs comprise a cascade of four impounding reservoirs. The three lower reservoirs, namely Fewston, Swinsty and Lindley Wood were formed between 1875 and 1879, by the formation of embankment dams with puddle clay cores. The upper reservoir, Thruscross, was completed in 1966 with the construction of a mass concrete gravity dam. The upper three reservoirs supply water to Leeds whereas the lowest reservoir, Lindley Wood, provides compensation to the River Washburn.

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

There are considerable benefits from being able to undertake iterative hydrology, hydraulic calculations and model testing at the same time. Changes to any one component can affect the others. For example, removal of spillway restrictions at Fewston and Swinsty changed the hydraulic characteristics, reduced the flood attenuation, and increased the required freeboard at Lindley Wood.

The five arch masonry bridges at Fewston and Swinsty significantly reduced spillway capacity at moderate discharges. Various schemes to preserve their appearance were considered but none were found to be practical and it was therefore decided to seek planning consent to remove them. The outline designs to pass the PMF through the cascade included:

- Fewston — A 2 span bridge, raised crest road and wave wall.
- Swinsty — A 2 span bridge, raised crest road and wave wall.
- Lindley Wood — Crest raised by 3 m.

Considerable out of channel flow was predicted at all three dams. The model testing provided excellent information on depths and velocities. YW adopted its normal practice of extending the spillway rating curves to flows 10% higher than anticipated and recording the test performance on video.

FLOOD REVIEW AND IMPACT OF FEH

In October 2000 *TEAM*, a working agreement between *E C Harris, Arup* and *MWH*, were appointed to carry out a feasibility review, detailed design and project management for implementation of the scheme. This was soon after the publication of the Flood Estimation Handbook (FEH) [2], which complicated the situation significantly.

The flood assessment techniques contained within the FEH are on a different basis to the Flood Studies Report (FSR) [3], and were publicised as being the "the replacement for the Flood Studies Report". Interim guidelines on their application were published by DoE, summarised as:

- If the overflow capacity is adequate to present standards (i.e. Reference 1), then do nothing.

  The overflow capacity had been found to be inadequate – hence 'do nothing' was unacceptable.

- If new or improved spillways are required, then follow one of the following three options:

  1. If practicable, then postpone work on spillways until new guidance is available.

     This was impracticable, since the recommendations were "in the interests of safety" and therefore mandatory, furthermore it was not known when new guidelines might become available.

  2. If (1) is not practical, adopt a 2-stage improvement, if this is technically, financially and environmentally acceptable. The first stage is to increase the spillway capacity using FSR. The second stage is increasing the capacity further, if subsequent higher standards are recommended.

     This approach was adopted, with the works designed to allow future crest and wall raising.

3. If (1) and (2) are not practicable, increase the capacity using FEH rainfall or worst case PMF.

Revised PMF

The FEH methodology was claimed to include latest thinking on catchment characteristics, which updated the Flood Studies Report. The new procedures were incorporated into a review of the flood hydrology, which resulted in an increased PMF from this “hybrid” approach. The following table compares previous and new PMF values and existing and proposed flood defence levels for the three reservoirs.

It was decided to adopt a precautionary approach and design to the higher flows and levels.

	Estimated PMF outflow (m^3/s)		Flood Defence Level (m OD)	
Reservoir	FSR	FEH “hybrid”	Existing	Required
Fewston	405	442	156.58	157.55
Swinsty	454	498	140.20	141.28
Lindley Wood	504	536	93.22	96.09

PROGRAMME OF WORK

It had originally been intended to improve spillway capacity sequentially, working upstream from Lindley Wood in 2001 and finishing with Fewston in 2003. The flood review had effectively lost a year from the programme.

It was decided to carry out the remedial works on the lower two reservoirs, (Swinsty and Lindley Wood) during 2002 under a single contract. This presented parallel difficulties of maintaining compensation discharges and water supplies, which were overcome by careful control of reservoir level. The works at Fewston followed under a separate contract in 2003, recovering the time lost.

The contracts were let by competitive tendering to a select list using NEC ECC Option A contract conditions. Both contracts were won by *Morrison Construction*, who were able to transfer staff, cabins and ‘lessons learnt’ from Swinsty to Fewston.

WORK CARRIED OUT

The main components of work at each dam are outlined below:

Lindley Wood

Lindley Wood dam is 330 metres long and was a maximum 21 m high with a capacity of 2920 Ml. Remedial works included:

- Raising of flood defence levels by about 3 m. This was achieved by construction of a new embankment above the existing one, thanks to the unusually wide crest. The new embankment comprised granular fill with side slopes of 1:2. An HDPE membrane was laid over the upstream face, terminating within the existing clay core at the bottom and rising above peak still water level at the top. The design of the crest raising was unusual in that the existing wide crest allowed the construction of the new embankment downstream of the existing access track. In extreme conditions both the track and existing valve towers will flood. Rather than opting for a scheme with higher capital costs that would ensure the track and valve tower did not flood, YW accepted this arrangement as a 'business risk' since it would not pose a threat to reservoir safety. There is no wave wall, however one could be built on top of the new embankment in future.

- Increase of spillway capacity by the demolition of existing footbridge, the construction of a new reinforced concrete headwall structure and by making provision for out-of-channel flow by creating reinforced grass revetments utilising proprietary pre-cast concrete blocks.

Swinsty

Swinsty dam is 460 m long and was a maximum of 20 m high with a capacity of 4655 Ml. Remedial works included:

- Raising flood defence levels by about 1.2 m, which was achieved by the construction of a new 2.25 m high reinforced concrete wave wall to replace the existing and raising the crest road level by approximately 1.2 m, in granular fill. A sheet pile cut off embedded into the existing puddle clay core and extending into the wall base ensures a continuous water barrier to above peak still water level. The wall can be raised by 0.5 m.

- Increase of spillway capacity by demolition of the existing five arch bridge and replacement with a new single span bridge with the soffit level set above the PMF level. Provision for out-of-channel flow by construction of additional bunding and provision of reinforced grass revetments utilising proprietary pre-cast concrete blocks. The replacement of the bridge at Swinsty was undertaken as a 'design & build' element within the contract, and designed to be lifted 0.5 m in the future. The main beams for the bridge were prefabricated and delivered to site as single 30 m long units;

- The crest road at Fewston is a public highway and as such these works are subject to the approval procedures of North Yorkshire County Council and the wave wall has been designed to provide vehicular impact containment to P2 level in accordance with BD 52/93 "The Design of Highway Bridge Parapets".

- Increase of spillway capacity by demolition of the existing five arch bridge and replacement with a new single span bridge with the soffit level set above the PMF level. The bridge is similar to Swinsty.

- Provision for out of channel flow by construction of additional bunding where necessary and provision of reinforced grass revetments utilising proprietary erosion control geotextile;

DESIGN ISSUES

Although many of the elements of the three designs were common to each, a number of issues required special consideration:

Revetment Protection System

Revetment protection systems were designed on the guidance of CIRIA Report 116 – Design of Reinforced Grass waterways. Maximum anticipated out of channel flow velocities for the three spillways are as indicated in the table below:

Reservoir	Maximum estimated out-of-channel velocity
Fewston	6.0 m/s
Swinsty	7.1 m/s
Lindley Wood	9.7 m/s

Flow velocities at Fewston and Swinsty resulted in geotextile erosion control matting and interlocking precast concrete blocks respectively to be chosen as the preferred method of protection. The peak velocities at Lindley Wood were anticipated to be in excess of those velocities covered by the CIRIA guidance (8 m/s maximum). However, one of the authors of that report confirmed that the interlocking pre-cast concrete block system could withstand sustained flows at velocities up to 10 m/s, if installed with sufficient attention to detail, hence this system was adopted.

Environmental Issues The Washburn Valley constitutes part of the Nidderdale Area of Outstanding Natural Beauty and planning restraints have required that as far as possible the existing landscape be preserved or enhanced. Detailed Planning Consent was sought for all three dams in a single application in order to reduce the chances of delays and permission was obtained with acceptable conditions. All new structures are required to be fully clad in natural stone work, including the bridges, and measures such as ecological surveys, archaeological studies and tree preservation strategies were employed in order to minimise the impact of the works.

Lindley Wood Cottage

This disused dwelling was originally intended for demolition as it was considered an obstruction to the dam raising works. However plans were altered when two bat colonies were discovered within the roof void. Bats are protected species and a mitigation strategy needed to be agreed with DEFRA in order that permission to remove the habitat could be given.

The most straightforward mitigation was to build another bat roost nearby, carefully replicating the conditions in the hope that the bats would move, however this would have meant delaying the work by at least one year and possibly longer. Alternatively, the raising could have been done by a complicated realignment of the crest around the house, in order that the structure might be left intact. The solution adopted in order to facilitate both the crest raising and the maintenance of the bat habitat was to build the cottage into the raised dam embankment. The ground floor was filled with lightweight concrete and the existing first floor became an electrical plant room. Landscaping around the house was designed to maintain flight paths and bat tiles were built into the roof to maintain access for the bats. The bats returned to breed in 2003, helpfully discharging the planning condition.

Recreation

The area is popular with ramblers and YW has promoted circular walks, which pass through the construction sites. Temporary footpaths were erected and maintained to segregate pedestrians from traffic.

All the reservoirs are active fisheries, which were able to continue in use during the work. New permanent tracks were built to allow access to the drawn down waterline in order to enable Fewston Reservoir to be restocked with rainbow trout.

Water Control Measures

YWS undertook to maintain water levels in the reservoirs within a pre-determined range below existing overflow weir levels so as to ensure that construction could not only proceed safely but also so that water supplies to Leeds could be maintained. The criterion for the upper limit was based on a 1% chance that the level would be exceeded during the critical construction period when work is undertaken on the dam or spillway. A procedure was formulated by *YWS*, *TEAM* and *Morrison Construction* whereby water levels would be monitored and contingency plans brought into action in the event of the reservoirs rising above various threshold levels.

The contingency plan was called into operation on one occasion during the works at Lindley Wood and it worked well. The contractor mobilised plant and materials to protect the open excavation over a weekend, scour discharge to the river was maximised and the bags of stone and clay were removed without getting wet the following week. The client accepted the financial risk of invoking the emergency measures and the incident was covered under the cost component schedule of the contract, including costs to accelerate the works back on to programme.

CONCLUSION

By the time the work at Fewston dam was nearing completion the project team of consultant, client and contractor were working so well together that they wanted to move straight on to the next dam upstream. Regrettably, all good jobs come to an end and this one was finished on time and below budget.

ACKNOWLEDGEMENTS

Thanks are due to Ian Farmery of TEAM, for permission to use some text previously published in Water Projects UK.

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Rehabilitation design of Acciano rockfill dam after the September 1997 earthquake

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SYNOPSIS. The Acciano rockfill dam was originally designed without taking seismic action into account. The area where it is located is now classified in the 2nd seismic zone, according to the current Italian regulations. On September 26th 1997, an earthquake of magnitude M_w=5.5, one of the largest seismic events of the last 20 years in Italy, occurred in that area and caused some visible damage to the dam. Subsequent investigation programmes and structural assessments were carried out to evaluate the residual safety margins of the dam in order to identify possible rehabilitation provisions to comply with the Italian standards for seismic design.
This paper describes the evaluation of the post – earthquake condition of the dam and outlines the assessments to validate the rationale of the rehabilitation project.

THE DAM

The Acciano rockfill dam is located in the centre of Italy (Perugia Province); it was built between 1976 and 1980 to impound water for agricultural use during period of deficient supply. The reservoir capacity is 1.7 million m^3.

The Acciano dam has a zoned embankment with a curvilinear axis; an internal central impervious core and external rockfill shoulders. The embankment is characterised by three berms, at different elevations: two are located at the downstream side and one at the upstream. The structure reaches a maximum height of 28.5 m, and it is 182 m long along the crest, at elevation 531.5 m a.s.l. The faces have a slope (Fig. 2) equal to 1:1.4 from the crest to the first berm (el.513 m), and 1:2.5 in the bottom part. The shoulders were built by dry compaction of two different materials: the zone above el. 513 m with rockfill and the part below with a gravelly sand. The core is of silty-clay.

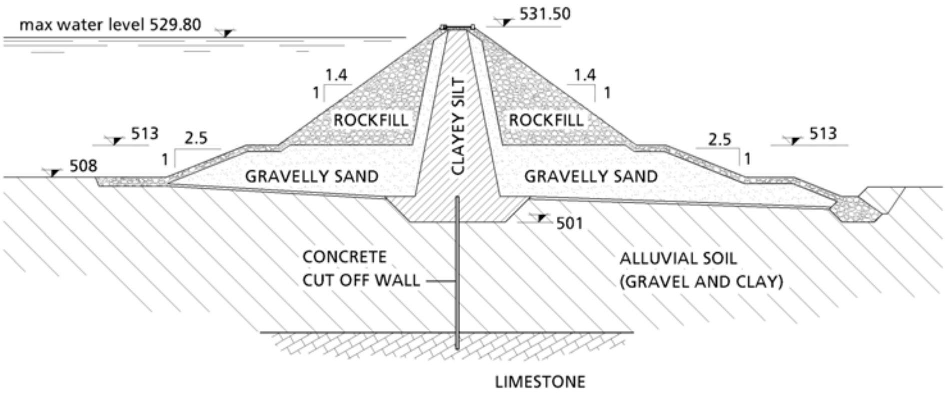


Figure 1. The main cross section

The embankment is founded over an alluvial soil, which is below the main section, about 20 m thick. Between the main section and the shoulders, the thickness of this alluvial layer decreases to zero and the dam is directly founded on a marly limestone rock. To reduce seepage, a concrete diaphragm wall 0.6 m thick was built below the core to bedrock. Cement grouting was carried out to enhance the hydraulic performance of the rock foundation, in particular at the abutments.

At the right abutment the rock is fractured to locally highly fractured: a grout shield 60 m long from the crest elevation and 30-40 m deep into the abutment rock mass was therefore added.

The dam has two outlet works, both located within the rock on the left abutment: a bottom outlet (el. 506.9 m a.s.l) and an overflow spillway (el. 528.50 m a.s.l.) which can release, at the maximum water level, a discharge outflow of 38.8 m^3/s and 86.2 m^3/s respectively, which overall corresponds to the 1000 year flood. The bottom outlet is a tunnel of precast reinforced concrete that, in its central part, lies within the dam body. In this part the discharge gallery is closed by two sliding gates, which are operated from the control tower located in the reservoir near to the left abutment.

The monitoring system comprises: a topographic collimation to observe planimetric displacement evolution and settlement of the crest and at the downstream berms; eight Casagrande piezometers placed downstream respect to the dam body and into the left and right foundation rock. The monitoring records did not show any anomalous response before the earthquake took place. The dam had been operational for 11 years and the reservoir reached 522 m a.s.l. water level before the earthquake struck.

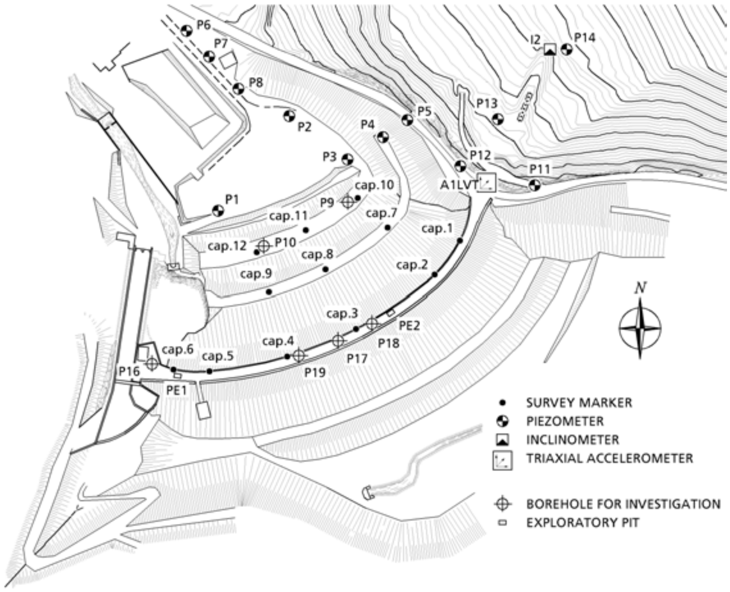


Figure 2. The instrumentation network (partial) and boreholes for investigation

OBSERVED EFFECTS OF THE EARTHQUAKE TO THE DAM

On September 26th 1997, an earthquake of magnitude Mw=5.5 occurred in the area of Nocera Umbra, one of the largest seismic events of the last 20 years in Italy, causing some visible damage to the crest structure.

Shortly after the event some wide cracks appeared in the asphalt paving of the crest: two longitudinal cracks, close to the crest edges, and two transverse ones, near the extreme ends of the dam, were observed (Fig.4). The upstream crack appeared the most significant and spanned the entire crest. Settlements up to 15 cm of the rigid reinforced concrete edges of the crest road were measured in the main cross section, where also a lateral spreading of about 10 cm was also found. The downstream lower berm settled about 2 cm. In the following month, settlements of the dam increased by only a small percentage, mainly due to the dissipation of the excess pore pressure in the foundation soil (a 0.05 MPa increase was measured in the alluvium shortly after the event). The following Figure 3 shows the chronological measurements of displacements on the crest and on the two berms.

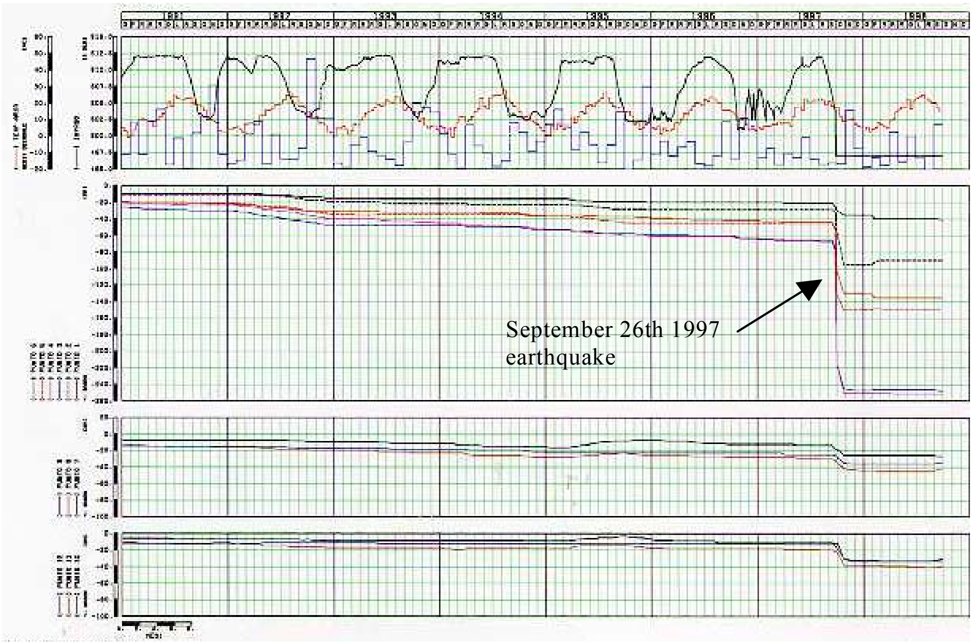


Figure 3. The chronological diagrams of displacements on the crest and berms (Midas® - Ismes Software)

In the following table, settlements of the crest and the two berms before and after the earthquake are reported.

Table 1. The measured vertical displacement values [mm]

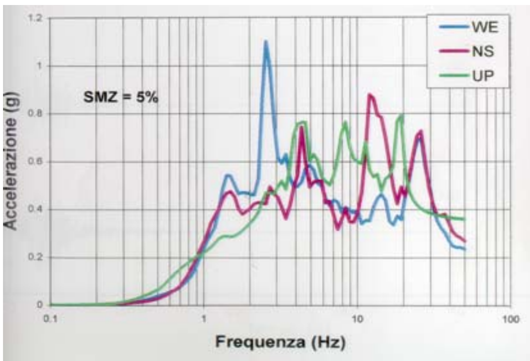
Marker		*from jan 1986 to sept,26 1997*	*After Earthquake*	*%*
Crest	P1	**-25**	**-16**	64
	P2	**-51**	**-84**	165
	P3	**-77**	**-171**	222
	P4	**-74**	**-177**	239
	P5	**-52**	**-97**	187
	P6	**-32**	**-58**	181
Berm 1	P7	**-18**	**-9**	50
	P8	**-32**	**-10**	31
	P9	**-26**	**-9**	35
Berm 2	P10	**-13**	**-18**	138
	P11	**-22**	**-17**	77
	P12	**-15**	**-18**	120

Two square exploration pits 3m wide (PE in Figure 2), dug one year later, did not show any evidence of deepening of the two transverse cracks. No extension of the observed crack path and of complementary damage has been observed.

Figure 4. The damage on the crest

CALCULATED EFFECTS OF THE EARTHQUAKE GROUND MOTION

A large number of stations of the national accelerometer network were operating in the Umbra-Marche region. The nearest to the Acciano dam was that of Nocera Umbra, 11 kmaway. From that record, a site spectrum has been derived as input at the dam foundation for use in structural assessments. Processing of the Nocera Umbra records was made adopting the Sabetta-Pugliese attenuation laws. Site amplification data, based on local measures of micro tremors, gave evidence that no local amplification need to be incorporated. The resulting response spectra (horizontal and vertical component) are depicted in Fig. 5 together with the corresponding accelerograms.



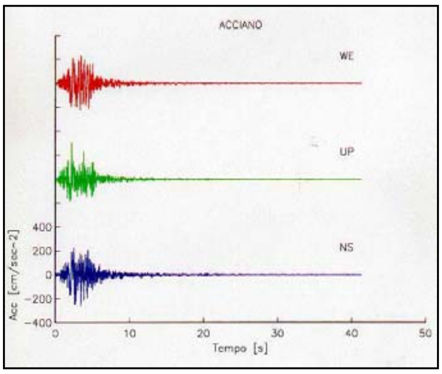


Figure 5. Calculated spectrum and time history below the dam

It may be appreciated that the highest accelerations concentrate within the interval 1.5-10 Hz, where the natural frequencies of the dam also fall. The dam crest could suffer accelerations up to 1.1 g.

A Finite Element (FE) model (Fig. 6), representing the dam body and a portion of the foundation rock, has been set up to determine: the actual distribution of accelerations within the dam body; the principal dynamic properties which are vibration modes and the associated frequencies. The material model is elastic with assigned decay law of material properties (damping and shear modulus). The parameters incorporate the stiffening contribution of the interstitial water and of the short duration of loading.

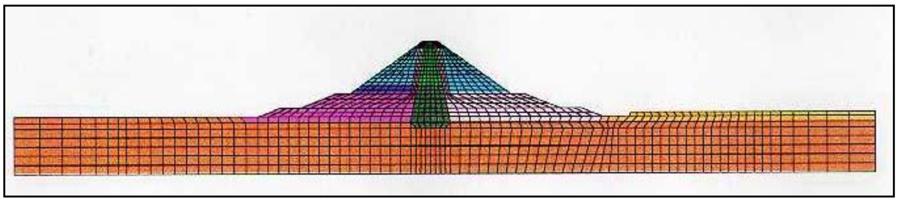


Figure 6. The F.E. mesh for dynamic analyses

The calculated accelerograms were applied at the base of the model. The earthquake (40 s duration) can reduce significantly the stiffness of dam materials: straining reaches the order of magnitude of about 0,01%, a threshold for a significant reduction in stiffness for many materials. This threshold was confirmed by resonant column tests run on specimens taken from the core material.

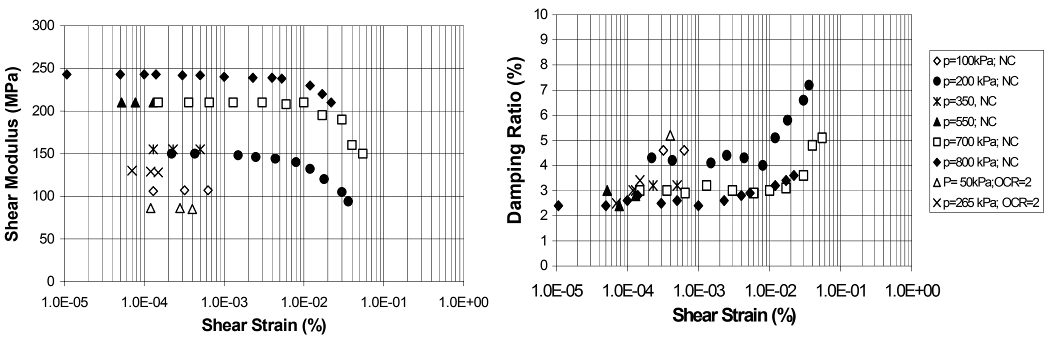


Figure 7. Shear modulus and critical damping by resonant column tests

As a further evidence of the decay, the modal analysis calculated 3.67 Hz for the first natural frequency before the seismic input is applied and 2.60 Hz at the end.

SEISMIC STABILITY EVALUATION

The evaluation of the safety margins after the shock were made by applying the Newmark method, which determines, within a limit equilibrium approach, the residual sliding displacement of a given portion of the dam body suffering a given acceleration record. In this case the accelerograms obtained by processing the actual records have been applied.

The most critical surfaces were determined again by the limit equilibrium method by Bishop (simplified with use of circular potential sliding surfaces), where the Italian regulatory seismic input was applied as a pseudo-static inertial contribution, based on a constant acceleration of 0.07 g. The approach is consistent to that used by the designer, who took into consideration static loads only.

In both assessments the same physical and mechanical material properties were used for the materials in the dam body and in the foundation. They are given in Table 2 and result from the design phase as well as from tests.

Table 2. Design physical and mechanical parameters

Material	**γ_d [kN/m^3]**	**c' [kPa]**	**φ' [°]**
Clayey silt	16.7	30	25
Rockfill shoulders	19.6	0	40
Gravelly sand	21.6	0	35
Alluvial soil (gravel and clay)	17.7	0	30

The water level was taken at the maximum operating (529.8 m a.s.l.) for conventional checks and to the much lower one (514.0 m a.s.l.) present when the earthquake took place.

The assessments confirm that the dam for most critical surfaces complies with the Italian regulation (1.4 and 1.2 for static and seismic conditions respectively). Critical surfaces located in the upper portion of the embankment have a reduced safety margin for static loads (1.17 compared to 1,4) and near to 1 for the seismic condition.

A thorough visual inspection of the embankment slopes did not show even local which could be attributed to sudden unstable conditions. It may be deduced that a higher shear resistance is available at the surface, where confinement due to overburden is a minimum (see Fig 8).

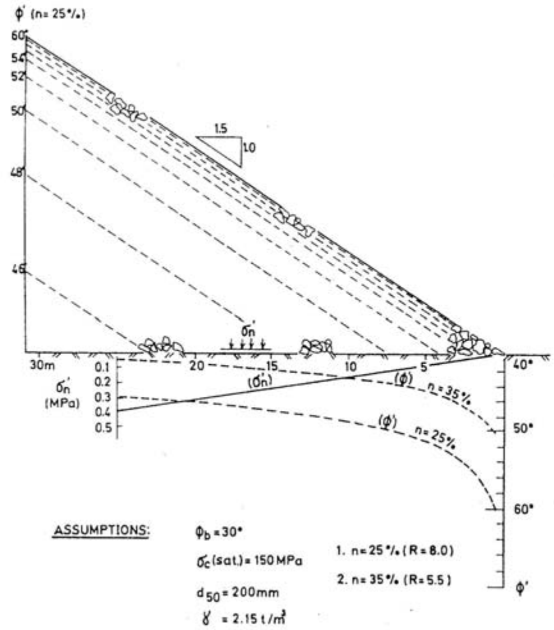


Figure 8. Available shear strength, as friction angle in a rockfill embankment approaching to the surface. σ'_n is the confining normal stress. Source NIT report

The effects of the actual earthquake have been evaluated by the modified Newton method (by Makadisi-Seed) The Newmark method allows the evaluation of the permanent displacement of a slope subjected to an earthquake, assuming that the motion occurs along arcs or planes as in the usual static analysis of stability. Direct integration has been used to compute the magnitude of the dynamic motions produced by the earthquake. The fundamental parameter in the analysis is the critical acceleration Kc, i.e. the pseudostatic acceleration corresponding to a unit safety factor against sliding in the limit equilibrium analysis.

The following steps were taken:

- For each critical surface the critical acceleration was determined at the centre of mass of the given portion of the dam body, defined as that bringing to unit the safety coefficient against sliding. This is the conventional value for unstable response to occur under the form of progressive displacements.
- By checking the critical value with the actual horizontal acceleration record of the given earthquake instant where the critical acceleration is exceeded are determined and the corresponding displacements cumulated.

The maximum critical acceleration value is calculated for critical surfaces originated nearby the crest and ending at the upstream portion of the embankment, and among them for those having the toe below the water table. In any case all the surfaces examined could withstand pseudostatic horizontal accelerations in excess of 015g.

The residual displacement reaches 15 cm, for the most vulnerable surface (Fig. 9) while, for most cases, such displacement is lower than 6 cm.

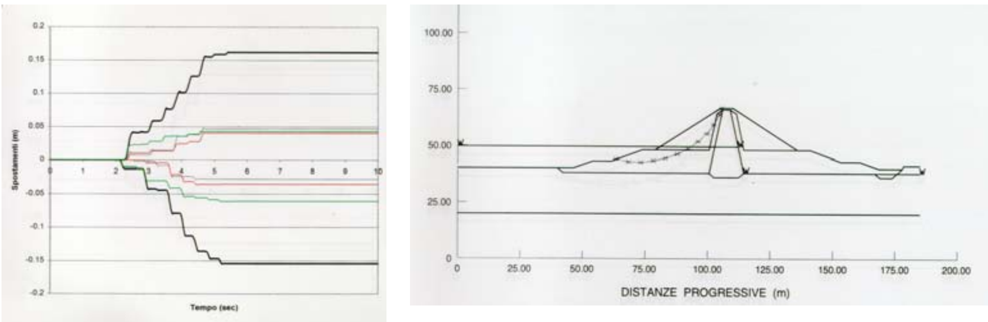


Figure 9. Residual displacements and a potential failure surface

The values obtained are modest if compared with threshold suggested in the literature (Lambe and Whitman for earth dams, NIT report), which are metric.

It may be concluded that the cumulative damage is moderate, and concentrated on the crest area. Considering that the actual earthquake can be associated to a SSE (rare), with a 475 year return period, the overall performance of the dam confirms the high safety margins incorporated in the Italian regulation even for static loads.By processing safety condition in terms of allowable acceleration, Paoliani concluded that millimetric to centimetric displacements can be associated with a OBE to SSE earthquake, which is characterised by PGA's well in excess (0.20-0.28g respectively) of that required by the Italian standards for dams (0.07g at the Acciano dam site). It was concluded that there were grounds for rehabilitating the dam.

EXPERIMENTAL ASSESSMENTS

Evidence of the state of critical materials in the dam body and foundations were acquired by an extensive site and laboratory investigation in support of the rehabilitation design. Emphasis was given to the core material, in order to ascertain its strength and stiffness properties. Self-healing and self-sealing capabilities, which mainly relate to the amount and the mineralogical composition of the clay fraction in the core material were of specific interest. They can counteract the possible formation of damage under the form of shear bands/microcracks, affecting therefore the fundamental barrier function of the core. Tests were also performed on the foundation materials, the alluvial soils and the rock itself.

Some *in situ* standard penetrometer and permeability tests (Lefranc and Lugeon) provided a framework for data obtained in laboratory on specimens. Test boreholes have been drilled from the crest to the foundation through the core, and some from the downstream berm (see Fig.2).

The laboratory test programme for specimens from the core consisted of: triaxial consolidated drained and undrained tests to derive frictional properties and undrained cohesion; direct permeability tests on undisturbed specimen to evaluate a possible anisotropy; load controlled oedometer tests to derive stiffness and permeability, to assess overconsolidation, and, finally, resonant column tests, for further assessment of dynamic properties to determine effects on the dam induced by the earthquake.

A few unconsolidated undrained tests were run on specimens of the alluvial foundation layer resting under the dam body. Uniaxial load tests were run on the foundation limestone rock to derive its mean strength properties.

RESULTS

The core is of a silty-sandy clay with gravel. The clay fraction increases, reaching 50% at increasing depths into the core. The upper material is much more dominated by the sandy and gravel fraction (Fig. 10).

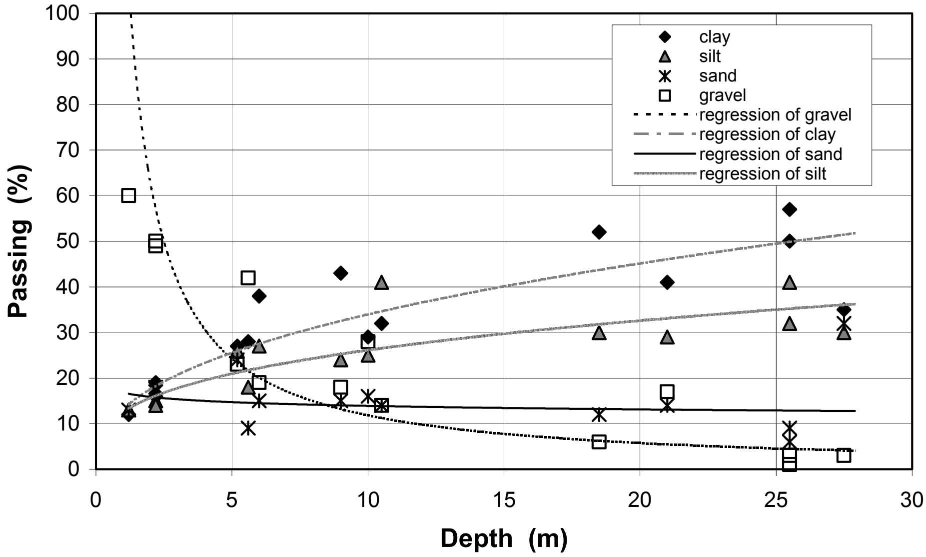


Figure 10. Material composition of the core by grain-size determination.

The clay is inorganic, of plasticity ranging from medium to high, CL (top portion of the core), CH (deeper core) according to Casagrande classification. The consistency index, 0.96, reveals a solid-plastic clay.

According to pocket penetrometers and S.P.T. tests, the core material is of good consistency, stiff in the upper part to very stiff in the deeper one, corresponding to a uniaxial compressive strength (UCS) ranging from 200 to 300 kPa.

The core material was found to be nearly saturated. The average water content is 24% and the wet density of the deeper core material is 19.3 kN/m^3, reaching 20-21 kN/m^3 in the upper, more gravelly portion. The core shows, from oedometer tests, some light overconsolidation. The estimated preconsolidation stress ranges from 400 to 500 kPa. Strength properties from drained and undrained consolidated triaxial tests run at several confining stresses in normally consolidated conditions can be described, according to the Mohr Coulomb criterion, as c' = 45 kPa and φ' = 23°. The same tests run on core upper material suggest a more marked frictional response, c' = 7 kPa and φ' = 34°, in Figure 12.

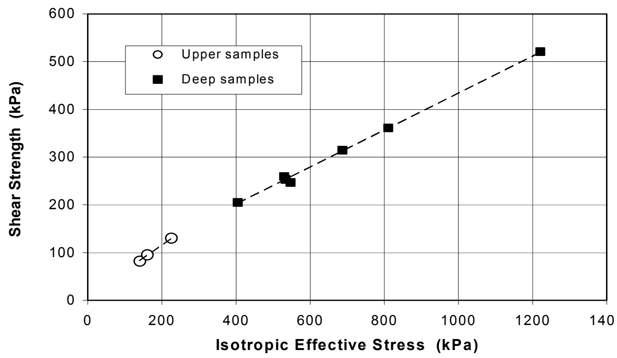


Figure 11. Shear strength vs. isotropic effective stress

Tests run on slightly overconsolidated specimens (OCR=2 corresponding to in situ conditions at the given elevation) show some, peak resistance. All considered, the overall strength envelope for the core can be defined by c'=15-45 kPa and φ'=27-23°, values adopted in the design stage are still represented.

Overconsolidation is weak and the general stress-strain response is quasi-ductile. Specimens collapse in triaxial tests at shear strains from 3.5% to 5% for overconsolidated specimens, and in excess for normally consolidated.

The undrained cohesion (c_u) has been found remarkably high, about 250 kPa. Some overestimation respect to values obtainable by other tests is due to the test bias. Upper specimens, which had to be reconstructed and consolidated to separate the gravelly fraction from the cohesive one gave lower, but still significant values (80-150 kPa).

The oedometric modulus is about M=30-15 MPa, the latter in the overconsolidated range within the prevailing stress state (up to 600 kPa). The consolidation coefficient is estimated as between 1×10^{-8} and 5×10^{-8} m^2/s. The shear modulus varies from 80 MPa a 250 MPa with increasing confinement stress (Fig. 12).

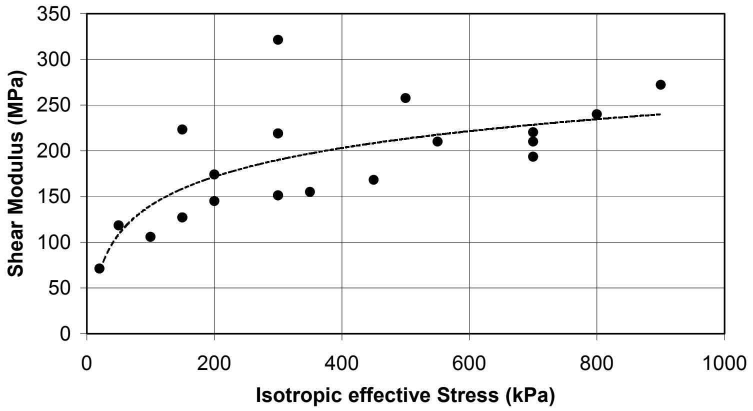


Figure 12. Shear modulus vs. isotropic effective stress by different tests.

Pin Hole test results indicate that the core material is not dispersive. The test has been run with distilled water; determination of chemical species in water taken at the dam site does not reveal any potential adverse effect with the clay minerals stability. Hydraulic conductivity from triaxial test run at several confining stresses and, indirectly, from oedometers, varies, within the stress range of interest (up to 600 kPa) from 1×10^{-10} (extrapolated) to 17×10^{-11} m/s (Fig. 13). No significant anisotropy of hydraulic conductivity has been observed. Lefranc tests indicate values from 5.0×10^{-9} m/s to 5.0×10^{-8} m/s for the core material.

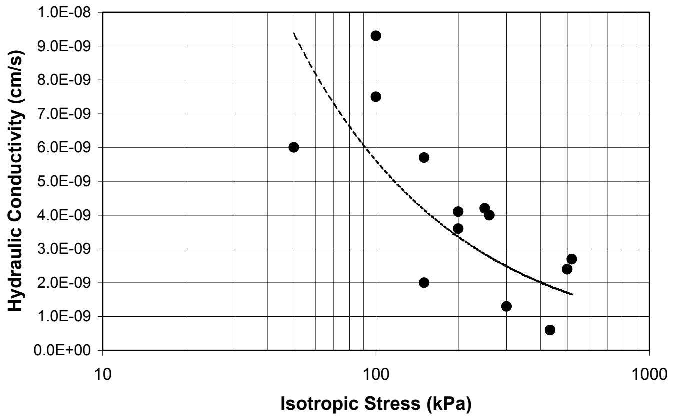


Figure 13. Hydraulic conductivity vs. the isotropic stress

Two principal soils have been identified in the foundation, one with a granular character, the other more cohesive. The latter has displayed undrained cohesion values of 70-80 kPa, and hydraulic conductivity (by Lefranc tests), similar to the average one of the core, 1.0×10^{-9} m/s.

DESIGN CONCEPT FOR REHABILITATION

The assessments of the impact of the Marche-Umbria 1997 earthquake and the properties of the core and cohesive foundation materials investigated by *ad hoc* laboratory tests and by *in situ* determinations revealed that the core retains satisfactory properties after the earthquake shock, which are very near to those adopted in the design phase. It is therefore justified to proceed with a seismic rehabilitation essentially based on providing additional confinement to the core, and higher margins of local safety against sliding to the slopes, by reshaping the embankment slopes. The above objectives can only be achieved by the addition of rockfill material.

The freeboard has been increased to 0.75 m, and the slopes shaped to reach l:2 upstream and 1:1.8 and 1:2.2 downstream. It is proposed to rebuild a small portion of the top of the dam body (the first two metres) . Some overburden is provided, at the downstream toe, to increase the factor of safety against piping.
Grouting is proposed to enhance the performance of the foundation materials, soils and rock, against seepage.

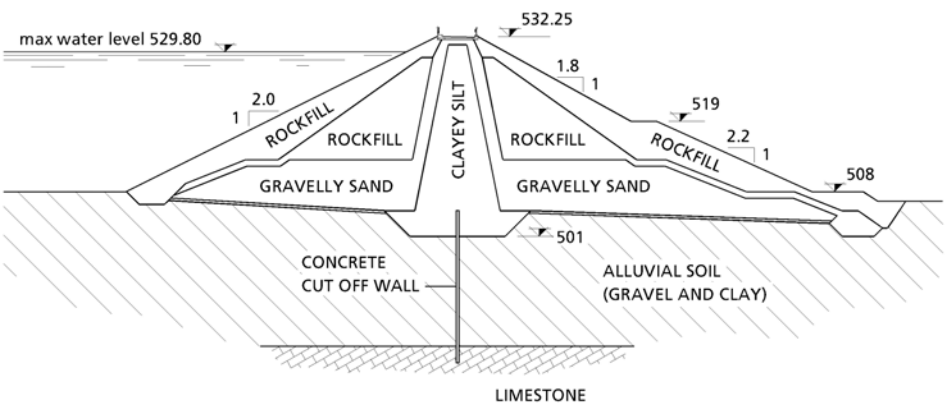


Figure 14. Rehabilitation design: main cross section

Data obtained from the tests allowed all the necessary analyses in support of the remedial works. These were basically:

- Stability checks of the dam body and foundation to comply with Italian standards.
- Seepage evaluation and checks for piping.

- Stability of structures within the dam body, such as the outlets, etc., and seismic checks on the structure of the gate tower.
- Check of punching of the concrete diaphragm into the clay core, during the earthquake motion and possibly reactivated by the consolidation effect of the alluvial soils due to the weight of the new rockfill material (more than 55000 m^3).

The design has proved compliant with regulations with regards to the above effects.

The assessments confirm that the rehabilitation project significantly improves the static and seismic safety margin with respect to the original configuration, varying from 40% for the surfaces located in the upper portion of the shoulders to 25% for deeper surfaces.

The evaluation of settlement of the dam, in the short and long-term conditions respectively, showed a maximum value of 6 cm and 11 cm. The maximum shear strains induced in the material core is 0.3% low compared with deformability of core material as observed in triaxial tests. The core material is able to withstand the overburden without displaying global or local damage effects. The check for local punching, before and after the seismic event, indicates that maximum shear stress (75-80 kPa) keep well below the undrained cohesion of the core.

CONCLUSION

The Acciano dam has provided evidence that factors of safety for seismic design incorporated in the Italian dam design code can effectively provide a significant seismic resistance capacity. The damage is confined to the crest of the dam and the condition of the core has remained suitable to allow remedial works to be implemented.
Such conclusions could only be made following a comprehensive testing and modelling programme outlining the critical role that such methods can play in assessing the current safety condition of existing dams.

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Lessons from a dam incident

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SYNOPSIS. Panel Engineers learn much when they are called out to deal with dam incidents. This paper attempts to share the lessons from an incident at an anonymous dam. Fifteen lessons are identified, which if followed, will lead to a greater understanding of the properties of dams and their behaviour if failure threatens. It is recommended that this knowledge be used to compile 'emergency handbooks' to equip those handling emergencies to take previously planned measures to minimise the risks to lives and property downstream and to release water quickly from threatened dams.

LEARNING FROM EXPERIENCE

Panel Engineers learn much when they are called out to deal with dam incidents. Our dams will present less of a threat if all concerned in dam safety learn from and react to these lessons. Taking a cue from the ICOLD (1974) publication 'Lessons from Dam Incidents', this paper attempts to share the lessons from a dam incident. The lessons are mostly not new. Some are already statutory, many appear in the embankment (Johnston et al, 1999) and concrete (Kennard et al, 1996) dam guides, and others were mentioned in the guidance on preparation of section 10 reports (Dams & Reservoirs, 2001). However, putting them together in the context of a dam incident makes their relevance and usefulness all the more obvious, and will, I hope, encourage all owners to prepare 'emergency handbooks' on their reservoirs to assist those charged with handling any emergencies to deal with them promptly and effectively, without making an already difficult situation worse.

ANONYMOUS RESERVOIR

The dam in question will remain anonymous, the lessons are not site specific and naming the reservoir serves no purpose. I would ask readers to try to live through the experience as I did and add it to their own experience, perhaps equipping themselves to deal with future incidents all the more competently. Of course, I recognise that my approach was far from perfect and many of the lessons that I learnt will not be new to all readers!

ANONYMOUS ACKNOWLEDGEMENT

I would like to say that the owners of the anonymous reservoir responded magnificently to the demands of the incident, maintaining close liaison with the police and emergency services, and providing, without hesitation, all the extra people and equipment needed to deal with it.

THE SYMPTOMS AND WHAT CAUSED THEM

The first lesson related to establishing the cause of the problem. I was called out because a hole had appeared in the downstream slope of a typical British dam. It was about one metre deep and about 800 mm in diameter, and looked to me like the surface expression of piping. We probed down and seemed to reach a solid bottom and we jumped on the bed of the hole and it didn't collapse, but I thought that perhaps the movements that had led to the formation of the hole had also formed some kind of temporary arch across the erosion pipe. No other reason for the formation of the hole seemed obvious. This and the fact that a rush of water in the culvert at the toe of the dam had been reported at about the time the hole had been spotted, led me to think that internal erosion was probably the cause. Had something triggered it, or had there been slow erosion for years that had finally manifested itself as the hole? BGS at Edinburgh reported no seismic activity and water level in the reservoir had been constant for months, and long-term slow erosion looked like the culprit.

I felt it was important to identify the cause of the problem because the incident could then be handled accordingly. But after pondering, experimenting and investigating and finding no other symptoms to convincingly confirm this diagnosis, I realised that it was not going to be easy to identify the cause. Worse, not knowing the cause, I would have to recommend further action after making a judgement on whether the hole had resulted from a single event that would not re-occur, or whether it was the result of some event that would re-occur or continue, possibly at an accelerated rate, and lead to escalating damage. Not a satisfactory situation for a supposed expert to be in, but a useful first lesson - there will be many unanswerable questions during the course of an incident, the first being what has caused the symptom triggering the incident, but even without knowledge, you must recommend appropriate action, usually erring on the side of caution.

LESSON 1 – It is rarely possible to quickly identify the cause of the symptoms that lead to incidents, consequently initial actions cannot respond to the cause, they need to be generally cautious

GETTING TO KNOW THE DAM
In an effort to see where the leakage causing the erosion was, the water level was kept high, better monitoring arrangements were put in place, and I studied drawings and read a paper about repairs done at the dam many years before. Here was lesson two - write down your experiences in a reputable journal. I found being able to 'talk' to my predecessors invaluable and eliminated much of the conjecture that inevitably arises during incidents.
LESSON 2 – Record your experiences

IMPACTS OF IMPROVEMENT WORKS AT OLD DAMS
The paper revealed that there was much stony fill in the dam. The hole may have resulted from settlement of this fill, because it had been wetted, perhaps for the first time, by water released after a huge thunderstorm from perforated pipes recently laid along the toe to drain the mitre. Lesson three – think carefully about the impact of supposed improvement works at old dams. You probably know relatively little about them, especially the fill in their shoulders, and therefore you know little about the impacts your safety works might have. Don't enter into even simple improvement works lightly. Incidentally, the perforated toe drains were replaced by unperforated ones, which reduced flows reaching the culverts during rain, as Figure 1 shows.
LESSON 3 – Think carefully about the impact of works to improve old dams

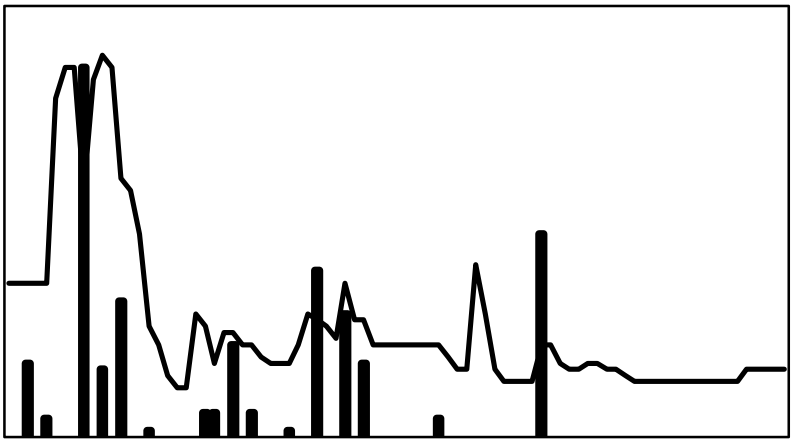

Figure 1 Rainfall and culvert flow over time

SAFETY FIRST
Then came lesson four. I realised that my forensic investigations had distracted me from the alarming fact that the real cause, whatever it was, might cause failure of the dam, releasing the water from the still full

reservoir on to the unsuspecting public downstream. I ordered the reservoir to be lowered forthwith.
LESSON 4 – Take precautions first, don't take chances.

RAPID DRAWDOWN
Lowering water level inevitably leads to the questions of how low and how quickly? On how quickly, I was very cautious because the paper had told the story of repairs to the upstream slope, and I didn't want to add to our woes by allowing water to escape from the reservoir through a slip of the upstream slope caused by too rapid a drawdown of the water level. The uncertainty could have been reduced if a rapid drawdown analysis had been done – lesson five. The guidance – 300 mm a day – is conservative in well-drained slopes (Reinius, 1948), but not conservative in clay slopes if the water level is drawn down a long way (Morgenstern, 1963). Modern numerical methods to analyse safe drawdown rates are also available (e.g. Dounias et al, 1996). All require knowledge of the fill in the upstream shoulder. In many British dams the shoulder fill is stony and well drained, consequently rapid drawdown rates will not cause failure. Some dams have clay upstream blankets as well as cores. Lowering the water level in these situations could leave a high water level between and the blanket may be ruptured.
LESSON 5 – Investigate fill in upstream shoulder and do rapid drawdown analyses

EMPTYING CAPACITY
The outlet pipework at the reservoir had an enormous capacity, though there was no rating curve. Lesson six – work out the opening v discharge relationship for the scour and other outlet valves. The maxima should be entered in the Prescribed Form of Record, Part 8, but a rating curve or table would be more helpful in an emergency situation.
LESSON 6 – Know how much water the emptying pipes can release

DOWNSTREAM RIVER CAPACITY
If I had not been constrained by drawdown failure worries, we could have lowered water level at a terrific rate, very re-assuring when there is a problem. However, the discharge might have gone out of bank and flooded properties downstream. Lesson seven – identify pinch points and low-lying properties near the river downstream, estimate likely in-bank flow capacity, send scouts out to pinch points when releasing water from the reservoir.
LESSON 7 – Know the capacity of the river downstream

CRITICAL RESERVOIR CAPACITY
The next question that arose was how far should we lower water level? Having postulated internal erosion, there was a possibility of a low-level

erosion pipe working back towards the reservoir, the symptomatic hole being a vertical tributary from it, and this could release the whole reservoir downstream. I thought that the consequences of this would decrease as the water level retained became lower; perhaps at some critical level there would be no significant damage downstream. The dambreak analyses (they had been done previously, otherwise another lesson would have been learnt) were dusted off and trials done with progressively smaller volumes of water escaping from the reservoir. The result? Even at 50% full, the number of properties at risk, while less than half of those when the reservoir was full, was still large, many up to 30 kilometres downstream. This confirms what we know (or it could be another lesson?), that the extent of dambreak impacts relates more to the river slope downstream of the dam, and therefore the flow velocity, than to the volume of water released. However, it would have been good to know about this before the emergency. At reservoirs with gentle downstream river slopes there might be a critical reservoir level at which no damage would occur downstream in a dambreak. Lesson eight – do dambreak studies with differing retained reservoir volumes to give guidance on how far to lower water levels in an emergency.
LESSON 8 – Know how much water needs to be released to reduce the threat to acceptable levels.

CONSTANT SURVEILLANCE
The emergency authorities had some direct questions. They needed to know when we would be able to tell them to evacuate people downstream. How would we know when the dam had started to fail? We expected that there would be accelerating movements, sounds of running water in the culvert, outbursts of dirty water into the culvert, subsidence of the crest and other manifestations of failure by internal erosion that we know about from the literature. The police noted that all these could occur at any moment. The order to evacuate depended on direct visual evidence. They instructed us to provide constant surveillance, with shelter and phones, at the dam. Lesson nine – faulty reservoirs should be under surveillance 24 hours a day, seven days a week. Provide experienced people, with a cabin and ready access to phones, they would be the ones who would trigger evacuation if failure seemed inevitable.
LESSON 9 – Faulty reservoirs should be under constant surveillance

DEFORMATIONS ON THE RUN UP TO FAILURE
The police questions about how would movements accelerate before failure occurred was a tough one. I have seen finite element analyses of movements prior to stability failures (e.g. Vaughan et al, 1989), but if internal erosion was occurring and the eroding water was somehow escaping without coming into view, what deformations could be expected and would there be any change in the rate at which they occurred. Also would they be

visible to the naked eye, or would they only be discernible by survey methods? Lesson ten – we should know more about deformations before failure, and what means would be required to monitor them.
LESSON 10 – How do dams deform on the run-up to failure?

SURVEY MOVEMENT MONITORING
We had set up monitoring pegs, read and co-ordinated in position and level daily by surveyors. As pre-failure deformation rates seemed (to me at least) something of an unknown, it was agreed that if the rate of deformation increased to be 25 mm or more between successive days, I would be called out to judge if failure was imminent. I relied on my innate knowledge of dams and earthworks to be able to make the judgement! I admit that by this time, when asked the question how likely did I think it was that failure would occur, I felt confident enough to say it was less that 50-50, mainly because there seemed no evidence of any serious signs of the causes of the damage, or of any changes in the dam's profiles.

The fact that we had simple pegs monitoring movement was re-assuring, and I recommend this is done in such situations, assuming that they don't develop rapidly, because it does provide evidence of malfunction or no malfunction, the latter being important in providing evidence to justify de-mobilising the emergency arrangements, when as often occurs, 'incidents' turn out to be non-incidents. Whether pegs should be permanently installed could be considered, though incident specific additions would probably be needed also. Lesson eleven – provide simple survey movement monitoring arrangements to monitor behaviour of dams during incidents.
LESSON 11 – Provide simple survey movement monitoring devices

FILL TYPES AND FAILURE MODES
While all these precautions were being dealt with, the matter of the cause of the incident, the 'hole', remained in question. If it was the top of an internal erosion pipe, the pipe below might be a sizeable cavern, and I felt that this precluded excavating into it. If a large hole was exposed, the excavator might fall into it. How would we stop an enormous hole forming, exposing the downstream side of the core, leading to its collapse and release of water, a terrible scenario? Could large quantities of filter be assembled and quickly shoved into the void if such a disaster struck? Such considerations had only one answer; don't dig into the hole until more is known about it. I have to confess that boreholes were to be done for other reasons at the dam, and this may have made such a decision easier.

However, if internal erosion were the cause, a knowledge of the fills, in the core and in the upstream and downstream shoulders, and the nature of the rocks in the foundation, would assist in assessing how vulnerable the dam would be to internal erosion. I knew something of the upstream fill because of the paper, and the core was puddle clay, but I knew nothing of the downstream fill, except that some, placed as part of the early repairs, was stony. The foundation, as seen in the culvert floor and in exposures near the dam, was open jointed sandstone.

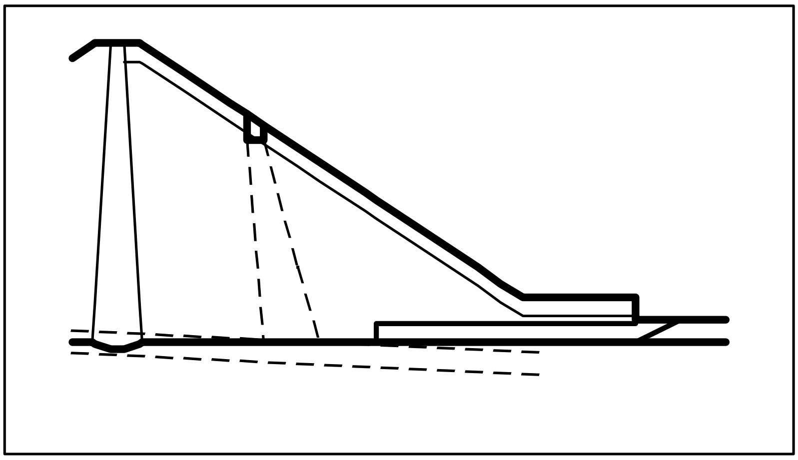

Figure 2 Erosion at interface between open-jointed foundation and fill; eroded materials carried downstream in joints in foundation rock, not visible in culvert. Subsidiary pipe leads to the hole on downstream slope.

As Figure 2 indicates, I postulated that water flowing in the open-jointed foundation rock might have slowly eroded through the base of the fill, including the core. Sediment in suspension was not visible in the culvert flows, but 'dirty' water containing eroded materials may have drained away below the foundation/fill interface and not been visible. An erosion pipe may have extended through the core, and the 'hole' may have been the top of a subsidiary pipe. This was worrying, but there were re-assuring signs, including no sign of local crest settlement (also being monitored by the surveyors) and no whirlpools in the reservoir (although they would not be formed if the upstream end of the erosion pipe broke out of the upstream shoulder at depth.).

Many of the anxieties about excavating into the dam and the potential for development of erosion pipes would have been dispelled if the properties of the fills were known. In our case the downstream fill was predominantly stone and gravel, not likely to be eroded, or to sustain open erosion pipes!

The property of the fills that can be most readily used is its in-situ permeability, easily measured in boreholes. Permeability values can be used

as a qualitative indicator of the texture of the dam fills. However, the permeability values can be used quantitatively in drawdown stability analyses of the upstream slopes to establish the safe rate of drawdown. In the downstream slope the permeability values can be used in the 'perfect' filter equation (Vaughan & Bridle, 2004) to determine the filtering potential of adjoining fills, the shoulder fill against the core fill, for example, thereby indicating whether the dam is capable of self-filtering or is vulnerable to internal erosion.
LESSON 12 – Know the properties of the fills in the dam, core and shoulders, particularly their permeabilities.

WARNING THE PUBLIC AND ASSISTANCE WITH EVACUATION
When we considered what to do should the dam fail, we expected that police and owners' staff would enter vulnerable areas and advise residents to evacuate, assisting them as necessary. However, we learnt that modern 'duty of care' obligations would preclude sending staff, including police, into a situation where their lives may be at risk. The concern was that the flood wave velocity, and therefore the period during which police and owners' staff could safely assist evacuees, was very uncertain. This made it impossible to devise any system of tracking and warnings that would be sure to get people clear without failing to meet reasonable 'duty of care' obligations. There may have been time to help those living far from the dam; those nearer would have to be assisted by other means - a 'sky shout' helicopter! People would be told to evacuate by a voice from the sky. Many wouldn't be expecting trouble and might not believe their ears. How would they know which way to go to be safe? Rehearsals might have helped, but were not advised in the context of a possibly imminent failure, they may have caused panic. Leaflets were precluded for the same reason.

An unsatisfactory situation, but it leads to a most important lesson, lesson thirteen – there is not time to safely evacuate those living close to a suspect dam after the flood wave is released, i.e. before failure has definitely commenced. They may have to be evacuated before failure has commenced. Some dam failures have been telegraphed by clear signs, (e.g. Baldwin Hills, ICOLD 1974) giving more time for evacuation, but this is not always the case.

Some of the advice and recommendations that the police gave may have been coloured by my earlier less than 50-50 decision. I certainly felt confident enough not to insist on any pre-failure evacuation - but this is a decision that we will be called on to make in future incidents. It would be more effective if it were pre-planned and those living in the floodway were made aware of it

LESSON 13 – The time available to evacuate those in the floodway will be limited, plan to start evacuation before failure has definitely commenced.

PREPARING FOR DAM DISASTERS

Although dambreak analysis had been done at the dam, the incident showed that the contingency plan to deal with such a situation was generic and not site specific. This leads to lesson fourteen – that it is necessary to make full preparations to deal with evacuations to avoid dambreak floods. The provisions of the Water Act (2003) empower the Secretary of State to call for 'flood plans' to be prepared at reservoirs and it seems likely that they will be required for all high hazard reservoirs, where large numbers would be at risk should a dambreak occur. The statutory requirement for flood plans should lead to effective plans being put in place for high hazard reservoirs. Owners will still need to make plans, appropriate to the hazard posed, for all their reservoirs. Clear plans are needed and the emergency services and the public should know the plans, and what actions they will need to take if a dambreak occurs. They may need to be made aware of them by rehearsals, signposts, talks, and whatever other means seem appropriate, and there may be a need for routine refreshers.

LESSON 14 – Make clear contingency plans to deal with dambreak risks, alert the public to them and train them to evacuate effectively

BEING PREPARED AND 'EMERGENCY HANDBOOKS'

And that leads to the final lesson – owners, advised by dam professionals, can do a great deal to reduce the impacts of dambreaks. We have worked together under the Act and the good practice that has been developed around it, to make dams safe. We cannot eliminate all risk but we can prepare to deal effectively with the residual risk.

We tend to think that dealing with emergencies comes down to mobilising the contingency plan and evacuating people at risk. But a great deal can be done by analysis and at the reservoir, completely within the owners' control, to be prepared for emergencies. This improves the possibility of being able to control situations effectively, reducing the numbers of lives at risk by prompt evacuation and perhaps completely averting dam failure. An 'emergency handbook' about the reservoir would do much to equip Panel Engineers and owners' staff handling emergencies to deal effectively with a crisis. They would be much more in command of the situation; they would not be experimenting. They would know the consequences of opening up valves and other actions, and could balance the risks.

The fundamental objective is to save lives by prompt evacuation and by emptying, or lowering the water level in, the reservoir as quickly as possible without exacerbating an already difficult situation.

A most difficult decision is ordering evacuation, because to be effective at most reservoirs it needs to commence before it is certain that the dam will fail. Present knowledge does not equip us to be able to quickly detect how close to failure a faulty dam is. Our trained instincts and the performance of the suspect dam probably help us on this. A few years of focussed research would likely lead to better guidance. All the other technical information needed for emergency handbooks could be assessed from published information, some of it dating from many years ago.

If we knew, as we entered an emergency situation, who should be evacuated promptly, how many turns of the scour valve would lead to out of bank flow at critical points downstream; how many turns of the scour valve would precipitate rapid drawdown failure of the upstream slope and how much water should be released to reduce the risk to an acceptable level; we would be in control. We could give the emergency authorities more specific advice. If our observations of the performance of the stricken dam warranted it, we could go into the 'risk zone' and take a chance on local downstream flooding and upstream slope failure if it would bring the water level down more quickly and thereby reduce the number of lives at risk.

Also, if we knew more about the fill in our dams, particularly its vulnerabilities to drawdown failure and internal erosion, we could provide permanent or emergency stand-by measures to deal with them, thereby further reducing the probability of occurrence of emergencies or failure.
LESSON 15 – Be prepared, by finding out more about our dams and assembling emergency handbooks to equip us to deal with emergencies as effectively as knowledge allows.

INCIDENT OR DISASTER?
The incident had a happy ending, the dam did not fail, the reservoir was refilled and has performed satisfactorily since. As indicated on Figures 3 and 4 below, I concluded that the hole had formed following deformation brought on by a combination of circumstances associated with new drains and wet weather. The heavy rainfall discharged from the (temporarily) slotted toe drain and wetted the stony fill, which had been kept dry by the deep layer of clayey fill on the surface of the downstream slope. The wetting caused some movement of the fill as points were wetted and collapsed. The clayey surface fill had been cut through at top and bottom of the slope for new drains, and may have stretched a little. There was some evidence in a trial pit of soil movement into voids in the stony fill, local internal erosion.

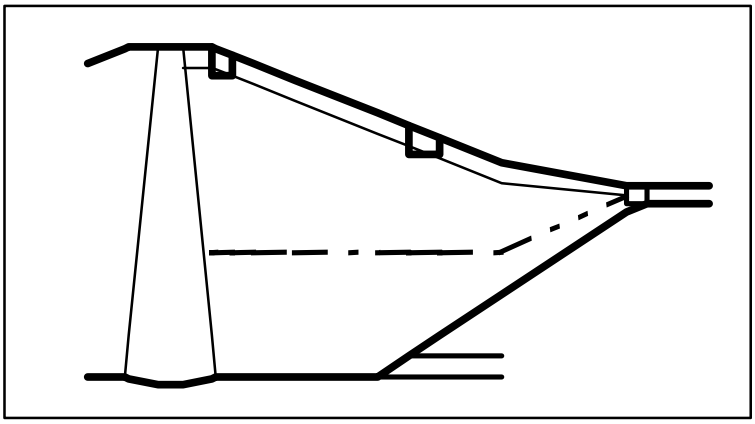

Figure 3 Cross section

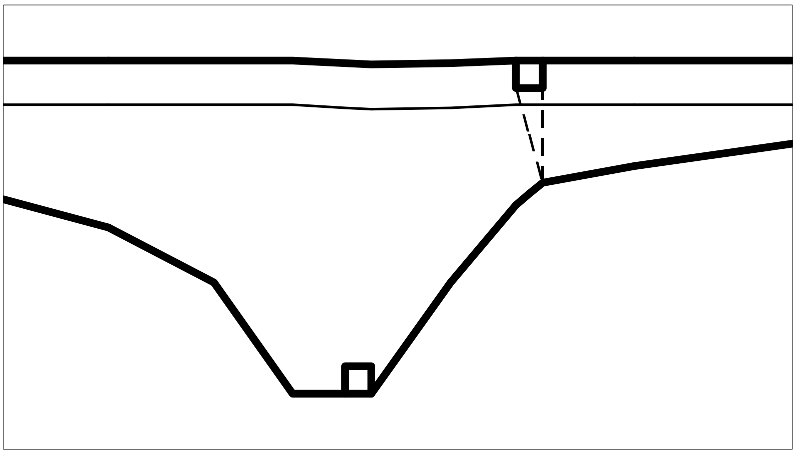

Figure 4 Longitudinal section

Figures 3 & 4 Conjectured mode of hole formation. Figure 3, cross-section, toe drain released water into stony fill, which may have settled slightly. Toe drain and crest drain cut through clayey surface fill, made slope less stable, allowed water in, surface fill moved. Figure 4, longitudinal section, foundation profile at hole position created differential settlement and adjustments in stony and clayey fill increased stress on clayey surface fill.

THE LIMITS OF OUR KNOWLEDGE AND USING WHAT WE KNOW

My final remarks relate to Panel Engineers and the expectations put on them. It would be easy to think that if you are appointed as a Panel Engineer, you must know enough about dams to respond expertly to any emergency. But as you have read, dealing with an emergency certainly revealed shortcomings in my knowledge, an admission shared, I imagine, by all but those entirely lacking in humility! While we will never know everything, it is disappointing that we do not routinely require safe drawdown rates and susceptibility to internal erosion to be checked using published techniques. Pre-knowledge on these issues would improve the safety of our dams and our effectiveness in dealing with emergencies. However, a proper sense of humility should not lessen our effectiveness in dealing with incidents. Those working with you will want clear and positive instructions as to what they are to do. I hope the lessons I've listed, when put into practice and assembled in emergency handbooks, will make that easier to achieve.

SUMMARY OF LESSONS

No	Lesson
1	It is rarely possible to quickly identify the cause of the symptoms that lead to incidents, consequently initial actions cannot respond to the cause, they need to be generally cautious
2	Record your experiences in a reputable journal, being able to 'talk' to predecessors is invaluable and eliminates much of the conjecture that inevitably arises during incidents
3	Think carefully about the impact of works to improve old dams
4	Take precautions first, don't take chances
5	Investigate fill in upstream shoulder and evaluate safe drawdown rate
6	Work out the opening v discharge relationship for the scour and other outlet valves
7	Identify pinch points and low-lying properties near the river downstream, estimate likely in-bank flow capacity, send scouts out to them when releasing water from the reservoir.
8	Do dambreak studies with differing retained reservoir volumes to give guidance on how much to lower water levels by in an emergency.
9	Faulty reservoirs should be under surveillance 24 hours a day, seven days a week. Provide experienced people, with a cabin or other shelter and ready access to phones.
10	We should know more about deformations before failure, and what means would be required to monitor them.
11	Provide simple survey movement monitoring arrangements to monitor deformations during incidents.
12	Know the properties of the fills in the dam, core and shoulders, particularly their permeabilities.
13	The time available to evacuate those in the floodway will be limited, plan to start evacuation before failure has definitely commenced.
14	Make clear contingency plans to deal with dambreak risks, alert the public to them and train them to evacuate effectively.
15	Be prepared, by finding out more about our dams and assembling emergency handbooks to equip us to deal with emergencies as effectively as knowledge allows.

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Comments on failures of small dams in the Czech Republic during historical flood events

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SYNOPSIS. During the catastrophic floods, namely those in July 1997 and August 2002 and also during local flood events, which occur almost every year in the Czech Republic, more than 100 failures of small dams were identified during last decade. After careful analysis of typical small dam failures, the reasons for dam collapses were found and assessed. During the flood events the most frequent failure of small dams was by breaching due to dam overtopping. The majority of small dam spillways suffer from insufficient capacity, inconvenient structure and arrangement. At some places spillways were blocked by broken gates, clogged jammed racks or floating debris. Moreover, in some cases the bottom outlets were not maintained and out of order. In the paper, several examples of unsuitable spillways and other dam appurtenances are shown.

INTRODUCTION

During the August 2002 flood, which affected the middle and western Europe, about 70 breached small dams were identified in the Czech Republic. This paper deals with the Blatna region in the south of Bohemia (see Figure 1), where 10 small dams were breached. The reasons for failures of two small dams in the area were analysed in more detail (dams of the Metelesky and Melin ponds). The failures caused disastrous damages in the villages of Metly and Predmir located downstream of the ponds, the breach outflow of two dams mentioned caused overtopping and failure of five small earth dams downstream of the ponds and finally flooded the town of Blatna (see Figure 2). In the analysis, the hydrological conditions in April 2002 were assessed in context with the capacity of bottom outlets and emergency spillways of both small dams. Finally the breaching mechanism was reproduced and the peak flood discharge was estimated based on comprehensive field data on the failure process during the night of 12th to 13th August 2002.

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

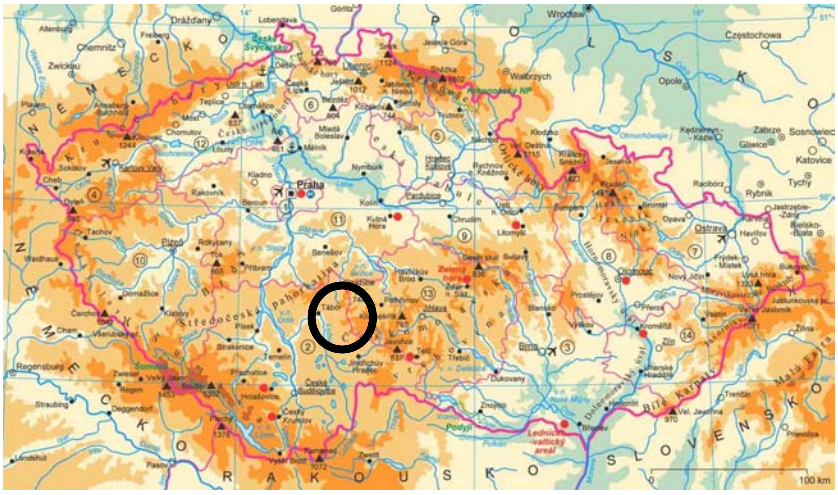

Figure 1: The map of the Czech Republic with the area of interest

Figure 2: Town of Blatna during the August 2002 flood

BASIC DATA

<u>August 2002 flood</u>

On August, 5th 2002 a cyclone developed above the Western Mediterranean, which proceeded northeast and reached the eastern Alps during August 6th. Heavy rainfall occurred over southern Bohemia with local showers of high intensity, which temporarily ceased on the morning of August 8th. After this cyclone the second one followed coming from British Isles to the southeast. On Saturday, August 10th the cyclone regenerated above Italy and continued to the north. During the 11th and 12th August the cyclone reached the Czech Republic, where the long lasting precipitation struck almost all the country. The most intensive rainfalls occurred in the mountainous regions to the southwest and northwest of Bohemia, and in the area of interest the three-day total was about 160 mm. On August 13th the rainfall intensity reduced and on 14th it completely ceased.

As the catchment soaked completely during the first precipitation event, the runoff percentage (runoff coefficient) during the second precipitation event was considerable. Due to the relatively long duration of rainfall an extreme flood was generated throughout the Vltava river catchment. Moreover, at some places local showers of considerable intensity caused runoff concentration at smaller streams. At bigger streams on downstream reaches the discharge exceeded the 500 year flood, and at smaller streams, especially at upper catchment portions it was estimated to be a 1000 year flood. This was the reason for the breaching of a great number of small dams with insufficient spillway capacity. Details of the dams breached during the August flood in the vicinity of Blatna are given in Table 1.

Table1: List of small dams breached in the Blatna region

Name of the pond	Dam height in metres	Total reservoir volume in thousand m^3	Reservoir area in hectares	Reason for failure
Belcicky	6.7	788	39.4	Overtopping
Buzicky	2.7	900	60.0	Overtopping of side dam
Dolejsi	2.6	334	30.0	Overtopping
Horejsi	4.0	232	22.4	Overtopping
Luh	3.8	48	6.0	Overtopping and improper outlet location
Melin	6.2	250	11.4	Slide of the downstream slope
Metelsky	8.5	1037	51.4	Overtopping at two places
Mlynsky	2.6	160	12.7	Overtopping
Podhajsky	2.9	225	15.0	Overtopping
Pusty	3.5	65	5.5	Overtopping

Details of the ponds studied

In this paper, the results of the analyses of only two ponds, namely Melin and Metelsky, are given. Both ponds are situated at the Metelsky brook about 12 km to the North of the town Blatna just upstream of the village Metly (see Figure 3). The catchment area of the pond Metelsky is about 15.5 km^2 and is covered by agricultural land (30%) and forests (70%).

The ***Melin*** dam is about 6.2 m high, the dam body is homogeneous, made of sandy clay with estimated hydraulic conductivity between 1.5 x 10^{-8} to 4.5 x 10^{-8} m/s. The upstream and downstream slopes are 1:1.5. The dam crest is uneven with 0.60 m differences in the crest level, and the lowest part of the dam crest is close to the bottom outlets. The dam crest is overgrown by trees and bushes. The root system of the vegetation disturbed the upper portion of the dam body, which is much more permeable than the lower part. The upstream slope of the dam is faced by stone pavement, and the downstream slope is grassed. The pond was equipped by one wooden bottom outlet with the maintenance and service shaft located at its upstream end (Figure 4). The dam is provided with two emergency spillways, one at the left bank abutment, the other at the right one. The total spillway capacity is 10.5 m^3/s for the water level at the minimum dam crest.

The ***Metelsky*** dam is about 8.5 m high with upstream and downstream slopes 1:2. The dam body is heterogeneous, created by upstream clayey blanket and sandy downstream shoulder. At the upper portion close to the dam crest the clayey sealing is missing or is degenerated by the root system of the plants grown on the dam crest (Figure 5). The upstream slope of the dam is faced by stone pavement, and the downstream slope is grassed. The pond is equipped with two wooden bottom outlets in a bad condition due to ruptures permitting seepage and rinsing of the sand from the dam to the pipes. At the left abutment the dam is equipped by an emergency spillway with a capacity of about 9.5 m^3/s. An auxiliary spillway (capacity 2.5 m^3/s) is formed by the local right bank road.

<u>Hydrological data</u>
Both ponds are constructed and operated as through-flow. The Melin pond is fed by three streams, the pond Metelsky is fed by two tributaries with total catchment area 15.83 km^2 with the peak level 712 m above sea water level (SWL). The N - year discharges at the dam sites are given in Table 2.

Table 2: N-years discharges Q_N in [m^3/s] at the dam sites

N	1	2	5	10	20	50	100
Q_N - Metelsky	5	7	10	12	15	19	23
Q_N - Melin	3.3	-	6.8	8.9	11.0	15.0	18.0

The flood hydrograph corresponding to the "natural" August 2002 flood at the dam profiles was derived using a rainfall – runoff model (Figure 6). The results of the modelling were compared with results obtained from the calibrated hydrodynamic model. The flood routing model calibration was based on the traces of the flood at the site. The flood routing in the area downstream of both ponds was considerably influenced by their collapse.

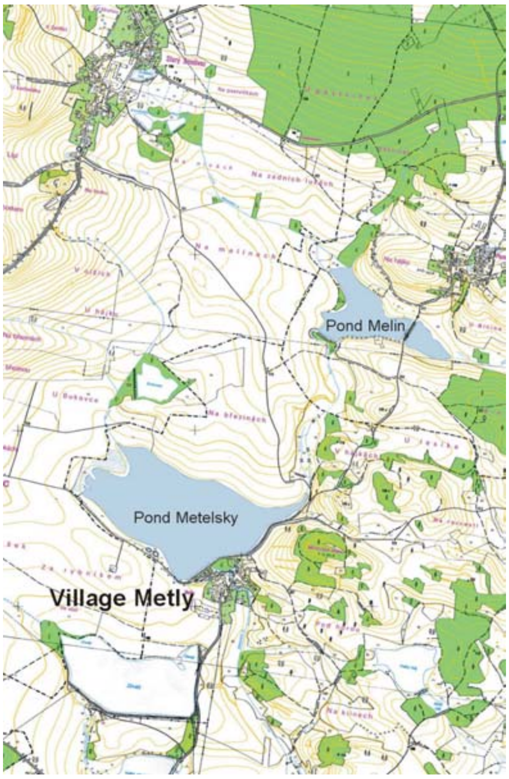

Figure 3: The detailed map of the ponds and Metly village

Figure 4: Melin – dam breach with remaining service shaft of the bottom outlet

Figure 5: Dam Metelsky - the cross section at the right breach

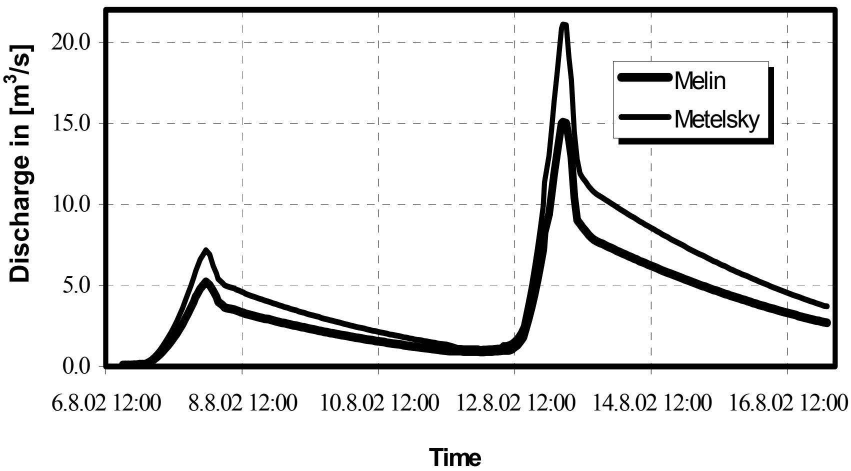

Figure 6: Derived flood hydrograph in August 2002 at the dam sites

ANALYSIS OF THE FAILURES OF THE SMALL DAMS

General comments

When comparing previous data about spillway capacities with flood hydrographs, it is obvious that the main reason for the dam failures was insufficient spillway capacity. Nevertheless, the purpose of the study was to provide a complex analysis of the event. Therefore, the following effects and their combination were assumed:

- dam erosion due to overtopping;
- loss of the dam body stability due to slide of the downstream face;
- internal erosion of the dam body.

The detailed analysis showed that the failures of both dams were caused by the combination of effects mentioned above. The analysis was carried out in following steps:

1. The reconstruction of the event using witness testimony provided by criminal police, local inhabitants and by the traces of water level at banks and upstream face of the dams.
2. The setting up of a numerical model consisting of rainfall-runoff, dam break and flood routing models. During this work, bottom outlet and spillway capacity rating curves were derived carefully.
3. The model calibration was based on the knowledge obtained in step 1. The calibration scenario resulted in the real flood and dam break discharges and possible reasons of the failures.
4. Finally, several aditional scenarios dealing with possible manipulation with bottom outlets combined with the temporary side spillway ‘on-site‘ installation were solved. The main goal of these scenarios was to prove that no measures were capable of averting dam failures.

The upper pond – Melin

Due to very low hydraulic conductivity of the homogeneous dam body, the seepage through the dam material was assumed to be very low. Anyway, the site investigation showed that the upper portion of the dam body of the thickness 0.3 to 0.5 m is composed of weathered grained humus material, the structure of which is disturbed by the root system of the vegetation (Figure 4). This material is of a significantly higher permeability. After the water level reached the higher position close the dam crest, more intensive seepage through the weathered layer probably caused the instability of relatively steep downstream face slope (1:1.5).

Detailed assessment of seepage conditions does not indicate suffosion trends in general. Nevertheless, the old wood pipe was found at the place of the breach, the rest of the pipe having been flushed down and dispersed downstream up to a distance 300 m from the dam site. As the wood pipe was quite old and damaged by cracks, it was concluded that the sandy material of downstream shoulder was flushed off by the pipe and caused the subsidence of the dam crest. This probably contributed to the dam failure.

The results of hydrological and runoff modelling indicated a rapid increase in the inflow discharge to the pond on the evening of 12 August 2002 with the peak discharge approximately 15 m^3/s (Fig. 7), which corresponds with 50 year flood (Table 2). The flood routing through Melin reservoir showed a transformed flood peak discharge 13.5 m^3/s and shift of the peak by 2.5 hours. At the same time, the spillway capacity was about 10 m^3/s at the water level at the dam crest.

The height of the wind driven waves was estimated to be between 0.55 and 0.60 m. The dam was locally overtopped by the wind waves for the period of 3 hours at the place with the lowest dam crest, i.e. at the location of the dam breach.

The Melin dam failure was caused by combined action of leakage through the upper portion of the dam below the dam crest and the wind waves overtopping the dam crest. These factors caused the dam failure in the section of the lowest crest, where its subsidence was probably caused by suffosion of the sandy material into the damaged wooden pipe. The process of breaching was accelerated by slides of the relatively steep downstream face of the dam. The dam break peak discharge at the dam site was estimated to be 150 m^3/s and this was verified by the calibrated flood routing model in the valley downstream of the pond. The resulting dam breach opening was of almost rectangular shape with the 5 m depth and 15 m width.

Dam at Metelsky pond

In case of the Metelsky dam, an overtopping was the primary reason of the failure. During the natural flood, the retention capacity of flood surcharge was exhausted due to malfunction of bottom outlets and unsufficient spillway capacity. At the same time the reservoir inflow increased considerably due to the breach of Melin dam located approximately 2 km upstream from Metelsky pond. Melin dam break peak at the inflow to the Metelsky reservoir was transformed to “only” 130 m^3/s, the flood wave volume corresponding to the Melin reservoir volume was about 600,000 m^3.

The detailed modelling of the event showed that the water level during the flood event was about 0.6 m above the spillway crest. At that time the Melin dam break wave entered the Metelsky pond and caused dam overtopping in two places. The resulting peak discharge was about 550 m^3/s, and the total volume of the flood wave was estimated to be 2.3 million m^3. The widths of two breach openings were 35 m and 27 m, the breach depth was about

7.7 m. Final dam break flood hydrographs compared with the hydrological flood are shown in Figures 7 and 8.

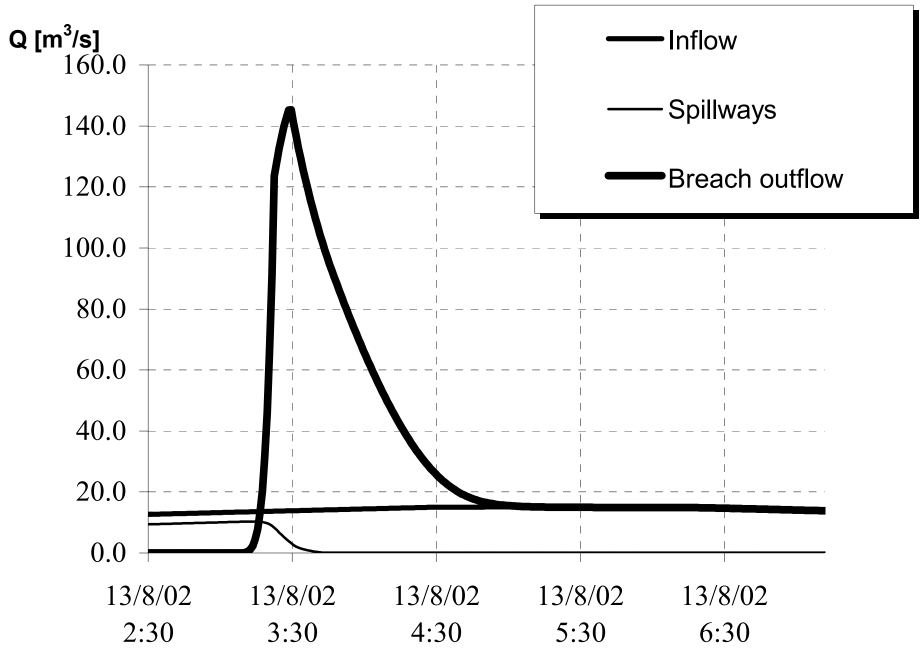

Figure 7: Melin reservoir - inflow and outflow

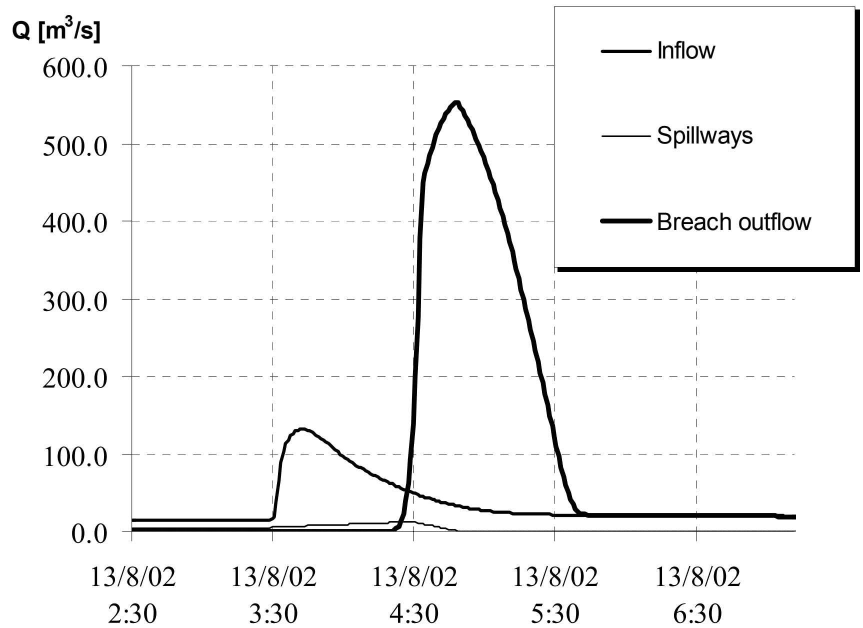

Figure 8: Metelsky reservoir - inflow and outflow

Several additional phenomena contributed to the dam failure and accelerated dam breaching. The 4 hours' action of wind waves of height 0.75 to 0.80 m and local overtopping caused local slides and disturbance of grass cover at the downstream face prior to the dam overtopping and this accelerated the destruction process of overflowing water. Moreover, the upper portion of the dam body was disturbed by the root system of the vegetation and by the action of animals. The left breach opening was located at the place of the old abandoned wooden bottom outlet pipe. The site investigation proved internal erosion and flushing of finer particles to the disturbed pipe (Figure 9) and consequent settlement of the dam body at this location.

Figure 9: The rest of cracked wood bottom outlet

The facts mentioned were not the primary cause of the failure, but accelerated the dam collapse and contributed to earlier overtopping. The important circumstance was inadequate technical safety surveillance of the dam and poor maintenance of dam body and equipment.

CONCLUSIONS

The causes of the failures of two small dams assessed can be summarised in the following statements:

- In case of Melin dam the failure was primarily caused by insufficient capacity of both spillways corresponding to 5years flood discharge (Q_5), while the peak flood was estimated as Q_{50} to

Q_{100}. Additional factors contributing to the failure were local slides of downstream slope, extreme seepage through upper loose portion of the dam, action of wind driven waves, potential privileged seepage paths and suffosion along the wood pipe outlet and malfunction of bottom outlet due to its improper structure.

- In case of Metelsky dam the failure was caused by an extreme hydrological situation combined with the breaching of the upstream dam Melin. The partial factors were practically same as in case of Melin dam.

The following conclusions and recommendations were put forward for further remedial and reconstruction activities at the sites of interest:

- Before the reconstruction of the dams, the revision and carefull surveillance of the state of dam bodies should be carried out. The restoration of the ponds cannot proceed in their present state. Careful assessment must be focused also on the design parameters of dam equipment, namely bottom outlets and emergency spillways.
- The reassessment of present safety classification of small dams should be done with respect to potential danger from insufficiently equipped small dams and based on the new dam safety standards and actual hydrological data.
- The manipulation regulations should reflect the optimal function of the entire reservoir system, which consists of approximately 20 small reservoirs in the Blatna region.
- The potential flood prone area specification should contain the inundation due to potential dam failures.

It is true that during local extreme flood events, on average from two to four small embankment dams (height less than 15 m) are overtopped and breached every year in the Czech Republic. During the extreme regional floods in 1997 and 2002 more than 100 small embankment dams failed and about 50 levees breached in the Czech Republic.

We recognize that the deficiencies mentioned in the structure, arrangement, parameters, operation and maintenance of small dams and their appurtenance are not rare phenomena in the Czech Republic (or in other countries). Remediating the present situation does, however, require time and money and it is also a difficult problem in relation to property and land ownership. Private dam owners (e.g. angling clubs) usually are not able to finance the remedial measures that are required. The state financial support is not systematic and is not steadily anchored in the present legislation, which in many cases is still not prepared for the private ownership of small waterworks.

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Detailed investigation of an old masonry dam

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SYNOPSIS. The gravity dam, described in this paper, is now over 100 years old. After such a long period of operation, a rehabilitation of the upstream face of the dam is necessary. During the last rehabilitation between 1965 and 1967 a protective shotcrete layer was applied on the upstream face. In the present rehabilitation the shotcrete layer will be maintained and a drained geomembrane will be installed. The geomembrane will be fastened by a system, which consists of inner and outer stainless steel profiles. These profiles are anchored onto the shotcrete layer.

Because the strata for fixing the anchors (the shotcrete layer) is now over 30 years old – and requires maintenance – major testing(e.g. georadar) of the shotcrete layer was carried out to give a guarantee of a sufficient bearing capacity of the anchor. The testing of the shotcrete is important to establish a basis for the design of the anchorage system.

This paper will describe the construction of the geomembrane lining with special consideration of the fastening of the steel profiles and the testing of the shotcrete layer.

INTRODUCTION

The dam was built by Prof. Intze between 1898 and 1900, and was one of the first Dams in Germany. The dam is used for the drinking water supply. The original dam had a height of 34 m and was designed as a gravity dam and is curved in plan (r = 176 m). In 1934 the dam was heightened to 38 m. The base width is 23.6 m and its width decreases to 4.5 m at the dam crest. The crest has a length of 215 m.

The dam consists of masonry with a volume of 47,000 m^3. The masonry consists of quartzite greywacke and a lime mortar with river sand and fly ash. Revetted masonry formed the upstream and downstream faces. The reservoir has a storage capacity of 2,855,000 m^3.

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

PREVIOUS REHABILITATIONS

There were two main rehabilitation activities in the past. The first was between 1950 and 1952, the second between 1965 and 1967.

Rehabilitation from 1950 to 1952

Investigations of the dam foundation showed that there was a high permeability on joints on the silty clayey slate, which were mainly rectangular to the dam axis. This explains a high water pressure in the dam foundation. Piezometer readings showed values up to 75 % of the hydrostatic pressure of the reservoir level. So after 50 years of operation the dam was rehabilitated by cement grouting, to seal the dam body and the dam foundation. The objectives were:

- Sealing the masonry and dam foundation
- Reduction of seepage water
- Avoiding the ongoing deterioration of the bond between the masonry and the lime mortar

Rehabilitation in 1965 to 1967

In the previous rehabilitation the sealing of the masonry was not satisfactorily achieved.

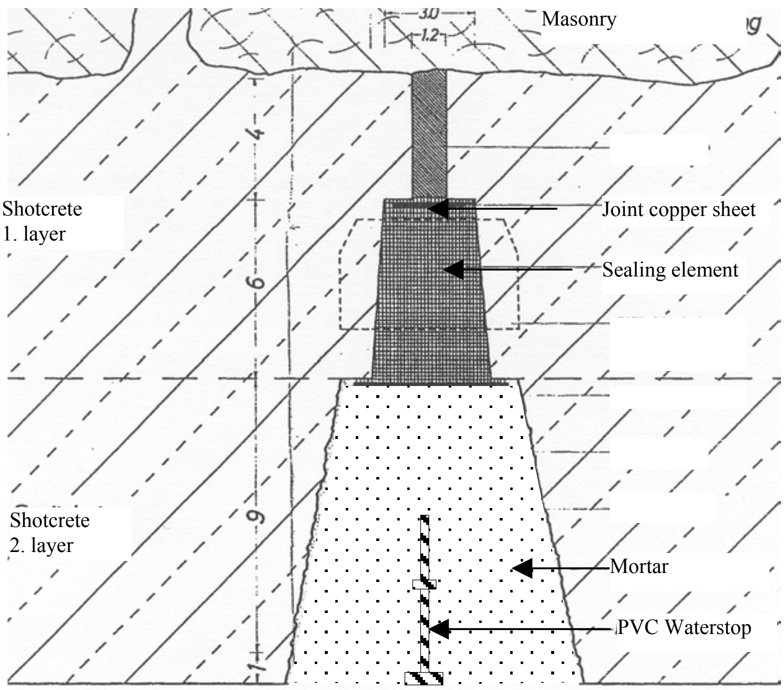

Figure 1. Cross section through joints of shotcrete layer

The main aspect of this rehabilitation was to seal the upstream face of the dam. The rehabilitation consisted of a reinforced shotcrete lining in panels of 5.0 x 5.0 m. Figure 1 and 2 show the shotcrete lining and the joints as a plan and as a photograph.

Each panel was anchored into the dam by 16 anchors with a diameter of 20 mm and a length of 2.50 m. The shotcrete was applied directly onto the masonry in two layers, each 10 cm deep. The joints between these shotcrete panels were formed with a sealing element and mortar, see figure 1.

Figure 2: photo of existing shotcrete lining

Despite these complex rehabilitations the sintering process deterioration was only slowed down slightly. As the joints were not really flexible joints the joints were disturbed due to thermal movement in the dam. Hence the water could leak through these joints into the dam.

CURRENT REHABILITATION

After over 30 years of operation since the last rehabilitation works in 1967, it was necessary to rehabilitate the upstream face of the dam. It was the wish of the owner to maintain the shotcrete layer. So the rehabilitation plan will be implemented by the following methods:

- Breaking out a gallery
- Installing drilled drainage holes
- Sealing the upstream face of the dam by application of a watertight geomembrane

A gallery in combination with drilled drainage holes is a very common method of reducing the subsoil water pressure and is thus not described in this paper.

Sealing concrete faces with a PVC geomembrane has been carried out now for over 40 years, especially on the upstream faces of dams, mainly either RCC dams or masonry gravity dams, e.g. the Brändbach Dam in Germany Veyhle/Jaup (2002) or Scuero/Vascetti (1996).

The geomembrane concept is in general quite simple: the synthetic impermeable liner extends over the whole area, which is to be sealed, not only joints or cracks. It is conceptually equivalent to a single waterstop, which covers the whole area. The geomembrane is attached to the structure as a separate element. The geomembrane is mechanically fastened by means of steel-profiles. The elasticity of the system allows it to bridge cracks that may develop due to external loads or changes in temperature.

The geomembrane consists of a flexible polyvinyl chloride (PVC) membrane, extruded in homogeneous mass from a flat die and heat bonded during manufacturing to a non-woven, needle punched geotextile (100% Polyester). The purpose of the geomembrane layer is to provide watertightness, while the purpose of the geotextile is to help to protect the geomembrane from puncturing and to give dimensional stability. The geomembrane is situated on the geogrid. This geogrid consists of a polymer–plastic, which serves as a drainage layer between the geomembrane and the dam. This construction enables a discharge by gravity of any drainage water that could infiltrate between the waterproofing liner and the dam body.

The Carpi PVC geocomposite is anchored by tensioning profiles on the shotcrete. The tensioning profiles belong to the patented Carpi-System, see figure 3. They are made of two parts, the inner and outer profile. The inner profile is anchored to the concrete structure by anchors. The outer profile is connected to the inner profile by means of a special adjustable threaded device. The connection of the two profiles creates a clamping effect to the geocomposite. Pre-tensioning gives the geocomposite best adhesion to the existing concrete surface.

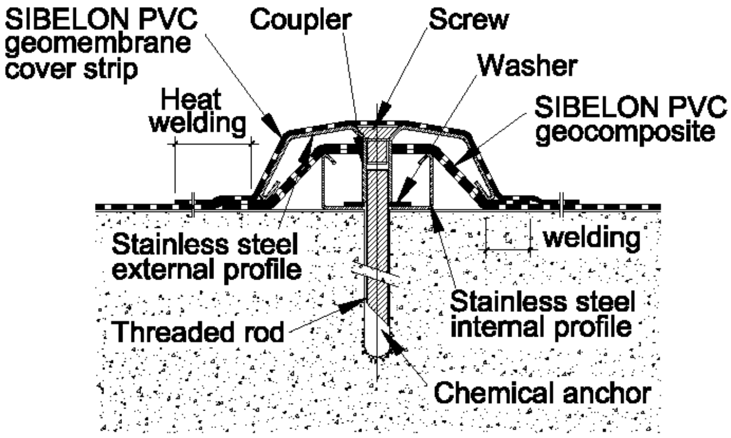

Figure 3: Cross section through pretensioning profiles.

INVESTIGATIONS OF THE DAM

As described in the previous section , the geomembrane is fastened onto the shotcrete lining by anchors. Hence the lining had to be tested in detail, as described in the following sections, to ensure a safe construction for the current rehabilitation.

Visual investigation

The whole surface of the upstream face of the dam was tapped with mechanical devices. A suspended platform was installed to allow access to the whole area of shotcrete. This first stage had the following objectives:

- Finding of areas where bond between the two shotcrete layers was poor
- Removal of loose shotcrete
- Marking on site of all repair areas

Findings

The visual inspection showed that the shotcrete itself was in good condition. Just one panel of the shotcrete layer was bad. In half of the panel there was no bond between the first and the second layer. Debris from the shotcrete was falling down during the tapping procedure. It was decided to remove the panel and to replace it completely.

In contrast to the shotcrete layer the panel joints were in a worse condition. This was because the joints were not flexible joints, see figure 1. Thermal movements of the dam caused high compressive stress in the joint material, which often resulted in cracks. More than two thirds of the joint material had to be replaced. The geomembrane needs a smooth subsurface with no protrusions, so these joints had to be reconstructed.

<u>Shotcrete testing</u>

For stability analysis of the anchoring system detailed knowledge of the shotcrete was necessary. Therefore five concrete cores with a diameter of 80 mm were drilled and analysed. The joint material was also examined.

Findings

- The concrete was classified as a concrete C20/25.
- All cores showed a steady microstructure. Also the transition zones between masonry and shotcrete and in between the shotcrete layers showed no voids.
- The geometry of the shotcrete layer could be confirmed: thickness 20 cm and two layers of reinforcement.
- In both the shotcrete and the joint material ettringite was found. Where ettringite fills the pores completely, an internal pressure arises and the microstructure is disturbed. As described above, the shotcrete is in good condition (no hollows, good microstructure), so the shotcrete will not be damaged by the ettringite. Figure 4 shows a microscopic view of the shotcrete. A pore with slight ettringite can be seen. Generally these very small pores are found in the shadow of the reinforcement. Further information on ettringite can be found in Jungermann (2000). In the joint material the pores were filled with the ettringite and the microstructure was destroyed. All the factors which lead to damage of the microstructure can also promote the formation off ettringite. The existence of ettringite crystals in concrete cracks is, as a rule, only a consequence and rarely the cause of the cracks, Stark/Bollmann (2000). This matches with the visual investigation, that the joint material was in a bad condition.

<u>Survey</u>

The entire dam was surveyed with a 3-D laserscanner. This scanner combines values from distances, angles and inclination to calculate 3D coordinates of each point of the dam. This method allowed a detailed design of the rehabilitation work considering the exact geometric condition of the dam.

Findings

This survey confirmed the general assumed dimension of the dam.

Figure 4: Microscopic view of shotcrete

Georadar

As mentioned above the geomembrane is fastened on to the shotcrete with anchors in vertical lines. To obtain detailed information of the shotcrete at each vertical line of a profile the shotcrete was surveyed by georadar, which had the following objectives:

- Recording of anchor plates of the shotcrete layer
- Recording of reinforcement (missing or overlapping reinforcement)
- Recording the structure of the concrete
- Recording the thickness of the shotcrete

In the Georadar technique electromagnetic impulses are radiated into the structure. Signals are reflected at interfaces between two materials (e.g. shotcrete - masonry) or at objects (e.g. reinforcement) and are registered at the surface. On the basis of the magnitude and the form of these reflections information on the structure can be obtained. The recording of the data is digital. First results can obtained during the measurement and a detailed evaluation including graphical presentation of the results will be made later. Figure 5 shows an example of the evaluation of the Georadar investigation

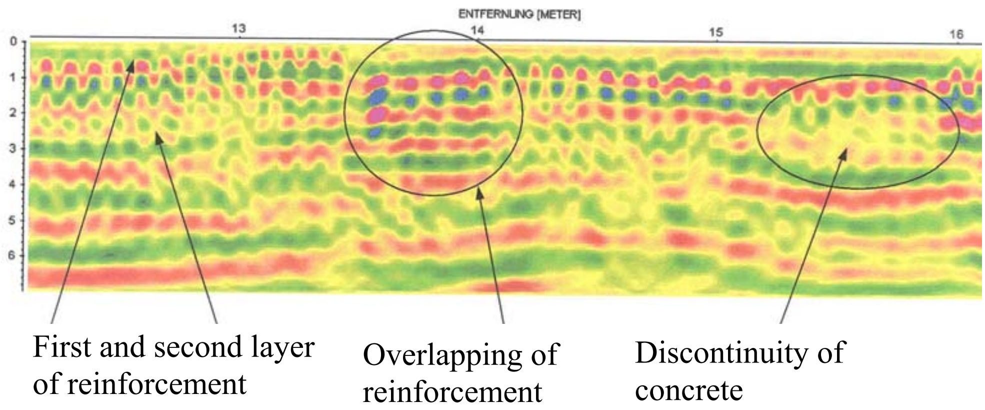

Figure 5: Example of evaluation of Georadar investigation

Findings

With the georadar survey the good condition of the shotcrete lining could be confirmed. There is a high consistency with the core drilling results, e.g. the thickness of 20 cm of the lining and the existing 2 layers of reinforcement. Discontinuities, e.g. the right side of figure 5, were in all cases minor.

CONCLUSION

As a result of all the investigations the main statements were:

- The shotcrete was classified as a reinforced concrete C20/25
- The lining has a thickness of 20 cm
- No big discontinuities were found
- The joints of the shotcrete lining had to be reconstructed

The results of all the investigations were satisfactory to decide that the geomembrane will be a safe and long-lasting construction. Parameters for anchor fixing were confirmed.

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The long term performance and remediation of a colloidal concrete dam.

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SYNOPSIS. Within Scottish and Southern Energy plc's (SSE) stock of concrete dams a small but nevertheless interesting subset is colloidal dams. SSE own three dams that are part formed using this technique. A brief history of colloidal dams is given followed by detailed information on particular problems at Loch Dubh Dam. Babtie Group (BG) is currently conducting studies leading to remediation proposals.

INTRODUCTION

Colloidal concrete or Colcrete depends on the production of a colloidal grout that is stable but highly fluid and can be injected into prelaid aggregate. The aggregate, from which all material below 1.5 inch must be excluded, is placed in position independently, and the grout is either poured over it and allowed to penetrate downwards, or introduced near the bottom through grouting pipes or channels and allowed to fill upwards. If correctly adopted the method ensures that the aggregate has point contact in all directions. The voidage is therefore less, and proportionately less grout, and therefore less cement, is required to fill it; thermal stresses are reduced, and cumulative contraction is prevented. Cement shortages were a significant issue during the early hydro development period and any reduction in use was sought.

This is a rare form of construction with known shortcomings in terms of performance when compared to conventional mass concrete, mainly due to the high water cement ratio required for placing.

LOCH DUBH DAM

Loch Dubh Dam is situated approximately 10 km north east of Ullapool and was completed in 1956. The dam is a concrete gravity structure of conventional profile, as shown in Figure 1 and set in a small steep-sided valley. It can be described as a medium sized dam in UK terms, due principally to its height of 20m (from lowest foundation). Much of the

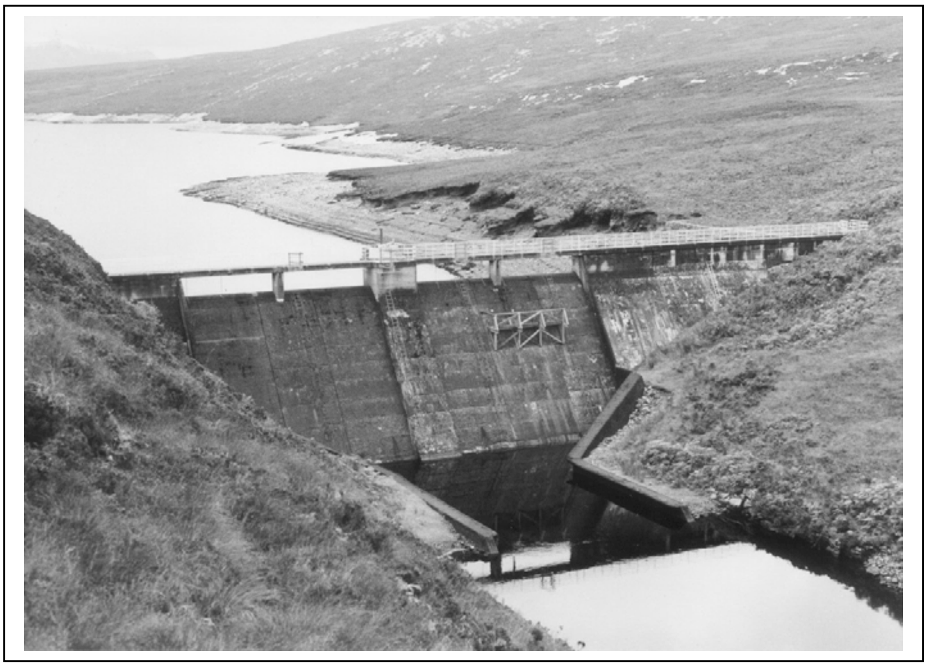

Figure 1: General View of Loch Dubh Dam

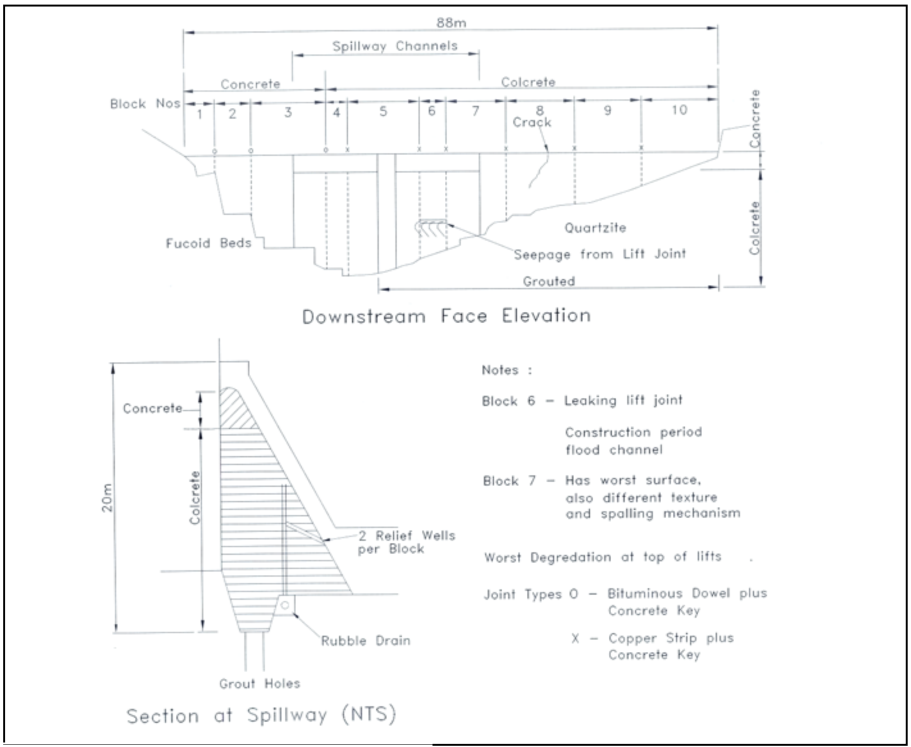

Figure 2: Principal Features

structure acts as the spillweir. The most remarkable feature of the dam is the method of construction using colloidal concrete. An associated feature that is equally unusual is the use of conventional mass concrete for a small number of monoliths on the right abutment. The reason for this decision is not clear although there is reference to a change in foundation rock type as a contributory factor (Refer Figure 2).

Where the dam is founded on quartzite, it is formed in Colcrete with a grout mix of 2 parts of sand to one part of cement by volume and aggregate in the range 37 mm to 225 mm. The overflow sill (upper 1.67 m) is formed in conventional structural concrete (19mm coarse aggregate).

Where the dam is founded on fucoid beds, the main body is formed in mass concrete (100mm coarse aggregate). The upstream and downstream faces have structural grade concrete 0.61 m thick, cast concurrently with the core.

OTHER COLLOIDAL DAMS

During construction of Mullardoch, which is a mass concrete gravity dam, changes in the economics of the day led to the design top water level being lowered and then raised again. The impact of this required a rather unique problem to be solved to thicken that portion of the dam already completed and to restore the 6.1 m removed from the original height. The method adopted for the thickening stage involved casting a slab over the downstream face of the dam, resting on precast concrete ribs on the sloping face but separated from it by a 0.91 m slot so that no bond could form between the two masses. After most of the contraction in the new concrete had taken place, the slot was filled with Colcrete, which was chosen as the material best suited to meet the requirements of minimum shrinkage coupled with good bonding qualities and, to a lesser extent, of high strength and high quality.

Difficulties were not surprisingly experienced during construction and rather than the more traditional form of Colcrete construction the coarse aggregate and grout were placed by “shooting” it into the slot through elephant trunking. Pre-mixing prevented the stones from breaking up and minimised the wear on the trunking. The water cement ratio of the grout was maintained at 0.9 with grout of 3:1 sand/cement mix and the coarse aggregate was crushed rock graded 89 to 63.5 mm. Test results on 57 cores taken demonstrated equivalent average cube strengths of 21.6 Nmm^{-2} and a density of 2387 kgm^{-3}. Current experience would suggest that although there are isolated areas of seepage and deterioration they are no more than those seen at other more traditional concrete gravity structures.

Tummel Bridge Aqueduct was substantially extended during 1957-1959 with the addition of a smolt bay using conventional concrete gravity construction and extension to the existing spillway portion of the aqueduct by buttressing. Colcrete techniques were adopted to form these sections which were formed directly on top of existing concrete gravity wall sections. Subsequently additional drainage holes have been drilled in an attempt to relieve water build up within the structure and to drain the interface between the concrete types. Deterioration of external surfaces due to freeze thaw action and spalling has progressed to an extent where localised concrete repairs should be carried out. The extent of this is perhaps greater than seen on conventional concrete gravity structures.

HISTORY & PERFORMANCE OF LOCH DUBH DAM

Over the years the structure of the dam has exhibited a range of defects. Some of these were not unusual on gravity dams while others were less common. Although disfiguring the external appearance none of the defects were judged immediately critical to safety, although regular monitoring was stipulated by Inspecting Engineers over the period.

The visual defects were surface breakdown and joint leakage while instrumentation revealed high pore pressures. Almost immediately these defects were attributed to the use of colloidal concrete.

The underlying process was believed to be a combination of high seepage through the mass concrete, poor drainage at the downstream face and the north facing aspect of this surface in an area prone to freeze thaw conditions.

These problems were obvious early in the dam's life, if only superficially, by the loss of surface finish. By 1964 the rate of surface erosion was causing concern and measures were put in hand to monitor the areal extent and depth of concrete degradation. These concerns increased during the 1970's and a limit of 75mm loss was set as a trigger for re-facing but by 1984 there was evidence that the process was slowing and to date the maximum loss is only locally at that limit.

Piezometers were first installed in 1968 (18No) as part of a research project into uplift pressures and revealed high levels in the downstream zone that indicated that enhanced drainage was required. The piezometers are simple standpipes. Many of them have been lost over the years and there are now only 6 in service. Although some isolated information is available from the early years the most consistent period of readings comes from the 1990's.

The trend of piezometer readings is not conclusive but shows signs of stabilising after increasing effective drainage in 1971.

Broadly speaking, the drainage gives an immediate drop in uplift of about 5m at full reservoir head (30%) with further dissipation towards the toe. The original series of vertical and horizontal holes have been reamed out where possible and supplemented with inclined connections. There is a possible link between improved drainage and decreased degradation of the surface although this is only a perception over time rather than a measured outcome.

Leakage from a lift joint, low in one of the central blocks, has been observed from the beginning. Accurate measurement of flow is not practical as it sits just above the drowned stilling basin and extends the full length of the joint. The rate of leakage is reported to be increasing.

There is a reasonable photographic record of the appearance of the dam from the 1960's to the present time (1961, 64, 66, 71, 72, 75, 81, 84, 2000 and 02) and this represents one of the best indicators of change in the degradation of the structure. Figure 3 shows a picture of a typical section of degraded face.

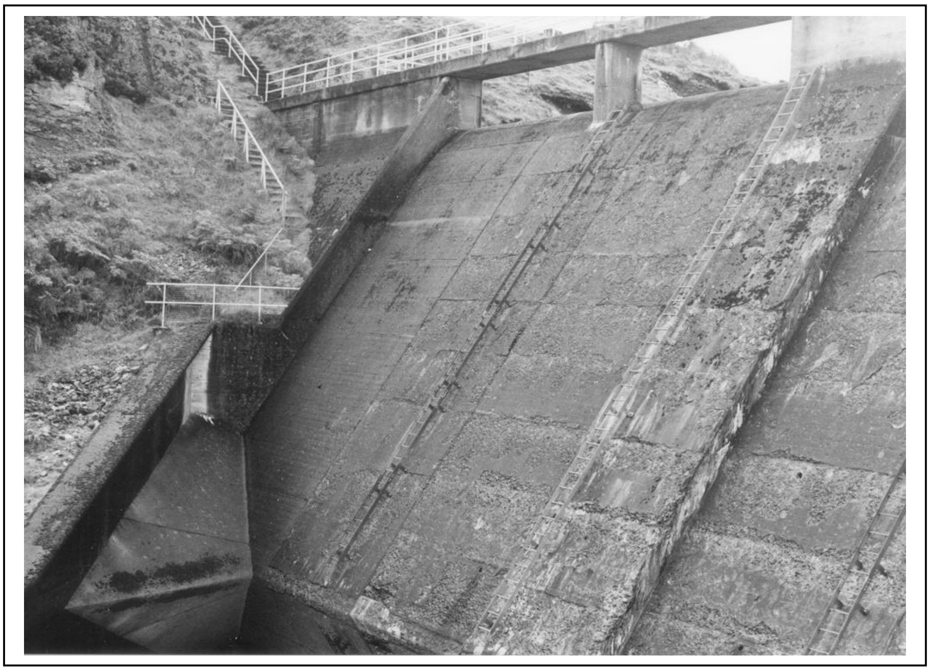

Figure 3: Varying face degradation, blocks 3, 4 and 5

The structure is routinely surveyed for settlement and lateral movement. There are no unexpected trends and the dam performs as would be expected of a gravity structure of this size.

The upstream face of the dam has been historically coated with a bitumastic paint although recently this has been allowed to degrade (in the expectation of an enhanced sealing coat).

While the possibility of making good the degradation of the downstream face by applying a finite surface thickness has been mooted it has always been accompanied by a concern that it might worsen the drainage/pore pressure regime even if a suitable application method could ensure adhesion and finish.

The current study has been triggered by the need to set a clear direction for the future maintenance of the asset.

DISCUSSION OF INDICATORS/INFLUENCES

The foregoing section on history describes the noted defects in general terms but as part of the current study the individual drivers were considered and analysed for likely impact. The principal indicators are as follows:

- Leakage
- Surface spalling
- Internal pore pressures
- Cracking

These physical outcomes are driven by a wide range of influences related to the design, construction and performance of this particular dam, namely:

- Colloidal concrete
- Acidic water
- Freeze thaw
- North orientation
- Construction sequence/standards/varying aggregate
- Porosity
- Face saturation
- Movement

The use of colloidal concrete is the first and most obvious deviation from standard gravity dam construction and is thus the prime suspect for poor performance. Although used on several dams in the UK and in the US during the 1950's the use of this technique was not sustained and it is now recognised as having poor strength and impermeability, factors that are likely to affect durability, particularly in an impounding function within a testing climatic environment. Thus the use of colloidal concrete might be expected to contribute directly to leakage and internal pore pressures. A secondary combination of saturation and weakness could also be contributing to the surface spalling. However, cracking is not likely to be a feature of this technique as it was recognised as reducing shrinkage effects.

Being an upland reservoir, acidic water from peaty soils is an influence with a known deleterious effect on cementitious matrix, the more so on weaker mixes. This might be expected to have an impact on the surface finish, particularly etching of the upstream surface but might also be impacting on conditions along discontinuities e.g. open lift joints.

The exposed surfaces of gravity dams in the Scottish highlands are prone to spalling due to frost action, especially if constructed during winter conditions. Additionally, saturated concrete is vulnerable to surface degradation during freeze/thaw cycles, the more so if weak and porous. Although not at a particularly high elevation (190m) the North facing aspect of this dam suggests that it might face the most severe freezing conditions. This might be supported by the difference in condition between the downstream face and the south facing, and submerged, upstream face.

Consideration of the construction details and sequence also reveals potential influences on performance and durability. In general there appear to be a number of variables in the way in which the work was carried out. This included considerable variation in aggregate size, aggregate type and lift height from pour to pour. Present condition also suggests variation in matrix texture (either cement, fine aggregate or additive). Sloping shutters are always an area of construction problems and may be more so with an injected cement matrix. This could explain differences between upstream and downstream face performance. Recognising the way in which colloidal concrete was formed it is perhaps not surprising to find the top edge of lifts showing a preference for early spalling. In some cases lift heights seem to be rather shallow for the technique. The combination of these influences might account for the most severe areas of degradation appearing on the downstream face. More particularly, in common with most gravity dams, a section of the structure was left low to pass construction period floods. A check on the records reveals that this corresponds to the badly leaking lift joint on block 6. While this surface may have suffered from exposure it appears also to have been compounded by the use of a very low lift in the pour immediately below the surface.

Although there is uncertainty over the reasons for the use of conventional concrete on the right abutment monoliths it appears to be related in some undefined way to the change in foundation rock type. However, it seems improbable that there was significant variation in loading response across the foundation thus cracking or movement due to uneven loading seems unlikely. In relation to the one notable crack (crest of block 8) it should be noted that the upper section of the dam (3.3m below crest) was constructed of conventional mass concrete throughout the length of the dam. As this was one of the wider monoliths it may be that the relatively slender block of conventional concrete suffered excessive shrinkage.

Thus it can be seen that there are a number of recognisable reasons for each of the defects indicated at Loch Dubh Dam. The key questions at this time relate to the significance of these processes and their likely progression with time.

Of the defects and processes identified the loss of mass on the downstream face was potentially the most serious in terms of safety of the dam and certainly the one that had been causing most concern over the life of the structure. The other defects (discrete leakage, pore pressures and cracking) although undesirable were not unusual and could be addressed by normal remedial works. However, all of the processes had consequences for the operational life of the structure, some with immediate impact on perceptions of care and others on long term maintenance cost.

Consideration of the above led to a programme of assessments and additional testing to determine the present impact of these processes and the likely consequences of further development. The defining indicators of current impact and future performance loss were identified as follows:

- Stability checks for assumptions of face loss
- Chemical change in seepage water
- Time dependant or event specific degradation of the surface concrete

DISCUSSION OF POTENTIAL OUTCOMES

SSE require a positive direction for future asset maintenance. This programme needs to be underpinned by a comprehensive review of the information available and a clear audit trail from adverse indicator to solution.

The key issue here was whether the defects and processes noted were largely superficial or whether there was a risk of long term erosion of safety margins or functionality. While safety is paramount, the operational function of a dam structure is also a serious issue for a commercial organisation.

Although possibly only cosmetic, the appearance of the degraded surface is an important issue of public perception for SSE. The impression given to senior company management was also relevant in that it reflects the level of investment and care applied to the dam stock of the company.

Upon site inspection in October 2002 and initial review of the records, an immediate impression was that the degradation process was in decline. This was very much a snap judgement and it was necessary to develop a realistic means of checking the validity. This impression did however seem to agree with the most recent periodical reviews carried out by Inspecting Engineers. Central to this was deciding whether the degradation process was time limited or time dependant.

While some information is available on concrete parameters there is nothing definitive within an advancing timeframe. Concrete cube strengths are available from construction in 1955 and there are a few core samples from the piezometer installation in 1968 that were tested for compressive strength. Together with recent tests it can be said only that strength is more than adequate for mass concrete but that there are significant variations across and through the structure (min approx. $15N/mm^2$)

TESTING PROGRAMME AND FURTHER ASSESSMENT OF HISTORICAL INFORMATION

The testing programme was designed to be as simple as possible and manageable with hand-held tools.

While overall compressive strength of the mass of concrete was of interest (for comparison with construction period tests) and would serve to indicate seepage related deterioration, the more important characteristic was the durability of the surface concrete, in particular, whether surface weakness was a one off construction feature rather than a function of continuous exposure.

It was decided that tests could be made for surface hardness and repeated after treating the surface in a defined way. It was also considered possible to core into the body of the dam from the downstream face sufficient to penetrate surface effects. In addition samples were taken of leakage flow for comparison with reservoir water.

Three test panels were exposed, each 1-2 m^2 in area. The approach chosen was to use a Schmidt hammer as a coarse indicator of surface hardness and to use this as a proxy for durability (it is acknowledged that this is using the Schmidt test outside its validated operating methodology). For each panel a matrix of hammer tests was carried out, firstly with the surface as found, followed immediately by a similar set after scabbling back to sound concrete. The tests were then repeated on this cleaned surface after a winter exposure.

Briefly summarising this exercise it was found that the exposed downstream panel was most degraded, followed by the upstream panel with the unexposed downstream panel least degraded. The follow-up tests on the cleaned surface revealed an immediate improvement with little deterioration between second and last tests.

The core strength tests revealed that there were variations in concrete strength but that these were not necessarily a function of seepage rather a reflection of general variability in concrete quality.

A review of stability revealed that the design section had a reasonable tolerance to loss of face material (no unacceptable reduction in FOS up to 150mm loss of material – current maximum of 75mm).

POSSIBLE / LIKELY REMEDIAL ACTIONS

There are broadly two approaches; firstly a universal upgrading of the structure that might be regarded as anticipating future problems. Alternatively, specific solutions to individual defects assuming that the extent of the problem has fully revealed itself.

The long-standing proposal has been to line the upstream face with a membrane. This would have the benefit of dealing not only with point leakage but with general porosity, the associated saturation and its link to face degradation. It would however require a full draw-down of the reservoir (although this would not be difficult as Loch Dubh does not feed direct to generating plant) and more significantly removal of sediment from the toe of the upstream face. This solution assumes that the degradation is essentially porosity/seepage driven and that loss of face material would continue without a global improvement in this characteristic.

The alternative on the upstream face is to deal with the one serious leak along the lift joint and to continue the practice of painting the exposed concrete surface. Depending on the level of the leaking joint in relation to sediment deposits the leak repair work could be carried out by diver without dewatering (using a mechanical clamping system with a strip membrane). Although a replacement bitumastic paint system is envisaged along with some limited concrete repairs to the upstream face, support for this solution depends on evidence that the degradation process is in decline.

In both cases the present level of degradation on the downstream face either has to be accepted or is treated in some way to reduce its visual impact.

As with many reservoir maintenance problems a definitive analysis is not possible and the chosen approach has to rely on an element of judgement.

CONCLUSION

Loch Dubh Dam has a number of visible defects that detract from its appearance and give an unjustified impression of neglect. Most of these features have been in evidence for the majority of the 48 year life of the structure. Despite concerns throughout its life that some of these defects were progressive the current assessment leads to the view that the processes are either in decline in the case of the face degradation, or can be addressed by relatively simple actions in the case of the severe joint leakage.

To be confident of the decision reached the situation has to be regarded from a number of perspectives: reservoir safety, asset performance, asset life and appearance.

In terms of reservoir safety the rate of degradation does not pose a threat to stability and the leakage is not unusual nor is it significant in uplift terms. The crack is not in a critical location and appears dormant.

As with all assets the residual life is of prime importance if the enterprise is to be sustainable. In the case of Loch Dubh Dam there is no reason why the structure should not remain effective for another 50 years even if current face degradation is doubled in extent. Routine maintenance and some remedial works are necessary to sustain that situation.

The appearance of the dam is one of perception and unless studied at close quarters does not raise any great concern. However, the philosophy of SSE is that all their structures should give an appearance of robustness and reliability even if not directly in the public eye. On this basis the current appearance of the dam does not meet the owner’s criteria and some action is necessary to improve the situation.

It is implicit that the critical indicators identified here are kept under review, not only in the routine surveillance of the structure but at times of statutory inspections such that the current assumptions are regularly reconfirmed or alternatively that a change of conditions requiring action is identified.

At the time of writing the decision has been made to proceed with the specific actions necessary to reduce the principal defects and specifications will now be prepared for leak sealing, sealing coat and downstream face stabilisation.

Some problems at small dams in the United Kingdom

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SYNOPSIS. The UK has a large number of small dams with diverse problems. Some have capacities of less than 25,000 m^3 and therefore fall outside the ambit of the Reservoirs Act, 1975. Many were constructed in the eighteenth and nineteenth centuries to impound ornamental lakes for stately homes. Whilst these reservoirs often require rehabilitation to meet modern safety standards the available funds are frequently tightly constrained.

This paper describes a number of recent case histories chosen to illustrate the breadth of issues which tend to arise, the nature of the solutions adopted and the lessons to be learnt. Some general principles are presented with regard to the rehabilitation of such structures.

INTRODUCTION

The average age of dams for which a construction date is given in the Building Research Establishment Register of British Dams is 100 years (BRE, 1994). 38% have a capacity of less than 100,000 m^3. Of these many are in private ownership and rarely generate sufficient income to pay for inspections under Sections 10 and 12 of the Reservoirs Act, 1975 or for improvements and remedial works.

In February 1986 the Department of the Environment wrote to Panel Engineers urging them to “keep expenditure to a scale justified by the risk” and stressing the importance of amenity, recreation and wildlife conservation. Inspecting Engineers therefore have to steer a careful path between permitting reservoirs to remain in an unsafe condition and imposing demands so onerous (and expensive) that the owners have no choice but to take the reservoir out of service. Of course there is sometimes the option to reduce the capacity of the reservoir to less than 25,000 m^3 and then discontinue it under Section 13. This is almost always to be preferred to abandonment under Section 14 because it dispenses with the need for a Supervising Engineer and 10 yearly inspections.

The following case histories illustrate some of the issues that arise:

Case History No. 1 – Upper Hartleton Farm Reservoir

Upper Hartleton Farm reservoir has a capacity of 59,000 m^3 and a catchment area of 11.5 km^2 . It was constructed in 1972 at the same time as Lower Hartleton Farm reservoir immediately downstream. The dams were built of silty clay and performed satisfactorily except for the regular appearance of cavities and internal erosion behind the spillway walls. The cavities appeared at the Lower reservoir in 1979 and 1998 and at the Upper reservoir in 1978, 1995, 2000 and 2003. Since the dams have shown no problems along most of their length it is thought that poor compaction of material adjacent to the spillway walls was the cause of the difficulties.

Figure 1. Axial wall on left side of spillway at Upper Hartleton Farm – leakage was taking place beneath the bottom of the wall.

Following the appearance of the most recent cavities at the Upper reservoir the Supervising Engineer recommended that the Section 10 Inspection due in 2004 be brought forward to 2003. The Inspecting Engineer was concerned about the repeated appearance of cavities over the years and about possible serious problems if heavy leakage were to coincide with the passage of a flood since the river passes through a town in a small culvert a few kilometres downstream of the dams. Consequently he recommended that the fill behind both spillway walls be dug out and replaced with well compacted clay. This was done in October 2003; during the work leakage channels were found in the excavation. At the time of writing there has been no further leakage.

This case history illustrates the importance of good compaction adjacent to structures and of keeping careful records of the behaviour of dams over a long period (in this case 30 years). It also illustrates the desirability of maintaining continuity of Inspecting and Supervising Engineers.

Case History No 2 – Weldon Lagoon, Corby

Weldon Lagoon was built as a flood alleviation reservoir by the Corby Development Corporation twenty five years ago. It had 1 No 1050 mm diameter pipe, 2 No 900 mm diameter pipes and three smaller pipes entering the Lagoon from an urban catchment of about 0.92 km^2 but only 1 No 600 mm diameter pipe controlled by a 225 mm x 225 mm penstock leading out. There was no spillway and with the water up to the crest of the dam the capacity of the reservoir was 30,415 m^3.

The reservoir filled almost to the crest in November 2000 and this prompted the Undertaker to seek an opinion from an All Reservoirs Panel Engineer. The inspection was made on 2 February 2001. The Inspecting Engineer expressed the opinion that the reservoir was a large raised reservoir and that a spillway was needed to pass the PMF (since there were houses downstream). He also said that the spillway should be designed and built before the autumn of 2001.

As well as a new spillway a clay filled cut-off trench was proposed for a length of 50 m along the axis of the dam to a depth of 1.2 m to cut off leakage. Because a spillway would increase the discharges at short return periods the Environment Agency withheld permission for the new spillway under Section 23 of the Land Drainage Act, 1991. However it was pointed out that under Section 23(6) permission is not required for works being carried out in pursuance of another Act of Parliament or any order having the force of an Act. However, everything possible was done to address Environment Agency concerns and, with this in mind, a labyrinth spillway was constructed so that the spillway sill could be set as high as possible.

Figure 2. New Labyrinth Spillway at Weldon Lagoon

There was uncertainty at the time of design regarding the validity of estimates for extreme events obtained using the Flood Estimation Handbook (MacDonald and Scott, 2000). Consequently the 10,000 year outflow, at which there was to be 400 mm wave freeboard, was calculated as 16.5 m^3/sec assuming a rainfall depth, in 83 minutes, half way between that in the Flood Studies Report (125 mm) and that in the Flood Estimation Handbook (213 mm). The 10,000 year rainfall depth obtained from the Flood Estimation Handbook was 73 % greater than ¼ world maximum (123 mm) and was thought to be excessive - this view subsequently gained support from another paper by Messrs MacDonald and Scott (MacDonald and Scott, 2001). The PMF outflow, at which there was to be nominal wave freeboard, was calculated as 25.8 m^3/sec.

The new spillway was completed in October 2001. On completion of the work a certificate of discontinuance was issued because the capacity of the reservoir was reduced to 17,650 m^3 (ie. less than 25,000 m^3).

The case history illustrates how discrepancies were dealt with between estimates made using FEH and FSR. In addition it shows that there may sometimes be conflicts between the requirements of reservoir safety and those of good practice in flood mitigation. It also illustrates the usefulness of labyrinth type spillways in those situations.

Case History No 3 – Shardeloes Reservoir

Shardeloes reservoir has a capacity of only 50,000 m^3 but a catchment area of 49.8 km^2 . Only 1 mm of runoff would suffice to fill the reservoir from empty. The reservoir is remarkable for having a spillway capacity of only 1 m^3/sec. Because the catchment is largely chalk this has been sufficient to ensure the survival of the reservoir since it was built in the early eighteenth century although it is thought that the dam must have been overtopped in the floods of March 1774 when boats could be rowed along the streets of the town downstream.

Because of the town downstream the dam is assigned to Category A as defined in the Institution of Civil Engineers booklet "Floods and Reservoir Safety" (ICE, 1996). The PMF is calculated at 186 m^3/sec for a saturated catchment. Strengthening the dam to withstand the passage of the PMF would however have been expensive and detrimental to amenity, recreation and wildlife conservation.

However the capacity of the reservoir is less than 1% of the volume of the PMF (5.65 Mm^3). This being so the failure of the dam in a major flood would not make a significant difference to flood levels downstream. It was therefore decided to apply American methodology as described in the article in Dams and Reservoirs on 'Small Reservoirs on Large Catchments' (Hinks, 2003).

Mathematical modelling was first carried out to determine flood levels downstream with and without dam break. A sunny day dam breach was expected to release water at a peak rate of about 11 m^3/sec. Coming on top of a flood of 3.5 m^3/sec the incremental flood depth in the town was calculated as 300 mm. This is considerably less than the figure of 600 mm permitted by American methodology and was therefore judged acceptable.

The new spillway is now being designed with a capacity of 3.5 m^3/sec.

The case history illustrates the relevance of American methodology for small reservoirs on large catchments.

Figure 3. Shardeloes Reservoir

Case History No 4 – Braydon Pond

Braydon Pond is a privately owned Category B reservoir impounding about 100,000m^3. It has a minor road along the crest and two spillways. The central spillway is carried beneath the crest road in twin concrete pipes which were installed by the Highway Authority in 1976. Unfortunately the pipes were surrounded with granular fill so there was considerable leakage downstream when reservoir levels were high. The pipes also became cracked and distorted under the weight of traffic. Eventually the pipes had to be dug out and replaced with new ones surrounded by clay.

The spillway at the right abutment was lowered by 300mm in 2000 to provide greater spillway capacity.

The most recent problem is a major slip in the crest road caused by heavy lorries using the minor road as a short cut. It remains to be seen whether the necessary repairs will be paid for by the owner of the dam or by the highway authority who own the road.

Case History No. 5 – Faringdon House Lake

Faringdon House Lake is an ornamental lake built in the grounds of Faringdon House in Oxfordshire in approximately 1770. The reservoir has a capacity of 33,000m^3 and a catchment area of 0.27 km^2. A particular feature of the reservoir is a spring fed fountain which discharges into the head of the reservoir under a head of 2m. The reservoir has been assessed as category D given that the failure of the reservoir would cause only minor inundation.

The reservoir was one of those that had slipped through the net of the Reservoirs (Safety Provisions) Act, 1930 and was not picked up until 1989 when a Section 10 inspection was instigated and supervising engineer appointed. The reservoir had been somewhat neglected up until this time. The principal defects related to a number of large trees, which had been allowed to grow unchecked on the embankment and the total lack of a spillway. The only outflow from the reservoir was via a 100mm pipe overflow at approximately 0.30m below the crest level. This passed through the 6m high embankment to feed an ornamental cascade on the downstream face. The owner was keen to maintain the essential character of the lake and the Victorian water garden at the dam toe and the remedial options were developed to take this into account. Relatively severe tree surgery removed much of the top weight from the larger trees and reduced the risk of toppling whilst a grass-lined spillway was constructed down the right abutment to carry flow in excess of the capacity of the 100mm pipe. The spillway was designed to pass the 150 year flood of 0.74 m^3/s, with a nominal freeboard and in most years the spillway will operate two or three times a year.

This case history illustrates the need to ensure that reservoirs are entered onto the register. If left unchecked this reservoir may well have failed during a 1,000 year rainfall event which occurred on the catchment in 1998. In addition it has been found that a sympathetic approach with a private owner will generally bear fruit in encouraging implementation of works in the interest of safety.

Case History No 6 – Marston Pond

Marston Pond is believed to date from about 1780. It now has a surface area of about 8 hectares and a capacity of 80,000 cubic metres. The dam is about 500 metres long with a maximum height of about 3 metres.

The dam was quite regularly overtopped and leaks develop fairly frequently.

The problem was to bring the dam up to modern safety standards at reasonable cost without spoiling the fishing and duck shooting in the reservoir.

The dam was classified as Category D on the grounds that there were no houses between the dam and the confluence with a larger river some distance downstream. The low height of the dam was also taken into consideration as was the extensive siltation which meant that the reservoir was generally quite shallow close to the dam.

Figure 4. Glory Hole Spillway at Marston Pond prior to lowering by 450 mm.

In order to pass the 150 year flood of 5.25 m^3/sec without overtopping of the dam the higher of the two spillways was lowered by 450 mm so that it was the same level as the other. After some debate it was decided to allow the owner to install stoplogs between 1 April and 30 September each year. This concession, which is subject to the instructions of the Supervising Engineer, will reduce the capacity of the spillway from 5.25 m^3/sec to 3.3. m^3/sec during the summer months but is expected to have a very beneficial effect on amenity, recreation and wildlife conservation in line with the Department of the Environment letter of 26 February 1986.

This case history illustrates the need to make compromises in order to achieve an appropriate balance between the demands of reservoir safety and those of amenity, recreation and wildlife conservation.

Case History No. 7 – Fawsley Estate Lakes

The Fawsley Lakes were constructed as a series of three ornamental ponds adjacent to Fawsley House in Northamptonshire in approximately 1850. Only one of the three reservoirs comes under the Reservoirs Act with a volume of 120,000 m^3 whilst the other two lakes are immediately upstream, on two separate tributaries, and have volumes of 22,000 m^3 and 23,000 m^3. All three reservoirs have suffered from deterioration over the years and in a statutory inspection some 5 years ago a significant number of items were recommended in the interests of safety. Because the two non-statutory reservoirs posed a risk to the statutory reservoir, items of major maintenance at these reservoirs were included in the recommendation in the interests of safety.

This statutory category B reservoir was deemed to have insufficient freeboard to pass a 10,000 year flood without overtopping of the embankment and there were concerns about seepages through the dam and alongside the walls of the cascade spillway. The spillways of the two upper reservoirs are both largely collapsed and there is significant erosion of the adjacent embankment fill and lack of freeboard. Works are now in hand to rectify these defects but are complicated by the fact that there are multiple undertakers with both the owner of the estate and the local fishing club undertaking work on an ad hoc basis.

This case history illustrates the need to consider non-statutory reservoirs or other constructions that may influence the safety of the statutory reservoir. In addition it demonstrates the potential problems of multiple undertakers in implementing recommendations in the interests of safety.

ACKNOWLEDGEMENTS
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The Discontinuance of Devils Dingle Ash Lagoon

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SYNOPSIS.Devils Dingle Ash Lagoon is the principal means of ash disposal for Ironbridge Power Station. The lagoon is impounded by an embankment constructed largely of PFA with an upstream clay core. Filling of the lagoon is entering the final stages and plans for the restoration of the site are currently being formalised.

This paper describes the proposed decommissioning of Devils Dingle Ash Lagoon and the measures taken to ensure that the reservoir will have its storage capacity reduced to less than 25,000m^3 and therefore fall outside the ambit of the Reservoirs Act 1975. The methods used to complete the filling and landscaping of the lagoon whilst maintaining and enhancing the important wildlife habitat that have established around the site are also described.

INTRODUCTION

The Devils Dingle Ash Lagoon has been the main means of ash disposal for the 1000 MW Ironbridge ‘B’ Power Station since it commenced operation in 1968. It comprises an embankment, constructed mainly of pulverised fuel ash (PFA) impounding a lagoon in a small tributary valley of the River Severn above the village of Buildwas, Shropshire. The embankment straddles the confluence of two small streams flowing down the valley.

The embankment was raised in stages ahead of the ash disposal requirement to a maximum height of 66m. The crest of the embankment has an approximate length of 570m. Approximately 3 million tonnes of ash were used to construct the embankment and another 2 million tonnes were used to fill the lagoon. A compacted clay embankment with stone drainage layers and an upstream rockfill berm was constructed for the initial impounding prior to the availability of the conditioned ash. The main body of the dam was then constructed from compacted PFA with a rockfill berm at the downstream toe. The upstream face of the dam was then sealed with a 3.5m thick clay blanket which is protected from wave erosion by a layer of coarse gravel and rockfill. A vertical wall drain was constructed downstream of the final crest line which connects with a horizontal drainage blanket located beneath the downstream shoulder of the embankment. A cross section of the embankment is shown in Figure 1.

PFA has been delivered to the lagoon in two ways. Conditioned ash with a moisture content of about 23% was delivered to the site by truck between 1967 and 1983. The remainder of the ash was slurried and pumped to the lagoon by pipeline. However, in December 2000 the pipeline delivering slurried ash was ruptured by slope movements along the valley between the power station and the ash lagoon. As a result the ash required to complete the filling of the lagoon and provide landscaping features is being delivered by road.

Figure 1: Cross section of the embankment

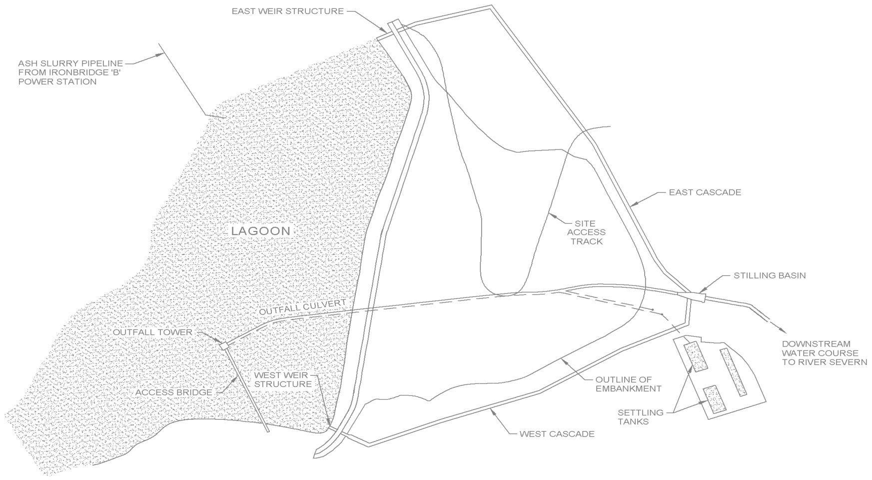

Figure 2: Plan of the embankment at Devils Dingle

Overflow Arrangements and Flood Control

During operation of the lagoon supernatant water is discharged over dam boards set in a slot in the side of the 4.6m diameter outfall tower. The water level in the lagoon can be varied by adding or removing these dam boards. Recently water within the reservoir has been held at 122.34mOD although the level can be raised to a maximum of 123.0mOD which coincides with the weir of the outfall tower. Access to the top of the tower is gained via a raised steel platform and walkway from the western (right hand) bank of the reservoir.

Decanted water drops down the tower to the base where a retained pool of water is used to dissipate the energy of the falling water. At the base of the tower a 600mm, reducing to 450mm, diameter pipe is set below in the main weir of the pool to discharge 'normal' flows to stilling ponds downstream of the dam. This pipe runs along the outfall culvert connecting the base of the tower to the downstream toe of the dam. Towards the end of the outfall culvert the pipe is diverted from the main culvert into a smaller secondary culvert that leads into the settling ponds situated just off the toe of the embankment.

During flood events water is initially discharged over the dam boards until the water level in the lagoon reaches 122.6mOD when flows also pass over two cascades located at either end of the embankment. The cascades are constructed of reinforced concrete and form trapezoidal channels with baffle blocks at regular intervals along their length. Each has been designed to discharge a flow of approximately 2m ^{3}/s and both discharge to the stilling basin at the toe of the embankment. A weir and venturi flume at the head of each cascade ensures that the design flow is not exceeded even under extreme PMF conditions. As part of the original design the cascade structures were model tested to confirm the arrangements.

As the water level in the reservoir continues to rise overtopping of the outfall tower weir set at 123.0mOD occurs. When the capacity of the 600mm pipe at the base of the tower is exceeded water discharges over the weir directly into the outfall culvert. The flood waters pass down the outfall culvert into the stilling basin before flowing back into the stream leading to the River Severn. The outfall culvert and stilling basin are designed for a maximum flow of 22m^3/s. The levels of the principal structural elements are summarised in Table 1 below:-

Table 1: Levels of the principal structural elements within the ash lagoon

Structure	Level (mOD)
Embankment Crest	124.30
PMF Flood Level	123.53
Outfall Tower Weir Level	123.00
Cascade Weir Level	122.60
Damboards (Typical weir level)	122.34

A recent hydrological assessment of the site undertaken by KBR reported the catchment area of the ash lagoon to be approximately 1.38 square kilometres with an average annual rainfall (SAAR) of 736mm. The peak inflows for the different flood events within the Devils Dingle catchment area are detailed in Table 2 below:-

Table 2: Peak inflows for the Devils Dingle catchment area during various flood events

Flood Return Period	Peak inflow
Mean annual flood	0.7 m^3/s
1,000 year flood	4.7 m^3/s
10,000 year flood	9.7 m^3/s
Probable Maximum Flood	19.1 m^3/s

Prior to the recent period of infilling with the ash lagoon, the attenuation within the lagoon results in the PMF peak outflows being approximately 16m^3/s, with 3.5m^3/s flowing down the two side cascades and 12.5 m^3/s flowing into the outfall tower and along outfall culvert.

Requirements of the Reservoirs Act

Discontinuance of a reservoir can only be certified if a Panel AR Engineer is satisfied that the impounded volume of a reservoir, excluding any flood storage, has been permanently reduced to less than 25,000m^3. However, in situations such as ash lagoons this volume should include any silt or ash deposits that would flow in the event of an embankment breach or failure. Therefore, the volume of ‘escapable contents’ should be considered in this case.

Ash lagoons such as Devils Dingle are usually operated under a number of interim certificates as the lagoon is being filled to its final level. When filling of the lagoon is completed the final certificate is issued and is immediately followed by a certificate of discontinuance as the lagoon would no longer have any storage available. However, in this particular case the owners of the site were keen that the restoration plan included at least one body of water in order that the wildlife habitat that had established around the lagoon could be retained.

Plate 1: The reservoir at the Devils Dingle Ash Lagoon with draw-off tower access walkway in the background

Given that the current surface area of the lagoon is of the order of 125,000m^2, a single pond with a volume restricted to less than 25,000m^3 would have an average depth of less than 200mm for discontinuance to be possible. In addition any underlying layer of fluid ash would also need to be considered in the calculation of 'escapable contents' and would further reduce the volume of stored water allowed in the final scheme. In order that the final restoration of the site could incorporate some form of stored water feature it was hoped that the ash at the lower levels had consolidated with time, encouraged by under drainage and through drainage into surrounding lower water table. Significant depths of 'fluid ash' would make it not possible to have any form of large ponds within the restoration plan.

Given the large surface area of the current reservoir and the likelihood that a layer of 'fluid' ash exists below the retained water level it was envisaged that 3 or 4 smaller separate water bodies each with escapable contents of less than 25,000m^3 of water and ash would have to be formed rather than a single pond. However, the construction of multiple ponds would have undertaken in such a way as not create a situation where the capacity of each lagoon was considered to be part of the sum of all the lagoons and therefore have a capacity in excess of 25,000m^3. The final design must therefore include lagoons, each one considered to be fully independent of its neighbours and with little likelihood, under any situation including instability, overtopping or piping, of failure of the dividing bunds.

It was considered that the dividing bunds must therefore be designed as engineered structures on a suitable foundation. However there would be no requirement to construct the bunds as linear features, or with uniform cross sections and so it is envisaged that the dividing embankments will be constructed to give the lagoon area as natural an appearance as possible. It is considered that the separating bunds would have to be constructed with typical crest width in the region of 30m and maximum slope gradients of 1V:6H in order to ensure that the dividing embankments remain stable and the lagoons remain independent features.

INVESTIGATIONS AND SURVEYS

In addition to a detailed topographic survey of the site, a bathometric survey of the lagoon was undertaken to determine the levels of the ash within the reservoir. A three dimensional computer model was then developed to determine the remaining void space and to establish a number of discontinuance options using varying quantities of PFA. This design flexibility was required as the actual volume of PFA available for disposal and landscaping is uncertain and largely depends on the operational life of Ironbridge Power Station and the requirement of PFA for other uses. The number of ponds created in the reservoir, the height of the controlling weirs and the height and topography of the ash bunds within the lagoon were all varied to establish the minimum volume of ash required to achieve the discontinuance of this reservoir.

A geotechnical site investigation was carried out to establish the condition of the previously deposited ash and to determine the depth of ash that could be considered to be fluid. Experience from other sites suggested that the low water table around the site and under-drainage may have caused the lower levels of the ash to have partially drained and consolidated. However, the upper two or three metres were likely to be unconsolidated with a high moisture content.

The method of investigation was determined by the soft nature of the ash deposits both in terms of the sampling methods proposed and the ability to move around within the lagoon. Some elevated areas in the lagoon close to the outlets had been ‘dry’ for many years and as a result the upper layer of ash had become relatively firm and vegetated. However, the level of ash in other areas closer to the embankment was considerably deeper and had been under water for significant periods of time. Due to the positioning of the slurried ash pipeline outlets around the lagoon the surface levels of the ash deposits varied by up to 5 metres. As a result it was decided to use a CPT (Cone Penetration Test) rig fitted with a Piezocone and mounted on a floating pontoon within the lagoon. The water level in the lagoon would be raised to the level of the two cascades (122.6mOD) by inserting dam boards in the outfall tower and this would enable the pontoon to access as large an area as possible including some of previously ‘dry ‘ areas. Immediately after the completion of the investigation the water level would be reduced to the lowest level possible in order to dry out as much of the ash surface as possible.

In April 2002 the first phase of ground investigation was carried out consisting of forty nine cone penetration tests positioned on a grid at 50m spacings. Each hole was continued to a maximum depth of between 10m and 16m or until ‘solid’ ash was encountered. In addition six continuous piston sampling holes were carried out to assist in the interpretation of the CPT holes and to enable the geotechnical characteristics of the deposited ash to be determined.

RESULTS OF THE INVESTIGATION

The results of the investigation enabled a depth profile of the ash to be plotted. The results indicated that the majority of the ash deposit had consolidated and drained and that there had been some cementing of the deposits. The results indicated that the ash composition and properties were relatively uniform across the lagoon. A relatively thin layer (<1m) of very soft ash was encountered at the surface of the deposits during the investigation. However, in the event of a breach in the embankment, the PFA deposits would be relatively stable and no significant flow of ash would be expected.

TRIAL FILLING AREA

As part of the preliminary design and prior to the construction of any permanent separating embankments, a trial filling area was established. The trial would not only allow an area of previously submerged ash deposits to be exposed and the proposed foundation to be examined but would also allow a ‘constructability’ trial to be completed. This would assist the contractor in choosing appropriate plant and methods for completing the remaining filling and the construction of the separating embankments. A site was chosen near to the eastern end of the embankment where the topography of the existing ash surface was suitable and where the trial could be undertaken safely in a position away from the outfall tower.

The trial showed that pushing conditioned ash into the upper layer of ash displaced the majority of the very soft ash deposits present and that the dry ash became founded on a suitable foundation layer. The trial also demonstrated that the method of placing the fill over the previously deposited ash fill was suitable and that a suitable founding layer could be established for the remaining fill and proposed separating embankments.

Following completion of the trial filling area two survey positions were constructed above the areas that had received the greatest depth of fill. The levels of these two survey stations have been recorded on a monthly basis to determine the amount of settlement taking place in the foundation and newly placed fill. The results to date indicate that no noticeable settlement in either the foundation or recent fill is taking place. Therefore, it is likely, given the granular nature of these ash deposits, that the majority of the settlement has occurred during the construction of the trial filling area.

Phase two site investigation

A second investigation was commissioned in November 2003 after approximately 18 months of filling the lagoon with PFA. To enable comparison with the first phase CPTs, eleven new CPTs were carried out in locations coincident with CPTs from the phase one investigation. These CPTs were carried out using a truck mounted rig and were taken to a maximum depth of 10m. Not all areas of the lagoon were accessible to this truck mounted rig as some areas remain under water.

The Phase 2 investigation was undertaken to determine the condition of the newly placed fill, the changes within the previously deposited ash fill and to establish the presence, or otherwise, of the soft layer previously identified at the surface of these deposits.

The results show that the soft layer was no longer present probably resulting from the method of filling, consolidation as more ash was placed above and the re-distribution of pore water pressures. Also the ash placed above that tested in Phase 1 had improved density and stiffness properties. Therefore, it is considered unlikely that the ash would flow if the embankment were breached.

POND LAYOUT

Based on the results and interpretation of the various investigation phases a preliminary restoration plan was developed. Three ponds are proposed, two of which are to be located close to the embankment adjacent to each of the cascade structures. A third pond is proposed towards the western edge of the lagoon adjacent to and north of the location of the existing elevated walkway to the outfall tower opening. The two ponds located close to the embankment are to have water levels of 122.6mOD controlled by the existing cascade weirs. The third pond will have a slightly higher water level controlled by inlet and outlet structures on the stream entering this pond.

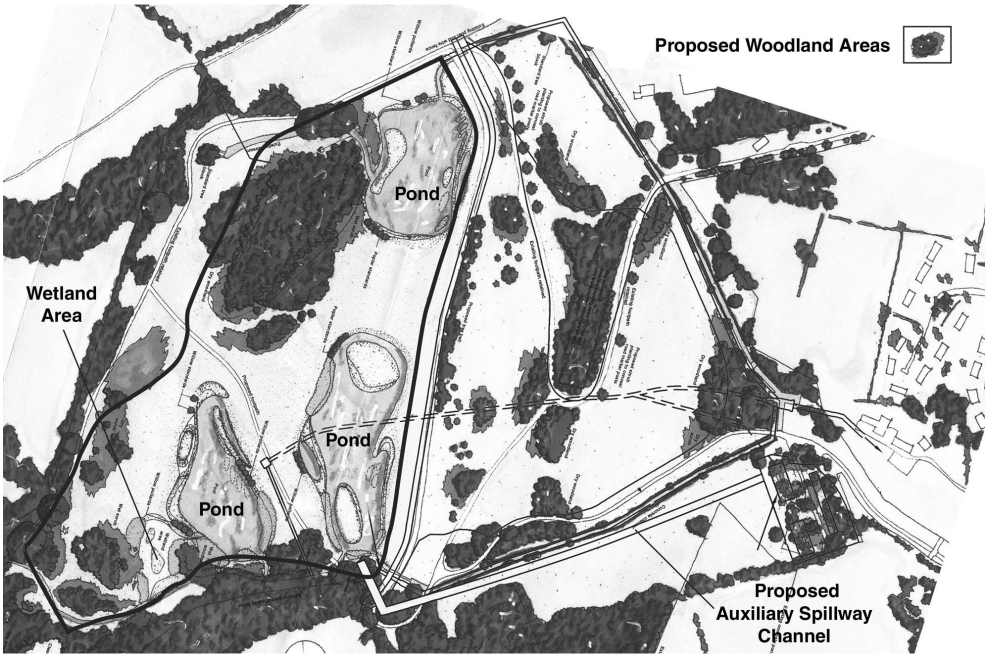

Figure 3: Plan of proposed restoration scheme

Each of the proposed ponds will have a specifically designed profile in order to try and establish a number of different aquatic habitats within the lagoons. Low lying areas close to the incoming streams will also be used to create new habitats such as wetland marginally areas.

Ecologists and landscape architects formed part of the project team that formulated the preliminary restoration plan for the site. Areas of young woodland and other vegetation that has become established within the lagoon area will be preserved where possible and new areas of both 'dry' and 'wet' woodland will be created around the proposed ponds.

PFA will also be used to construct additional landscaping areas on the downstream face of the embankment create a more natural landform and to mask the concrete features on the embankment that for hydraulic reasons tend to follow straight lines. Planting of selected shrubs and trees on the downstream face will also help to disguise the embankment.

MODIFICATIONS TO EXISTING STRUCTURES

In order to return the site to as natural an appearance as possible it will be necessary to carry out modifications to the existing structures associated with the lagoon. Sequencing of the necessary modifications to the overflow tower, cascades and stilling basins must be programmed such that no works are undertaken on these structures prior to the satisfactory discontinuance of the reservoir. As further ash is deposited in the existing lagoon both the volume of retained water and surface area of the reservoir are reduced. Although the further filling reduces the volume of the reservoir, the benefit of the flood attenuation provided by the lagoon is also reduced. A detailed programme of ash deposition, construction and modifications was therefore developed to ensure that a 'less safe' condition is not created during this process.

During the early planning and preliminary design stages of this scheme the issue of how the site will respond to flood events during and on completion of the restoration plan have had to be addressed. The original proposal for discontinuance includes the decommissioning of the outfall tower and culvert by sealing both ends and filling the void with a PFA/cement grout as this will reduce the future maintenance requirements of the site. The decommissioning of the outfall tower will reduce the discharge capacity of the site to the combined capacity of the two remaining cascade structure approximately 4m ^{3}/s which equates to a 1000 year flood event.

Although the reservoir will no longer be subject to the Reservoirs Act 1975 and have a requirement to safely pass the PMF event, the owners were keen that the restoration plan should include measures to protect the embankment against overtopping and possible erosion from storms greater than the 1,000 year event. This would particularly important when maintenance and inspection regimes would be stepped down and in the long term when the site may possibly be sold. The potential for blockage of the existing cascades will also be more likely given the large number of trees and other vegetation to be planted around the proposed ponds

Therefore, it was decided to construct a reinforced grass auxiliary spillway down the right mitre of the embankment, adjacent to the western cascade channel, to give additional spillway capacity. The weir of this structure will be designed in such a way that the discharge capacity of the combined spillways will again be able to pass a PMF event safely and therefore the embankment will be protected from overtopping. Flood flows will pass via a reinforced grass channel into a newly constructed stilling basin at the toe of the embankment where the existing settling tanks are located. The construction of the auxiliary spillway is planned early in the programme prior to discontinuance of the reservoir in order that the adequate discharge capacity is always available during the works. This also will provide greater flexibility in the timing of the remaining ash placement, pond formation and modification of existing structures.

Outfall Tower and Access Walkway

The outfall tower and access walkway will become redundant in the proposed scheme. Removal of the raised walkway will be achieved by the construction of a large ash bund adjacent to the walkway from the western bank of the lagoon out towards the outfall tower. This bund will provide a working platform from which the access walkway will be dismantled and the supporting piers broken down. The bund will also allow access to the top of the outfall tower. It is proposed that the downstream end of the outfall culvert is sealed with a concrete bulkhead and the entire outfall tower and culvert be filled from the above using a PFA cement grout. This will ensure that there will be no long term maintenance issues associated with the outfall tower or outfall culvert.

Plate 2: Eastern cascade channel on the left mitre of the embankment

Spillway Cascades

Following the completion of the restoration plan the two cascades located at either end of the embankment will be in almost continuous use as these structures will control the level of the ponds. The structures are likely to be largely unchanged, however, some screening of the cascades using various types of vegetation will be undertaken in order to reduce the visual impact of these linear features.

Stilling Basin

The main stilling basin will still be required following discontinuance of the reservoir as flows from the east and west cascade channels will enter either side of the stilling basin. As the stilling basin will no longer receive flood flows from the outfall culvert some minor works are proposed to mask the sealed entrance of the outfall culvert and reduce the visual impact this feature.

Main Embankment

The placing of additional ash and topsoil on the downstream face and selected planting is proposed to create a more natural appearance and to create more rounded features and break up the straight lines of site that exist.

Settling Lagoons

The area currently occupied by the settling lagoons will be modified and will be used as a stilling facility for the reinforced grass auxiliary spillway. Measures will be taken to obscure the view of both the auxiliary spillway channel and stilling basin from the village of Buildwas located close to the toe of the embankment.

Pipeline

A 2.5km long pipeline exists between the power station and the lagoon through which slurried ash was pumped up to the lagoon. The pipe varies in depth considerably over its length being some 10 to 12m deep in places. Small land movements adjacent to the pipeline are thought to have caused cracking in the pipe leading to release of water and ash into and onto the surrounding ground on a number of occasions. This release of fluid may have lubricated the surrounding ground to encourage larger slips. As a result of these problems the pipe has not been used for a number of years and all PFA is now transported to the site by road.

It is anticipated that the pipe will be decommissioned as part of the reservoir discontinuance by grouting the pipe with a cement PFA grout. The pressure the pumped grout will need to be adjusted where there are fractures to ensure leakage from the pipe will be minimised.

PROPOSED PROGRAMME

The restoration scheme has already started with the deposition of ash in selected places within the lagoon in line with the final proposals. The construction of the auxiliary spillway is due to commence in the spring of 2004. Further filling of the lagoon, construction of the ponds and decommissioning of various structures is planned for 2005 and 2006 together with the final landscaping of the site.

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Bewl Water spillway remedial works

I DAVISON, MWH
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SYNOPSIS. Bewl Water is a major impounding reservoir in South East England, UK. In the mid-1990's an external inspection confirmed that the crest of the spillway shaft was suffering from severe cracking. A paper, by one of the authors, presented at the British Dam Society Conference in Bangor in 1998 described the investigation that was carried out to determine the cause of the cracking. This paper describes the remedial works contract that was undertaken to replace the pre-cast concrete crest blocks that were suffering from Alkali Silica Reaction, and describes the extensive temporary works over the top of the spillway shaft, the methods used to remove the existing pre-cast units and the design of the new units. The new crest units have been designed to allow the crest to be raised by 350 mm without the need for further major temporary works over the top of the shaft.

BACKGROUND

General

Bewl Bridge Reservoir is a large, raw water reservoir situated approximately 10 km south east of Royal Tunbridge Wells in Kent. The reservoir is primarily filled by the Yalding Pumping Station with additional water from the original pumping station on the River Teise and the relatively small natural catchment of 1.9 km^2.

A 30.5 m high earthfill embankment with a central rolled clay core retains the reservoir. A bellmouth spillway shaft located within the reservoir was designed to discharge floods up to a Catastrophic Flood of 115.5 m^3/s. Water is abstracted from the reservoir via pipework within a 36 m high reinforced concrete draw-off tower adjacent to the spillway tower. Impounding of the reservoir started in 1976 and was full by mid 1978.

Spillway Shaft

The overflow structure, as shown in Figure 1, consists of a vertical shaft from the reservoir bed up to the full supply level of the reservoir. The lower section of the shaft is 3.5 m in diameter internally and has 500 mm thick reinforced concrete walls. Over its upper 7.75 m, the shaft flares out to give a weir crest diameter of 10.8 m. The crest is formed by a series of 32 pre-cast concrete blocks that form the lip of the weir which is divided into quadrants by anti-vortex piers that prevent a vortex forming in the shaft when it becomes submerged. A 1500 mm deep reinforced concrete beam boat fender surrounds the weir reducing the size of waves impinging the weir and preventing boats from getting too close to the overflow crest. The boat fender is located 2 m away from the weir crest and is supported by four radial beams spanning out from the rear of the anti-vortex piers.

At the base of the tower the shaft turns through a 90° bend into the discharge tunnel which passes under the dam before discharging into the river downstream. The tunnel is a horseshoe in section and is approximately 3.3 m in diameter.

THE PROBLEM

The Supervising Engineer carried out a detailed inspection by boat of the cracking and spalling of the pre-cast concrete blocks around the spillway crest in 1995. In February 1996, McDowells Consulting Engineers carried out a preliminary investigation of the cracking, and recommended further investigation of the boat fender and upper section of the shaft, including removal of core samples from the crest blocks and *in situ* concrete. The cause and extent of the cracking were unknown and it was unclear whether the condition was deteriorating. The Inspecting Engineer considered that ASR was a possible cause and recommended repair or replacement.

THE INVESTIGATION

A detailed investigation was carried out using roped access techniques during the summer of 1997. This investigation included carbonation testing, concrete sampling, a covermeter survey and ultrasonic pulse velocity testing. In addition, a visual crack survey and a photographic record of the inside and outside of the tower were carried out. The concrete samples taken from the *in situ* and pre-cast concrete around the crest of the shaft were tested in a laboratory. Reference 1 gives details of this investigation.

CAUSE OF CRACKING

The investigation concluded that the cause of the cracking was due to Alkali Silica Reaction (ASR) in the pre-cast weir units around the top of the shaft. The units had been secured by dowels during construction, and as the units swelled, due to the ASR, the radial geometry of the units caused them to be forced outwards. This in turn caused the *in situ* concrete to which they were secured also to move outwards and, as a result, tension cracks appeared on the flared section of the shaft and on the anti-vortex piers.

REMEDIAL WORKS DESIGN

Three remedial works options were considered:

1. "Do Nothing"
2. Replace all the crest units thereby removing the problem and preventing further expansion.
3. Selective replacement of units and provision of expansion joints which would allow further expansion of the blocks without causing further distress to the *in situ* concrete.

Monitoring of the crack widths was carried out throughout the winter of 1997 and 1998, by measuring tell-tales with a vernier gauge, to try and determine if the expansion of the concrete was still taking place. A more sophisticated system of monitoring had been considered that would have allowed smaller amounts of expansion to be measured but the estimate of the costs of installing and maintaining the system was not significantly less than the cost of the remedial works.

No signs of movement were detected, although this was possibly due to either the movements being smaller than the accuracy of the vernier gauge or the temporary suspension of the expansion. ASR requires moist conditions to occur and during the monitoring period the reservoir level was below the level of the pre-cast units.

Bewl Water is one of Southern Water's most important assets and since under the worst case scenario deterioration of the spillway could lead to loss of storage in the reservoir, the "Do Nothing" option was discounted early on.

The main portion of the cost of the remedial works was the temporary works required by safety considerations to operate over the water on the outside of the shaft and over the 36 m drop on the inside of the shaft. The difference in costs between the selective and complete replacement of the units was not significant and therefore a decision was made to replace all of the units.

Although replacement of the pre-cast units would prevent further expansion of the top of the shaft, the *in situ* concrete directly under the units had already been cracked, and it was considered that ingress of the reservoir water into the cracks could lead to corrosion of the reinforcement. It was therefore decided to apply a waterproof membrane onto the outside of the shaft using a Flexcrete cementitious coating.

During the inspection several areas of exposed reinforcement were identified on the upper sections of the anti vortex piers. It was decided to carry out a detailed cover meter survey and determine the steel condition by removing the concrete from areas with very low cover. The concrete would then be reinstated using a cementitious compound and the whole anti vortex pier would be then grit blasted and coated with an anti carbonation coating, to reduce the risk of further deterioration of the reinforcement due to lack of cover.

FULL SUPPLY LEVEL RAISING

As a separate project but at the same time as the remedial works design was being carried out, MWH was asked by Southern Water to consider the potential for raising the top water level of Bewl Water by reviewing the amount of wave run-up. During this study a potential for raising the top water level by 350 mm was identified, without raising the embankment crest level or carrying out work on the draw-off tower. This would increase the reservoir storage by 7%, a valuable addition to Southern Water's assets.

It was proposed to carry out the raising by increasing the height of the pre-cast crest units on the spillway shaft. However, full investigation of the effect of the raising and approval of the scheme by the Environment Agency were not possible within the time scale of the remedial works contract, so the shape of the new crest units were designed so that they could be easily raised in the future.

As discussed previously, the major portion of the cost of any works on the spillway is temporary works due to safety considerations. To reduce the risk of working over the centre of the shaft the units were designed so that the future units could be lowered into place and secured from a floating barge without the need for accessing the inside of the shaft.

The blocks to be installed under the remedial works contract were designed with a 200 mm step on the upstream side and a socket in the upstand. The pre-cast section that could be added later to raise the top water level has a corresponding step and socket so that the two blocks can be bolted together and then grouted in place. Sealing of the downstream joints could be carried out by roped access techniques without the need for erecting scaffolding on the shaft.

REMEDIAL WORKS CONSTRUCTION

<u>General</u>

The contract was awarded to Brent Construction following a competitive tender procedure and work began at the beginning of the summer in 1999. Bewl Water is not only a major water supply resource shared by Southern Water and Mid Kent Water but it is also a major amenity used by thousands of visitors each year. The reservoir and its surrounding footpaths are used for fishing, sailing, walking and riding. It was therefore essential that extra care was taken to ensure that no interruption or disturbance was caused to the other users of the reservoir. An excellent working relationship was developed between the Southern Water staff, especially the local rangers, and the contractor. At the end of the contract Brent Construction were presented with a Customer Care Merit award by Southern Water. The contract was complete on programme and under budget.

Temporary Works

Access to the shaft for materials and plant was by barge equipped with Hi-ab crane. Daily access for personnel was by smaller boats.

During the investigation stage of the project, a scaffold walkway had been erected around top of the boat fender. This platform was maintained throughout the remedial works contract, and, in addition, a scaffold platform was erected over the void of the shaft with another hanging walkway suspended around the outside of the weir crest to allow access to both sides of the working area. The central platform effectively blocked the only spillway facility at Bewl Water so the works were programmed for the end of the summer when the water levels are normally low and falling and sufficient storage existed in the reservoir to contain low return period floods without interfering with the works.

Removal of Existing Weir Units

During the design stage consideration was given to the method of removal of the existing pre-cast units. Due to the restrained expansion of the pre-cast units it was considered that a sudden release of any in built stresses could cause problems. Therefore slots were required to be cut through the units prior to their removal. This was carried out by diamond drilling a hole through the base of one of the units nearest an anti-vortex pier in each quadrant. A diamond rope saw was then threaded through the hole and a slot cut upwards through the block (see Figure 2). No sudden release of stresses was noticed.

Further holes were then drilled in the base of each block at the intersection with the *in situ* concrete. A hydraulic jack was then inserted into the hole and pressure applied. The joints on all three sides fractured cleanly and the units were all able to be lifted off in one piece by the crane on the barge.

This operation left a good clean and smooth surface that required little preparation before the new units could be installed. With the units removed it was possible to see that some of the cracking in the *in situ* concrete had penetrated more than half way through the thickness of the shaft walls validating the decision to apply an external waterproof coating to prevent further deterioration.

Installation of New Weir Units

The new pre-cast units were installed using the crane on the barge. The units were placed on thin metal shims and all units were located and levelled prior to any grouting of the vertical or horizontal joints (see Figure 3). Two vertical movement joints were installed in each quadrant, consisting of a 25 mm thick compressible joint filler surrounded by a joint sealant. All other joints were filled with a non-shrink cementitious grout and then pointed with an SBR mortar.

To prevent sliding of the pre-cast units on the smooth surface of the *in situ* concrete, vertical dowels were inserted through holes in the units into the underlying concrete and then grouted in place.

PRESENT SITUATION

The new units have now been in place for four years (see Figure 4) and recent observations made from the balcony of the adjacent draw-off tower indicate that the surface of the replacement units are without deterioration. During this period the reservoir has spilled and this confirmed the quality of the workmanship as spilling was uniform around the perimeter.

The protective coating on the anti-vortex piers is free of defects, maintaining adhesion and without noticeable cracking.

The low water levels in 2003, exposed the coating to the external surface of the spillway shaft, and again, this remains free from defects. No signs of seepage have been noted on the inside of the shaft at high water levels and it appears that the application of the coating has been successful.

ACKNOWLEDGEMENTS

The authors are grateful to Southern Water for permission to present this paper on the remedial works carried out at Bewl Water.

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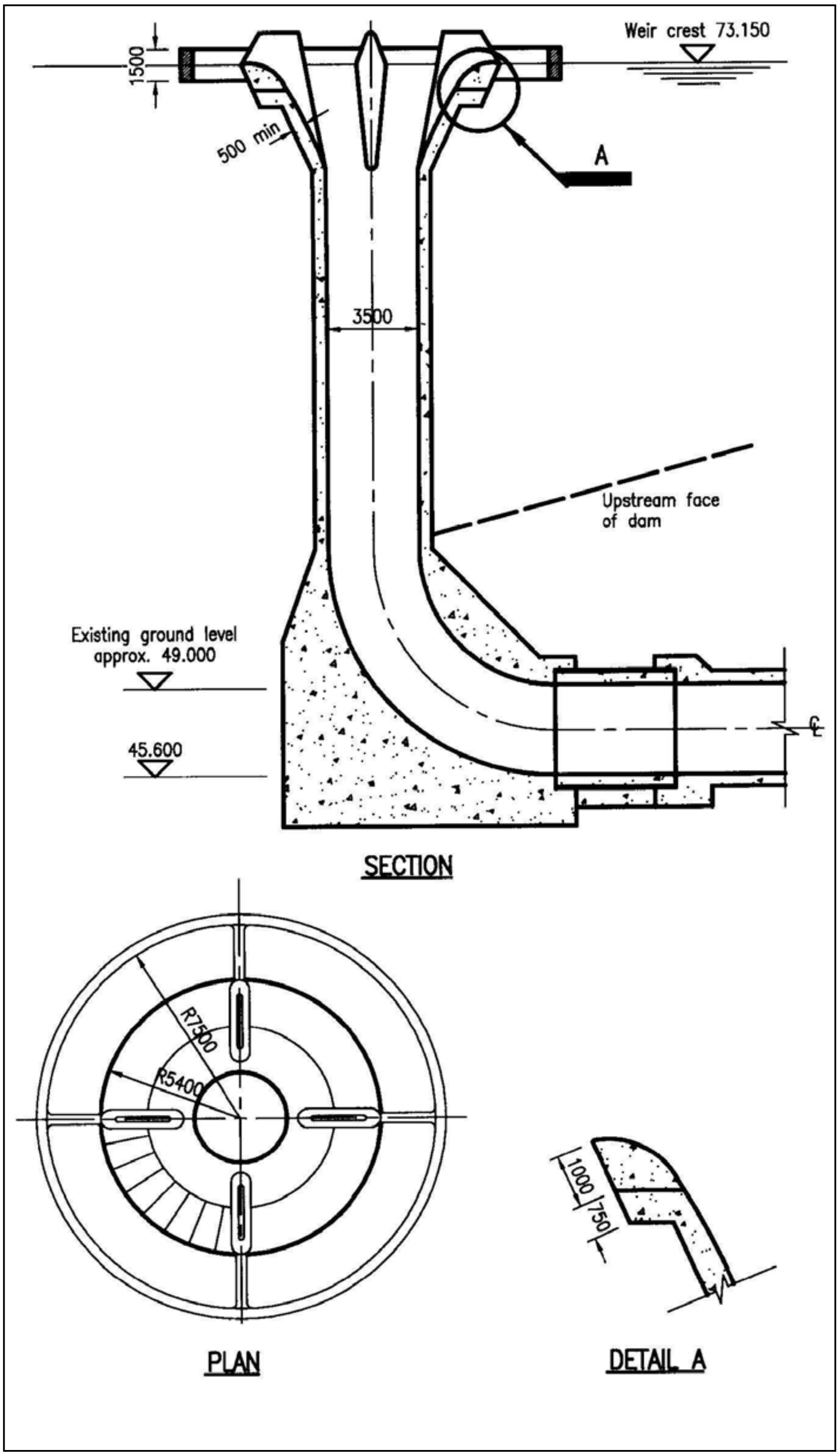

Figure 1 - Detail of the Overflow Structure

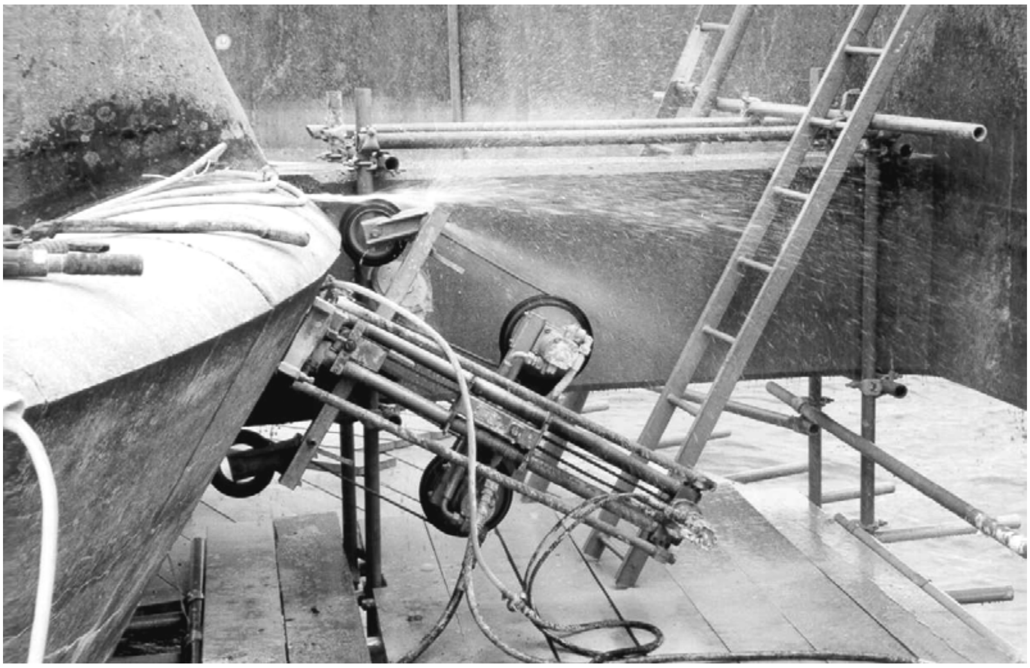

Figure 2 - Saw cutting between the existing blocks

Figure 3 - New blocks in place

Figure 4 - Completed spillway crest

Walthamstow Reservoirs No. 4 & No. 5 embankment protection

C.B. PECK, Thames Water RWE, UK

SYNOPSIS.
Walthamstow reservoirs Nos. 2, 3, 4 and 5 are situated in the Lee Valley, north-east London. Reservoirs Nos. 4 and 5 fall within the provisions of the Reservoirs Act 1975, have a common top water level and share a common embankment with Reservoir Nos. 2 and 3 which lie at a lower level. No. 2 and 3 reservoirs are used as settlement lagoons for wash water from the nearby Coppermills water treatment works and were in danger of becoming "silt" bound. The reservoirs are also within a Site of Special Scientific Interest and support fish, birds and wildfowl, including migratory species.

An inspection of the common embankments in 1998, revealed a general lack of protection, including evidence of wave action undercutting the lower toe. Recommendations were made to provide protection to the whole length of the embankment, namely the shore of Reservoir Nos. 2 and 3.

The project involved sinking a chain of timber stakes 3m from the lower toe. A geo-mesh lining was then secured to contain "silt" dredged from Reservoir No. 3. Reeds were then planted in the "silt" to consolidate the protection and enhance the environment. Timber platforms were provided for anglers.

BACKGROUND

Walthamstow reservoirs Nos. 2, 3, 4 and 5 are a chain of reservoirs situated in the Lee Valley, north-east London (Fig. 1). Reservoirs Nos. 2 and 3 were constructed in 1863 and Nos. 4 and 5 in 1866, under the powers of the East London Act of 1853. The two sets of reservoirs share a common embankment.

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

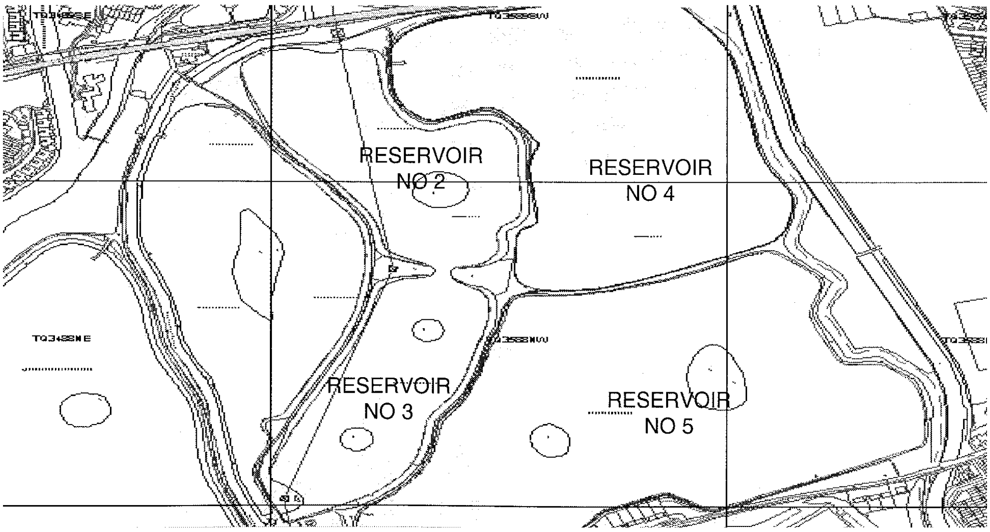

Fig. 1 Location plan of Reservoirs Nos. 2,3,4 & 5
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Reservoirs Nos. 4 and 5 are statutory reservoirs, falling within the provisions of the Reservoirs Act 1975, and have a common top water level and share a common embankment with Reservoirs Nos. 2 and 3, which lie at a lower level (Fig. 2). Reservoirs Nos. 4 and 5 are operated as raw water storage reservoirs and provide a key supply route for stored water to Coppermills water treatment works, as the final two reservoirs in the gravity chain. Reservoirs Nos. 2 and 3 are used as settlement lagoons for washwater from Coppermills water treatment works and are not classified as statutory reservoirs due to the volumes they hold. These two lagoons were in danger of becoming "silt" bound.

All the Walthamstow reservoirs form part of a designated site of special scientific interest (SSSI), which has also been designated as a special protection area under the EU Birds Directive. This SSSI supports a wide variety of fish, birds and waterfowl, including migratory species. In particular they provide a habitat for a colony of herons, which have bred at the reservoirs since 1928. The environmental management and development of the site is work in conjunction with English Nature, who act as guardians of the environmental legalisation. The reservoirs are also used by anglers and birdwatchers as part of the recreational facilities managed by Thames Water Utilities.

Fig. 2 General view of Reservoir No. 2 next to Reservoir No. 4

Over the past 15 years, three separate incidents have occurred at Walthamstow Reservoirs Nos. 4 and 5, which have affected reservoir safety. The incidents were a downstream embankment slip in 1988, crest settlement between 1986 to 1992 and seepage through the embankment in 1996. Remedial works have been carried out to solve the problems caused by the incidents.

Hydrographic surveys carried out in 1994 and 1998 on Reservoirs Nos. 2 and 3 showed them to be heavily "silted, with a significant increase in "silt" **1* levels between the surveys, as a result of washwater discharge from a newly constructed granulated activated carbon (GAC) / sand separation plant. At the time of the project the inlet to Reservoir/Lagoon No 3 was almost completely blocked with sand and "silt" (Fig. 3). Reprofiling of these reservoirs was identified as being required in the immediate future to maintain their effective use. Issues concerning contamination within the "silt", drying out, transportation and special landfill requirements, ruled out the option of removing the "silt" from site.

**1 the term "silt" referred to in the paper is a general term covering the sediment found in the reservoirs*

View Synopsis

Fig. 3 Silted inlet of Reservoir (Lagoon) No. 3

During the statutory inspection dated 20th November 1998 of reservoirs Nos. 4 and 5, recommendations were made to inspect the outer embankments. The subsequent inspection, carried out at water level, revealed a general lack of suitable protection including evidence of wave action undercutting the lower toe. The final inspection report recommended that all areas where erosion had taken place, were to be reinstated and erosion protection provided to the whole length of the external bank of Reservoirs Nos. 4 and 5. This is the internal bank to Reservoir (Lagoons) Nos. 2 and 3. This protection was required around the top water level in Reservoirs (Lagoons) Nos. 2 and 3, whose water level is usually constant at around 7.81m above ordnance datum Newlyn (AODN). A project was initiated in September 1999 for the design and construction of 830 metres of bank protection works.

SCOPE OF WORK
Three options were considered, two of which addressed embankment protection only, and one of which addressed embankment protection and washwater treatment as a secondary output.

Option 1: Removal of "silt" from Reservoir No. 3, placing and stabilising it along the external banks of Reservoirs Nos. 4 and 5.

The protection to the external banks in this option, would be provided by dredging the "silt" from Reservoir No. 3 and placing it on the banks to form a "silt" shelf, in which reed beds would be planted. When established, the reed beds will help to keep the "silt" in place and will also provide an environmental enhancement to the area. The length of embankment protected by the reeded "silt" shelf can be identified in Figure 1.

Option 2: Installation of a 2m wide layer of crushed rocks along the external banks of Reservoirs Nos. 4 and 5.

Option 3: Installation of precast concrete mats along the external banks of Reservoirs Nos. 4 and 5.

Options 2 and 3 only addressed the matter of protection of the existing reservoir banks and were unlikely to be favourable from an environmental point of view to English Nature, whose approval was required for any works carried out on these reservoirs.

Option 1 was chosen as it was the only option that, as well as meeting the primary objective of providing protection to the external embankments of the statutory reservoirs, also provided other benefits. Dredging the shallowest part of Reservoir No. 3, will help to maintain the effective use of the reservoirs as settling lagoons for the treatment of the washwater from Coppermills advance water treatment plant. Other benefits included not having to import permanent works materials, which would have created an impact of additional traffic on the restricted local roads leading up to the site. Finally the chosen option was more likely to receive the required environmental approval for the works from English Nature, which in due course was attained. The reed beds provide a new facility for birds such as herons, who already use the site, and also attract new species of birds, and they have also provided biodiversity enhancements to the SSSI.

LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

SURVEYS & TESTING

Hydrographic Surveys

Hydrographic surveys were carried out on Reservoirs Nos. 2 and 3 to determine the levels and volumes of "silt" in the reservoirs. The hydrographic survey carried out in 1994 covered all of Reservoir No. 3, but only part of Reservoir No 2 and therefore did not provide a figure for the volume of "silt" in Reservoir No. 2. The survey revealed that there was approximately 50,000m3 of "silt" in Reservoir No. 3, which was equivalent to 73% of its volume.

The later survey in 1998 covered Reservoir No. 2 as well as No. 3 and took measurements of the top and bottom levels of the "silt", which enabled the depth of "silt" to be calculated. In Reservoir No. 2 the "silt" depth varied up to a maximum depth of 1.1m, and in Reservoir No. 3 up to 2.1m. Topographical/hydrographic CAD drawings were produced by Thames Water's Survey Group, which were then used to estimate the volumes of "silt" in the two reservoirs, which are shown in Table 1.

Table 1 Results of Hydrographic Survey in 1998 for Reservoirs Nos. 2 and 3

Reservoir	Reservoir Capacity at Design TWL (m3)	Volume of "Silt" in Reservoir (m3)	Percentage of "Silt" in Reservoir (%)
Walthamstow No 2	77,000	31,020	40
Walthamstow No 3	68,000	59,400	87

The output from the hydrographic survey carried out in 1998, was primarily to give an indication of the rate and pattern of the build up of "silt" in Reservoirs Nos. 2 and 3, but it was also used to determine where best to dredge the "silt", which was used to form the "silt" shelf.

"Silt" Testing

To provide information on the nature of the "silt" at the base of the reservoirs to the Contractor Land and Water, several disturbed samples were recovered using 'grab' sampling techniques from eight separate locations in Reservoirs Nos. 2 and 3. The results from particle size distribution analyses and Atterberg Limits indicated that the material fell into two distinct groups. Two samples contained no fines, one being sand and the other gravel. The remaining six samples had a fines content varying from between 95 and 99% and liquid limits varying between 135 and 285%. All the samples, bar one, had a liquid limit of 262% or greater and organic contents around 20%. By a combination of test results and visual description these samples were classified as organic "silt"/clay of extremely high plasticity.

The results from the hydrographic surveys and the "silt" testing were used by the Contractor Land and Water and Thames Water, to determine where best to dredge the "silt", that was used to form the "silt" shelf.

DESIGN & CONSTRUCTION

Access to Reservoir Embankments

As there have been minor slips along the external embankments of Reservoirs Nos. 2 and 3 in the past, there was a need to avoid moving any heavy plant along the top or on the embankments, to reduce the risk of causing any further slips. The Contractor was able to carry out all the works from the water using floating craft/machinery (see Fig. 4), with only Land Rovers and small vans being used along the top of the embankments when carrying out the planting of the reed beds.

Fig. 4 Placing of "silt" shelf using machinery on floating craft

Dredging of "Silt"
The Contractor carried out pre and post dredging hydrographic surveys. This information was used to estimate the amount of "silt" dredged and where in Reservoir No. 3 it was dredged from.

The dredging of the "silt" from the reservoirs was carried out using a hydraulic excavator floated on a barge. 3750m3 of "silt" was dredged from Reservoir No. 3, firstly from the area around the outlet from the culvert that brings the washwater into Reservoir No. 3, as this is the location where the larger heavier sand/gravel particles settle out first and had formed banks that were visible above top water level (see Fig. 3). Removing the material from around the outlet helped clear a path for the washwater, creating a more distributed settlement pattern through Reservoirs Nos. 2 and 3. These more coarse particles were a better material for forming the protection shelf being built up all along the 800m length being protected. When the material around the outlet had been exhausted, further "silt" material was dredged from the deepest areas of "silt", identified from the hydrographic survey of Reservoir No. 3. The chemistry of silt samples from Reservoir No. 3 indicated the expected organic rich conditions and elevated sulphide, ammonia, zinc and copper.

There was some existing concrete "rip-rap" embankments protection around the water level on the internal embankments of Reservoirs Nos. 2 and 3. This existing protection was left in place and the new protection shelf formed over the top of the "rip-rap".

"Silt" Shelf Retaining System
A "nicospan" revetment system supplied by MMG, retained the "silt", which formed the 3m wide dredged "silt" planting shelf for the reed beds on the reservoir embankments. The retaining system utilised a geo-mesh lining retained by sinking a chain of timber stakes 3m from the lower toe. "Nicospan" is a prefabricated, double weave revetment fabric made from strong UV stabilized monofilament yarns that are heat sealed to form a series of open pockets each having a width of 220mm, so that posts can be placed into them. The geo-mesh was selected to allow water to pass through but retain the "silt" (Fig. 5).

The posts for the "nicospan" revetment were driven in using a small piling hammer, converted for 100mm posts, mounted on an excavator. The posts were driven, at 500mm centres, into individual pockets of the "nicospan" to progressively "tighten" the revetment. The line of the revetment was agreed with Thames Water's site staff to offer maximum toe restraint to the embankment, but also offer the most ecological benefit. Anchor poles were driven to the rear of the “nicospan” at 100mm centres and, wired to the “nicospan” using galvanized fending wire. The excavator, used to install the retaining system, was secured to a floating pontoon (Fig. 5).

Fig. 5 View of revetment system being placed

The Contractor designed the form and retaining system for the “silt” shelf (see Fig. 6) and Thames Water’s Geotechnics Group checked whether the new shelf would affect the stability of the reservoir embankments. The analyses were undertaken for slope angles of 1 in 2 and 1 in 3 using the methodology suggested by Morgenstern and Price (1965) and conservative soil parameters (c' = 0kPa and phi = 37 degrees for the gravels and c' = 3kPa and phi = 20 degrees for the London Clay). The results for failure surfaces within the gravels and the London Clay both with and without the silt shelf are summarised in Table 2

Table 2 FOS Results for failure surfaces within gravels and London Clay with and without the silt shelf

1 in 3 Slope				**1 in 2 Slope**			
Failure in London Clay		Failure in Gravels		Failure in London Clay		Failure in Gravels	
A	**B**	**A**	**B**	**A**	**B**	**A**	**B**
1.92	1.80	2.22	2.08	1.36	1.27	1.53	1.40

(Case A FOS without silt shelf and Case B FOS with silt shelf)

The conclusion was that the effect of the "silt" shelf on FOS was minimal and that even with the most onerous combination of a steeper slope and with the failure surface entirely in the London Clay, an acceptable FOS was obtained.

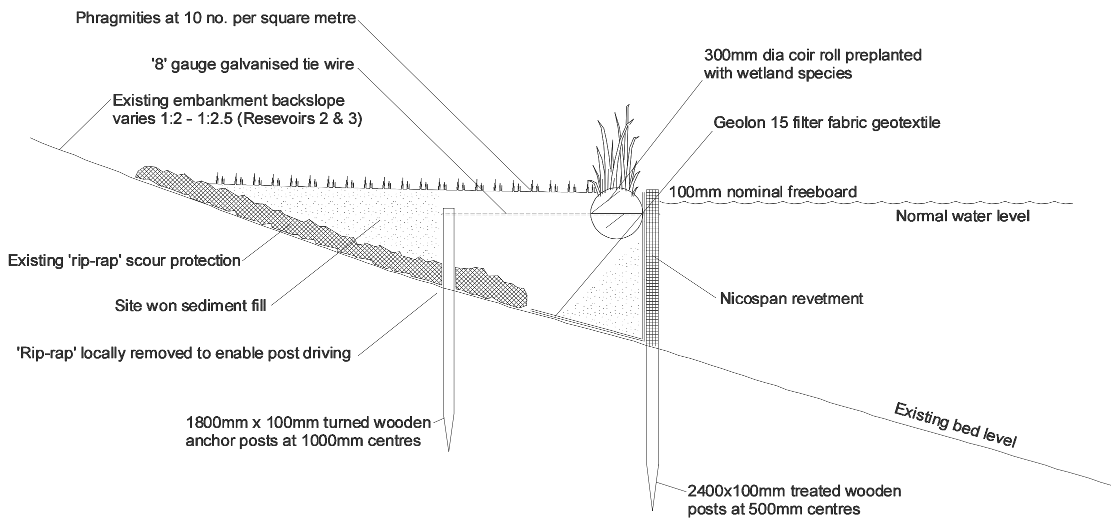

Fig. 6 – Section of "silt" shelf

At the start of the construction period the Contractor formed a short section of the proposed "silt" shelf, which demonstrated the effectiveness of the design, before progressing with the rest of the required 800m length.

After the "silt" shelf was completed (see Fig. 7), reed beds were planted during late spring 2000, which was the best time of the year for their establishment. The reed beds were planted in the "silt" to consolidate the protection and enhance the environment. Timber platforms were constructed at intervals to provide "swims" for the anglers that use the reservoirs.

Fig. 7 View of placed “silt” shelf

<u>Reed Beds</u>
The depth that the reed beds sit in the water was important to their surviving and maturing, and the Contractor formed the “silt” shelf to a level of 8.00m AOD, which allowed for 0.20m settlement of the “silt” shelf. This level ensured that the roots of the plants were always submerged. The finished level of the “nicospan” revetment was 50mm above the top water level. The density of the reed beds planted was ten plants per square metre. The reed beds were planted during May, which was the best time of year for their establishment (Fig. 8). Also the Contractor’s design included the use of pre-planted reed “coir” rolls and mattresses, which minimised the chances of die back or natural waste of the reeds.

As these rolls were placed at the front of the shelf and the reeds in the rolls were established, they prevented erosion of the “silt” shelf whilst it consolidated and the planted reeds behind established themselves.

Fig. 8 Planting of reed beds

There is a thriving bird life on the Walthamstow Reservoirs, and the newly planted reeds would be susceptible to damage by the birds, therefore to minimise the damage, netting was placed over the reeds as protection. This type of netting is proven to deter wildfowl interest.

Fishing Platforms ("Swims")
There were ten wooden platforms built out into Reservoirs Nos. 2 and 3 along the 800m of the "silt" shelf and they were approximately 2m long by 3m wide. The swims provided a new safer access to the waterside, and the reed beds either side helped to conceal the outline of the fishermen to the fish. A plan and section of a platform is shown in Fig. 9.

ENVIRONMENTAL ISSUES WITH CONSTRUCTION

There were a few environmental issues identified at the early stages of the project, which were dealt with by the Contractor in a responsible way. There was a need for the Contractor, whilst dredging, to prevent disturbed suspended solids from passing further downstream and into the River Lea. This was done by erecting a geofabric boom sediment curtain at the outlet from Reservoir No. 2, for the extent of the construction period. This boom curtain was designed by the Contractor.

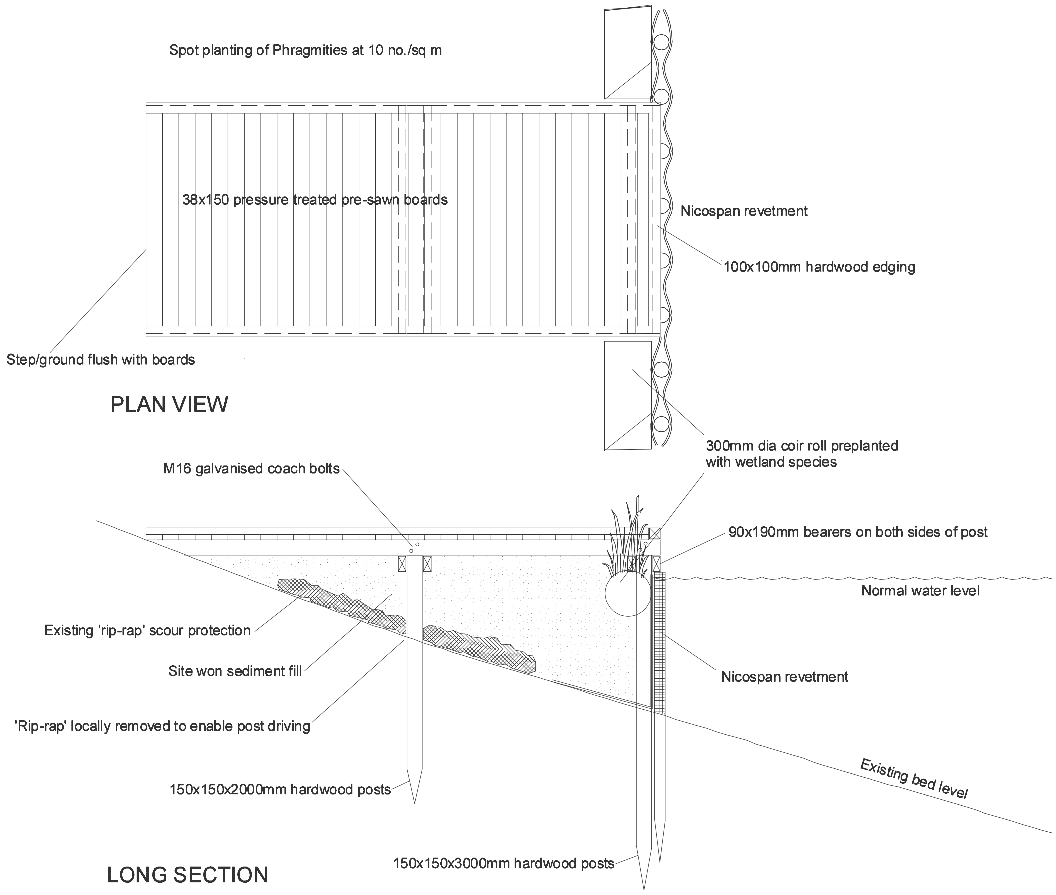

Fig. 9 Section of fishing platform

There are fish and other aquatic life within the reservoirs on the site and the Contractor monitored the dissolved oxygen and ammonia levels in Reservoirs Nos. 2 and 3 at least twice a day during the dredging. If the levels fell dramatically this would be likely to affect the fish and so the Contractor had on site emergency aeration equipment that could be immediately deployed to improve the water quality. The aeration was from a blower feeding a 1m diameter diffuser ring, but this equipment was not actually required to be used.

CONCLUSION

The “silt” shelf and reed beds were completed in May 2000, taking six months to complete, and apart from some minor secondary planting of reeds in early 2001, the reed beds are fully established and along with the “silt” shelf are fulfilling their function of protecting the embankments of the statutory reservoirs and providing biodiversity enhancements to the SSSI.

ACKNOWLEDMENTS

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SESSION 7
INCIDENTS AND REHABILITATION CASE HISTORIES (PART 1)

Chairman Jonathan Hinks
Technical Reporter Iain Hampson

Papers presented

1. Ericht & Dalwhinnie dam refurbishment and protection works
   K.J Dempster, M Gaskin, R.M Doake & D Hay-Smith

2. Wave assessment on Loch Ericht
   D Hay-Smith, R Doake, W Allsop & K McConnell

3. Papan Dam studies and remedies
   C Makinson

4. An incident at Ogston Reservoir
   A Hughes, P Kelham, D Littlemore & S Harwood

5. Rehabilitation design of Acciano rockfill dam after the September 1997 earthquake
   R Menga, M Eusebio, R Pellegrini & R Patacca

Papers not presented

6. Marmaric Dam investigations and remedial works
   L Spasic Gril & J Sawyer

7. Bewl Water spillway remedial works
   I Davison & K Shave

8. Remedial drainage to Laggan & Blackwater gravity dams
   R.P Wallis, A.C Morison & R Gunstensen

Martin Airey (Mott Macdonald)
The questions are related to the Ericht and Dalwhinnie papers. I think they are primarily perhaps for Debbie but as Debbie has departed, hopefully Kenny will be able to handle them. They are really prompted by your closing remarks Kenny - "is it all believable".

My first question – The increase in wave heights at both Ericht and Dalwhinnie was brought about by the use of a fetch line that more or less extends along the full length of the reservoir and does not take into account the proximity of the shoreline. This is a significant difference from the old Saville approach of the effective fetch, and one asks the question, is this totally applicable with a reservoir which is approximately 20 times longer than it is wide?

My second question - Was there any specific wave data available for the site (historic data or even anecdotal evidence given that the reservoir has been in existence for about 70 years), that suggests that waves of this height could actually occur?

The final question concerns the starting point, the starting level of the reservoir at the onset of the flood or wave incident. Was any probability analysis done as to what the most likely starting point would be? Clearly, if it was a situation that was some way lower than top water level, it could have a significant impact in reducing the scope of the works.

Kenny Dempster (Scottish & Southern Energy)
With my technical expert off to a wedding and my panel engineer unable to attend the conference, the asset manager must have broad shoulders! So, I can maybe give some answers to some of the points. I guess the fetch length is following the guidance, but I ask the question of the wider audience, is that correct? Perhaps this is not just for me to answer. In my personal opinion, there must be some doubt in assuming such long banana shaped fetches. However, there is always difficulty in going away from guidance – even though it is just guidance.

On the second point –there is no specific wave data available at Loch Ericht, although we can raise a few anecdotal points. The reservoir is a hydro generation reservoir and is traditionally kept low; we do not like to spill water because we do not make money on it! If you do carry out semi-statistical analysis on the reservoir water level, more often than not it is one or two metres below spillway level. This could be the starting point for a PMF flood estimation rather than spilling long-term average base flow. Indeed, for one or two of our other structures where we are facing stability issues, we are looking at re-evaluating the PMF level with the revised starting point. This differs from the guidance of "Floods and Reservoir Safety", so we are very much advocating moving away from guidance where we can provide full supporting evidence for doing so. I guess that has answered the third point at the same time.

Jonathan Hinks (Halcrow Group Ltd.)
I would like to ask a question of Roberto while I have the chance. You mentioned 0.5g as peak ground acceleration. This is presumably at the base of the dam?

Roberto Menga (Enel. Hydro S.p.A-ISMES Division)
This is not at the base of the dam.

The peak accelerations measured at the Nocera Umbra accelerometer station were about 0.5g along all three directions. (0.562g Horiz Long., 0.510g Horiz. Trans., 0.46g Vert.).
Processing of the Nocera Umbra records, adopting the attenuation laws, and taking site amplification data into account, based on local measures of micro tremors, PGA of 0.23g (W-E), 0.26g (N-S) and 0.31g (Vert.) were obtained. These values exceed those required by the Italian standards for dams (0.07g at the Acciano dam site), i.e. PGA 3 times the correspondent regulatory value.

Jonathan Hinks (Halcrow Group Ltd.)
So the acceleration at the base of the dam would be about 0.2g?

Roberto Menga (Enel. Hydro S.p.A-ISMES Division)
Yes.

Alan Brown (KBR)

(1.) Of Debbie/ Kenny - How was the required wave freeboard calculated, and had they estimated the quantity of wave overtopping? This may allow a reduction in wave freeboard if the embankment could sustain limited wave overtopping?

(2.) Of Colin - How will the diaphragm wall be secured into the abutments to ensure the interface is watertight, and how would overhangs (which are likely to be present with such steep valley sides) be dealt with?

(3.) Of Mr Riha – How many people were killed in the 1997 and 2002 floods (noting that he quoted damage in the 2002 floods as 2 billion Euro).

Kenny Dempster (Scottish & Southern Energy)

Quite a short answer I guess – Yes, the wave overtopping was considered and no, it wasn't tolerable with grass covered embankments supporting concrete core walls. A core wall was considered to be at risk from erosion and wouldn't stand consistent collapses so wave wall overtopping and remedial measures were deemed to be required.

Colin Makinson (Jacobs GIBB Ltd.)

I didn't touch on this because of the time restraints.

There are definitely vertical valley walls at depth and above the crest. There was no actual trench cut in this very hard limestone but certain overhangs were treated during the original construction by an ad hoc collection of steelwork and concrete infill.

The project has to make sure that the diaphragm wall makes a watertight connection with the rock, and the initial specification was for a 1m key trench into the rock face. This becomes more and more problematic when you consider the practicalities, because each panel will in fact cut into the rockhead as a triangle. The original panels were designed to be 9m long and upwards, so that some of these triangles were going to be enormous.

With the shorter 3m panel length we had the advantage of smaller triangles into rock, reducing the total volume of milling of the bedrock. Providing we do get a reasonable depth of penetration (however variable) we believe that, with rock contact grouting to 5m depth, we can achieve a watertight membrane, linked to and reinforcing the existing core and grout curtain.

Jonathan Hinks (Halcrow Group Ltd.)

Thank you Colin. Yes the sides of the gorge are extremely deep there and when we visited a few months ago there were rocks of several tonnes weight falling and landing on the face of the dam. We were asked to wear hard hats!!

J Riha (Brno University of Technology)

Sixty were killed in the floods in 1997. Seventy small (< 10m high) dams failed in the 2002 floods. Sixteen were killed in these floods (8 of whom were young ; the other 8 could have been heart attacks rather than drowning).

Ian Gowans (Independent Consultant)
Asked a question concerning the relationship between the planning process and works in the interests of safety, quoting some of his own experience.

Kenny Dempster (Scottish & Southern Energy)
You have obviously dealt with a much easier consent process than we have been going through. The consent process is Section 36 of the Electricity Act and it is presented to the Scottish Executive. There are a number of issues in the consent process. This one should have been and I quote "a very simple consent process" but here we are almost two years further on and we still await consent. I guess you could look at various reasons as to why is there a delay and I could come up with three reasons. One is a lack of understanding within the consent and emergency planning team of the Scottish Executive of what "measures to be taken in the interests of safety" mean and what the Reservoirs Act 1975 actually means. We have now got beyond that, and had dialogue to explain the position and the project drivers.

The second point is the objection we received to the scheme In broad terms, this is questioning the legality of what we are endeavouring to do. I don't really want to say much more about it but that process is now very time consuming. It is involving a legal review within the Scottish Executive and they have their own speed and time processes.

The third aspect is the simple fact that the Scottish Executive Emergency and Consents planning team are over-stretched. They are dealing with windfarm consents, other hydro consents and overhead line consents. This is just a small consent matter that fits in with everything else or so it would seem. However, it does create a dilemma between matters raised in the interest of safety and the consent process which is delaying the works and which is delaying the Company's spending £1 million on a capital project or a safety project.

Ian Gowans (Independent Consultant)
I will come back on that because only two days ago I got some very strong phone calls from Elgin. Once again some heavy flooding has occurred and that is only 50 miles from York as opposed to Dalwhinnie. Every 5 years major flooding takes place but no lives were lost. If we are serious about having to deal with this, then surely the priorities of human lives comes first, before the environmental objective of holding streams back?

Andy Hughes (KBR)
Just one comment. I have been in a similar situation where, let's say, crazy Government bodies have tried to delay schemes. I suggest you just send a simple letter to the Scottish Executive inviting them to take responsibility for reservoir safety and that might clarify the situation.

Jim Claydon (Yorkshire Water Services)
A question for David Littlemore about Ogsden. We have a similar situation with Larner Johnson valves discharging into a stilling basin, one of which has been replaced because it suffered wear in the same way as Ogsden. It will be replaced by a sluice gate - but that is by the way. If you had not been in the fortunate position that the Larner Johnson valve you took out was available for refurbishment what would you have put in its place?

David Littlemore (KBR)
I am sure a butterfly valve could have worked but perhaps Andy Hughes wants to comment on that.

Andy Hughes (KBR)
I think what is important to understand, and what was very evident when we did the analysis at Ogsden, was that the actual location of the valve just downstream of a 90^{o} bend led to a situation where you have got some very peculiar hydraulic characteristics within the water flow - a spiralling flow across the gate of the butterfly. If the butterfly valve had been fitted with a mechanism forming a linkage between the gearbox and the butterfly that stopped the butterfly closing, then we wouldn't have the situation we had. It was also a lesson to me, and I suggest it should be to all of you, in that when I made a simple request of the supplier of the valve as to why this had happened, he could not answer me. He said he had no technical expertise to answer that question - and that was from a major international valve manufacturer. So don't assume when you purchase anything that people are expert in their field – sometimes they are not! I personally would not have put a butterfly valve in that situation. I would have replaced it with a Larner Johnson valve.

SESSION 8
INCIDENTS AND REHABILITATION CASE HISTORIES (PART II)

Chairman Dr Andy Hughes
Technical Reporter Kenny Dempster

Papers presented

1. Lessons from a dam incident
   R Bridle

2. Some problems of small dams in the United Kingdom
   J.L Hinks & P.J Williams

3. The Washburn Valley reservoirs – spillway improvements
   J.R Claydon, D.L Knott & I.C Carter

4. Challenges and limits – the feasibility of underwater rehabilitation work
   C Heitefuss, H.J Kny & U Moschner

Papers not presented

5. Detailed investigation of an old masonry dam
   A Jaup

6. The long-term performance and remediation of a colloidal concrete dam
   K Dempster, J Findlay

7. The discontinuance of Devil's Dingle ash lagoon
   A Hughes & D Littlemore

8. Walthamstow Reservoirs No. 4 & No. 5 embankment protection
   C.B Peck

Mr Riha, (Czech Water Structures Institute)
Let me briefly introduce you to a few dam incidents during the 2002 flood in the Czech Republic. The flood came in two cyclones, the first one was slower but it filled all available retention storage and the second one was of longer duration. The larger river discharges were estimated between 500 to 1,000 year floods. Other smaller rivers ranged from 50 to 100 year floods. Prague, the capital was quite disastrously damaged including flooding of the subway and total losses reached about 2 billion euros.

The first example is from one of the biggest dams in Czech Republic, Orleague Dam. The gravity dam is about 100 metres high and the result of the extreme overspill put the associated hydropower plant out of operation. The power station discharge capacity, which was designed to pass the flood was lost and the water level in the reservoir reached more than 1.5 metres higher than it was ever assumed. The result was that the water flowed into ventilation openings, filled the dam and damaged all observation points and instrumentation. The water flowed out via other ventilation openings in the downstream face, so it was good for newspapers and TV who reported that the dam was cracking. As a result of this increased storage the flooding which reached Prague was reduced so the losses at Orleague Dam probably decreased losses in Prague.

The second example is from the south of Moravia in the vicinity of Perno Town located close to the boundary with Austria where a 25 metre high embankment dam with concrete blocks has been constructed. The problem was that the bridge deck was almost flooded and blocked by floating debris. There was then a serious danger of the dam then being overtopped. Just a couple of centimetres remained between the flood level and the deck, so the floating trees and blocks had to be picked out during the flood to maintain freeboard. After this rehabilitation of the fill at the crest and some experimental modelling of the flow behaviour and spillway arrangement are to be carried out.

Finally, I'd like to also mention that about 70 small dams, which means dams lower than 50 metres in height were also breached during the 2002 flood. Some failures were initiated by internal erosion that caused subsidence of the dam crest that then allowed overtopping. Wooden pipes traversed through the dam body and during the years some material was probably flushed through these pipes. One small dam was breached in this way. The second effect was downstream breaching initiated by the so-called domino effect. On one river system 8 dams breached due to the problems at one of the upstream dams. The village of Metly was engulfed with a volume of 1 million cubic metres and half of the village was flushed out during the flood. The breach at Metly was combined with sufficient spillway capacity but insufficient profile and overtopping at two places hence two breaches of 30-35 metres. At another dam the spillway completely filled with bushes and trees and the resultant breach developed over 9 hours, the asphalt road resisting for quite a long time. Finally another dam which was about 8 m high was breached due to improper maintenance and gate operation.

Peter Vaughan (Imperial College)
I was very interested in the last set of photographs on erosion damage due to the floods. I have had for some time some slight interest in trying to connect the amount of damage which occurs through a flood in terms of erosion on the ground that it is a way of tying in the risk element to a much longer timescale namely that of rainfall where right or wrongly we think that we know what the probability of a certain size of storm is. I wonder if any work is being done to record the amount of shifts of rivers as a consequence of these floods. My own contribution such as it is, is to buy a rather few rather old ordnance survey maps of the north of England that discover that the boundary of counties which were plotted in about 1870 is roughly where it is now and the rivers have not moved in other words, although there are signs in the occasional place of movement. It would be attractive to have a longer time base to work with and to predict rainfall and the size of floods then.

Mark Morris (HR Wallingford).
Look at the work being carried out in the University Catholic de Louvain (UCL – Yves Zech/ Sandra Soares) in Brussels. As part of the EC IMPACT Project they are looking at the effects on river morphology of extreme floods. This is more in the short term than the long term, but their work does look good, and addresses issues such as changing river plan form and large scale erosion/deposition of material during extreme events such as breach/ dambreak. New modelling approaches are being considered along with sediment transport relationships under these extreme conditions on the effects on river morphology of extreme

floods, more in the short term than the long term which does look good, changing the river plan form.

Alan Brown (KBR)
A question in relation to the failure of the small dam in the Czech Republic. Mention was made of the amount of money that was the physical damage, but was there any loss of life ? Also had people already been evacuated because of the floods resulting in reduced loss of life?

Mr Riha (Czech Water Structures Institute)
I have of course forgotten to mention the 16 casualties, some of which were due to heart attack or some form of shock, 8 people drowned out of those 16. This compares to the floods in 1997 when about 60 people were drowned. Improved organisation with the village mentioned evacuated in 2002.

Jim Millmore (Babtie Group)
A question to dam owners in the audience and it really follows on from Rod Bridle's previous point about the police reluctance to be involved in assisting with the evacuation of people following the incident in which he was concerned. We're spending quite a bit of time and money producing flood plans in this country and if the implication is however that the police are going to do nothing to help, are we embarking upon the wrong strategy?

Keith Gardiner (United Utilities)
United Utilities had discussions with Lancashire emergency planners, following the Rivington incident. This incident will be written up in the next issue of Dams and Reservoirs, so I can at last talk about it. But only about what happened on site.

The Lancashire Major Incident Control Group now invite UU along to their regular meetings. Its now been recognised that the police will take charge of evacuations in most circumstances. This incident has brought things out into the open and allowed open discussion with local emergency services. But, as Rod Bridle said, you need to know quite a lot about the susceptibility to drawdown, you need to know where all the vulnerable sites are downstream etc.

It's a huge subject about which not much has been said publicly to date, and I'm not sure that it will, the way the Reservoirs Act 1975 has been amended to make us produce flood plans and then make us not tell anybody about them. So these matters are far from settled. There's a long way to go.

Rod Bridle (Independent Consultant)
While we tend to think that a contingency plan is a plan to evacuate people from the dambreak floodway, a better and more achievable plan is to make sure you never have to evacuate people. This might be done by changing the probability of failure to be 'hardly ever', or by prolonging the time to failure so that fewer, or preferably no, lives are at risk. Actually the KBR FN plot shows half a person at risk. I'm still wondering who that half person is! But the principle is to attend to the condition of your reservoir long before anything goes wrong. This is within your control and likely to be more successful than relying on evacuation of people at risk. Evacuation puts huge pressure on unsuspecting people who are going to find it difficult to evacuate in accordance with plans. They are at terrible risk. We should do everything we can within our control, and the ideal approach is to

take steps that alter the condition of the dam that shift the location on the F-N plot of as many reservoirs as possible to the left, that is to locations where lives at risk are zero.

Colin Hunt (Bristol Water).
The approach that Bristol Water have taken is to carry out dam break analysis on all their relevant reservoirs. In accordance with DEFRA instructions, a controlled document is held by Bristol Water plc. Another controlled document has been offered to the Environment Agency.

Bristol Water have invited all the Emergency Planning Officers from the respective Local Authorities to look through the controlled documents so that they can draw up Emergency Evacuation Plans.

In the event of a major incident the Emergency Planning Officer will take responsibility for any evacuation leaving Bristol Water to deal with reservoir structure.

The Local Authority has a responsibility through their Emergency Planning Officer to put in place an Emergency Plan to deal with these types of incidents.

Peter Kite (Independent Consultant)
Question for Rod Bridle. What emergency plans existed before the emergency incident and who took responsibility during the incident in terms of making sure that the off site people were informed and finally were any exercises undertaken before or any planned in the new emergency plans that are being drawn up?

Rod Bridle (Independent Consultant)
Dambreak studies had been done for the reservoir in question. In fact we were able to resurrect the model to examine the consequences of differing reservoir volumes at the commencement of dambreak, in what turned out at this reservoir to be a vain attempt to identify the ‘critical volume’ at which a dambreak would not put lives at risk. As I said the owners were brilliant and they coordinated all the communications with the emergency authorities. Everyone responded well. However, as I said, there had not been any rehearsals, and the folks who were at risk had not the vaguest idea they were at risk at all.

Mark Morris (HR Wallingford)
Just following up on that point, has anyone done any work to assess the difference and effectiveness of action plan when people are aware or unaware in advance and if there’s a difference who takes that responsibility in deciding to keep this in or out of the public domain?

Alan Brown (KBR)
In response to the comment as to why likely loss of life is a fraction of one, note the important difference between :

1. Population at risk (PAR) which is the population where
- the water depth, above natural ground level, is greater than 0.5m (thus including those in houses who could step out into the flood)
- the product of depth and velocity is greater than 0.5 m^2/s

2. Likely loss of life

- The PAR multiplied by the fatality rate, which depends on water depth and velocity, whether a warning had been given and the effectiveness of the warning. In steep valleys with no warning it will approach 100%; in wide valleys with shallow gradients it may be as low as 1%.

The reasons for likely loss of life as a number less than 1 is simply the output from the multiplication of PAR and fatality rate

Information on how the fatality rate may vary with quality and amount of warning is given in a report by Wayne Graham of the US Bureau of Reclamation (BOR, 1999, A procedure for estimating Loss of life caused by dam failure. DSO-99-06. Sept 43pp at www.usbr.gov/ssle/dam_safety/risk/Estimating%20life%20loss.pdf). This highlights the need for education of the public and others, so that if a warning is given the population would understand the need to make for high ground.

Rod Bridle (Independent Consultant)
How did the other papers relate to improving the safety of our reservoirs? Christian Heitefuss told us how to improve emptying valves to empty reservoirs quickly, an excellent means of improving safety and reducing risk to lives.

Jonathan Hinks and Dave Knott did not mention probabilities of failure, but can I ask if they know what the probability of failure from internal erosion is at their dams? Are we concentrating over-much on overflow capacity and not enough on internal erosion?

Jonathan Hinks (Halcrow Group Ltd)
I am very pleased at the emphasis placed at this conference on internal erosion. In the past a lot of attention has been given to slips because we understand them and know how to calculate the factors of safety. However Foster, Fell and Spannagle say that only 4% of dam failures are attributable to slope instability whereas internal erosion accounts for 48% of failures. It is important that we understand as much as possible about internal erosion.

The figures on which I based the above remark are at the top of page 222 of the paper and the Foster, Fell and Spannagle references are given on pages 230 and 231.

Andy Hughes (KBR)
Certainly in terms of the Rivington incident having two operable 600mm diameter scours was a valuable feature.

Christian Heitfuss (Ruhrverband – Ruhr River Association)
Yes, I can say that I've seen quite a few bottom outlets all over the world which seem to need more care and that some owners do not know yet what kind of problems they will run into. If you look at valves, about 100 year old, made from cast iron, affected by spongiosis and graphitisation, you cannot exactly say if that old cast iron will last 10 or 20 or 30 years. You have to do something about it and that has been our approach in the last 20 years: to remove the old valves and replace them by new and safer ones.

I am not in a position to talk about the risks of other modes of failure at these dams. But I would just make one comment. We had a reservoir management plan to reduce the risk of spill during construction to less than 1%, as is usual. Of course the risk event happened immediately during construction of the works. Somehow very low levels of flood risk don't seem what they used to be, bearing in mind as well some of the other speakers' comments, so perhaps food for thought.