The reservoir as an asset

THE BRITISH DAM SOCIETY

The reservoir as an asset

Proceedings of the ninth conference of the British Dam Society held at the University of York on 11–14 September 1996

Thomas Telford

Conference organised by the British Dam Society

Organising committee: I C Carter, D P M Dutton, J L Hinks (Chairman), Dr A K Hughes, A C Robertshaw and Dr P Tedd

Published by Thomas Telford Publishing, Thomas Telford Services Ltd, 1 Heron Quay, London E14 4JD

First published 1996

Distributors for Thomas Telford books are
USA: American Society of Civil Engineers, Publications Sales Department, 345 East 47th Street, New York, NY 10017-2398
Japan: Maruzen Co. Ltd, Book Department, 3–10 Nihonbashi 2-chome, Chuo-ku, Tokyo 103
Australia: DA Books and Journals, 648 Whitehorse Road, Mitcham 3132, Victoria

ISBN: 0 7277 2528 9

Printed in Great Britain by The Cromwell Press, Melksham, Wilts.

Preface

The 1996 British Dam Society Conference held in York is the ninth of the society's biennial conferences. The theme for the conference "The Reservoir as an Asset" is reflected in the papers principally in terms of asset management and reservoir safety management. These conference proceedings contain a valuable collection of 28 papers by 45 authors on a large range of subjects in the field of dam engineering including embankment dams, concrete dams, service reservoirs, investigations and reservoir safety standards.

With little recent construction of large reservoirs in Great Britain, papers on British dams are confined to remedial works and investigations, mainly concerned with increasing spillway capacity. The first use of a proprietary fusegate system in this country is presented. A significant proportion of the reservoirs in Great Britain are service reservoirs. The particular problems and remedial works associated with ageing service reservoirs both in Britain and in Hong Kong are addressed.

Papers by British consulting engineers describe major dam construction projects overseas, including a 150m high concrete faced rockfill dam in Greece, roller compacted concrete dams in Jordan, a concrete arch dam in Lesotho and a number of embankment dams. Aspects of reservoir sedimentation in New Zealand are described.

The operation of the Reservoirs Act 1975 from an owner's and a Supervising Engineer's view are discussed. Legislation in Europe, and the development of dam safety legislation in Indonesia are described.

Discussion at the conference and the Geoffrey Binnie lecture given at York by Theo van Robbroeck, President of the International Commission on Large Dams, are published in the Society's Journal, Dams & Reservoirs.

Contents

Embankment dams

Investigation, instrumentation and monitoring

Concrete dams and service reservoirs

Dam safety, standards and reservoir management

The design of an auxiliary spillway at Dove Stone reservoir using tipping fusegates.

K D GARDINER, Bechtel Water Technology, UK.

SYNOPSIS
The existing overflow at Dove Stone reservoir had insufficient capacity to pass the design flood. The recommended solution was to provide an auxiliary spillway with a 55m long weir. Significant cost savings were achieved by using tipping fusegates to reduce the weir length to 9.8m. Model testing was necessary to prove the design and to optimize the final arrangement to produce additional savings.

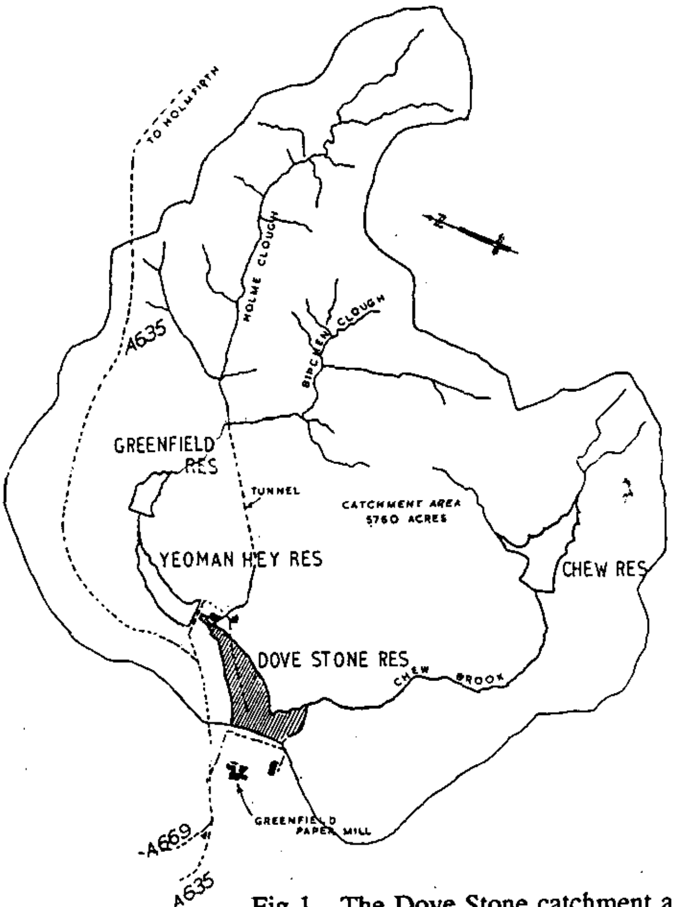


Fig 1. The Dove Stone catchment area.

The reservoir as an asset. Thomas Telford, London, 1996

INTRODUCTION

Dove Stone is the lowest reservoir of a group of four, in the Greenfield and Chew valleys, on a tributary of the river Thame, 9km east of Oldham, Greater Manchester. See Fig 1. The dam, designed by G H Hill & Sons, is an earth embankment with a puddle clay core, concrete cutoff and a grout curtain. Construction was completed in 1967. The dam is designated Category A as defined by "Floods and Reservoir Safety - An Engineering Guide". Yeoman Hey and Greenfield reservoirs were fitted with concrete tipping gates in 1980-82. (Ackers and Hughes 1988)

The embankment is 540m long with a maximum height of 31.4m. The reservoir storage capacity is approximately 5 million m^3 and the area at top water level is 334×10^3 m^2. A supply of 16Ml/day is made to a paper mill immediately downstream and up to 35Ml/day can be supplied to Buckton Castle water treatment works which serves the Ashton-under-Lyne and Oldham areas.

The existing overflow consists of a 24.4m dia bellmouth spillway with a masonry weir, discharging into a 3.66m dia vertical dropshaft. This is connected by a 90deg vertical bend and transition to a horseshoe section tunnel beneath the embankment. The tunnel discharges into a stilling basin and thence to Chew Brook. The valve house is constructed immediately alongside the drop shaft and this structure and two anti-vortex piers reduce the weir length to 61.3m.

THE PROBLEM

A report on the overflow capacity of the group of four reservoirs in 1980 concluded that during the Probable Maximum Flood, the Dove Stone embankment would be overtopped. The PMF outflow was $299m^3/s$ and it was calculated that the overflow tunnel had a maximum capacity of $163m^3/s$.

SOLUTIONS

After consideration of many solutions, the options were reduced to two in the final Planning Note submitted to North West Water in 1993. The cheapest option, to lower the existing weir by 2.5m, had already been rejected because the loss of yield from the reservoir would be unacceptable. There were also objections on environmental and amenity grounds. The final options required the construction of an auxiliary overflow either at the north or south end of the embankment. The option to construct the spillway at the south end of the dam required the construction of a 56m long weir on natural ground set to operate at 0.3 PMF. This option was rejected because the access road to the sailing club would have been routed through the spillway which, although it operates only rarely, would be subject to wave splash. Also, additional land would need to be purchased.

The recommended solution proposed the construction of a 55m long weir and tumbling bay on the embankment at the north end, immediately behind the existing overflow. A 14m wide, 60m long spillway discharged the additional flood water onto the adjacent steep hillside to join Chew Brook near the stilling basin. See Fig 2.

The Planning Note was given to Bechtel Water Technology, BeWT, (then North West Water Engineering), as a design brief in November 1994. BeWT had already had some contact with Hydroplus International on the use of their tipping fusegates to maintain the storage in reservoirs where the cill level was to be lowered, and invited them to consider the use of fusegates at Dove Stone. The primary use of fusegates is to increase storage in an existing reservoir by allowing use to be made of the freeboard between the overflow weir and the dam crest. At Dove Stone reservoir it was realised that the 55m weir length could be shortened considerably if the overflow cill could be lowered, and the storage in the reservoir maintained using fusegates. Three gates each 2.15m high and 3.23m wide were proposed, giving a weir length of 9.8m after the gates have tipped.

In the recommended option, the weir was set at the 0.2PMF level and would come into operation for floods with return periods greater than about 1 in 150 years. The weirs on the fusegates were also to be set at this level. Fusegates tip when a chamber connected to the underside of the gate fills with water producing an uplift force that rapidly reduces the factor of safety against overturning from about 2.0 to less than 1. By setting the top of this chamber at a different level for each gate they can be made to tip at different flood outflows.

MODEL TESTING

The chosen location for the auxiliary overflow was immediately behind the existing bellmouth spillway. Therefore the hydraulic conditions at the approach to the new weir were not possible to predict accurately and a model test was considered essential. The contract to construct and test the model was awarded to Salford University Civil Engineering Ltd.

The model was built such that both the recommended option and the alternative solution using fusegates could be investigated. The natural hillside down which auxiliary flood flows will pass and the access road that runs across it were also constructed. To verify the calculated capacity of the existing overflow arrangement, the bellmouth weir, dropshaft, tunnel and stilling basin were accurately modelled, together with part of Chew Brook. The chosen model scale was 1 in 21.

Existing overflow

The hydraulic behaviour of the existing overflow was first investigated. The tunnel first began to choke at 153m^3/s and at the maximum allowable reservoir level of 218.37m AOD the outflow was 175m^3/s. This was in line with the calculated values.

Despite the presence of the valve house and two anti-vortex piers, flow circulated around the bellmouth in a clockwise direction. The section of weir adjacent to the circular retaining wall was fed entirely by water flowing behind the valve house at up to 5m/s at peak flows. It was also revealed that above 75m^3/s the flow emerging from the tunnel portal impacts directly upon the opposite spillway wall and that flows greater than this were likely to cause considerable damage.

Long weir solution

The 55m long weir solution was then modelled. The requirement for a tumbling bay behind the weir meant that the weir itself was positioned about 6m in front of the wavewall and thus approached within 8m of the bellmouth. The tumbling bay was 12.5m wide and fed into the 14m wide spillway channel which was to be constructed across the natural hillside adjacent to the north abutment. See Fig 2. It was found that the auxiliary weir came into operation at 57m^3/s, very close to the 0.2PMF value, and that the weir could be shortened to 46m before the maximum allowable reservoir level was reached. This reduction on its own would have produced sufficient saving in the prototype to pay for the model testing .

The proximity of the existing and auxiliary weirs and the circulatory flow were found not to affect the performance of either overflow.

Fusegate solution

After the long weir had been tested, the model was modified for the fusegate solution by extending the south wall of the spillway parallel to the north wall until it met the line of the long weir. The fusegates were not modelled, but were replaced by small plain handstops that had their crests at the same level as the long weir. The fusegate weirs are labyrinthine and therefore would pass more flow than the handstops, however, since at maximum flow all the gates would be tipped, it was considered unnecessary to construct scale model fusegates. The handstops could be taken out in any order to determine the optimum tipping sequence.

The handstops were removed at flows predetermined by flood routing through the reservoir carried out by Hydroplus. Flows were increased up to the design outflow flood of 300m^3/s in stages and the handstops were taken out at flows of 175, 195 and 220m^3/s.

Although the entire length of the weir in this solution was very close to the existing bellmouth, this arrangement was found to be satisfactory and would pass the design flow at a reservoir level 100mm below the maximum allowed.

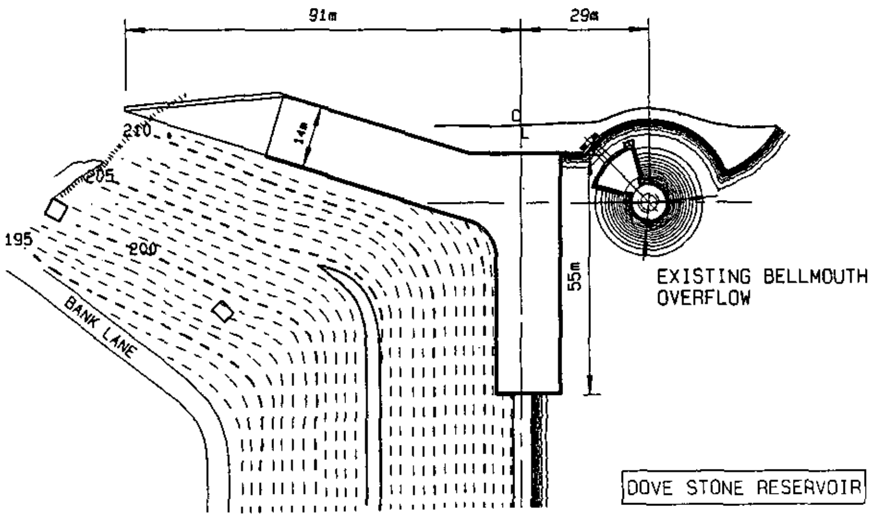


FIG. 2 PLAN SHOWING LONG WEIR SOLUTION

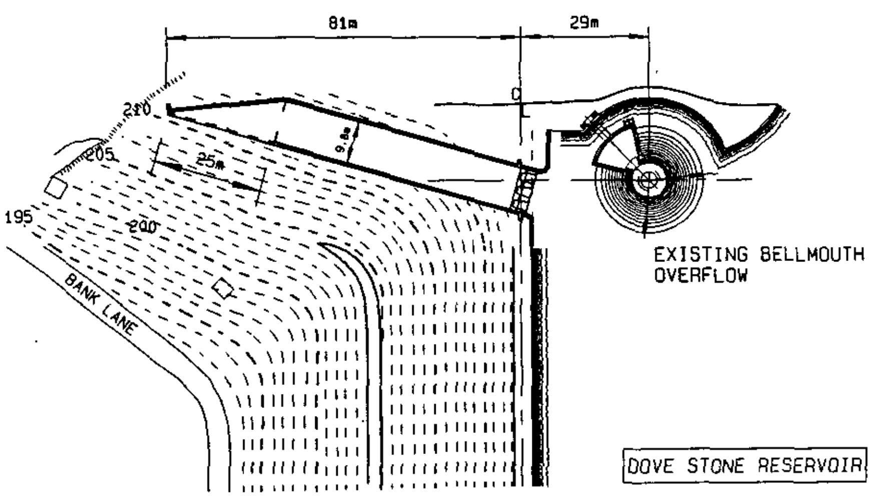


FIG. 3 PLAN SHOWING FUSEGATE SOLUTION

Modified fusegate solution

In the original fusegate layout, it was found that as the gates were removed the water profile measured immediately upstream became irregular. Water piled up at the gate nearest the valve house if the other two gates were removed first. It was recognised that this asymmetry of flow could cause difficulty in setting the correct tipping level in the prototype. Therefore, to improve the approach conditions, the fusegates were moved as far downstream as possible without impounding water beyond the clay core. The wingwalls were moved back to the line of the wavewall and curved to prevent the flow separation witnessed with the original layout. The kink in the spillway was removed and the gates positioned at right-angles to the walls. It was also decided to reduce the width of the spillway to the width of the gates. See Figs 3 & 5.

These modifications proved very successful. The upstream profile remained horizontal at all flow rates, independent of the order of gate removal. However, the height of the cross waves that occur in the supercritical flow in the downstream channel was found to vary with the removal sequence. Water levels were plotted along the southern wall, for all flows and tipping combinations, to determine the minimum height to which it must be constructed to avoid overtopping. It was found that the narrower channel did not require a significantly higher wall to contain the flow. Therefore savings can be made on excavation and concrete volumes in the prototype. The height of the north wall is fixed by the depth of the excavation into the hillside.

Downstream conditions

The Planning Note had recommended the construction of a rockfill bund down the mitre to protect the embankment from auxiliary flows, but model testing revealed that this would not be required. The spillway as modelled discharged all flows onto the hillside clear of the mitre. However, early in the model testing it was observed that a considerable volume of water would flow along the access road, Bank Lane, back towards the dam. This was prevented by changing the alignment of the road to obtain a high point near the mitre that this flow could not cross. Unfortunately, this realignment cannot now be carried out as the road would need to be closed for six weeks and access was denied along the private road that would provide the only alternative route for local residents. Therefore, since the flood flow along Bank Lane cannot be contained, it will be directed down a 50m wide section of the lower embankment slope protected with a grassed revetment system, to discharge beyond the toe. See Fig 4. The expected maximum flow down Bank Lane is approximately $7m^3/s$.

During the final stages of testing, the oblique wall at the end of the spillway channel was hinged to ascertain the optimum angle. The flow is supercritical in the spillway, culminating in a hydraulic jump in the triangular apron area whatever the outflow rate. By increasing the angle of the end wall, the width of the water sheet flowing down the hillside could be reduced, but this caused the depth of the hydraulic jump to increase requiring an increase in the wall height. It was found that if the apron width is less than 27m the flow is just contained between the two inspection chambers which stand proud of the hillside. For a width less than 20m, the flow is directed across the roof of the pumping station next to the tunnel portal. See Fig 4. The final width chosen was 25m. At this width the flow surge that occurs when a gate is removed does not cause flow to impinge upon the further inspection chamber. Flow surges were not perceived to be a problem in the model; the flow regime was little different to the steady state condition.

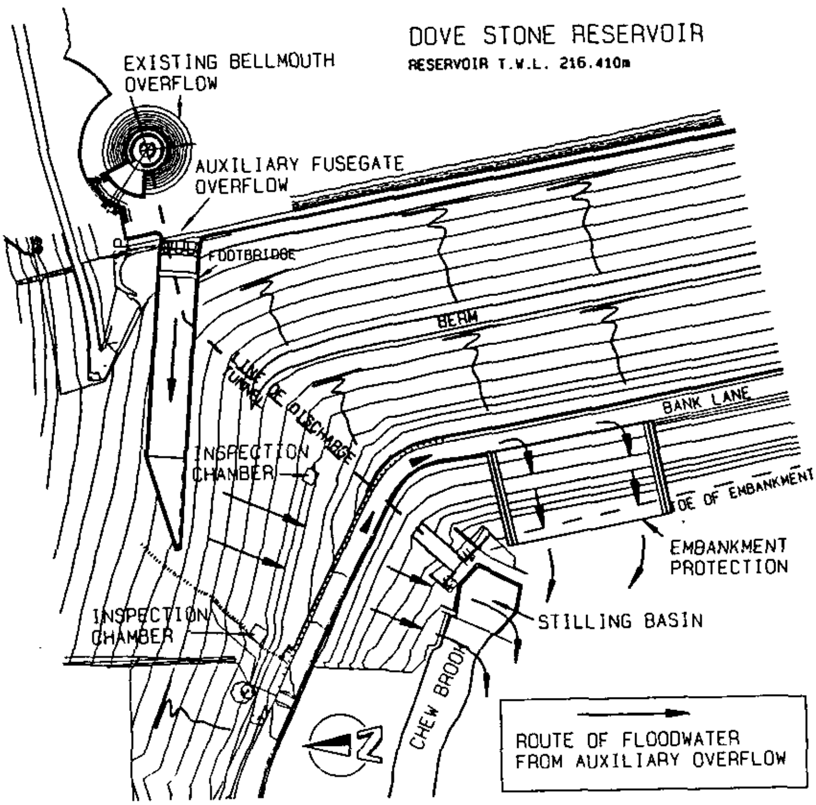


FIG.4 EXISTING AND PROPOSED AUXILIARY OVERFLOWS

With the fusegate option, the flow down the hillside is less than with the long weir option for total flows up to 175m^3/s, when the first gate tips. Thereafter, proportionately more flow uses the hillside route, especially after the peak of the flood has passed since the gate cill is 1.59m below the existing bellmouth weir. The maximum proportion by which the hillside flow in the fusegate option exceeds that for the long weir is approximately 15%, just after the first gate tips.

ADVANTAGES

The alternative solution, as optimized using the model, was approximately £100,000 cheaper than the original recommendation including the cost of the fusegates. At the time of writing, December 1995, this solution is under construction. The total cost of the capital works is approximately £800,000.

The other major advantage of the fusegate solution was that the entire construction could be founded on natural ground. If the long weir solution were constructed, it would be founded partly on the embankment and partly in the abutment. The water pressure on the weir cill would need to be resisted by the embankment and some degree of articulation would need to be introduced within the structure to accommodate differential settlement. Also the effect of seismic forces on such a structure would need to be considered carefully.

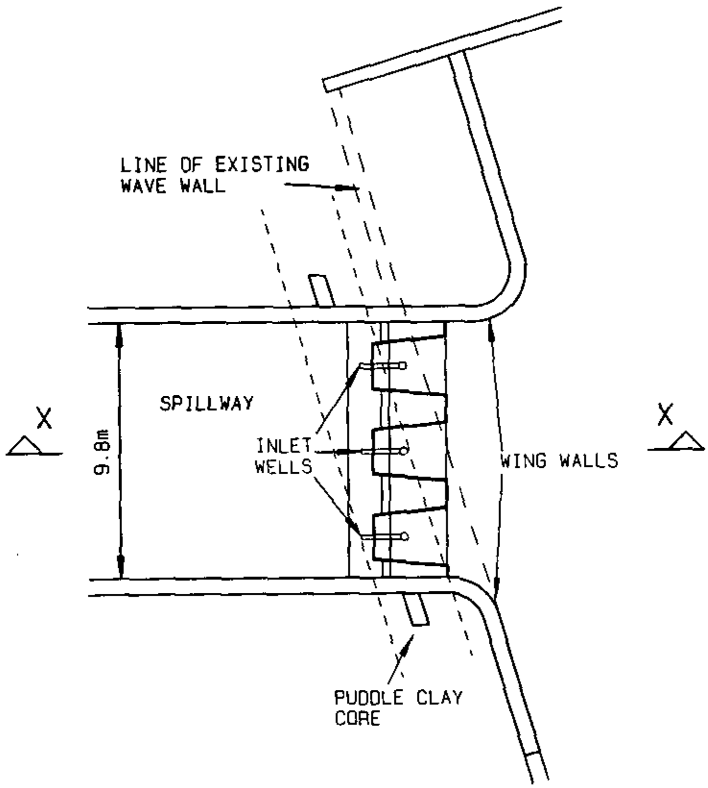
With the alternative solution, the thrust on the gates and the wingwall is taken by the long spillway channel. At the abutment, the concrete cutoff wall projects above the formation level of the spillway at the north wall, but this will be broken out locally during construction and replaced with puddle clay to give uniform foundation support across the channel width. The structure will be sealed to the clay core by puddling against a concrete key cast onto the sidewalls. See Fig 5.

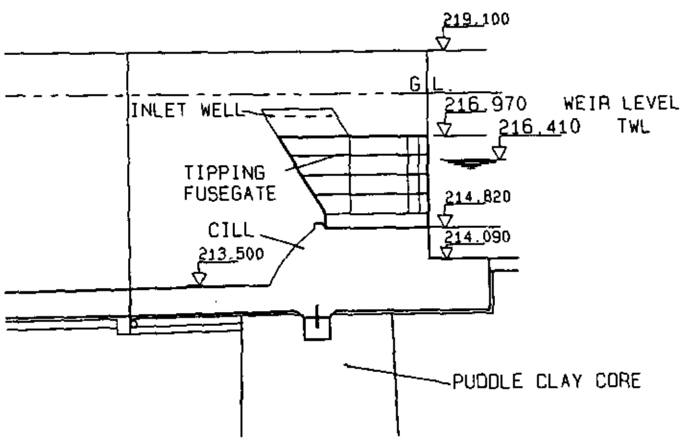
PUBLIC SAFETY

The flow immediately downstream of the reservoir will be slightly greater with the fusegate solution than with the long weir solution, but it has been calculated that at no stage, even immediately after a gate has tipped, will the outflow be greater than the inflow. Therefore the flood will be no greater than if the dam had not been constructed.

The most obvious danger to the public will be caused by flows passing across Bank Lane, the only access to the sailing club, the public car park and several farms and houses. The auxiliary overflow will operate only during floods with return periods greater than 150 years, so this will be a once-in-a-lifetime event. Wave splash will occur from time to time but this will be piped into the stilling basin. Signs are to be erected on either side of the affected section of road, warning vehicle drivers and pedestrians not to attempt to cross if water is flowing down the hillside.

Once auxiliary flow becomes established, velocity of flow across the road will be up to 16m/s and the flood will be accompanied by much turbulence and spray as it hits the barrier alongside the road. It is considered unlikely





SECTION X:X

FIG.5 FUSEGATES

that anyone would attempt to cross in these circumstances. Prolonged flows are likely to cause extensive erosion of the hillside and could possibly wash out the road, but the dam will have been protected.

The eventual resting place of a tipped fusegate has also been considered. The prospect of a steel fusegate weighing 2.5tonnes tumbling down the hillside is daunting, however it is impossible that anyone could get in its way as it will be preceded by a $20m^3/s$ torrent. Once the gate reaches Chew Brook, it will be in a flood of approximately $190m^3/s$, which is far in excess of the downstream channel capacity, and the presence of the gate will not increase the hazard to people and property downstream significantly. It must also be remembered that this will be an extremely rare event, only occurring once in about 10,000 years.

CONCLUSION

The use of tipping fusegates has produced a cost effective alternative to the original design and has resulted in a structure which can be founded entirely on natural ground.

ACKNOWLEDGEMENTS

The author is grateful to North West Water Ltd for their kind permission to publish this paper, and to Dr A K Hughes, who has been retained by BeWT to certificate the works under the Reservoirs Act 1975, for his suggestions and comments.

REFERENCES

Ackers J C and Hughes A K(1988). Tipping gates for auxiliary spillway control. Proceedings of a symposium by BNCOLD - University of Manchester, Paper 4.5. BNCOLD.

Walshaw Dean Reservoirs - Spillway Improvements

P C HARRISON, Babcock Water Engineering Ltd, UK
J DRABBLE, Babcock Water Engineering Ltd, UK

SYNOPSIS. An extreme rainfall event over the Walshaw Dean valley caused considerable damage to the masonry byewash channels of the three reservoirs. A subsequent flood study and physical hydraulic models proved that there were significant shortcomings in the discharge capability of all three spillway systems. The Lower reservoir required modifications to the existing spillway, together with a new supplementary spillway at the opposite end of the embankment. The Middle and Upper reservoirs required improvements to the weirs, tumblebays and upper spillway channels. This paper outlines the key features of the design and construction of the spillway improvements.

INTRODUCTION. The three Walshaw Dean reservoirs are situated in the Pennines approximately 15 km north west of Halifax and were designed by G H Hill and constructed between 1901 and 1915. They are conventional earth embankments with puddle clay cores and cut off trenches with details documented in comprehensive record drawings.

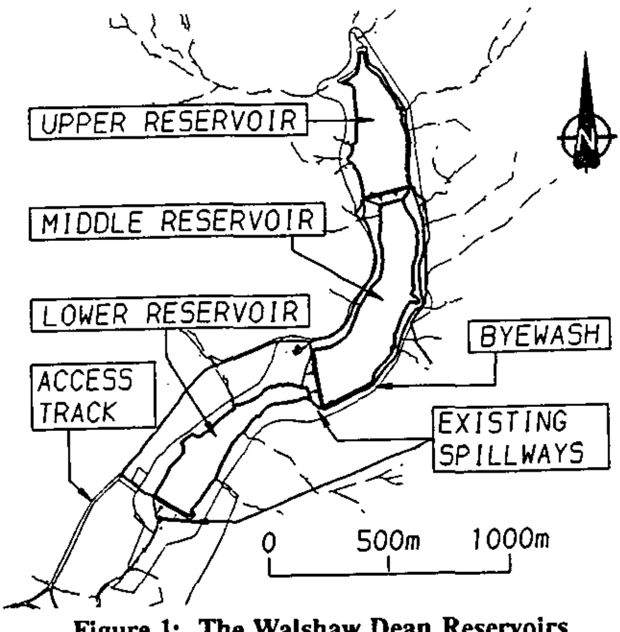


Figure 1: The Walshaw Dean Reservoirs

The reservoir as an asset. Thomas Telford, London, 1996

The Middle reservoir suffered from early settlement and was one of the first reservoirs in the UK to be subjected to remedial grouting of the foundations in 1915. Further grouting of the Middle reservoir puddle clay core was undertaken in 1938 following substantial settlement of the crest.

On 19 May 1989 193mm rainfall was recorded in a rain gauge located beside the Middle reservoir. The majority of this rainfall is reputed to have fallen during approximately 2 hours, but this storm intensity has been the subject of some controversy. A subsequent statutory inspection under the Reservoir Act 1975 by All Reservoirs Panel Engineer John E Massey, recommended that the reservoirs owner, Yorkshire Water (YW), commission a flood study and physical hydraulic modelling of the three reservoirs. These were undertaken by consultants Mott MacDonald and Hydraulic Models Ltd under Mr Massey's guidance.

Due to stilling basin limitations at the toe of the embankment, the Lower reservoir spillway was found to be capable of safely discharging 25 cumec, whereas the PMF overflow was 117 cumec. It was decided that the shortfall could most effectively be provided by the construction of a supplementary spillway with a capacity of 93 cumec on the right hand abutment. The Middle reservoir spillway was found to be capable of safely discharging 24.6 cumec, whereas the PMF was 67.6 cumec (excluding 35 cumec passing through the byewash channel). Improvements were achieved by providing a new longer weir of ogee profile, and constructing a new deeper tumblebay and upper spillway channel. The Upper reservoir was found to have a shortfall of 37 cumec and improvement works are similar to the Middle reservoir.

Due to the need to secure the embankments and works during construction, and also maintain some storage in the valley, improvements were made to one reservoir at a time, commencing with the Lower reservoir and working up the valley. The first 52 week contract commenced in November 1993, the Middle reservoir contract commenced in June 1995, and with a short overlap, the Upper reservoir contract commenced in April 1996.

WALSHAW DEAN LOWER RESERVOIR

Design. Hydraulic calculations for the side overflow weir, tumblebay, channel and energy dissipator were verified by a 1/30th full size scale model. Testing allowed optimum shapes and sections to be developed and verified. With a peak entry velocity of 14.8m/sec into the energy dissipator, the initially proposed USBR type structure with chute blocks was found to be inadequate and extensive erosion damage would have

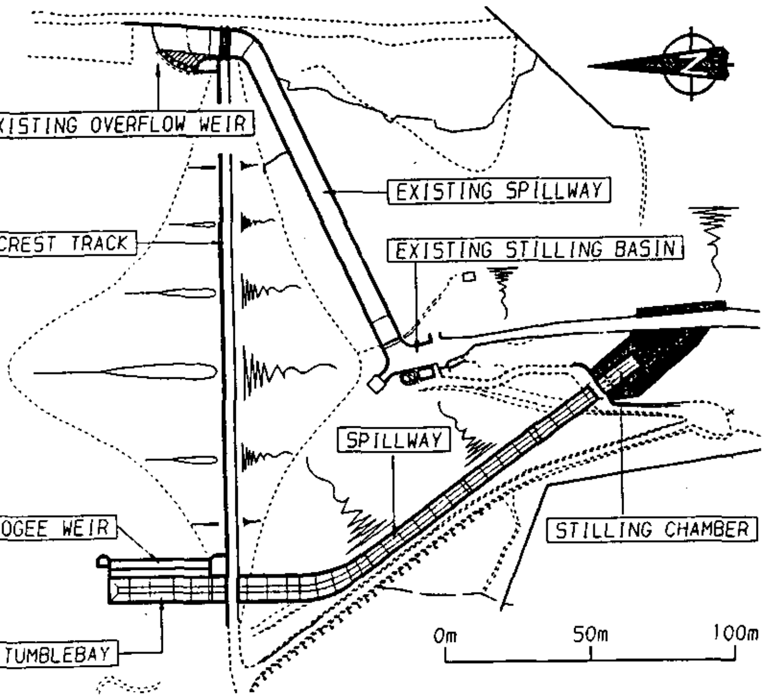


Figure 2: Walshaw Dean Lower Reservoir Spillway

resulted. A compact hybrid solution was developed consisting of an open topped chamber of sufficient proportions to promote the formation of a standing wave at the downstream end of the spillway. Excess energy was dissipated by the standing wave, with lower velocity flow discharged over the top of the structure. In order that a permanently flooded structure did not pose a safety risk a small outlet port was provided at chamber bed level, which is adequate to discharge short return period floods without inducing overtopping.

Tumblebay. The design was subject to the following requirements:

* Capability of discharging the PMF
* Resistance to uplift
* Water exclusion
* Resistance to the effects of moorland water, high velocity flow and frost

Alternative systems to counter uplift pressures of 57 KN/m^2 were considered, including rock anchorages, self weight and extended toes mobilising the weight of backfill. Rock anchorages were seriously considered given the competent nature of the rock foundations and their relatively low cost. However, they would be exposed to moorland

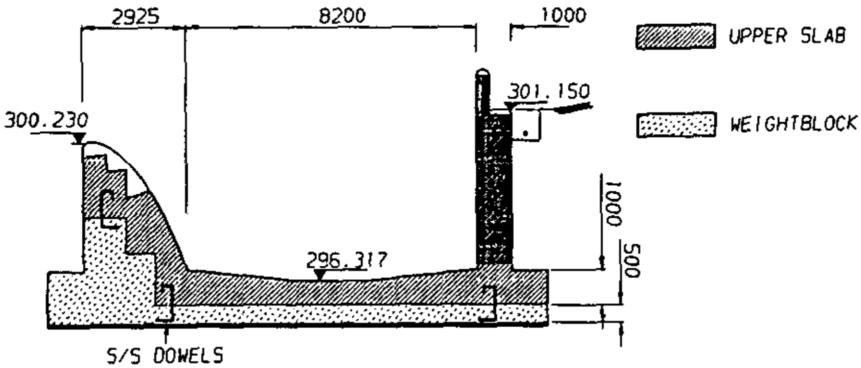


Figure 3: Lower Reservoir Tumblebay Section

water, and corrosion could put the structure at risk. The client, YWS, was also apprehensive about the commitment to monitor the performance of the anchorages in the longer term. Toes extended beyond the rear of the Tumblebay walls may be used to counter uplift forces when the weight of backfill is taken into account. However, to rely exclusively on this mechanism was considered to be unsatisfactory as it is not feasible to ensure that all future excavations follow appropriate precautions. Self weight was considered to be the most reliable option, but in view of the potential problems associated with thick concrete sections and susceptibility to moorland water, it was necessary to control the heat of hydration whilst maintaining the strength of the mix. The solution developed was to split the base slab into 2 layers as shown in Fig 3. The lower slab, termed the weightblock, comprised a low heat C35 concrete utilising 70% cement replacement with GGBS. The upper slab comprised C40 concrete with air entrainment at 5% and 50% cement replacement for resistance to frost, moorland water and high velocity flows. The two slabs were dowelled together using stainless steel reinforcement. The walls were cast using a similar mix to the upper slab except that the air entrainment was omitted. Toes were added to the rear of the retaining walls to add to the total resistance to uplift. These features contribute 15% to the total uplift resistance required and were designed to allow removal of backfill to the Rear of Wall Drains for maintenance.

Ogee Weir. Forming the 35m long ogee profile to fine tolerances by casting concrete in-situ was considered to be impractical. Precasting as specified allows high quality concrete blocks finished to close tolerances of form to be accurately positioned on site to achieve the required tolerances for the weir crest level and profile. Details of the weir blocks are shown in Fig 4. Staggering of the joints on plan was

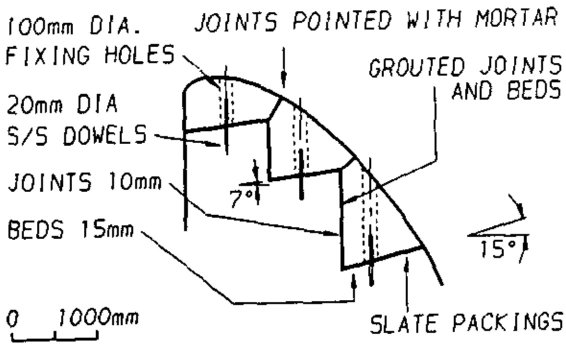


Figure 4: Precast Concrete Ogee Weir Construction

introduced in order to minimise any seepage. All joints were pointed with mortar which acts as a permanent shutter during the subsequent grouting operations to seal the joints between and beneath each block. Slopes of 7.5° and 15° across the beds were specified to encourage grout flow and minimise entrapment of air.

Temporary Works. Construction of the new overflow works at the Lower Reservoir benefitted in respect of reservoir safety from retention of the existing overflow facilities and the early demolition of the 0.61m high by 23.47m long existing overflow weir section to promote earlier release of rising floodwaters during the construction period. Following drawdown to accommodate a 1 in 20 year flood, a line of sheet piles (refer to Fig 5) were driven into the puddle clay core along the crest centreline prior to any excavation within the embankment. The design accommodated a flood rise to a level of 300.73mAOD until construction

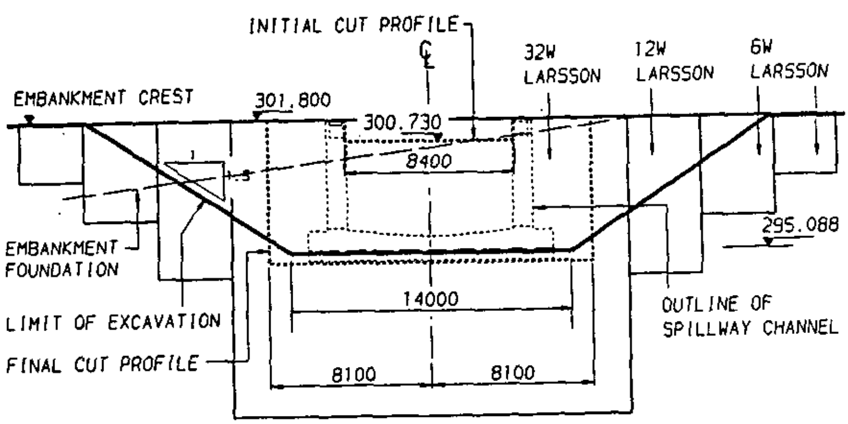


Figure 5: Sheet Pile Wall Through Puddle Clay Cut-Off

of the core crossing was commenced. This was controlled by an 8.4m wide slot cut out of the sheet piles central to the new spillway once excavation within the embankment profile had commenced. Temporary earth banks were required by the specification to contain overflow and direct any flow over the sheet pile wall into the new spillway. Construction of the core crossing required that the sheet piles be cut back to their final profile and excavation be completed. The specification stressed that the Contractor should expedite this element of the works and submit a detailed programme for scrutiny. However, the temporary banks ensured that at this stage, any water potentially infiltrating or overflowing into the channel would not escape to scour at the mitre.

Stilling Chamber. The initial design of the 15.5m long by 5.2m wide chamber was significantly modified following the discovery of 5.2m artesian pressures at the site. Proximity to the embankment toe, see Fig 2 and linking of artesian pressure to falls and rises in reservoir level in site observations resulted in the Panel Engineer recommending that the design should counter the pressure using self weight alone. Arrangements were considered incorporating a variety of materials to ballast the structure; heavy ore aggregates, lead and steel. Steel Ballast was favoured on the basis of cost, extent of excavation and durability and the design incorporated an 800mm deep compartment sandwiched between 2 structural slabs, into which steel slab and billet was to be placed and sealed with grout. See Fig 6. However, the contractor, Johnston Construction Ltd, demonstrated that railway track could be packed to a sufficiently tight density in order to achieve the required self weight and this was used instead. It was feared that artesian pressure could damage the foundation following excavation when a considerable element of the overburden was to be removed. Following a drawdown of the reservoir inclined relief wells were drilled from both

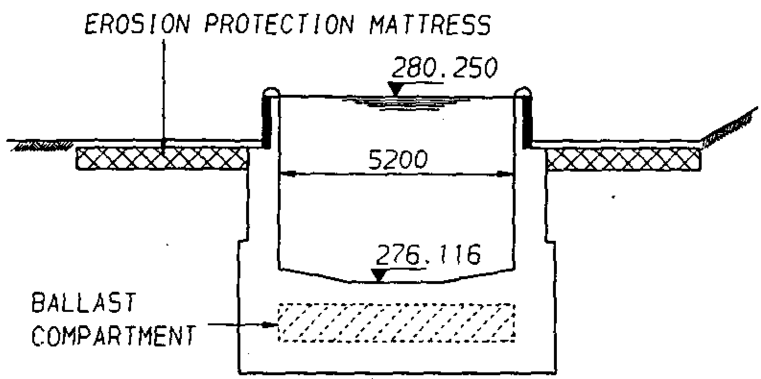


Figure 6: Stilling Chamber Cross Section

sides, extending beneath the proposed structure to intercept water drainage paths and locally reduce the pressure in the aquifer. The relief wells were sealed with grout when the stilling basin achieved the required self weight. Open stone asphalt was utilised in the discharge channel linking to the stream course for resistance to erosion when exposed to high flow velocities.

WALSHAW DEAN MIDDLE RESERVOIR.

Design. Tumblebay and weir designs were very similar to those adopted for the Lower Reservoir. The ogee weir has a length of 24m and has a drop of 3.5m. The tumblebay increases from 6m to 9m wide. The transition to the existing spillway resulted in 43m of the upper part of the spillway being reconstructed. The retained spillway walls were raised over the remaining length of 44m.

Geological Fault. Archive drawings from construction of the reservoir indicated the existence of a geological fault crossing the spillway channel just downstream of the core. Following first filling of the reservoir it appears that this fault was associated with leaks issuing into the draw-off tunnel, mitre and toe. As a precaution against any movement of the fault, additional movement joints were detailed in the channel to both sides of its anticipated location.

Crest Protection. All works on the Middle and Upper reservoir spillways were on the opposite side of the valley to the access road, thus requiring all plant and materials to be transported across the embankments. In order to minimise potential damage from earthmoving activity, sites were chosen close to the works for disposal of surplus excavated materials. The settlement history and narrow crest track prompted the provision of various measures to restrict

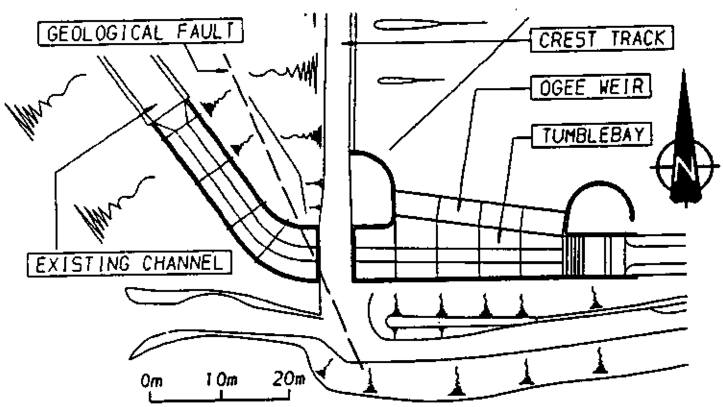


Figure 7: Walshaw Dean Middle Reservoir Spillway

loading of the crest. The solution adopted utilised railway sleepers laid side by side over the full length of the crest on a 125mm road stone bedding and overlaid by 125mm road stone. It was considered that the railway sleepers would distribute loads at a low cost whilst imposing minimal loads. Sleepers were also utilised to retain the roadstone and provide kerbs to prevent damage to the wave wall and to restrain wagons from driving off the crest.

Byewash Protection. Flood flow along the byewash would normally enter the tumblebay area, so temporary works were necessary to divert the flow to facilitate construction and in the interests of reservoir safety. The Panel Engineer recommended that these temporary works should be designed and specified by the Engineer. Material excavated to form a side overflow into the reservoir from the byewash was used to form a bund within the byewash channel, and was sealed with puddle clay at the upstream face against the pitched channel walls.

Embankment Crossing. Archive drawings show that the core trench was filled with concrete beneath the spillway and a trial hole excavated behind the abutment wall at design stage encountered the concrete at a depth of 800mm. In order to provide 97% security against exceedance during the construction period, the Panel Engineer recommended that drawdown be provided to safely accommodate a 1 in 35 year flood. For operational reasons, YW required that the drawdown be split between the Middle and Upper Reservoirs. The specification required the temporary works to:

* Support the puddle clay core
* Provide erosion protection
* Prevent overspill below normal overflow level until the spillway channel was completed
* Protect the excavated surfaces upstream of the core against heave due to rising reservoir levels
* Avoid creation of water paths through the core and embankment shoulder.

Temporary works designed by the contractor, Henry Boot Construction Ltd, comprised of sheet piles finishing close to the concrete shoe behind the crest bridge abutment. Requirements were set out in the Specification for emergency works including prompt attendance and material stockpiles. These stockpiles comprised quantities of large sandstone boulders, geotextile and hooked dowel pins. The materials were considered to be suitable for control of erosion should a storm threaten to overspill during construction of the channel.

WALSHAW DEAN UPPER RESERVOIR.

Work at the Upper Reservoir was very similar in nature to that undertaken at the Middle Reservoir, the main differences being due to the absence of a geological fault and the lesser depth of excavation. The weir length is 20m, the tumblebay width 9m and the length of spillway channel reconstruction 60m. Specified requirements for ensuring safety of the Reservoir during construction only varied in that timescales for completing the works requiring drawdown were minimised. This change was considered to be beneficial from the viewpoint of water resource management whilst avoiding the necessity to progress construction of the channel over winter.

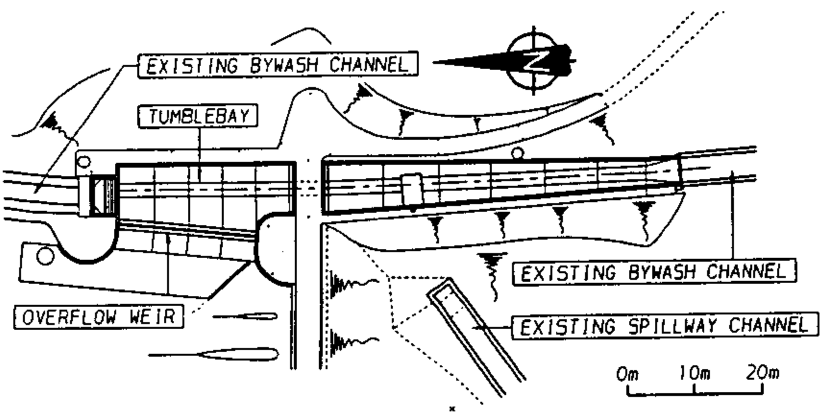


Figure 8: Walshaw Dean Upper Reservoir Spillway

ACKNOWLEDGEMENTS

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

REFERENCE

Wood, J.N. (1946). Discussion on The Leakage of a Clay Core-Wall, Transactions of Institution of Water Engineers, 51, pp 118-121.

Spillway Capacity Augmentation on Three UK Dams

E McKenna, Associate, Robert H Cuthbertson & Partners, UK

SYNOPSIS
This paper describes different approaches to the problem of augmenting the spillway capacity of three Category A (General) dams in Central Scotland. In each case the basic design constraint was the need to contain, within the limits of available freeboard, the flood surcharge resulting from the Probable Maximum Flood together with the appropriate concurrent wave surcharge. In one case the solution (perverse as it may seem) involved raising the overflow sill level; in another a moderate lowering of the overflow sill level was permissible, while the third case retained the overflow sill at its original level.

CARRON DAM (Central Region)
Carron Valley Reservoir is the principal water resource of Central Regional Council. The reservoir is impounded at its eastern end by Carron Dam, a composite gravity/embankment dam on which the overflow weir formed the top of the central gravity section.

A statutory inspection of the reservoir confirmed long-standing doubts about the stability of the gravity section of the dam and the associated hydrological analyses demonstrated that the overflow weir could not discharge the PMF at an acceptable flood lift.

It was proposed that the structural inadequacy of the gravity section be corrected by providing pre-stressed rock anchors to increase the Factor of Safety against overturning. The scale, and expense, of the proposed remedial works resulted in a comprehensive review of reservoir yield and catchment water resources to examine the possibility of operating the Carron Valley resources conjunctively with the resources of Loch Lomond and Loch Turret both of which had direct links into the Carron Valley supply area. The review concluded that the economic benefits would be maximised by raising the full supply level of the reservoir by 0.45 metres. Any further raising would incur additional costs in diverting a public road running alongside the reservoir.

The economic analysis indicated that the cheapest option for providing sufficient free-board above the raised full supply level was to reconstruct the overflow weir on top of the gravity dam and provide a continuous wave-wall over the full length of the dam.

A "Creager" type profile for a design head of 1.07 m was selected for the overflow weir. The overall weir length was defined by the 62.73 m width available between the stilling basin side walls. Within these leading parameters, the general arrangement of the overflow weir was dictated by structural considerations in respect of the required distribution of rock anchors, defined as follows:

- an access bridge had to be carried across the overflow necessitating the provision of piers, the number and/ or thickness of which should be minimised to give the greatest possible clear waterway length.
- the rock anchors stabilising the overflow section of the dam would be contained within the piers. The number of piers should be maximised to minimise each anchor load.
- the overflow section of the dam was split into monoliths by 4 No contraction joints. It was essential to continue these joints through the new work so that their presence dictated possible pier locations.
- the piers had to be reasonably uniformly disposed on each monolith to give an acceptable distribution of total anchor load.

Within these various constraints the arrangement shown on Fig. 1 was devised.

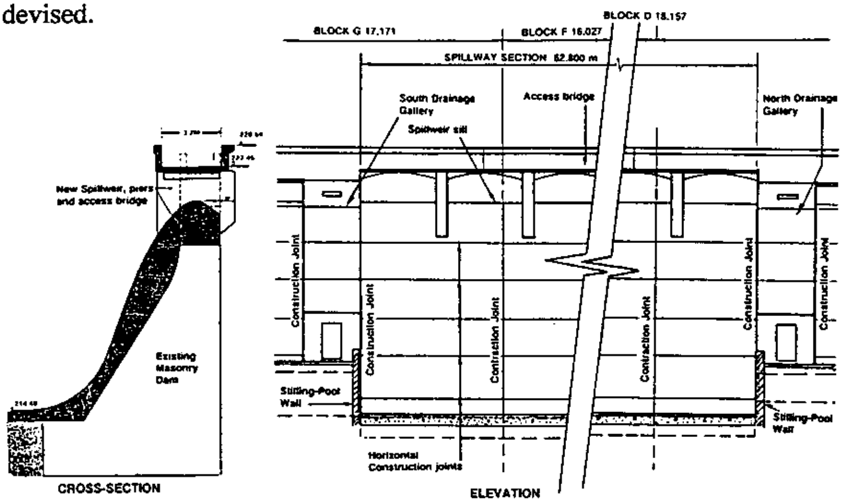


Fig. 1

The weir shape was model-tested to determine the correct stage/discharge curve. Various shapes of pier nose were tested to determine their effect on contraction of the nappe. In addition, pressures on the downstream face of the dam were measured to ensure that at maximum discharges, when the actual head on the weir would be in excess of the design head, there would be no tendency for the nappe to spring clear of the dam and thus introduce negative pressures with the attendant risk of inducing cavitation.

At PMF discharge the peak overflow rate is 150.70 m^3/s at a head of 1.175 m over the weir.

GLENSHERUP DAM (Fife Region)
Glensherup Reservoir is located on a tributary of the River Devon which is impounded behind Castlehill Dam at the south end of Glendevon.

The hydrologic and hydraulic studies carried out as part of a statutory inspection of the reservoir revealed that the reservoir overflow weir, comprising a 9.14 m long flat sill discharging to a steep, curved and narrowing channel, was inadequate to discharge the routed PMF within the limits of available free-board of 1.832 metres. The wave surcharge for the appropriate seasonal condition was calculated at 0.8 metres leaving a margin of 1.032 metres available to accommodate the flood surcharge, without raising the top of the dam or providing a continuous wave-wall.

Topographical constraints, particularly steeply rising ground at the left-hand end of the dam where the overflow was located, restricted the selection of possible options for augmenting the overflow capacity. Some options which would be physically possible within these constraints, e.g. a bellmouth type overflow, were rejected on the grounds of cost. Construction of a wave-wall along the 200 m length of embankment was rejected for similar reasons.

Eight options for increasing overflow capacity were considered before concluding that the most practicable option was to provide a side channel spillway with an overflow weir located alongside the existing outlet channel some 11 metres downstream of the dam axis, and connected to the reservoir by a forebay excavated in the reservoir embankment with the reinforced concrete floor of the forebay keyed-in to the embankment clay core. A standard USBR "ogee" profile weir discharging to a 4 m wide channel controlled by a transition section at its downstream end was adopted, all as shown on Fig. 2.

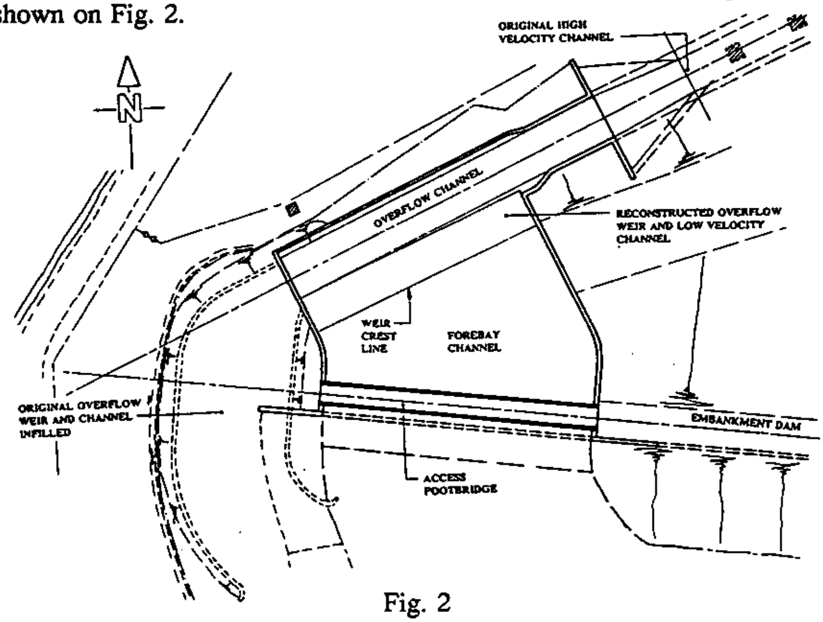


Fig. 2

Given that the permissible flood surcharge level had been defined it was necessary to consider the optimum weir length. Since any loss of yield arising from lowering of the full supply level could be made up by increasing pumping from Castlehill Reservoir downstream it was considered that the full supply (overflow) level could be reduced thus permitting use of a shorter weir for the same flood surcharge. An attempt was made to identify the economic optimum weir length but no minimum was found for reductions in overflow level of up to 0.9 metres at which point the PMF would produce a head of 1.932 m over a weir 11.5 m long. It was eventually decided to adopt a 20 m long weir requiring the overflow level to be reduced to 284.520 m AOD, a reduction of 0.468 m.

BADINSGILL RESERVOIR (Lothian Region)

A statutory inspection of Baddinsgill Reservoir revealed overflow and freeboard deficiencies identical to those found at Glensherup. Both dams were designed by the same engineer.

At Baddinsgill the problems were compounded by very steeply rising ground adjacent to the overflow weir and a steep, 1 in 6 gradient, spillway channel running down the left mitre of the dam. The upper end of the spillway channel was severely curved and tapered in width from 24.38 m at the weir to a 2.75 m base width trapezoidal section at the upper end of a long cascade channel.

The distance between the dam axis and the cascade channel and the difference in elevation due to the steepness of the channel precluded adoption of the solution used at Glensherup. In addition the spillway channel turned through an angle of 48° at its upper end with the result that supercritical flow gave rise to significant cross-waves, even at very low flows, and these waves caused water to overtop the shallow trapezoidal channel and run dow the dam mitre. It was considered necessary to achieve sub-critical flow over the curved part of the channel.

The flood surcharge permissible within the available freeboard was 0.75 m for the winter PMF of 68.9 m^3/s or 1.06 m for the summer PMF of 74.9 m^3/s. The difference in permissible flood surcharge arises from consideration of the appropriate concurrent wave run-up.

Physical constraints precluded any lengthening of the overflow weir along the line of the dam axis. One option considered was to arrange a side weir overflow channel with the weir at 90° to the dam axis and projecting upstream into the reservoir basin. However, to discharge the PMF within the permissible flood lift would have required a weir about 44 metres long. Since there were also potential foundation problems due to the steeply sloping sides of the reservoir basin other solutions were investigated.

The provision of a labyrinth weir constructed on the line and within the length of the original overflow weir emerged as the cheapest option.

The new weir, as shown in Fig. 3, comprises a 3-bay labyrinth weir with an overall length of 24.384 m. The weir discharges to a low velocity section within which the flow is turned through 48° before entering a transition section leading to the upper end of the original cascade section which has been retained.

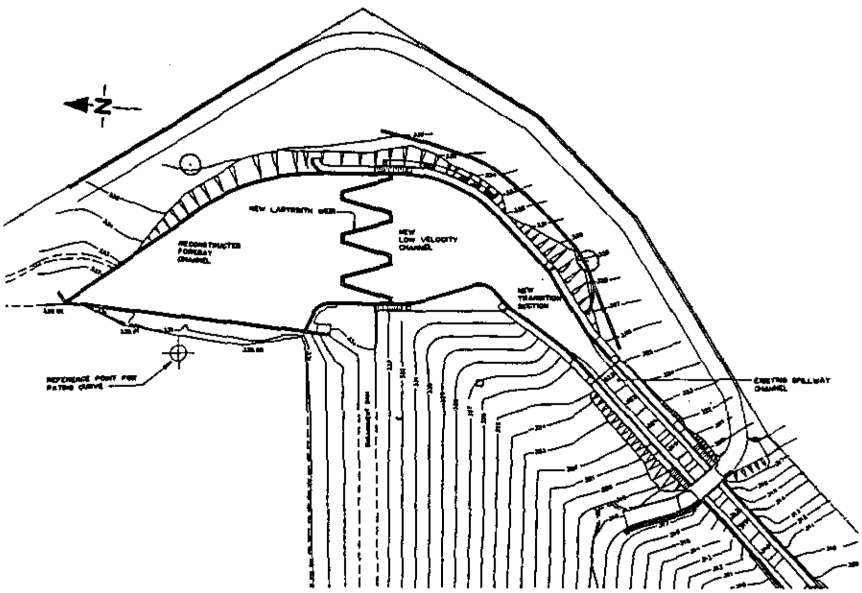


Fig. 3

The proposed arrangement was subject to comprehensive tests on a 1:20 scale physical model of the whole arrangement including approach channel. An alternative 5-bay labyrinth was also tested and while it showed some improvement in the stage/discharge relationship it was considered that the improvement would not justify the additional expense of the 5-bay layout.

The final weir rating curve was based on water level measurements taken at the entrance to the approach channel - shown on Fig. 3. Routing the critical winter PMF revealed that the maximum reservoir water level was 0.856 m above weir level, or some 0.1 m in excess of the permissible maximum. However, the undertaker required that a new access road be constructed along the top of the dam and the opportunity was therefore taken to increase the freeboard by a minor raising of the level of the top of the dam. The total surcharge level (flood lift plus wave run-up) is 334.171 m AOD while the top of the dam is now nowhere less than 334.200 m AOD.

ACKNOWLEDGEMENTS
The author wishes to thank Central, Fife and Lothian Regional Councils for permission to prepare this paper.

The refurbishment of Winterburn Reservoir

L. MORGAN, British Waterways, UK
J.M. WARRINGTON, British Waterways, UK
P. HOWLETT, British Waterways, UK

SYNOPSIS.
Winterburn Reservoir was constructed 100 years ago to provide feed water for the Leeds and Liverpool Canal and is a Grade II listed structure situated within the Yorkshire Dales National Park. This paper describes the investigations, model tests, design and construction works implemented to bring the reservoir up to the 1975 Reservoirs Act standards.

INTRODUCTION.
British Waterways is a statutory body responsible for 89 reservoirs dating back to the 18th Century. Winterburn Reservoir, feeding the Leeds and Liverpool Canal, is one of the youngest reservoirs owned by British Waterways. In 1990, the Reservoir Inspecting Engineer confirmed that the existing overflow was incapable of passing the probable maximum flood (PMF) safely. This was because of a restriction caused by a masonry and brick twin arch bridge which carries a farm access across the top of the spillway. Such a restriction would cause a backing up of water within the reservoir and subsequent overtopping of the dam. Consequently, in order to comply with the requirements of the Act, works were proposed to increase the spillway capacity. A number of alternatives were considered;

i) raising of the dam crest level to attenuate the PMF by routing the flood through the reservoir
ii) replacement of the wave wall with a higher water retaining structure to attenuate the PMF by routing the flood through the reservoir
iii) construction of an emergency spillway on the downstream face to attenuate the PMF by increasing the available discharge capacity
iv) construction of a supplementary spillway to attenuate the PMF by increasing the available discharge capacity
v) modification of the existing spillway to attenuate the PMF by increasing the available discharge capacity

Because the reservoir lies within the Yorkshire Dales National Park, it's Grade 2 listed status and the environmental impact of the works on the local community, it was agreed that options requiring major alteration of the dam fabric or involving a large number of HGV movements through rural areas would be unacceptable. In addition British Waterways regards its duties' in protecting the heritage of the canal network seriously and therefore schemes for further development were narrowed down to options which would minimise effects on the environment and landscape and which would maximise the preservation of the existing heritage. Consequently the schemes taken forward for further development comprised a combination of options for raising of the wave wall and modification to the existing spillway.

MODEL TESTING.

Preliminary design calculations indicated that the section of spillway beneath the existing bridge would require substantial lowering in order to pass the PMF. A model test was therefore commissioned to confirm the calculations and to test varying combinations of raising of the wave wall with lowering of the spillway (Fig 1).

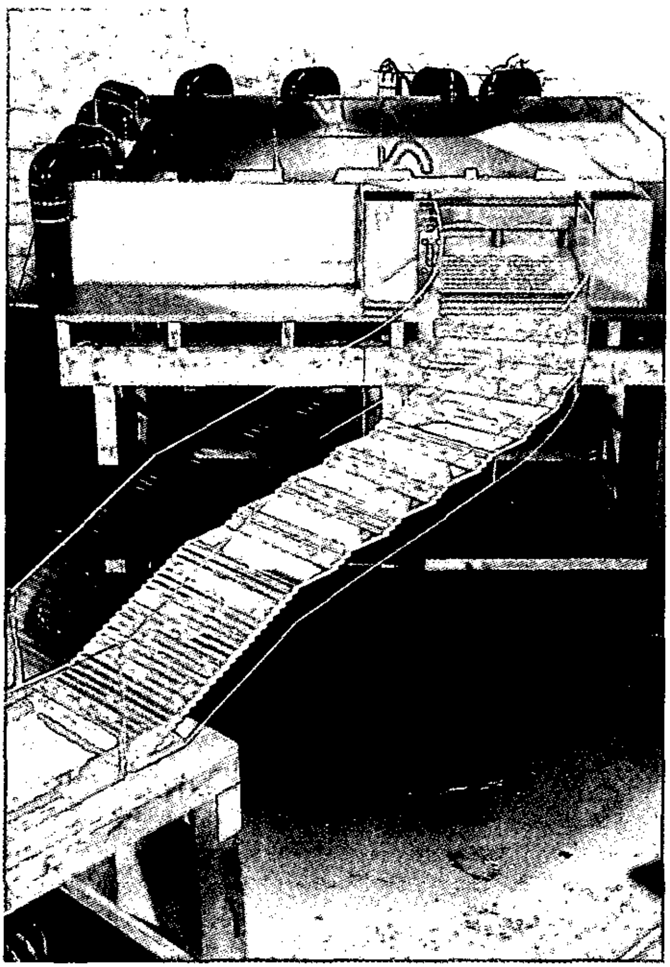


FIG 1. SPILLWAY MODEL

The model testing regime involved close liaison between the design team and the laboratory, with designs developed in the drawing office being tested in the model and modified based on the feedback from the lab. The model indicated that, for intermediate and large flood levels, out of channel flows and significant splashing would occur at each of the third points along the length of the spillway. Works would therefore be required to prevent erosion of the dam embankment at these locations. The findings from the model test demonstrated that a compromise solution between a modest raising of the dam wave wall by 400mm combined with a reduction in the level of the spillway channel beneath the bridge of about 2.0m would result in an arrangement capable of passing the required PMF.

DESIGN.

Structural design was required for the wave wall and all elements affected by the lowering of the spillway channel invert. The wave walls were designed to retain water to an increased level either as reinforced masonry for reconstructed walls or as masonry faced reinforced concrete for lengths of new wall. Walls were designed to maximise the use of existing materials to comply with heritage requirements and blend in with existing structures.

Amendment to the bridge and channel to increase outflow to 260 m^3/s was required, the existing arrangement being capable of passing only 110 m^3/s. The design philosophy adopted was to agree a maximum allowable reservoir level dictated by wave wall height and then to use this to determine the invert level of the channel. As a consequence, the level difference of weir crest and wave wall became 3.3m at which stage the bridge arches acted as orifices. Unfortunately, the discharge co-efficient of the twin arches could not be predicted accurately given the close proximity of the curved weir. Initial calculations indicated a requirement to lower the invert by 1.5m. Model testing confirmed that a minimum of 1.9m was required, modified to 2.16m to coincide with a step in the cascade spillway.

The ideal time to undertake the works was towards the end of the summer by which time the reservoir would generally be depleted after supplying the canal. The works therefore commenced during July 1995 by which time the reservoir capacity was reduced by 50% providing sufficient storage to cater for the 1 in 25 year storm.

CONSTRUCTION.

The original bridge and spillway channel were found to be of substantial construction, more so than had been indicated in the structural investigation. Foundations were brickwork or mass concrete on rock with no soft materials encountered (Fig. 2). This enabled temporary works to be scaled down bringing significant time and cost savings. The clay core was identified beneath a shallow trough within the spillway channel bed and running under blanked off arches to each side wall (Fig. 3). These arches had foundations on rock with massive masonry blocks infilling the arches over the core and free to settle with any consolidation of the clay.

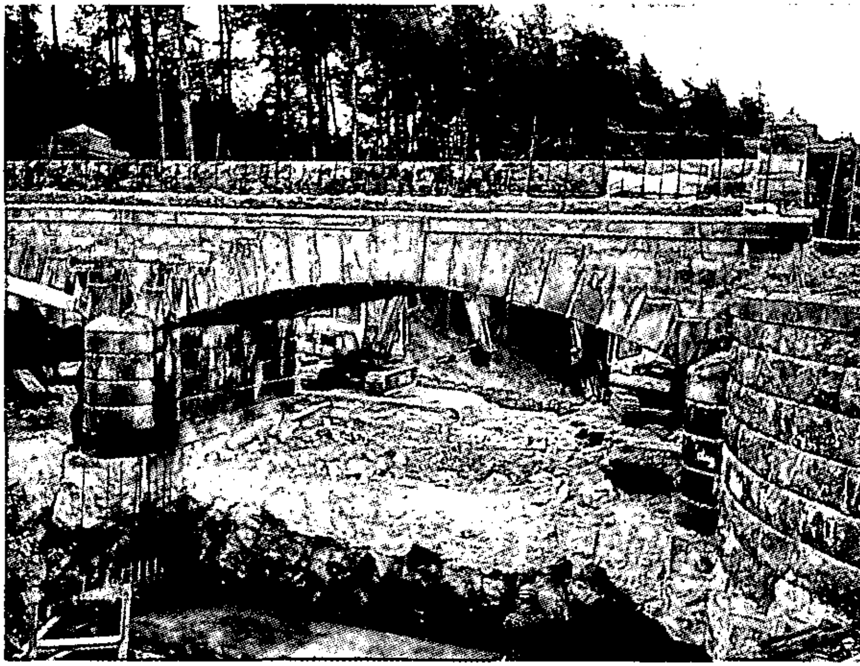


FIG 2. SPILLWAY BRIDGE EXCAVATION

A new invert of reinforced concrete was provided, constructed in bays to maintain side wall stability. At the core position the bays were amended such that the new slab spanned over the clay core (Fig. 4). Pipes were provided in the slab for future grouting of voids above the core. Removal of the original floor and steps presented no major problems other than at locations where the blocks were larger than anticipated. Some of these, up to 900mm deep, were broken up to facilitate removal of the remainder.

CONCLUSIONS.

The main problems experienced during the project were principally encountered during the construction phase, especially the matching of new masonry work into existing. Limitations on the availability of as constructed drawings and site investigation data made design work more difficult, although in the event the ground conditions were found to be much more favourable than had originally been anticipated. This enabled economies in construction to be achieved, especially in the spillway channel wall underpinning works and replacement of the channel floor. The designers of the dam, Edward Filliter and Henry Rofe of Leeds, had clearly made the best

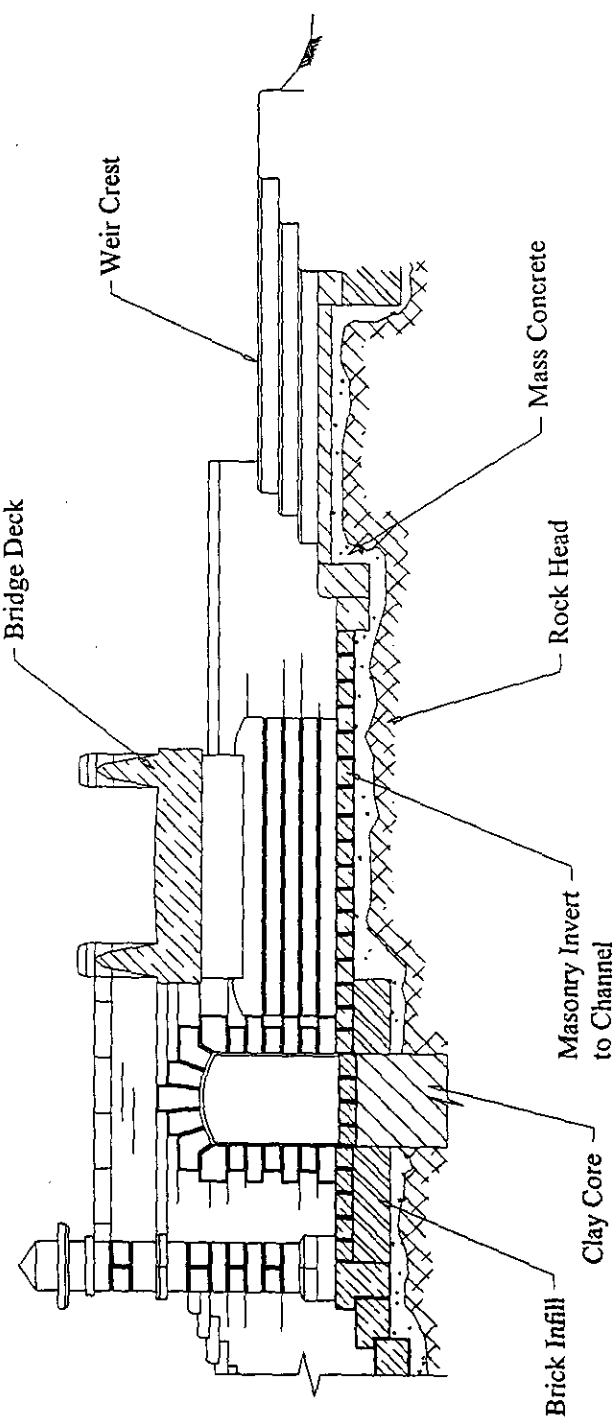


FIG 3. SECTION THROUGH ORIGINAL CONSTRUCTION

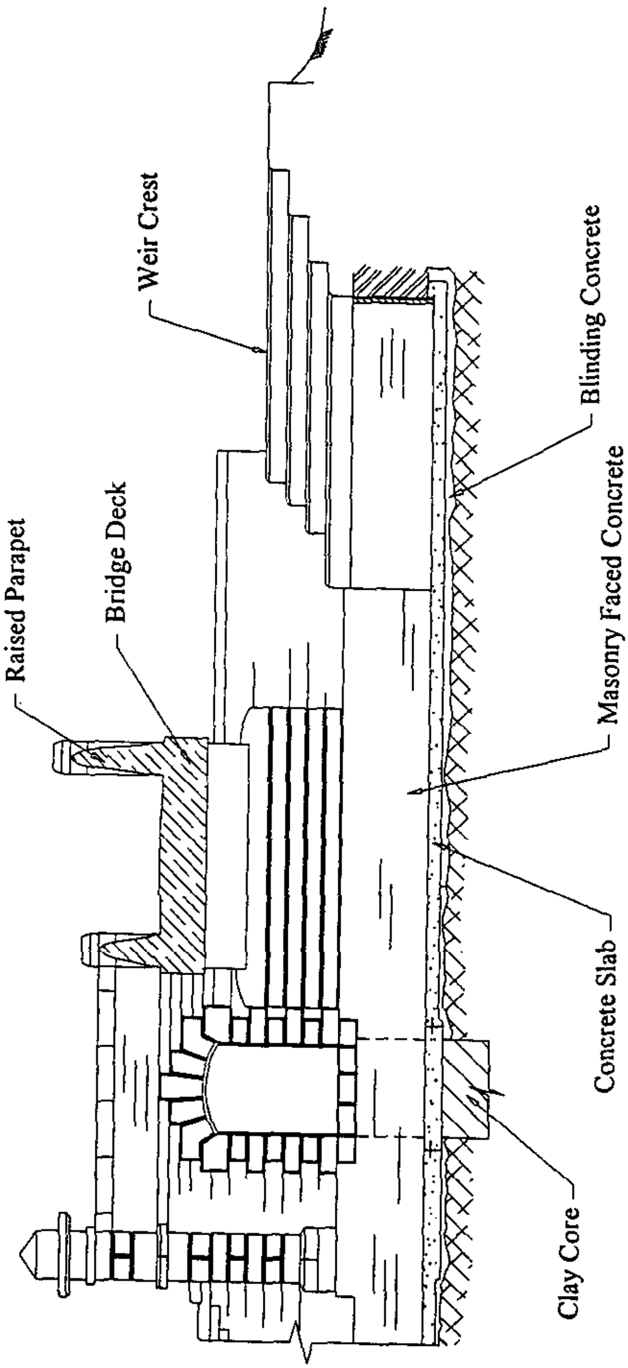


FIG 4. SECTION THROUGH LOWERED CHANNEL

use of the available site by positioning the important structures along the east abutment of the dam where rock head was high and it's quality good.

The scheme adopted, at a cost of £300,000, represented the best compromise between reservoir safety, waterway operational requirements, financial considerations and waterway heritage and is a good example of how a number of apparently conflicting requirements can be successfully resolved.

The Hydroplus Fusegate System - four years on.

S.CHEVALIER - Project Engineer - Hydroplus International
S.T.CULSHAW - Area Manager - Hydroplus International
J.P.FAUQUEZ - President - Hydroplus International

SYNOPSIS.
The Hydroplus fusegates can be utilized for increasing live storage capacity of dams with ungated spillways and/or for improving spillway safety without sacrificing storage capacity. They are designed to rotate (tilt) in succession when the reservoir level reaches predetermined elevations thereby preventing the flood from overtopping the dam. This paper discusses varying approaches in the use of the system to solve the problem of safety worldwide including the Dove Stone reservoir in the North West of England.

INTRODUCTION.

Under normal conditions fusegates increase the active storage area of reservoirs. The labyrinth weir configuration allows for increased discharge at lower heads than conventional spillways. The fusegate design allows for the fusegates to overturn during high or extreme flood events thereby allowing more flow through the spillway. These systems can increase the active storage of a reservoir and increase the discharge capacity for extreme events so that the design flood may pass without overtopping the existing dam and thus reducing or eliminating the need for increasing the dam height.

THE CONCEPT.

The Hydroplus system consists of one or more adjacent but independent fusegate units resting on the overspill sill and held in place solely by gravity forces. The fusegates are overtopped by moderate floods. In case of larger floods, some elements are overturned by rotating about lugs located at the downstream edge of the sill. The number of units activated away is graded to suit the hydrograph of the particular expected flood and match the required discharge flow.

Each unit sits freely on a concrete slab cemented into the crest and consists of three parts :

* The bucket (top part) made of steel or reinforced concrete with can be given a labyrinth shape to increase the overspill length so that moderate floods can be discharged with a relatively small head on the lip.

* The bottom chamber base made of steel or reinforced concrete.

* The well : a funnelled conduit into the chamber which begins to fill for a given reservoir level. Wells are set at different elevations.

When reservoir level reaches a given elevation, water flows through the well into the chamber. Pressure builts up in the bottom chamber and when sufficient vertical uplift has been mobilized, the fusegate rotates about the abutment blocks located at the downstream edge of the sill. The triggering system is very precise and leaves an ample safety margin until the reservoir level reaches the predetermined elevation activating a given element.

The fusegate on the spillway crest is acting in three different ways depending on headwater level :

* As part of the dam, as long as the reservoir level does not exceed the fusegate bucket top edge.

* as a weir, when headwater level is higher than the fusegate top edge but lower than the predetermined level causing the fusegate to overturn.

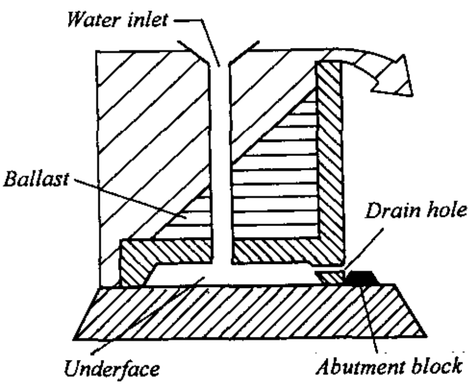
* As a "fuse" or a regular spillway gate when headwater reaches this predetermined level and triggers the uplift.

Once the flood has receded, the overturned fusegate is simply lifted back on the sill or replaced with an identical new unit if seriously damaged.

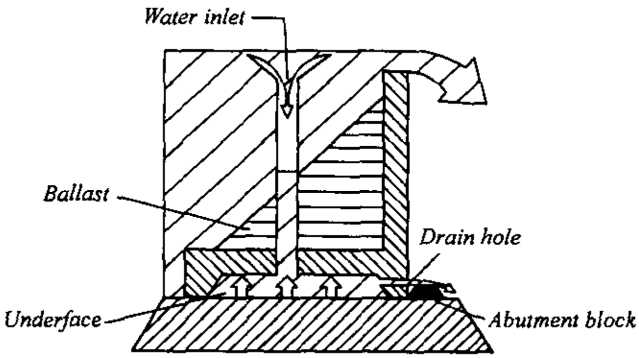
Extensive theoretical research has been conducted on fusegate performance when spilling and rotating and thorough model testing programs were performed to validate the mathematical approach.

Testing of fusegates was performed at the Tennessee Valley Authority facilities at Norris (Tenn.). These tests validated very precisely the behaviour of the Hydroplus fusegates under various circumstances and conditions.

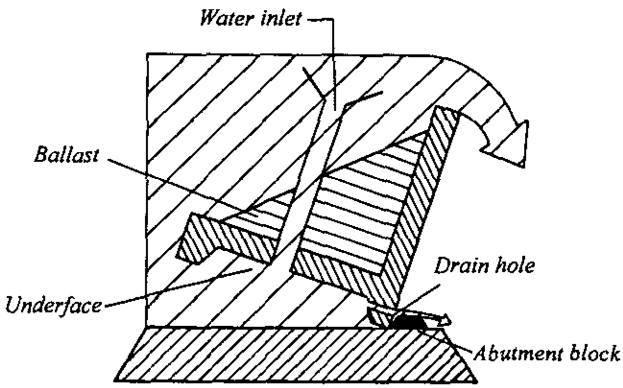
Also, Ice has been the subject of specific research at the Institute for Marine Dynamics of the National Research Council, in St John's, Canada and at the Scientific Research Institute of Energy Structures, in Moscow, Russia.



Water is spilling over the crest



The inlet stack admits water



The fusegate tips off

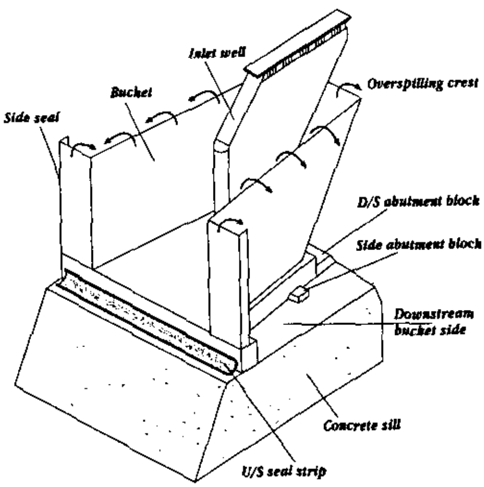
FUSEGATE CONFIGURATIONS.

The fusegate system may form a labyrinth or straight crest to the weir.

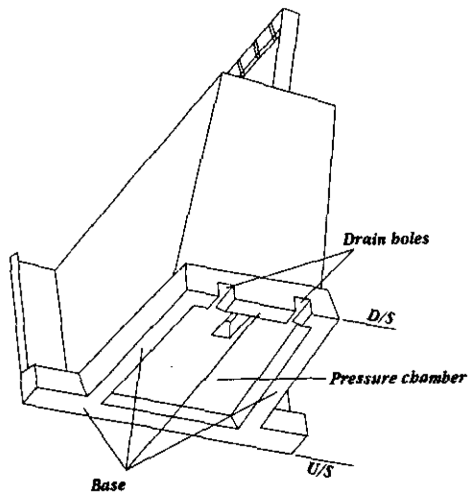
1 - Labyrinth Crest Fusegates
A bucket shape is used to form a labyrinth crest capable of overspilling a nappe equivalent to 30 - 150% of the fusegate height. The bucket is open at the upstream side. An alignment of buckets forms a labyrinth crest which allows moderate floods to spill with a reduced head on the crest. The developed length of the labyrinth is approximately three times that of a concrete sill length. The bucket height is commonly 1m to 6.5m, depending on the clients needs.

2 - Straight Crested Fusegates
There are three types of straight-crested fusegates :
Emergency Fusegates are designed to overturn before or just after being overtopped. They are used as back-up to conventional gates in the event of very large floods or gate malfunction. They may be 1m to 10m in height.

Intense Cold Environment Fusegates are designed to be very stable and their upstream face are shaped and finished to reduce ice loading, friction and abrasion.
High Heads Fusegates are designed to withstand high heads on the fusegate crest. They are constructed to obtain the best discharge coefficient and may be large (20m wide) and capable of remaining stable when spilling under a head of up four times their height.



Upstream view of labyrinth crest fusegate



Bottom view of labyrinth crest fusegate

CASE STUDIES.

Case 1.
Lussas Dam in the South-Central France provides irrigation water for agriculture on the Lussas plain. The dam is an earth construction 19.5m high. The spillway is situated on the left bank and is 36m in length.

The runoff pattern is governed by sudden, heavy storms on the relatively large catchment (26.2 km) which accounts for a large design flood. The spillway was modified and the fusegates installed in two summer months of 1991 to increase the reservoir storage capacity. The reservoir was successfully filled in time for crop irrigation soon after. During a flood in the winter of 1994 one fusegate overturned at the correct water elevation and was replaced soon after for the reservoir to fill for the next crop season.

* - New storage capacity	420,000 cubic metres
* - Sill length	35m
* - New spillway capacity	250 cubic metres per second
* - Fusegate number	10
* - Fusegate height	2.15 metres
* - Fusegate width	3.5 metres
* - Storage increase	30%
* - Spillway capacity increase	15%

Case 2.
The Wanakbori pick-up Weir in Gujarat State in India was built to collect water to irrigate two hundred thousand hectares of agricultural land. Subsequent constructions of several dams upstream resulted in higher flood control and so reduced by 2 metres the water head available at the weir. No conventional method was cost effective to restore the requested head on the 678 m long spillway.

The Hydroplus solution consisted in the installation of thirty three straight crested high overspill concrete fusegates two meter high and twenty metre long.
The first fusegate tilt for a flood of about 20.000 cubic metres per second which correspond to a 6 metres overspill over the fusegates.

* - New storage capacity	84,000,000 cubic metres
* - Sill length	675 metres
* - New spillway capacity	47,000 metres per second
* - Fusegate number	33
* - Fusegate height	2 metres
* - Fusegate width	20 metres
* - Storage increase	100%
* - Spillway capacity increase	nil

Case 3.
Caillaouas Dam is at an altitude of 2,170 metres in the French Pyrenees. It is a masonry structure at the end of small but very deep natural lake. Three straight crested fusegates have been installed. to improve dam safety. A flap gate has also been installed to discharge moderate floods. The fusegates give the extra capacity to discharge the design flood. The combination of a flap gate and fusegates presents the two-fold advantage of good control of the

reservoir level. Because of the area in which the dam is located, the fusegates were transported and installed by helicopter. The operation manual for the client includes requirements for ice conditions.

* - Storage capacity	25,400,000 cubic metres
* - *Sill length*	7.4 metres
* - New spillway capacity	29.3 cubic metres per second
* - Fusegate number	3
* - Fusegate height	1.5 metres
* - Fusegate width	1.6 metres
* - Storage increase	nil
* - Spillway capacity increase	190%

Case 4.
The Shongweni Dam is located in Kwa Zulu in Natal in the Republic of South Africa. The dam was originally built in the 1920's, the ungated spillway was 126 metres. The original design flood was rated at 1,250 cubic metres per second. After inspection in 1988 the design flood was re-evaluated at 5,000 cubic metres per second. The new safety requirements were not satisfied for two reasons;. (a) the dam structure was inadequate and (b) the spillway could not discharge the re-evaluated design flood. The client examined four alternatives but settled on lowering the spillway by 8 metres. The spillway was cut back 7.9 metres and ten 6.5 metres high fusegates were installed. The result was that the reservoir level was lowered only by 1.4 metres. The first fusegate is designed to tip for a flood that occurs every 240 years. Each fusegate weight was some 120 tonnes and since the cableway safe working load was 5 tonnes the structural design had to be adapted to meet the weight limitation.

* - Storage capacity	6,600,000 cubic metres
* - Sill length	126 metres
* - New spillway capacity	5,000 cubic metres per sec.
* - Fusegates number	10
* - Fusegate height	6.5 metres
* - Fusegate width	9.73 metres
* - *Storage increase*	*nil*
* - Spillway capacity increase	235%

Case 5.
The McClure Dam located in New Mexico, USA is an earth structure with a maximum head of 38.6 metres and a crest length of 234 metres. The dam was built in the 1930's and raised

three time in 1936, 1947 and 1988. In 1988 the 17 metres spillway was reconstructed. With two other gravity dams, McClure supplied domestic water to the city of Santa Fe until a large crack was found in the foundation of Two Mile dam. In February 1995, the operator was allowed by the State authorities to transfer the water rights to McClure dam. In April 1995, eight steel fusegates were installed on the Ogee crested sill. The Hydroplus concept had successfully increased live reservoir storage capacity by 0.62 hm^3 and spillway discharge capacity by 8 per cent.

* - *Storage capacity*	3,820,000 cubic metres
* - Sill length	17.3 metres

* - New spillway capacity	507 cubic metres per sec.
* - Fusegate number	8
* - Fusegate height	2.15 metres
* - Fusegate width	2.13 metres
* - Storage increase	500 acre-feet
* - Spillway capacity increase	8%

Case 6.
The Dove Stone Dam is an earth structure situated under Saddleworth Moor 8 km east of Oldham in Lancashire. Calculations and 1:21 model tests indicated that the existing bellmouth in the reservoir could pass approximately 160 cumecs. The required discharge for the PMF being approximately 300 cumecs an auxiliary spillway was required to meet safety criteria. Three fusegates were installed into the new 9.8 metre wide auxiliary spillway. The fusegates are designed to tilt off the spillway 0.53, 0.58 and 0.63 metres above the lip of each fusegate respectively. The fusegates are designed to rotate in succession between the 1 in 10,000 years flood event and the PMF thereby preventing the flood from overtopping the dam.

* - Storage capacity	5,050,000 cubic metres
* - Sill length	9.8 metres
* - New spillway capacity	282 cubic metres per sec.
* - Fusegate number	3
* - Fusegate Height	2.15 metres
* - Fusegate width	3.23 metres
* - Storage increase	Nil
* - Spillway capacity increase	42%

CONCLUSION.
Hydroplus Fusegate systems are being used Worldwide including the United Kingdom, China, South America, United States of America, parts of Africa and South East Asia and areas of Europe. The system has proved and established its versatility in application, in the use of materials, methods of construction and logistics.

The Hydroplus Fusegate system is a method by which dam safety and/or storage capacity can be achieved in a cost effective manner.

REFERENCES.

Lempérière F., Bessière C. (1992). Hydroplus submersible fusegates for surface spillways. Modification of dams to accomodate major floods. Twelfth Annual USCOLD Lecture Series, Fort Worth, Texas.

Lempérière. F.(1992) Overspill fusegates. Water Power & Dam Construction..

Bessière C. (1992) Hydroplus submersible fusegates for surface spillway. International Symposium on Dams and Extreme Floods, Granada Spain, Tomo I, Topic A : Design.

Lempérière.F. (1992). New solution to improve Spillways of existing or Future Arch Dams. International Symposium on arch dams, Hohai University, Nanjing, China. Proceedings of the Symposium.

Philippe M., Winton. J. (1992) Overspill fusegates. ANCOLD, 1992 Conference on dams, New South Wales, Australia, Recent developments in dams Engineering. Conference papers.

Brenac J.L. (1993). Hydroplus fusegates. ESHA Info N° 9, European Small Hydro Association..

Fauquez. J.P.(1993) Hydroplus, the fusegates principle. International Conference, Hydropower, Energy and the Environnement, Stockholm. - Proceedings.

Lempérière.F. (1994) Enlarging the capacity of existing dams. Structural Engineering International, SEI Volume 4, Number 2.

Falvey H.. T, Snell E.F.A., Rayssiguier.J. (1994) Increasing Shongweni dam discharge capacity with a Hydroplus fusegate system. Dam Safety Modification and Rehabilitation. Fourteenth Annual USCOLD Lecture Series, Phoenix, Arizona..

Snell E.F.A., Walford B. and Gibbons.(1994) F.J.G. Shongweni to have record fusegates. International Water Power & Dam Construction..

Trissler M. and Crawford.C. (1994) Installation of a fusegate system at Shongweni dam. Hydropower and Dams.

Hakin W.D., Trissler. M. (1994) Use of fusegate system to improve safety at Shongweni dam. International Commission on Large Dams. Eighteenth Congress on Large Dams, Durban.

Falvey H.. T., Treille Philippe.(1995) Hydraulics and Design of Fusegates. Journal of Hydraulic Engineering Vol.121, N° 7, © ASCE. USA.

Sikora J., Ferns L., Bailey F. , Gavillet.M. (1995) McClure dam spillway modifications with fusegates. Proceedings of 1995 Annual Conference of the Association of State Dam Safety Officials, Atlanta, Georgia, USA.

Lempérière.F. (1995) Cost effective improvements in fill dam safety. Hydropower & Dams.

Gulati.T.(1995) Augmenting canal capacity through application of Hydroplus Fusegates at Mahi project of Gujarat. International R&D Conference, Water & Energy 2001, Needs Development Utilisation, New-Delhi, India,. Proceedings : Water Ressources, Volume II.

Rayssiguier.J. (1995) Hydroplus Fusegate. A substantial contribution to increasing dam storage capacity. Symposium "Water in East Africa, french technologies, Nairobi. Kenya, Proceedings of the Symposium.

Aït Alla. A.(1995) Fusegates contribute to dam safety. International Water Power & Dam Construction.

Yong.Liu (1995) Application of Hydroplus fusegates in flood-diversion areas of Huaihe river basin. Hydropower & Dams.

Gulati.T.(1995) Augmenting canal capacity through application of Hydroplus Fusegates at Mahi project of Gujarat. Hydropower & Dams.

Great Northern Reservoir Works

R.B.BINNIE, Crouch Hogg Waterman Ltd., UK
I.C. MACDONALD, Crouch Hogg Waterman Ltd., UK
M.C.SWEENEY, Crouch Hogg Waterman Ltd., UK

INTRODUCTION

The Great Northern Reservoir is one of the Whittle Dene Group of Reservoirs and lies immediately North of the B6318 Military Road some 9 km North East of Corbridge in Northumberland.

The project, carried out on behalf of North East Water plc (NEW), comprised the construction of a new intake and valve tower, embankment crest regulation, construction of wave walls, spillway modifications and upstream slope protection. Other elements of the Works included low pressure grouting to sections of the clay core of the reservoir embankment and valve tower, refurbishment of existing masonry valve tower including repointing the tower and culvert, refurbishing the existing valve and constructing a new access platform and installing counterfort drains to the downstream face of the West and South embankments.

The Draw-Off Works were required to supply raw water to a new 135 Ml/day Treatment Works at Whittle Dene, and included 800 m of 1200 mm diameter steel pipeline downstream of the reservoir. The flood protection works and spillway modifications were required to satisfy safety recommendations made following a statutory inspection under the Reservoirs Act 1975. Of particular interest were the design considerations in locating the intake within the solum of the reservoir, the valve tower within the existing reservoir embankment, and the pipeline crossing of Hadrian's Wall and the adjacent Roman Vallum (earth rampart).

PRELIMINARY WORKS.

The Great Northern Reservoir was drawn down to allow ground investigation and surveys of the reservoir solum and embankments for design purposes. Other aspects of the project which were addressed at an early stage in the design were the pipeline crossings of the Military road (B6318), which runs on the line of Hadrian's Wall, and the Roman Vallum downstream of the reservoir. English Heritage were informed of the proposed pipeline route and NEW funded an archaeological dig in advance of the main contract to ensure that Scheduled Monument consent was obtained for the work to avoid delay in the overall scheme. No traces of Roman structures were found.

The reservoir as an asset. Thomas Telford, London, 1996

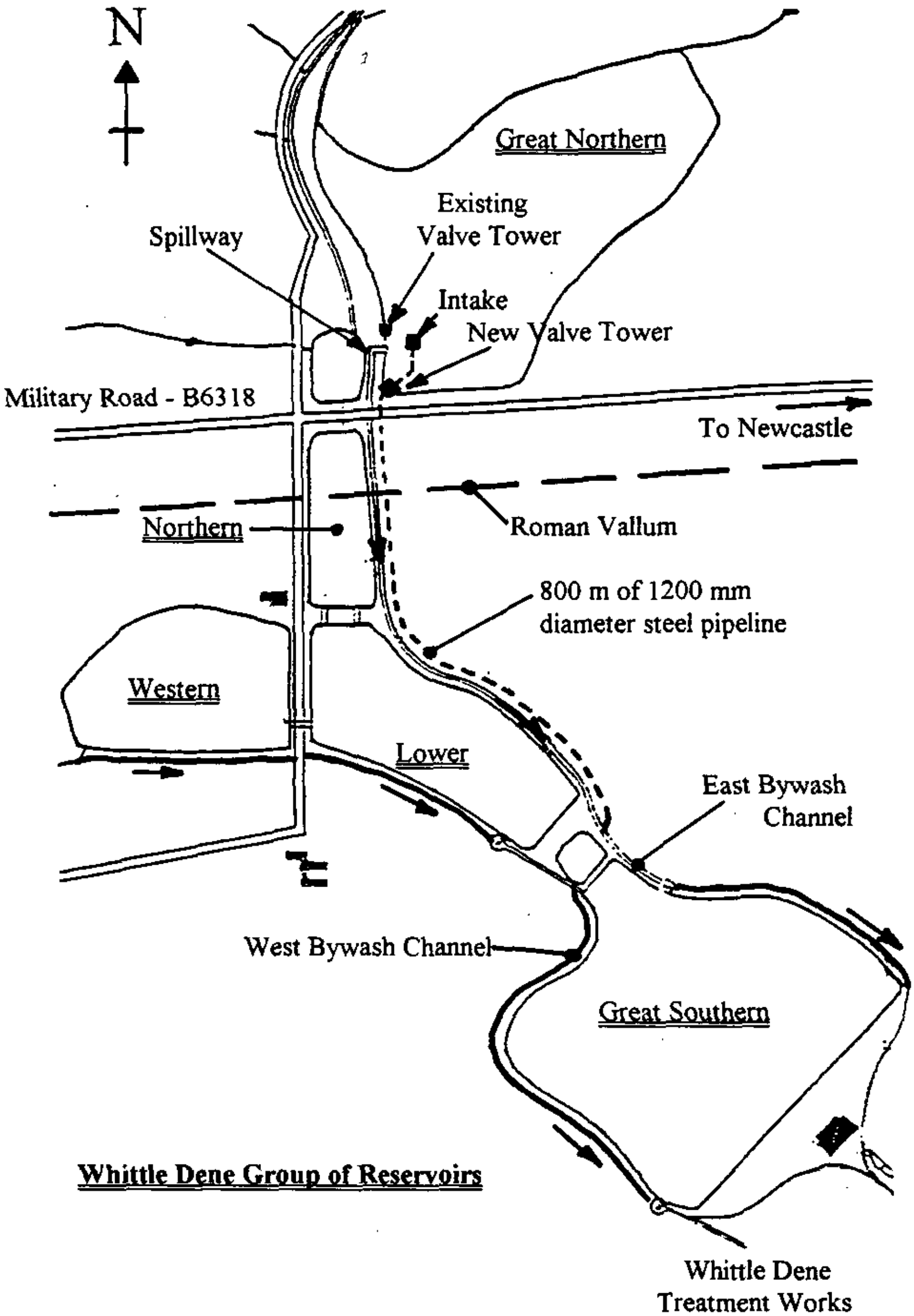


Figure 1. - Great Northern Reservoir Works

DETAILED DESIGN - DRAW-OFF WORKS

The arrangement of the intake chamber and valve tower structure were influenced by the existing ground levels and poor soil conditions in the reservoir solum. Initially, consideration was given to locating the valve tower at the middle of the South embankment. However, this would have led to significant depths of excavation for the crossing of the Military Road directly downstream of the embankment. The valve tower was therefore re-located to the junction of the South and West embankments which allowed excavation depths at the road crossing to be kept to a minimum.

The principal design consideration was the method of installing the pipeline and valve tower in the 8m high embankment of the reservoir without compromising the integrity of the earthfill structure. Having considered mini-tunnelling and thrust boring techniques as possible options, conventional open trench excavation was adopted for the pipeline installation together with a steel sheet piled containment box to house the valve tower. The valve tower took the form of a 4m diameter, 8m high epoxy coated steel tube and was encased in concrete with the steel sheet pile box retained as a permanent shutter. The piling arrangement keyed into original ground and incorporated sheet pile wings extending into undisturbed core material.

Careful consideration was given to the selection of the sheet pile section to ensure adequate driving characteristics through the embankment material, the method of excavation within the piling arrangement, installation of waling frames, provision of clearance tolerances for locating the valve tower and external loading on the steel liner tube during concrete lifts.

Steel was selected as the material for the 800m of 1200mm diameter piping in preference to ductile iron on the basis of economy and maintenance considerations. The 1220mm O.D. welded steel main was protected with an internal bitumen lining and an external reinforced bitumen enamel wrapping and was cathodically protected.

The diameter of the steel valve tower was determined by the requirement to house main supply and guard valve sluices and an access tee in the pipework arrangement. Due to the significant cost of adopting twin 1200mm diameter sluice valves and the limited area available at the embankment the pipework was tapered to 1000mm diameter within the valve tower. The specification for the 1000mm diameter sluice valves had several operational criteria to satisfy. Portable actuation was incorporated as the principal means of valve operation since no electricity supply is available at the reservoir. In addition, independent manual operation of each valve was provided within the valve tower by means of a handwheel mounted on a headstock.

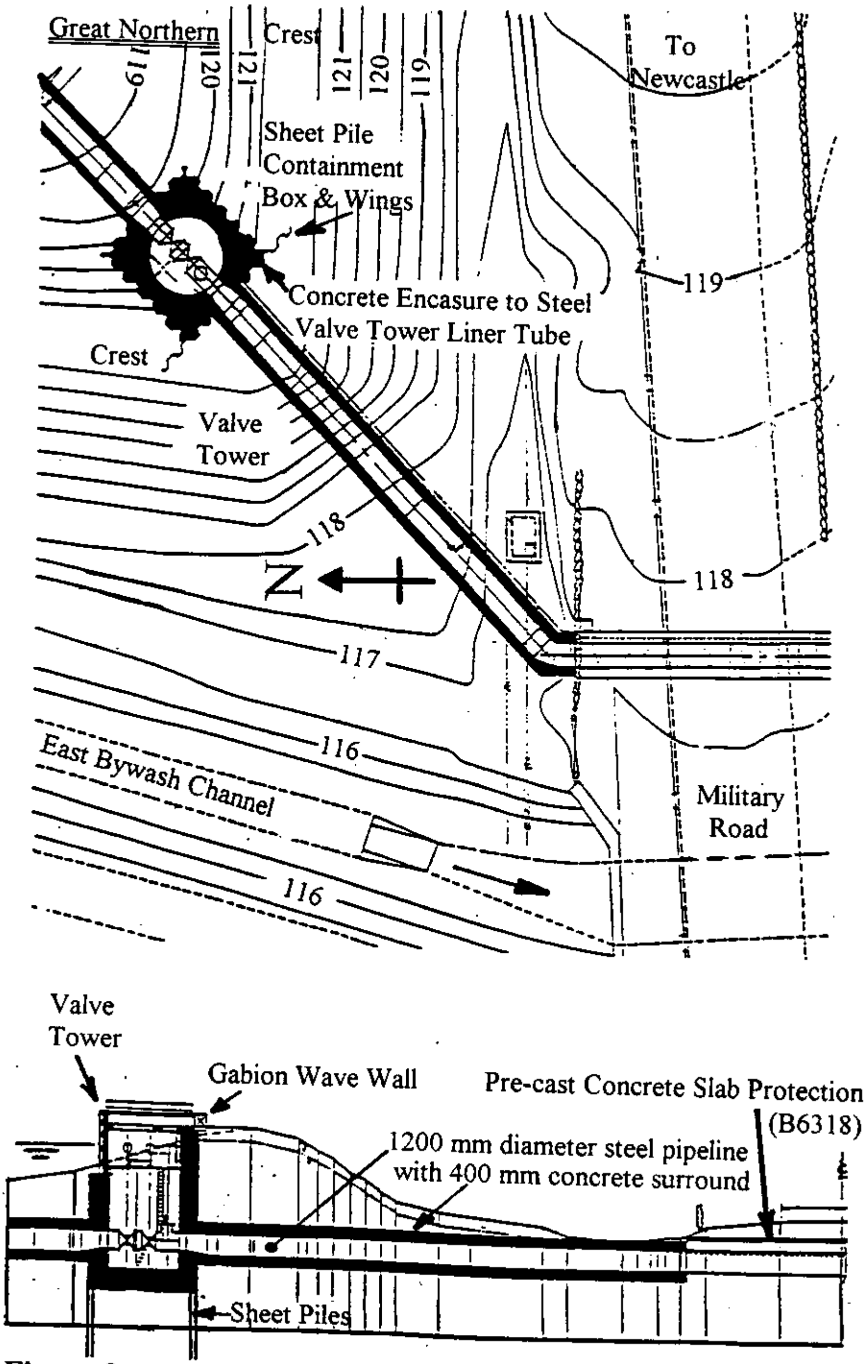


Figure 2. - Plan and Section of New Valve Tower

In general a minimum 900mm cover was maintained to the pipe crown throughout its length. The various design aspects examined included preventing flotation of the main within the reservoir, incorporating low head air valves in the system and maintaining a falling gradient crossing the line of the vallum and over the last 100m of the 800m length where a rising ground profile existed. A particularly important aspect of the pipeline installation was the speed of construction required across the heavily trafficked Military road which involved employing a temporary traffic light diversion system.

DETAILED DESIGN - FLOOD PROTECTION WORKS

The existing spillway arrangement discharged into a bywash channel below the toe of the main West embankment. This channel had inadequate carrying capacity to convey the peak PMF which is the design condition. The modifications entailed the extension of the spillway by bridging the East Bywash channel to discharge into the adjacent Northern Subsiding Reservoir. The demolition of the lower section of the spillway was carried out in two stages to provide support to the existing structure.

A stability analysis of the Great Northern Reservoir's main West embankment was considered necessary due to large areas of material loss on the upstream face and evidence of seepage along parts of the downstream toe. The analysis employed the results from the ground investigation which had revealed relatively homogeneous material for the full cross-section rather than the distinct puddle clay core shown on the record drawings of the embankment construction.

A system of counterfort perforated fireclay drainage pipes were placed in gravel filled trenches on the downstream face and toe of the embankment in an attempt to lower the phreatic surface line through the embankment and improve its stability. The drains were led to a series of manholes where flows could be gauged.

A number of other remedial measures were recommended by the Inspecting Engineer in the interests of safety to ensure the integrity of the embankments and containment of wave action and flood waters. These included regulation of the embankment crest to above still water flood level, placement of gabion mattresses on the upstream face of the West embankment which was subject to erosion damage, and provision of gabion wave walls on the West and South embankments. The gabions adopted were of galvanised weldmesh construction filled with local selected stone.

CONSTRUCTION

The Great Northern Reservoir Works were constructed by M.J.Gleeson Group plc between 12th November, 1990 and 1st November, 1991. At an early stage during the construction period, further inspections by the All Reservoirs Panel Engineer identified a requirement for additional repair work which was not covered under the main contract. The elements comprising the additional works contract included low pressure grouting to lengths of the reservoir embankment and the new valve tower, repointing of the existing masonry valve tower and culvert within the reservoir and constructing a toe berm on the upstream face of the West embankment.

The refilling of the Great Northern Reservoir was carried out at a rate of not more than 300 mm/day. Early in the procedure a leak was observed at one of the inspection manholes on the downstream toe of the West embankment on the line of the secondary draw-off pipe. It became apparent that the Contractor, in the course of installing the counterfort drainage system, had damaged a 3" diameter cast iron vent from the old 24" diameter draw-off pipe from the reservoir. The vent pipe was reinstated to the crest of the embankment.

CONCLUSION

The project value was £1.4 million and spanned a 3 year period from inception to completion in advance of the commissioning of Whittle Dene Treatment Works which was inaugurated by H.R.H. The Princess Royal in May, 1992.

ACKNOWLEDGEMENTS

The authors are grateful to North East Water plc for their permission and encouragement in the publication of this paper.

Construction of a concrete face to a rockfill dam: Messochora Dam, Greece

P.J.WILLIAMS, Sir William Halcrow & Partners Ltd

SYNOPSIS. The following paper discusses the principal aspects of the design and construction of the concrete membrane for the Messochora dam in Greece.

INTRODUCTION. The Messochora dam is a Concrete Faced Rockfill Dam of height 150 m and crest length 350 m constructed between 1992 and 1995. The dam is situated in a narrow gorge on the upper reaches of the Acheloos River in the north western part of Greece. The Acheloos River has a total length of 265 km and extends from elevation 2393 m in the Pindos mountain range to the Ionian Sea. The catchment area of the Messochora Dam is 633 km^2. The reservoir has a total storage of $358x10^6$ m^3 and a live storage of $288x10^6$ m^3 between elevations 731 m and 770 m. The layout of the dam is shown in Fig.1.

The purpose of the dam is to provide river regulation and storage for hydropower generation (160 MW at 235 m head). Rockfill construction was favoured because of the relative seismicity of the area. The lack of suitable sources of material for the construction of an impermeable core led to the adoption of an upstream concrete membrane design. The concrete membrane was constructed by the slipforming method, with strips up to 250 m long.

The site suffers from particularly poor road access and temperature extremes, ranging between +40°C and -10°C. Low temperatures and snow cover for much of the winter months meant that construction virtually ceased between mid-December and the end of February.

DESIGN AND CONSTRUCTION

The upstream and downstream face slopes of 1:1.4 (V:H) were established on the basis of general experience. The concrete face slab was designed with a varying thickness in accordance with the formula $t=0.3+0.003H$, where t is the perpendicular slab thickness in metres and H is the water head in metres below top water level. Reinforcement of 0.4% of concrete area each way was provided at the centre of the slab with additional reinforcement at the edge of the strips to prevent cracking. The "as constructed" slab was an average of 50 mm thicker than the design thickness due principally to surface irregularities.

The reservoir as an asset. Thomas Telford, London, 1996

A total of 29,500 m^3 of concrete was placed in the 49,600 m^2 facing slab. A granular drainage layer of less than 75 mm material was used to form the upstream face with a less than 400 mm material in the transition zone between the rockfill and the drainage layer.

The construction programme required that the rockfill of the dam be completed to full height before construction of the face slab commenced. This requirement made construction of the 5.5×10^6 m^3 embankment simpler, but gave rise to slipform strips of up to 250 m long. In order to prevent unravelling of the drainage layer at the face during the winter period the surface was protected with a 75 mm thick layer of unreinforced shotcrete. Although the shotcrete cracked during this period it provided adequate protection and no significant surface erosion occurred. Prior to construction of the facing slab strips, the areas of shotcrete were tested to confirm that they were still bonded to the surface and, where loose, were removed before concreting.

An upstream toe plinth was constructed to provide an impermeable connection between the face slab and the foundation rock and was also used for the drilling of the curtain grouting. The toe plinth was orientated to provide a perpendicular joint surface to the face slab at all locations. The peripheral joint between plinth and facing slab was formed with a waterstop and surface sealant as shown in Fig.2.

The vertical joints between the facing slab strips were constructed with copper waterstops to the underside face. Those joints toward the abutments, where joint opening was expected, were also provided with a surface joint seal similar to that provided at the perimetric joint. No shear reinforcement was provided between the face slab and the toe plinth or between adjacent facing strips.

The following 30 MPa concrete mix was adopted for the facing slab:

Cement		300 kg
Water		144 kg
Sand		750 kg
Aggregates	20 to 40 mm	530 kg
	5 to 20 mm	610 kg

Admixtures : Plasticiser & Air Entraining Agent

This mix proved workable during both summer and early winter conditions with adjustments made to the plasticiser to ensure a 5 cm slump at the slipform, without adjusting the water cement ratio. Air entrainment was designed as 4% to provide protection against freeze/thaw conditions. It had originally been intended to adopt a higher cement content but this was reduced to control the concrete placing temperatures below the specified 28℃.

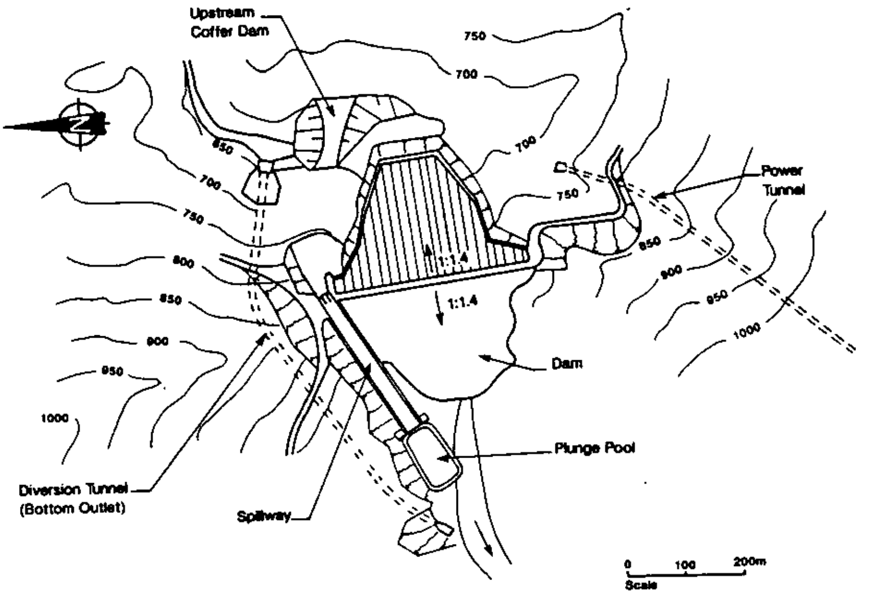


Fig 1 : Plan of Messochora Dam

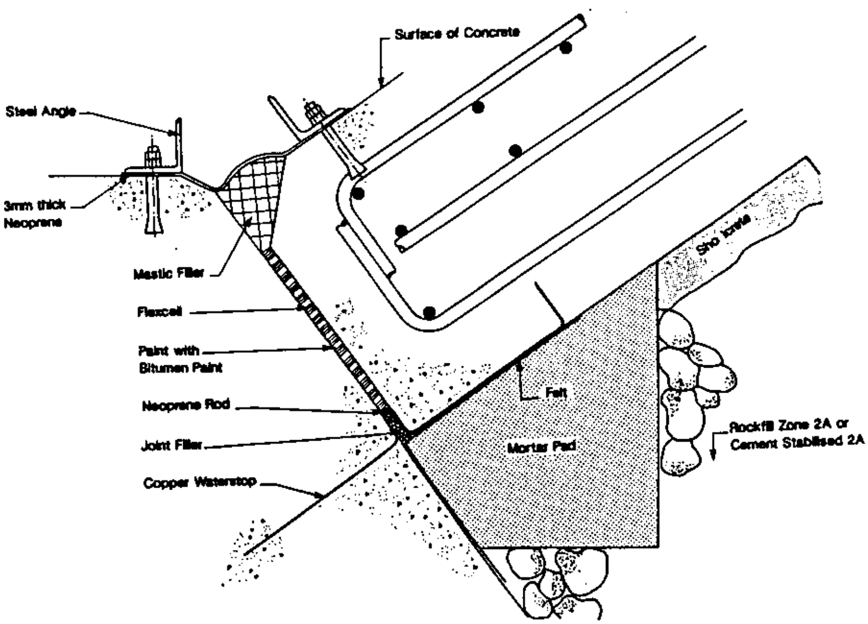


Fig 2 : Peripheral Joint Detail

The facing slab was constructed by the slipforming method in strips of 15 m width, working sequentially from the left abutment. The triangular portions of the lower part of the strips at their joint with the toe plinth were treated as starter bays to provide a full 15 m wide construction joint for the slipforming. The starter bays were constructed by hand in advance of the slipforming of the strip above. A version of the traditional method of slipforming was adopted, in which the concrete delivery conveyor and slipform were supported on wheels running on rails fixed to the braced side forms. The slipform was moved up the dam face by means of hydraulic jacks mounted on the slipform.

PERFORMANCE

The original programme for the construction of the facing slab called for concreting to commence at the end of May 1994, with completion by early November. This programme, comprising 24 weeks, was designed to make full use of the summer construction period and aimed for completion before winter when snow and low temperatures would be expected to preclude concreting. In the event construction of the face slab did not commence until the end of July 1994. Construction of the face slab continued from the left abutment until low temperatures required the suspension of slipforming after completion of strip 15 (numbered from the left abutment) in early December 1994. It was not until early May 1995 that the weather improved sufficiently to enable the slipforming of strips to resume and to be completed by mid July 1995.

The overall performance attained in constructing the concrete face slab largely followed the scheduled programme. Teams assembled at the commencement of the slab construction were assigned to one of the five sub-tasks, comprising mortar pad construction, reinforcement fixing, waterstop placement, side form fixing and concreting. It proved difficult to realise the maximum potential rates for certain of the preparatory tasks such as waterstop fixing and side form placing, as progress on these was largely determined by progress on other activities such as reinforcement fixing and concreting. The performance achieved by the teams associated to each of the sub-tasks is summarised in Table 1. Whilst the concreting was carried out on a two shift basis (each 12 hours), the other sub-tasks were undertaken on a day shift only.

Mortar Pads - This sub task included cleaning down of the strips, removal of loose shotcrete (particularly on the line of the mortar pad), setting of survey pins, fixing of mesh reinforcement, setting of chutes for mortar delivery from the crest and finally concreting of the pad. Progress on the mortar pad construction was delayed at times because of the risk of injury to personnel engaged on other tasks on adjacent strips when cleaning down. The erection of wire mesh safety fencing between bays failed to eliminate the problem.

Table 1 - Concrete face construction performance

Sub-Task	Personnel in Team	Productivity	Shifts
Mortar Pads	8	50 m/day	1*10hr
Reinforcement Placing by Hand	10	60 m^2/day	1*12hr
Reinforcement Mat Prefabrication	10	700 m^2/day	1*12hr
Reinforcement Mat Placement	10	600 m^2/day	1*12hr
Waterstop Fixing	8	60 m/day	1*10hr
Sideform Fixing	8	50 m/day	1*10hr
Concreting by hand	15	320 m^2/day	2*12hr
Concreting by slipforming	15	1100 m^2/day	2*12hr

Reinforcement - The placing of reinforcement fell into two distinct categories, that placed by hand and that placed by means of a trolley running on the face of the dam. The reinforcement to the starter bays could not be easily prefabricated because of the triangular shape nor was it possible to use the placement trolley in the starter bays. For these areas the individual bars were delivered to the starter bays from the abutments. In general the reinforcement to the strips was prefabricated in 14.5 by 6 m mats and placed by means of a trolley spanning the strip. All reinforcement was delivered to the crest from a prefabrication yard and lowered down by trolley to its final position. The mats were supported on steel bar chairs and pins driven into the rockfill surface through the shotcrete.

Waterstop - The sub-task comprised the laying of a bituminous felt layer on the mortar pad, the transfer of copper waterstops from the crest in 6 m lengths and the brazing of those lengths to form a continuous strip. In general the progress on the fixing of the waterstops was dependent on the progress on the fixing of the reinforcement and it was not always possible for the team to achieve their full potential.

<u>Side forms</u> - The erection of the side forms and the rail ready for slip forming included the fixing of anti crack reinforcement at the edge of the strip. As with the waterstop, progress on the erection of the side forms was largely dependent on the progress of the preceding tasks.

<u>Concreting</u> - All of the starter bays and the two abutment strips less than 15 m wide were concreted by hand. Where access on the abutments permitted, concrete was delivered to the bay by pump. Where this was not possible concrete was transferred to the bay by chute from the crest. The remainder of the facing slab was concreted by slipforming in 15 m wide strips. In general the slipform was lowered down the slab from the crest using the hydraulic climbing jacks in reverse. Slipforming rates showed a distinct improvement as the teams became more experienced. The average concreting rate for the longer strips was 28 m^3/hr with slipforming rates varying between 1.5 m/hr at lower elevations and 5 m/hr towards the crest.

EQUIPMENT

The rockfill embankment was constructed to elevation 733 m (2 m below the crest level) before construction of the facing slab. This temporary crest had a width of 13 m and provided more than adequate space for all equipment operation, materials storage and access. During the majority of face slab construction operations it was possible to maintain through access along the crest. A 50 tonne mobile crane was used for moving the following equipment:

reinforcement placing trolley (8 tonnes)
slip form conveyor section (6.5 tonnes)
slip form screed section (6.5 tonnes)
reinforcement mats (up to 5 tonnes)
winches and materials

The reinforcement placement trolley and slipform equipment were fabricated by the contractor JV Meton-AEGEK to designs prepared by Brasilos SA. This equipment was based on similar equipment used successfully on other projects world-wide.

A particularly useful piece of equipment used on the face was a 1.2 by 2 m equipment sledge. This was constructed to deliver materials and equipment up and down the face. The sledge was controlled by a single winch (3 tonne) at the crest and was provided with steel runners for use on the reinforcement mat. The sledge could also be fitted with wheels for running on the finished concrete surface. A larger rubber wheeled equipment trolley was used on the shotcreted face of the embankment for cleaning down and for delivery of chutes for placement of mortar pads. This trolley was raised and lowered by means of a winch mounted on a truck. Adjustment to the rear wheels to make the trolley "steerable" proved particularly beneficial.

CHUTES

Steel chutes were used for the delivery of concrete down the face from the crest to the slipform equipment. The chutes were fabricated from 0.55 m diameter oil drums, cut longitudinally, braced with steel bars and provided with hooks to enable successive chute sections to be hung down the face as a string. Initially the chutes were formed from third parts of the oil drum circumference. However, problems were experienced when delivering concrete down these chutes as the sides were frequently overtopped.

The revised chute arrangement avoided spillage, with concrete feed rates up to the 30 m^3/hr required. Hessian sheeting was placed over the chute section and kept damp to control concrete temperatures. The 0.8 mm thick steel of the chutes suffered considerable wear due to the friction of the concrete and it was found that the chutes developed holes or became too thin to use after approximately 2-3,000 m^3 of concrete delivery. The particular advantage of this chute arrangement was that the 0.9 m section lengths allowed simple horizontal and vertical adjustment of the line and the comparative light weight sections could be easily and safely manhandled on the face. In order to support the chute strings, every fifth chute was tied down to the concrete face of the adjacent slab.

REINFORCEMENT PLACING TROLLEY

The placing trolley was used to deliver prefabricated reinforcement mats from the crest of the dam to the embankment face ready for fixing. The trolley comprised a rectangular three dimensional lattice frame, running on wheels in channel rails fixed to the mortar pads and spanning across the 15 m strip. The trolley was moved up and down the face by means of two winch units. These frame mounted units each consisted of a hydraulically powered winch with an 8 tonne capacity. They were controlled from a dual hydraulic power pack with mechanical linkage to ensure synchronisation between the operation of the two winches. The arrangement of the placing trolley is shown in Fig.3.

SLIP FORM

The slipforming equipment included a conveyor discharge beam and a slipform beam incorperating a finishing platform. The concrete from the chutes was fed into a hopper at one end of the conveyor and the concrete discharged across the face of the strip by means of a movable inclined blade. The conveyor section was linked to the slip form at each side by means of a hydraulic jack, with the jacks pushing the conveyor and pulling the slipform. Self-adjusting wedges at the conveyor and the

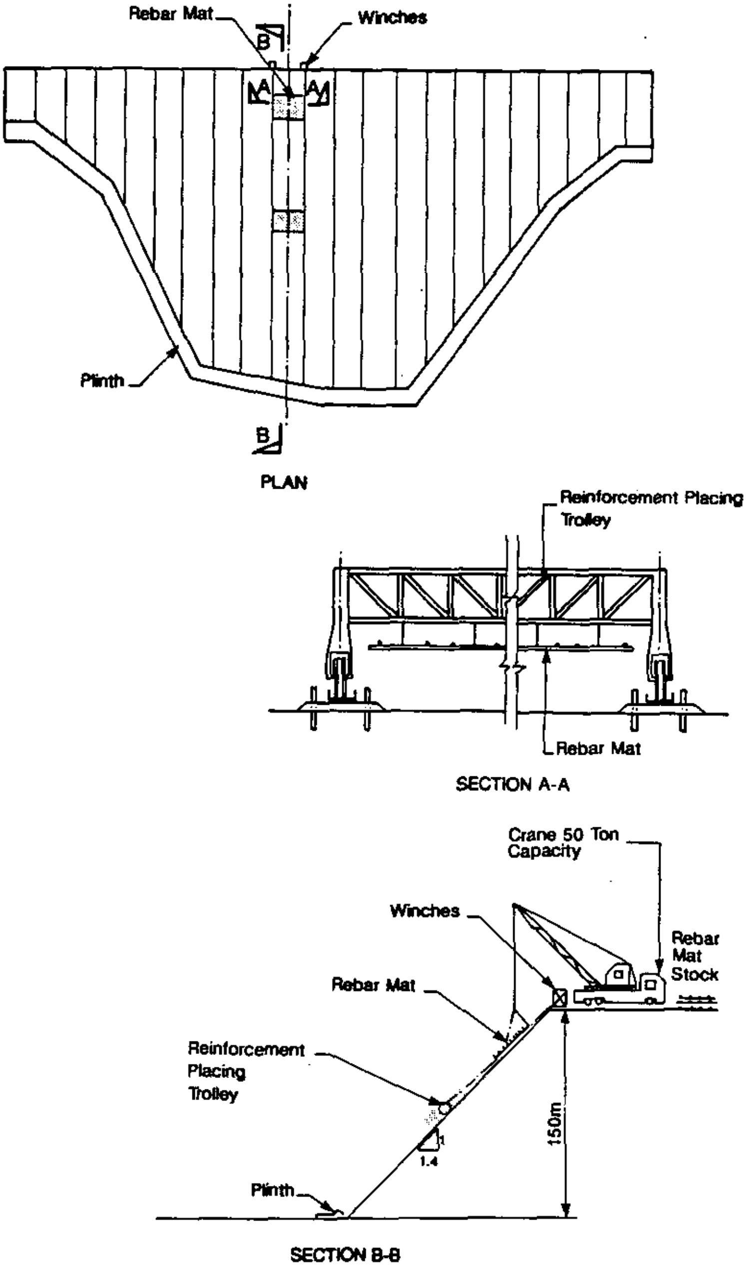


Fig 3 : Reinforcement Placing Trolley

slipform prevented sliding on the rails and allowed the equipment to climb.

Details of the slipform are shown in Fig.4. Once constructed there were very few modifications made to the slipform. These were generally minor in nature and did not change the overall performance. The 4 m space between the conveyor and the slipform provided sufficient space to compact the concrete with immersion vibrators and in general the concrete was being slipformed between 1 and 2 hours after batching. The 0.4 m stroke of the hydraulic jacks and the 1.0 m long slipform face proved adequate to achieve the slipforming rates required.

The finishing platform arrangement provided suitable access to allow personnel to complete the wood float finish without undue discomfort. Plastic sheeting was dragged behind the slip form to provide cover from the sun and rain. Water curing was carried out behind the plastic sheeting where the concrete surface had gained sufficient strength.

CURING

Water curing of the completed face slabs was continued for 1 month following construction. The curing was carried out by wetting the surface of the concrete continuously by means of perforated steel pipes. Immediate curing of the lower sections of the longer slabs during slipforming was achieved by dragging a perforated pipe approximately 15 m behind the slipform. Initially the finished slabs were covered with hessian strips in addition to water curing. This procedure had to be suspended in October when high winds caused the hessian to lift and roll transversely across the face of the dam, thus negating any benefits and deflecting the flow of curing water.

CRACKING AND JOINT MOVEMENT

Transverse shrinkage cracking was observed on all of the slabs at a spacing varying between 6 and 20 m, with most of the cracking occurring within 6 weeks of construction. In general the cracking was more regular on the longer strips with little cracking on the shorter abutment strips. Inspection showed the majority of cracks to be between 0.1 and 0.2 mm wide with the maximum being 0.3 mm. No treatment of the cracks was undertaken as it was anticipated that they would largely seal by autogenous healing.

The opening of joints between strips was monitored. In general the joints had only opened between 2 and 4 mm at 6 months after construction. The only joint to exceed this range was that between strips numbered 3 and 4 at the left abutment which opened to 7 mm approximately 2 months after construction

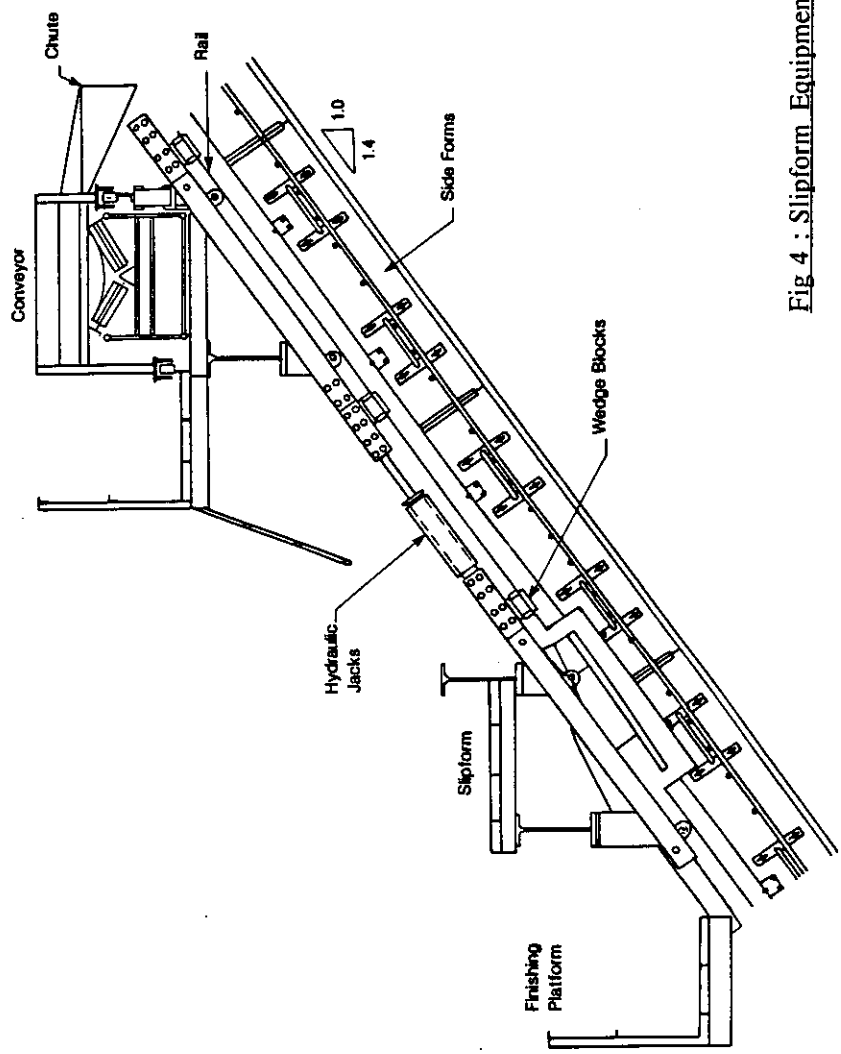


Fig 4 : Slipform Equipment

RUNOFF

Methods to control run off down the face of the dam were introduced in October when the frequency and intensity of storms increased for the winter. The purpose of these measures was to reduce erosion of open areas of the rockfill face and to limit water contamination when placing concrete. The method adopted consisted of nailing a thin galvanised steel sheet, bent to form an "L",to the surface across the strips at an angle. The water from these gutters was then collected to drainage pipes to run down the face of the embankment.

CONCLUSIONS

The concrete faced rockfill dam construction proved to be well suited to the site conditions at Messochora because although poor road access and extremes of weather frequently disrupted the construction programme this did not adversely affect the integrity of the dam. For a dam the height of Messochora it would be usual to construct the embankment and facing in two or even three stages. Considerable benefits, including efficiencies of scale, were derived from the decision to complete the dam to full height prior to the construction of the concrete membrane.

ACKNOWLEDGEMENTS

The author would like to thank the Public Power Corporation (PPC) of Greece and AEGEK SA of Greece for permission to publish this paper. Particular acknowledgement is given to Binnie Black & Veatch with whom Halcrow associated in providing construction advice and assistance.

Embankment Dams in Nigeria

J W FINDLAY, Babtie Group, UK
J W KERR, Babtie Group, UK

SYNOPSIS. The authors review a selection of topics related to the value of reservoirs in tropical Africa and the means by which they are created and remain effective. This is presented against the background of their own experience and the workload of their firm in Nigeria. Available water in Africa is invaluable and reservoirs can play a major part in achieving this. As in most regions of the world inappropriate development is counterproductive but it is arguable that appropriately sized storage can only be an asset to populations that often have few other sustainable water resources.

INTRODUCTION

Babtie Group have worked on a range of reservoir projects in Nigeria since the early eighties and during the course of this activity staff have been exposed to the particular pressures and rewards of providing resources in a water stressed environment.

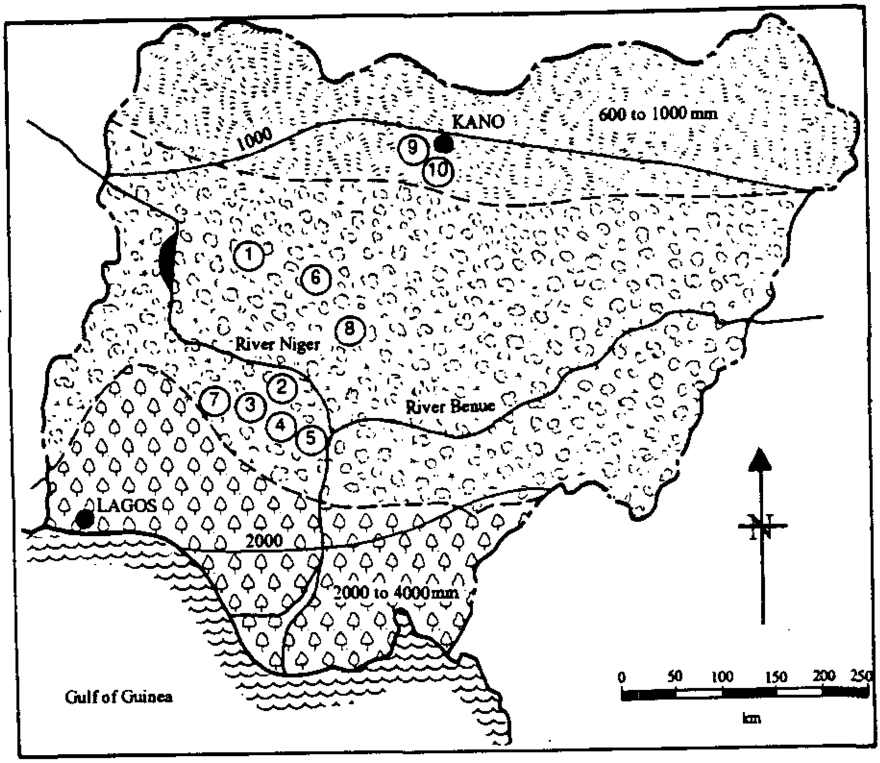
This experience has covered earthfill, rockfill and concrete dams mainly in the North of the country (Fig. 1). The work has involved design & construction and remedial & improvement works as well as studies and safety inspections. Clients include turnkey contractors and development organisations as well as government departments. The basic details of the various dams are shown on Table 1.

The nature of the geomorphology in the north of Nigeria results in many dams being embankment structures and it is this aspect that is the basis of the paper. With some exceptions the terrain is gently undulating and foundation conditions are formed of residual soils.

Limited water availability is the reason for reservoir development but the frequency and intensity of its availability also plays a part in the construction and maintenance of the embankment structures that are often the most cost effective means of storage provision in semi-arid areas.

From the yield of the catchment and the related flood aspects, through the needs of construction to the effects of climate on the finished structure, precipitation plays a crucial role in the construction of embankment dams.

The reservoir as an asset. Thomas Telford, London, 1996



<u>Key to dams</u>
(refer to table 1 for details)

1. Kontagora
2. Isanlu
3. Egbe
4. Ekuku
5. Kabba
6. Kagara
7. Offa
8. Tagwai
9. Tiga
10. Ruwan Kanya

<u>Key</u>

Steppe

Savannah

Rain Forest

1000 Isohyetal

Fig. 1. Map of Nigeria

Table 1. Dam details

NAME	TYPE	HEIGHT	LENGTH	RESERVOIR SIZE
DESIGN & CONSTRUCTION WORKS				
KONTAGORA	EARTH EMBANKMENT WITH CLAY CORE	20m	1000m	17Mm3
ISANLU	ROCKFILL WITH HDPE MEMBRANE ON U/S FACE	19m	150m	1.5Mm3
EGBE	MASS CONCRETE	5m	80m	$0.5Mm^3$
EKUKU	MASS CONCRETE	8m	100m	$0.6Mm^3$
KAGARA	EARTH EMBANKMENT WITH CENTRAL CORE	30m	1700m	$48Mm^3$
REMEDIAL & IMPROVEMENT WORKS				
OFFA	COMPOSITE GRAVITY EMBANKMENT	7m	250m	$1.5Mm^3$
TAGWAI	EARTH EMBANKMENT	30m	1500m	$50Mm^3$
TIGA	EARTH EMBANKMENT	47m	5800m	$1968Mm^3$
STUDIES & SAFETY INSPECTIONS				
TIGA	EARTH EMBANKMENT	47m	5800m	$1968Mm^3$
RUWAN KANYA	EARTH EMBANKMENT	22m	3700m	$58Mm^3$

AFRICAN RESERVOIRS - ASSET OR LIABILITY

By the simplest of definitions every reservoir is an asset in the sense that it is an element of infrastructure. Whether it is truly an asset or just a liability in economic or human terms depends on how it performs.

Reservoirs are unarguably useful but is this sufficient reason for their creation. Useful to whom; to local, regional or national populations or to limited commercial or political interests? These are questions that should be asked in determining the value of a scheme. Further, does it provide a reliable resource in the quantities predicted. Does it fit with its environmental setting. Are the consequent effects acceptable. Will it be designed and built to last?

The answers to all these questions will be strongly influenced by the relationship of the respondent to the project. Sadly they often give cause for concern and engineers cannot just restrict their interest to technical aspects of dam performance nor must they leave environmental problems unchallenged.

Carefully handled, construction of medium sized reservoirs in dry areas of Africa can be a major benefit and can overcome most of the objections that are currently raised in opposition to dam based schemes. Politically they have the attraction of being conspicuous development in areas of need , although this is open to exploitation on religious or tribal grounds. They serve to economically store varied and unreliable run-off. When conditions suit they offer gravity supply that removes the costs and ongoing maintenance of electro-mechanical plant, drawbacks which often invalidate groundwater schemes. As a readily accessible water resource on their margins they have advantages to those living closest to the development irrespective of ability to pay for piped supply. Finally, they can offer improvements in local shallow groundwater storage that maximises the availability of clean seasonal water from village wells (Findlay,1994).

Sizing of the reservoir is crucial and has to cope with the variability of annual run-off and evaporation, thus very small reservoirs are not always reliable. Conversely, very large storage reservoirs on major rivers can have significant effects on downstream agriculture and fishing.

The basic perceptions from the engineering and environmental communities can be respectively summarised as follows.

- provision of any stored water is good
- dams introduce untenable problems

The reality is generally somewhere between the two opposing views and depends as much on the climatic and geographic situation as it does on the human need for water or the need to preserve the environment. That there is an urgent need for more water resource development in Africa is clear but it is not always possible to achieve this without some element of the water environment suffering as a result. Falkenmark (1989) has introduced the concept of water stress in the African context. This looks at the overall availability of water in a region and compares it with the basic needs of the population.

Growing populations require water not just for their basic survival needs but to increase food production and in the longer term to meet social development aspirations. There is only so much water available from the global water cycle in any region and in some African countries this figure is pitifully small. It is not just the long term shortage of water but the severe fluctuations in annual supply that cause the water stress problems common in the last decade. As the population increases the options for development of the water resources reduce. In some cases this critically limits the application of technology to the problem.

Although Nigeria is the most populous country in Africa with correspondingly steep predicted growth rates, the water availability from consideration of the global water cycle is sufficient under medium or high yield situations to support the predicted size of population until well into the next century. This holds true for most of the West African nations.

This does not mean that efforts to minimise stress of the water system due to population growth should be reduced but does open the way for technological solutions to the availability and use of the water in those countries. In particular the use of water storage is recognised as an essential part of the process.

The primary benefits of a reservoir in Africa can be stated as :

- a means of retaining seasonal river flow for later use
- a necessary storage facility to provide sustainable raw water resources throughout drought years
- a vital source facility for irrigation schemes to accommodate the varying water demands during the crop seasons.
- a means of mitigating the effects of floods.

Set against these primary advantages there are problems associated with reservoir construction that can in some circumstances cancel out the benefits to be obtained. These include: insufficient or unreliable catchment yield, sedimentation and poor construction that limit the planned volume of storage; displacement of communities with resulting problems of relocation, loss of good agricultural land and subsequent pressure on land elsewhere; changes in natural flood cycles with effects on traditional irrigation methods, loss of silt nutrients, lowered soil moistures and poor fishing that can destroy long-standing local economies.

In Nigeria there is evidence that each of these effects has played a part in previous poor scheme performance.

Lal (1985) describes the generally poor appreciation of the processes leading to soil erosion and notes that African soils can be highly susceptible to erosion, especially when combined with the high erosivity of tropical rains. This is particularly damaging at times when vegetative cover is poor and is a feature of West African conditions. Change in land use is a major cause of accelerated soil erosion with mechanical land clearance causing the greatest run-off and erosion. Grazing is also significant as is the practice of burning-off vegetation which affects erosion and infiltration rates. Soil loss rates for Nigeria are reported at up to 500 $t/km^2/yr$ with figures for reservoir sedimentation estimated in the range 0.05 to 20 Mt/yr. (Oyebande,1984). Obviously an integrated approach to soil and water management is essential for any new scheme.

Adams (1985,1991) describes the downstream effects of large dam based irrigation schemes in Northern Nigeria and highlights the reasons for the poor performance of some of the largest schemes even when judged solely on irrigation performance. More critically he notes high social, environmental and economic costs associated with the schemes. This can be attributed in part to the poor supply reliability of the irrigation infrastructure, including dams. While noting the need for storage dams on the seasonal rivers of Northern Nigeria, to allow for dry season irrigation, he illustrates the problems that very large reservoirs can have on the traditional forms of small scale irrigation and wet season agriculture based on the flood plains downstream of the dam. Sizing and reliability are thus crucial to success of storage schemes.

These matters require to be addressed from the very beginning of a scheme if full advantage is to be taken of available water resources to improve the quality of life and self sufficiency of the African peoples.

However, it is not just a case of getting the concept, planning, environmental balance and economics right. The structures have to be well engineered for their purpose and situation if they are to function for the whole life of the scheme. To ensure reliability the design, construction, operation and maintenance of schemes has to match the care given to environmental planning.

TECHNICAL ISSUES OF DAM CONSTRUCTION

Babtie Group have worked in Nigeria since the early 1980's, most often in a design and construct role with Biwater International, and the bulk of this work has been for the provision of storage reservoirs for water supply. The structures designed for this role have been varied and include concrete, rockfill and earthfill dams. Notable technical features of these projects include the articulated overbank spillway at Kontagora Dam (Berry et al, 1988) and the use of an HDPE membrane on the upstream face of Isanlu Dam (Berry et al, 1988).

Perhaps the most vulnerable of dams in the Nigerian environment is the embankment dam and it is this category on which the technical aspects of this paper concentrates.

Much of the material available for embankment construction is generally described as laterite. This has a vast range of sub-types from low permeability clayey material to freedraining granular soils. River bed and flood plain deposits can also yield a range of sands for filters as well as pure clays for use in cores. With these resources the dam designer in Nigeria often has a range of options for embankment design. However, the available fill is dependant on its physical condition and placement methods for satisfactory performance. Climatic conditions play an important part in this process, particularly the influence of rainfall which determines not only the natural condition of the fill but also when it can be placed.

The relationship between the parties on site is also crucial. Methods often have to be developed to deal with circumstances on site and ensure economic placement of fill to give the designers intended performance.

Construction Phase

The placement of fill to a consistent standard is crucial to the success and safety of all embankment dams. Rainfall and temperature have a major influence on this aspect of construction. In wet, temperate climates control is usually hindered by excessive rainfall, leading to onerous specification restrictions on times of placing, both in anticipation of rain and for an appropriate drying out period thereafter.

In the semi-arid climate of sub-Saharan Northern Nigeria the problem is the constant searing heat of the dry season which normally occurs from September through to May or June of the following year. Midday temperatures regularly rise to over 140 F in the direct sun.

During the wet season certain dam elements such as concrete works, rock armouring and grouting work for example, can usually be progressed.

However it is extremely unproductive and eventually not physically possible, to continue with embankment earthworks during this period. Unless the borrow pits conveniently lie on a self draining slope they usually become waterlogged very rapidly. Although daily rains are typically only of 2 to 3 hours duration and the ambient temperature is adequate to ensure some drying out of the material thereafter, it is often found that access to the borrow areas is not feasible. The laterite soils of the haul roads only require a slight increase in surface moisture to make them treacherous and totally unsafe for large earthmoving plant.

Embanking works must therefore be achieved during the dry season and ways have to be found to compensate for the extreme heat and drying winds to ensure that fill placement is achieved to a consistent, acceptable standard.

The problem of placing consistent fill is described from practical on-site experience, under the following key headings:

a) moisture content control
b) fill conditioning
c) optimum compaction

Moisture Content Control. Control of the moisture content of cohesive fill material is central to the efficiency and ultimate success of any embankment dam construction. This is not just because of the technical aspects influencing optimum compaction, but also because the condition can vary substantially between borrow areas and from day to day. In situations where material is being won concurrently from two or more borrow areas, it is vital that a regular and structured approach is adopted to ensure adequate monitoring of the material. This can have a significant effect on a contractor's operations when considering the large volumes of material involved.

On a recent dam project in Niger State, over 850,000 cu.m. of fill were required. On this particular site the borrow areas were located over a wide area and whilst being individually quite extensive, were nevertheless rather shallow, typically being only 1 to 2 metres in depth. It would have been advantageous to have had deeper borrow pits with less surface area to avoid drying out. In the circumstances it was therefore very important for the contractor to plan the extraction and use of the material particularly carefully, so as to minimise the exposure of the material to the severe drying effects of the wind and sun.

On this site, natural moisture contents of material in the borrow areas, although quite variable, were typically between 11 to 13 %. On-site laboratory "Maximum Dry Density/Moisture Content" testing indicated an optimum value of 16 to 17 % moisture content for core material and 12 to 13 % for shoulder materials. There was therefore a need to raise the natural moisture content by about 4 or 5 % before placing. This is a substantial increase in moisture content and required significant fill conditioning operations to be carried out.

Fill Conditioning. One of the most important requirements of any fill conditioning operation is an adequate supply of water. Most rivers in Northern Nigeria are seasonal and water has to be stored for dry season use. The easiest way to achieve this is to temporarily dam the river. Experienced judgement is vital in the preparations prior to the main embanking period and before the river actually dries up. Adequate bunds have to be formed across the river while it is still flowing. The great risk in this period is forming the bunds too early and losing them to late wet season floods which are all too often as severe as they are sudden. Equally, it can be risky to delay too long in case the last of the base flow in the river is missed.

For fill conditioning it has been found that the most effective approach is to carry out the moisture content conditioning in the borrow areas and thereafter run the material directly to the embanking operations at the dam site. This approach separates the time consuming and plant intensive conditioning work from the more congested working area of the dam embankment itself.

Ideally, large water pumps and sprinkler systems should be available to achieve a steady, even application of water to the selected material. The most obvious disadvantage to this approach, is of course the high costs associated with trying to establish such a fixed installation when the actual locations of the materials being won are constantly moving. More typically a contractor will mobilise a number of water tankers. However problems can be encountered very quickly if inappropriate plant is used. The lateritic soils of the region very rapidly turn to a slick surface when excess moisture is applied.

Wheeled vehicles, no matter how rugged and apparently versatile, become bogged down very quickly. Care has to be taken to try and avoid flooding the surface of the material. Tracked machines can be used but these usually have the limitations of having to tow the water tank, which is itself often wheeled , but more significantly, they are slower than wheeled vehicles. Often the compromise that achieves the most productive results is to simply limit the rate of application of water at the expense of increasing the number of passes required.

However, it is not only the application of water which is required but the means of introducing it uniformly throughout the material. Various items of plant have been employed for this operation. Much depends on the layer thickness being conditioned. Agricultural type, tractor towed harrows can be effective on thin layers. However, for layers of the order of 200 to 250mm thick, road graders have been applied to this task with successful results when under the control of experienced operatives. The grader blades are used at an angle to turn the layers over.

Although all fill conditioning situations are different, a typical requirement can be in the order of 6 to 8 passes of the water tankers followed by a similar number of passes by the graders to achieve a uniform moisture content increase of approximately 4 %. This is a time consuming process and means of limiting or avoiding such high inputs should always be sought.

Optimum Compaction. Site laboratory testing establishes the maximum dry density/moisture content relationships of the fill to be used and while this is an essential guide to the conditions required for optimum compaction the site engineer must be aware of the influence of the actual plant in use on the insitu results to be obtained.

Trial embankment tests are a worthwhile investment of time and effort when compared to the resources used during the main works. These trials should indicate the combination of layer thickness; number of watering and conditioning plant passes and also the number of compactor passes likely to produce the most efficient overall operations on the dam embankment .

It is at this stage of the works that reappraisal of the design assumptions may be required in light of the embankment trial results and the available plant which has been mobilised by the contractor. Maximum dry densities may not be the key design requirement in the clay core for example, where moisture content and plasticity may be of more significance. It is essential therefore to establish clear construction guidelines for technical supervisory staff on site, setting out unambiguously the various design parameters, target values and tolerance ranges, for all material zones within the structure.

Long term performance

Having constructed an embankment dam to the designers intentions it cannot then be forgotten and left to perform without maintenance and remedial works. This is one of the major problems in Nigeria especially with small and medium sized dams. The geographic disposition of the assets of many organisations means that responsibility for monitoring and maintaining the structure is often so delegated that it is forgotten or rendered impossible by the distances and lack of transport. This results in minor defects developing into major problems that can compromise the function of the reservoir.

The principal cause of problems with the performance of embankment dams in Nigeria is the deterioration of the structure and its appurtenances. Sometimes these problems can be related to poor control of construction but it is the lack of maintenance that causes most problems. Climatic conditions also play a major part in the performance of an embankment dam but the effects can be addressed by careful design detailing and regular monitoring.

There are a number of features that have major influences on the performance of the dam, many of which also have dam safety implications.

Crest Profile. Some older embankment dams in Nigeria have very variable crest profiles. Variations of up to 2m have been observed and this is generally accompanied by oversteepened upper slopes and very narrow crest widths reduced in some cases to 50% of design dimensions. This has serious implications in terms of freeboard but also influences stability through loss of section. There are a number of reasons postulated for the original problem but the variability also leads to further problems of degradation and maintenance.

Settlement, if not properly allowed for in design or as a result of poor construction control, can result in inadequate crest levels. The importance of careful interpretation of the available fill characteristics is obvious and will lead to adequate allowances but the rigorous site control of placement cannot be overemphasised as a means of ensuring that settlement predictions are met. The pressures of seasonal construction and the need to maintain high production rates sometimes put a severe strain on the fill monitoring process, particularly when climatic conditions require careful moisture content control. The control of the fill conditioning processes described in the previous section is essential to the ultimate performance and safety of the dam and cannot be overlooked even though volume production may be going well.

Inaccurate setting-out of the embankment slopes at foundation level can lead to oversteepening of the upper slopes and narrowing of the crest in an attempt to meet design crest levels. Sometimes this has obviously failed and the combination of excessive settlement and poor geometrical control results in severe reductions in freeboard. This is particularly likely where the logistics of construction required embanking to proceed on several fronts and consequently foundation level at the deepest part of the valley is not established at the start of the works.

Good control of new works eliminates these problems but with existing dams there is little that can be done to improve this situation without major works to the downstream face. Often this is not possible because of drainage, landform or cost. This only leaves a reduction in the top water level as a means of restoring acceptable safety margins against overtopping and this carries a severe penalty in terms of storage.

The crest of an embankment dam in Nigeria is rarely given formal surfacing and this coupled with the inevitable local traffic along the dam leads to rutting and ponding which further aggravates the crest profile problem. Although the cost of a formal road surface is rarely justified it is prudent to include a graded stone sub-base below the surface fill to avoid the worst of the rutting that is formed in unstrengthened fills. Such a layer also improves the drainage characteristics of the surface and limits the potential for rutting and ponding. The crest profile should of course fall to the upstream side to avoid erosion of the downstream face. However there is no substitute for regular monitoring of the crest condition and for carrying out timely repairs.

Repairs to rutting and ponding require provision of well specified materials and adequately sized plant. Too often all that is available is some locally dug laterite and manual labour. In these circumstances any infilling is largely cosmetic and leads to further softening of the crest.

Downstream slopes. Erosion is the biggest risk to degradation of the downstream slope. The intensity of tropical rainfall is such that even vegetated slopes can be eroded. This is aggravated by the oversteepening mentioned earlier and by the tendency for vegetation to die down and be burnt-off at the end of the dry season. Provision of stone protection is a means of preventing erosion but comes with a heavy financial penalty and can disguise seepage indicators that may be the first sign of serious internal problems.

Direct rainfall on the slope can cause sheet erosion but more often variations in the surface profile concentrate flows and lead to gullying. Once this is established it can accelerate dramatically and gullies in excess of 3m deep have been recorded. Runoff from crest depressions further aggravates this problem and can lead to deep gullies cutting into the crest profile. The best solution to this problem is to ensure a well vegetated layer with adequate resistance to local conditions. Conventional seeding of topsoil has not been particularly successful due to the need to provide significant quantities of water evenly on the slope without washing out the seed or soil. It is not possible to rely on the natural soil moisture as won and rainfall intensities are such that they wash out the seed. Babtie Group experience has been to strip vegetated topsoil from borrow areas towards the end of the dry season and to rely on the germination of intermixed native seeds in the soil to establish dependable growth during the first rains. Should gullies be allowed to form the remedial works to these can be difficult to carry out.

Compacting fill material is almost impossible within the confines of a sloping gully and erosion tends to reappear on the same alignment within a short period. A more permanent repair using a graded crushed rock has proved successful but requires heavy plant to implement.

Upstream slopes, The biggest problem on the upstream face is the stability of the rip rap protection. Significant variation in general stone sizing has been noted and this together with oversteepening of slopes on the approach to the crest can lead to loss of protection under wave action. The probability of resources being available for providing and placing heavy stone at a later date is low and design should therefore use a no damage criteria.

Drainage. The intensity of rainfall during tropical storms is capable of causing severe erosion to natural formations and can wreak havoc on an embankment.

Provision of robust surface water drainage at the downstream toe of the embankment is essential to avoid erosion along the mitre and to avoid waterlogging of the toe across the valley bottom. Two factors conspire to make conventional methods ineffective

- runoff sediment accumulations block drainage media and pipes and infill ditch sections
- runoff flows erode natural ground and undermine drainage structures

The natural ground profiles tend to concentrate these effects close to the toe of the embankment. This is where designers often detail drainage provision but the effects noted above can render these useless. It is therefore essential to use lined ditch sections where valley slopes are steep and to keep french drains away from the toe where run-off material tends to collect and block the drainage media.

SUMMARY / CONCLUSIONS

The expectation of performance from water supply schemes in developing countries will always be high but the difficulties facing such schemes are significant. The dam engineer cannot afford to relax the standards of design and construction on the grounds that these are low tech solutions. It is essential to the beneficiaries that schemes perform and continue to do so despite prevailing circumstances. It is also essential to the profession that it is seen to be addressing need with appropriate solutions. In particular, water resource engineering cannot be viewed in isolation, there is an indivisible relationship with land management and agriculture.

The reservoir for water supply in Nigeria is therefore an essential part of development infrastructure but is only an asset if it has been well conceived, can be properly constructed and carefully maintained.

In remote situations there is a much better chance of avoiding deterioration by design and construction than in effecting repairs at a later date. The reasons for this are many but centre on the management structure of the responsible authorities. Maintenance requires staff continuity, availability of resources, transport and experience. These inputs are seldom available in Nigeria.

Designers and constructors of dams for storage of water must always keep in mind the level of monitoring and maintenance work that a structure is likely to receive in the course of its life. This is particularly relevant to the value of storage reservoirs in the Nigerian situation. A well designed and built dam has a reasonable chance of remaining in effective and safe service with minimum maintenance provided the parties are aware of the prevailing deteriorating forces and the standards necessary to resist these. While proper provision for monitoring and maintenance should be encouraged the limitations of the local situation should be recognised and unrealistic obligations should be avoided.

The environmental lobby has justifiably recognised that there can be great disadvantages with large reservoir schemes and considerable criticism of the engineering community has been offered in recent years. The dam engineering profession must continue to present an honest, well argued and robust defence. For the foreseeable future there will continue to be a need for new dams in many locations throughout the world. However, ever greater care and vigilance will be demanded in their planning, execution and maintenance. These are healthy demands and the profession should respond to them positively.

If this can be achieved then the reservoir as an asset will be assured.

ACKNOWLEDGEMENTS
The authors take this opportunity of recognising the contribution of their colleagues in Nigeria in developing the experience on which this paper is based.

REFERENCES

Adams W M (1985). The downstream impacts of dam construction: a case study from Nigeria. Trans. Inst. Br. Geogr. N.S. 10: 292-302

Adams W M (1991). Large scale irrigation in northern Nigeria: performance and ideology. Trans. Inst. Br. Geogr. N.S. 16: 287-300

Berry N S M, Millmore J P, Smith A & Findlay J W (1988). Kontagora Dam: Aspects of design and construction of an embankment dam relevant to a remote rural area of West Africa. Proc.Instn. Civ. Engrs, Part 1, Vol 84: 855-874

Berry N S M, Eadie H S & Macgregor I R (1988). Isanlu Dam, Nigeria. Proc. Instn. Civ. Engrs, Part 1, Vol 84: 875-894

Falkenmark M (1989). The massive water scarcity now threatening Africa - Why isn't it being addressed? AMBIO Vol. 18 No. 2: 112-118

Findlay J W (1994). Reservoir development in semi-arid countries - A human ecology perspective. Proc. 9th Conf. British Dam Society, Exeter.

Lal R (1985). Soil erosion and sediment transport research in tropical Africa. Hydrological Sciences Journal, 30, 2, 6: 239-256

Embankment dams in the developing world.

A D M PENMAN, Geotechnical Engineering Consultant, Sladeleye, Chamberlaines, Harpenden, AL5 3PW, UK.

SYNOPSIS. Increasing demand for food and water requires provision of reservoirs to store water for irrigation and municipalities. Embankment dams are the cheapest and most environmentally friendly means for storing water and three in various parts of the world are discussed in relation to river basin development for the benefit of mankind.

INTRODUCTION.

It has been said that civil engineering has played a more important role than that of medicine in the reduction of child mortality and other premature deaths, by provision of clean water and hygienic sewerage arrangements. Both require increased water supply, and are making a major contribution towards increase of population, which is currently running at a rate of 90 millions a year. This enormous number of new people (the population of the British Isles is about 57 millions) being added to the world's burden every year, requires a civil engineering response to try to provide the basic necessities of food and water.

Unlike the equable climate and moderate, well distributed rainfall enjoyed in Britain, many parts of the world suffer extremes of temperature and severe rainfalls that occur only during limited seasons. The temperature and hours of sunshine during the growing season may be advantageous for the production of rich crops if there were sufficient water for irrigation. In these areas there is often also acute shortages of water for industry and the population in general. River basin developments can be engineered to save the damaging flooding and enable the annual rainfall to be more equitably distributed to the considerable benefit of mankind.

SOUTH AFRICA.

Water, in many parts of South Africa, is a serious problem. President Mandela has said that the new South Africa needs more dams for hydro-power, irrigation, industry and domestic supply. Clean drinking water for each citizen within a reasonable distance from home will be the aim. There are many schemes in the country but only one, that of the Komati River Basin development, will be discussed.

The Komati and Lomati rivers rise in the Eastern Transvaal Highveld and flow through Swaziland then back into the South African Republic to join the Crocodile River at Komatipoort, before flowing into Mozambique and the Indian Ocean. The Komati basin has an area of 11 100 km^2 with an annual runoff of $1.43x10^9$ m^3 of which the Lomati contributes $0.364x10^9$ m^3. Considerations of development of the basin to ensure reliable water supply both for existing irrigation and the creation of new areas for sugar cane, plus a guaranteed supply for watering cattle, had been discussed since 1940. Seven dams were proposed, one of them on the Lomati. After considerable work and discussion, a treaty between Swaziland, South Africa and Mozambique was signed on 13th March 1992 to enable the 1st phase of the development to begin. This requires the formation, mostly in the South African Republic, of the reservoir on the Lomati with the construction of the 50 m high Driekoppies dam. The 2nd phase will require the construction of the Maguga dam on the Komati in Swaziland. The Driekoppies dam is 2.4 km long, with a concrete gravity section of 316 m, containing a 150 m long simple weir spillway. A cross section for the embankment is shown by Fig.1. The core and shoulders will wrap round the ends of the concrete, and the river has been diverted to pass through the concrete section during the construction of the rest of the dam.

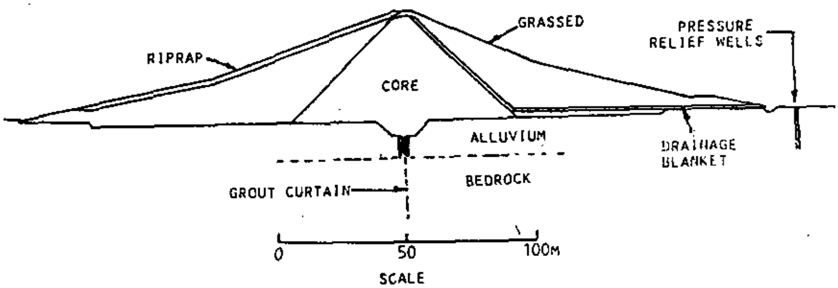


Fig. 1. Cross section of Driekoppies dam.

<u>Difficulties Encountered.</u>

At the site for the Driekoppies dam, the Lomati had, during the geological past, eroded three channels in the gneiss bedrock. These are now buried under 10 to 15 m of alluvium, and the river runs at the junction between gneiss and granite bedrocks, with most of the bed on the granite, to the right side of the wide valley. The alluvial sands were very loose, with a density of about 1.422 tonnes/m^3 over an area of 72 500 m^2 under the dam, to a depth of about 9 m. To avoid risk of liquefaction, the density was increased by lowering the water table and use of dynamic compaction. 38 pressure relief wells were placed at 20 m centres along the line for the downstream toe. Dynamic compaction over the deepest parts, was effected by a 14 tonne weight of 1.8 m dia., dropped 20 m. The first pass created holes 0.75 m deep. S.P.Tests were used to assess the improvement produced, and results are given in Fig.2. Energy used varied from 150 tm/m^2 for sand depths of 5 m, to 300 tm/m^2 for 9 m depth. For the 2nd pass, the energy was reduced to between 80 and 100 tm/m^2.

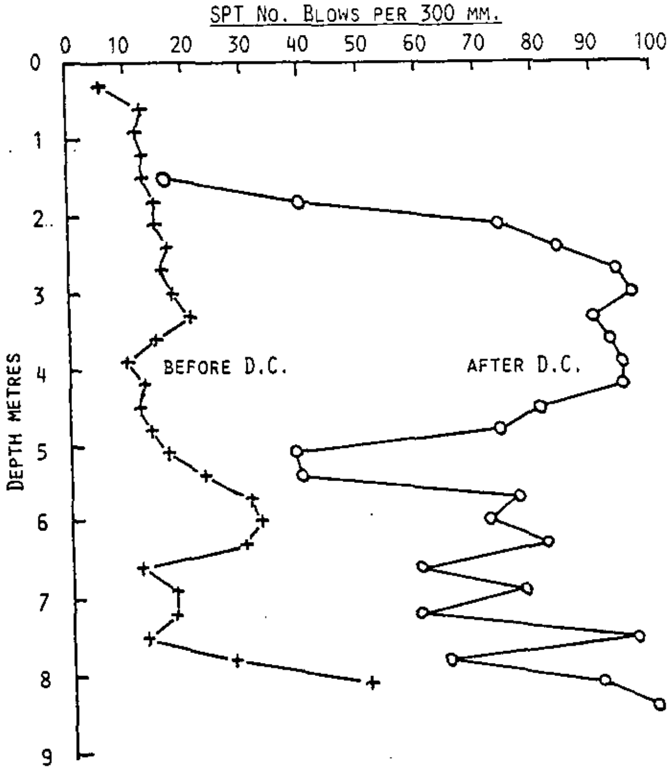


Fig. 2 Results from SPT made before and after dynamic compaction of the foundation alluvial sands.

The reservoir area of 1 870 ha, extends up to and slightly beyond the boundary between South Africa and Swaziland. The land is owned by tribal chiefs on behalf of his people and it required a good deal of work to identify all the tracts of land. 330 houses on the lower lying parts of Schoemansdal village have been replaced by new houses in a settlement built 5 km away. 260 graves had to be moved from areas that would be inundated. All interested people were located and exhumation and reburial completed by the end of 1993.

CHILE.

As a broad generalisation, the central plains of this narrow country, lying between the Andes and the Pacific, are covered in rich volcanic ash, and are crossed by rivers flowing from the mountains into the sea. From before the days of the conquistadors, little canals had been dug to carry water from the rivers to nearby fields, resulting in a band of irrigated land, several km. wide bordering each river. This left fairly wide areas of dry lands between the rivers, and there were proposals for reservoirs in the foothills of the Andes to supply feeder canals running north and south, to take water to systems of small canals for the irrigation of these dry areas. In the region of Chillán, 400 km south of Santiago, two reservoirs were planned, on the rivers Nuble and Diguillin. Both sites were at narrowings of the valleys; the one on the Diguillin formed by a succession of flows of lava from the Chillán volcano which had spilled into the valley, closing it off and forming a lake. It had overtopped and eroded a narrow gorge in the old left side of the valley, which was to be filled by the dam, to re-form the ancient lake. A snag was that each flow of molten lava formed a collection of loose blocks, (each the size of the granite sets used for the pavé of the old north European roads), on the surface, caused by the surface solidifying while the lower material was still moving. The next flow covered this loose, very hard material, so that the complete plug contained several very permeable layers. These were most difficult to grout, because diamond drills were shattered as they became jammed by these movable blocks of glass hard lava. A full site investigation and detailed dam design was completed, but it was eventually decided to obtain water from the Laja Lake by canal, rather than build the dam. Various political difficulties caused the Punilla dam on the Rio Nuble also to be abandoned, so wasting all the work that had gone into its completed design.

Limari Basin Development.

This area lies approximately between 30° 15' to 31° 15' S. and 70° 50' to 72° 10' W., with the town of Ovalle at its centre, about 350 km N of Santiago. More than half of the 12 000 km^2 basin is 1000 m above sea level, rising to 4500 m in the High Cordillera of the Andes. The Rio Hurtado comes down from the N.E. and the Rio Grande from the S.E. to join near Ovalle to form the Limari which throws all that fresh water into the Pacific. The Rio Huatulame comes from the south as a major tributary, joining the Rio Grande at Paloma. As in other parts of Chile, many small canals ran from the rivers to irrigate the nearby fields. It was found that there was a total of 677 of these uncontrolled canals, with open offtakes, spread out along the river network. The Departamento de Riego de la Direccion General de Obras Publicas had begun looking at the possibilities for development of the basin from shortly after its formation in 1914, with a view to providing irrigation for much more land. They had built the Recoleta dam on the Hurtado river in 1935 to impound 96×10^6 m^3; a capacity increased in 1940 to 100×10^6 m^3 by raising the crest of the spillway by 1.1 m. In 1938 the 85 m high Cogoti dam was completed on the Huatulame to impound 150×10^6 m^3. This concrete face rockfill dam has been subjected to four major earthquakes ranging from 7.1 to 7.9 on the richtar scale. The 7.9 event in 1943 caused an instantaneous settlement of 0.4 m, equal to the settlement that had already occurred since completion, resulting in a total settlement of 0.8 m (the dam has now settled a total of 1.1 m). The face slab was constructed in 10 m square panels, connected by generous copper water stops, and it has moved without developing any leakage. All that was necessary after the 1943 event, was to lower the spillway to reinstated the crest freeboard. These events have been described by Arrau et al (1985).

La Paloma Dam.

The third large reservoir of the scheme was formed by the 96 m high Paloma dam on the Rio Grande 23 km upstream from Ovalle, completed in 1967, impounding 740×10^6 m^3. The 24 year rainfall record showed that the three reservoirs were adequate for 37 860 ha of irrigation annually, of which 9 192 ha was double cropped, making a single crop equivalent of 47 052 ha. This would increase agricultural output by a factor of 2.86.

At the site the valley was cut into the granodiorite bedrock, with a deep central buried valley filled with alluvium, and the river running on the left side, giving a left abutment of the granodiorite rock. The alluvium consisted of a permeable gravel, `fluvial moderno', 14 m thick, underlain by more than 100 m of a badly weathered sandy gravel know as `fluvial antiguo', that was slightly cemented by calcite. The appearance of this very old alluvium was very like a normal sandy gravel, but the cobbles were full of incipient cracks. When two pieces were banged together, they broke apart into pieces the size of sugar cubes. This would clearly affect the ultimate strength of the alluvium, and samples taken very carefully in a triple tube core barrel were tested in a triaxial apparatus made in an Ovalle workshops. A series of 12 drained tests showed that the values of the angle of shearing resistance in terms of effective stresses, ϕ', varied with the confining pressure, σ_3. With σ_3 = 245 kN/m^2, ϕ' = 56°, and with σ_3 = 981 kN/m^2, ϕ' = 38°. An average from the 12 tests gave ϕ' = 45.8° and the material was considered to be adequately strong for the low stresses that would be imposed by the embankment dam. The site would clearly be unsuitable for a concrete dam.

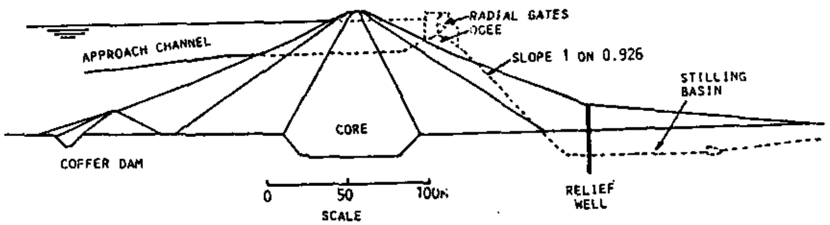


Fig. 3. Section of Paloma embankment dam with the outline of the spillway in the left abutment shown dotted.

The spillway had to be able to pass 6 500 m^3/s, so the design was for a gated structure, with 8 gates each 12.5 m wide and 6 m deep, to be built into the granodiorite left abutment. Model tests were made at the Hydraulics

Laboratory of the University of Chile, with particular emphasis on the shape of the ogee under the gates, to ensure best flow during all stages of discharge. Fig.3 gives a cross section of the dam, with the spillway and chute superimposed, to show its position in relation to the adjoining embankment, and Fig.4 is a photograph of the whole downstream face of the dam and spillway.

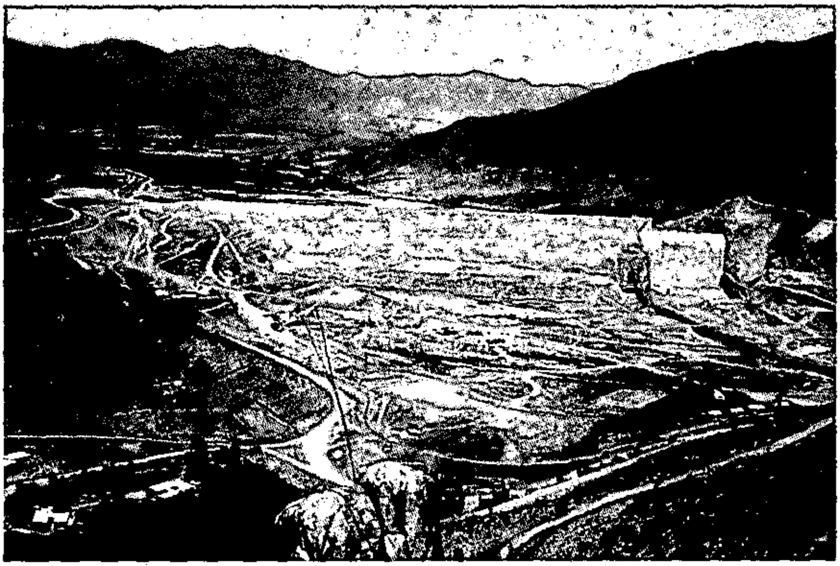


Fig. 4. Photograph of the whole downstream face of Paloma dam and spillway.

Difficulties Encountered.

The Rio Grande valley carried the main road and railway to Ovalle as well as the river, and a canal from the Cogoti reservoir running along the contours on the left side, that was 35 m above the crest of the dam as it crossed the dam centre-line. The railway was to be moved east into an adjoining valley and extensive tunnelling was required to form the new route. This work had not been completed when the contractor moved on to the dam site to begin construction. He was requested to make a temporary diversion for the railway on the right side, and if possible, take it over crest level. Unfortunately a tributary into the Rio Grande, crossed by a railway bridge, limited the distance upstream for the beginning of the diversion. Using the maximum gradient that the railway could tolerate, that could possibly be negotiated by the express passenger and heavily laden goods trains that constituted the heavy traffic on the route, the contractor excavated a new line for the track, slanting up the valley side, but having to cross the centre-line for the dam some distance below crest level. It was hoped that by the time construction neared completion, the new permanent route would be open. But this was prevented by a rock fall in one of the tunnels before there had been time to fit the concrete lining, causing extensive remedial works that took some time to complete. Meanwhile the dam construction reached full level, and a gap had to be left for the railway to pass through. Careful inspection of Fig.4 shows the dip in crest level on the right side (left side of the photograph), and Fig.5 shows a passenger train passing through the gap.

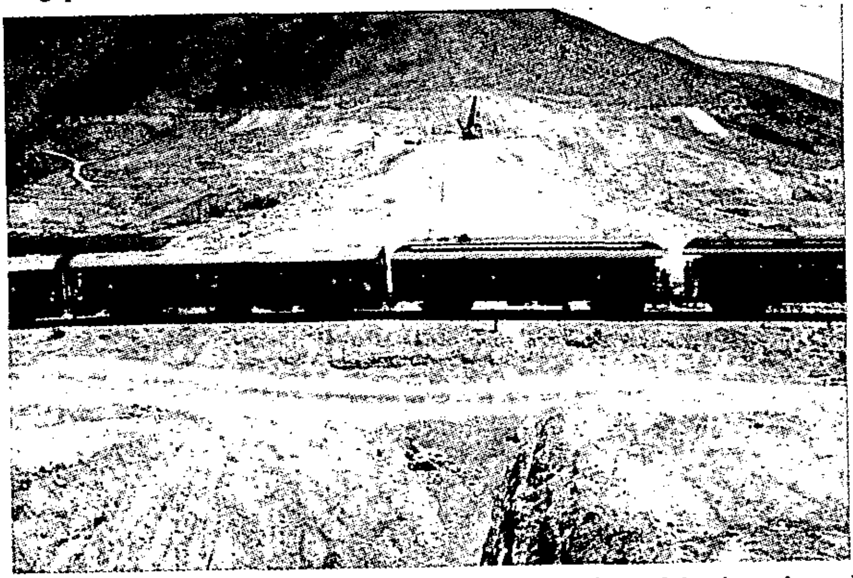


Fig. 5. A passenger train passing across the centre-line of the dam, through the gap on the left side. The crane is standing on the level part of the crest that is level with the camera.

During the model tests to study the shape of the ogee under the gates, a measurement had been made of the angle formed by the water freely discharging from the downstream edge of the model. It was found to be 47.2° to the horizontal, and it was suggested that the spillway chute should not be made steeper than that. Somehow this became understood to mean that the chute should be at an angle of 47.2° (1 on 0.926) to ensure free discharge from the gates. Since the spillway was to be excavated in the rock of the left abutment, adjacent to the dam, which had a downstream slope of 1 on 2, it was considered necessary to move the chute downstream as far as possible, so a complex approach channel and gate structure was designed to enable the top of the chute to be 70 m downstream of the dam centre-line. The excavated rock face forms the left side of the approach channel, but the right side required a mass concrete gravity wall, 32 m high, downstream of the dam axis, tied in to the impervious core to prevent escape of reservoir water when at normal top water level and against the gates. At normal T.W.L. there is a head of 21 m above the channel floor, acting on this concrete gravity `dam', and 23.5 m at flood level.

The granodiorite forming the left abutment contained three suites of joints, one running upstream-downstream almost vertically, one dipping outwards, towards the river at an angle of 20° to 25° to the horizontal, generally parallel to the topography, and a third suite with a downstream component of up to 10°. Excavation for the approach channel and chute would clearly intercept the Cogoti canal, so a tunnel was constructed for it, far enough back into the hillside so that the designed excavated steep slope above the channel would not affect it. Unfortunately the slope could not be excavated to the design angle, so another tunnel had to be built, even further back into the hillside.

The excavation for the chute at a slope of 1 on 0.926 to a depth of 65 m below the approach channel floor, was a massive operation, providing a lot of rock for the dam shoulders, and taking the chute well back into completely unweathered rock, so that it was not considered necessary for it to be lined with concrete. In 1980, when the spillway was discharging only 300 m^3/s, it was found that the rock was eroding in such a way that fear of the ogee overturning, resulted in a recommendation to place a concrete lining. It was used by Schnitter (1993) as an example of the unfavourable effects of cavitation erosion under moderate but unfavourable conditions, illustrated by a photograph taken from a postcard in 1981. During the mid 1980s, a concrete lining was placed to protect the rock of both the chute and the stilling basin.

MOROCCO.

Of the many schemes for improving the lot of mankind by providing water for irrigation, industry and the general supply for towns in Morocco, only one small project will be mentioned. It is the provision of a reservoir on the river Mikkès to supply water for drinking and for industry to the town of Meknes, and irrigation in the Mikkès valley.

The site for the dam to form the required reservoir is about 32 km from the town. The reservoir will be supplied from a catchment area of 864 km^2 over which the average annual rainfall is 580 mm. The spillway will have to cater for a 10 000 year flood of 2000 m^3/s, although during the period of dam construction, a 50 year flood of 605 m^3/s would seem a reasonable allowance for the diversion works. To obtain the required storage volume for the reservoir of $170x10^6$ m^3, the dam is to be 51 m high above the valley floor.

The bedrock underlying the dam and much of the reservoir is Miocene marl, and through thrust faulting, some Triassic marls, of a distinctive red colouring containing rock salt deposits, outcrop in the valley of a tributary that will feed the reservoir. The watertight nature of the marls will prevent leakage from the reservoir. There is some alluvium in the bottoms of the valleys. At the dam site, this consist of gravel overlying the marl, with sand above the gravel, and the area generally is covered with a normally consolidated silty clay. The site had been chosen where the valley is most constricted, with a flood plain about 150 m wide and abutments of the weathered marl, rising at 1 on 2.4 on both sides, flattening to 1 on 4.7 on the right abutment.

The dam had been designed to be of homogeneous earthfill of the Brazilian section, ie. it has a vertical filter and wall drain rising above the downstream edge of the cut-off, from a drainage blanket lying under the whole of the downstream shoulder, to ensure that reservoir water cannot raise the position of the phreatic surface in the downstream shoulder. The downstream slope will be 1 on 3.5, with a slightly flatter slope of 1 on 4 upstream to allow for drawdown conditions. The dam will be built of, and stand on the silty clay. Laboratory tests on samples of the silty clay have shown values of $\phi' = 33°$ to 28°: a lower bound value of 26° was used for design purposes. A 20 m wide cut-off trench has been taken right through the alluvium layers and is well keyed into the fairly impervious marl bedrock. A locally quarried crystalline limestone will be used for riprap for protection on the upstream slopes. Dam construction is currently progressing.

Difficulties encountered.

The water table in the gravel and sand alluvium is more or less at river level and in order to safely de-water the alluvium under the dam site for the construction of the wide cut off trench, it was necessary to make slurry trench vertical cut-offs, both upstream and downstream of the main open trench excavation. Construction of these vertical cut-offs was by a well established method and no serious difficulties were experienced.

The approach channel and spillway chute require an excavation into the right abutment on the upper, flatter slope. To minimise risk of creep movements that might affect the village that lies further up the valley side, the back slopes of the excavations will be additionally stabilised by rock anchors taken deep into the unweathered marl. Triassic salt outcrops could cause undesirable salinity of the reservoir water, and it is proposed to trim and seal the outcrops, particularly those that will be below top water level, with layers of compacted clay at least 1.5 m thick.

Despite these minor difficulties, the whole scheme will be of great benefit to the people of the region, particularly those resident in the town of Meknes.

REFERENCES

Arrau L, Ibarra I & Noguera G. (1985). Performance of Cogoti dam under seismic loading. Concrete Face Rockfill Dams, Symp. Proc.,Detroit. ASCE. pp 1-14.

Schnitter N.J. (1993). Dam failures due to overtopping. Proc. Int. Workshop Dam Safety Evaluation, Grindelwald, vol 1, pp 13-19.

Design, Construction and Environmental Aspects Kau Sai Chau Dam, Hong Kong

D GALLACHER, Senior Partner, R H Cuthbertson and Partners, UK
J D RIDLEY, Senior Environmental Consultant, Axis Environmental Consultants, Hong Kong

---

SYNOPSIS
Kau Sai Chau dam was constructed as part of a public golf centre on Kau Sai Chau Island in Hong Kong forming a reservoir to supply irrigation needs for the golf course. The dam is located within the Kwat Tau Tam tidal inlet on the north east side of the island. Baseline environmental studies identified the Kwat Tau Tam inlet as an important and diverse estuarine habitat due to the presence of mangrove communities on the tidal mudflats in the area of the proposed dam and reservoir. The dam was relocated further upstream within the inlet from its original location to minimise the overall loss of mangrove communities with the dam and the appurtenant works being re-designed to maintain the required storage volume and to compensate for the loss in direct catchment area. The dam construction programme required close consultation and co-operation between engineers, environmental scientists, and experts in mangrove biology. Construction commenced in October 1994 and was completed in October 1995.

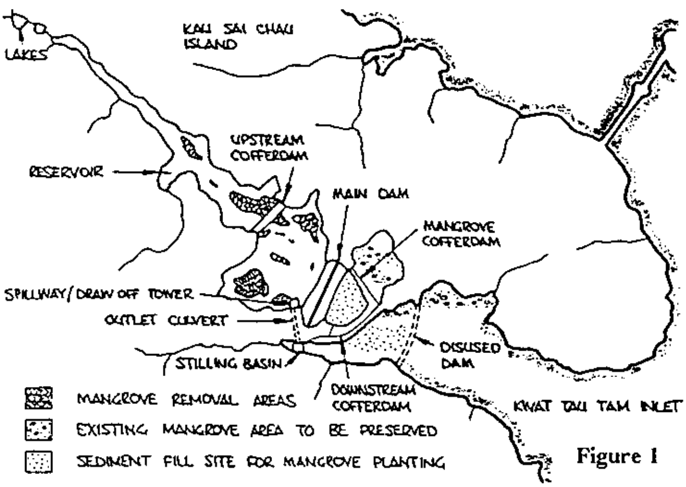


Figure 1

1 INTRODUCTION

Kau Sai Chau Dam and Reservoir were constructed as part of the development of a public golf centre by the Royal Hong Kong Jockey Club (RHKJC) on Kau Sai Chau Island off Sai Kung Peninsula in the New Territories, Hong Kong. The function of the reservoir is to supply the irrigation needs for the golf centre.

Planning, project management and construction supervision of the project were carried out by the RHKJC with the support of international consultants in golf course design and construction, dam design and construction, civil and structural engineering, and environmental science.

The dam is constructed in the Kwat Tau Tam coastal inlet on the northern side of the island (Figure 1), and the reservoir formed by the dam has a capacity of 440,000 m^3 capacity with a surface area of 7.5 hectares. The dam comprises an embankment of 22 m total height at the deepest point of the foundation bedrock and a crest length of approximately 180 m. The main section of the dam is constructed of rockfill with a central reinforced concrete core wall. Fissure grouting of the dam foundations was carried out to depths of up to 15 m into the bedrock to reduce seepage losses and to prevent intrusion of saline groundwater.

Temporary cofferdams were constructed both upstream and downstream of the dam and provision was made for stream course diversion by a large diameter pipeline from the small reservoir formed by the upstream cofferdam feeding into a reinforced concrete culvert through the Right abutment. The culvert is incorporated in the design of the overflow works which include a vertical overflow shaft spillway within the reservoir. The draw-off works which comprise a pump well and a pump house are located adjacent to the spillway with a connecting reinforced concrete bridge to the reservoir shore at the Right abutment.

From its inception the project was subject to rigorous control, particularly environmental, by Hong Kong Government departments including:

Environmental Protection Department (EPD)
Agriculture and Fisheries Department (AFD)
Architectural Services Department (ASD)
Geotechnical Engineering Office (GEO)
Water Supply Department (WSD)
Secretariat for Recreation and Culture (SRC)

Non-government environmental organisations including Friends of the Earth (FOE) and World Wide Fund for Nature (WWF) were also invited to contribute to the conceptual planning of the project.

2 ENVIRONMENTAL ASPECTS

An Environmental Impact Assessment (EIA) for the overall development was carried out at the conceptual design stage as part of the planning process. Detailed design for the dam and reservoir and other engineering aspects were not finalised at the above stage. It was, therefore, possible to feed back the results of the EIA process into the design as part of an iterative design/assessment/redesign process and, in this way, environmental design criteria for the dam and reservoir were incorporated as an integral part of the overall design.

Mangrove habitats ("mangals") in the area of the dam and reservoir were a key environmental issue of the project. The Government also has a strong commitment to preservation of mangals and, therefore, a special mangrove planting and maintenance programme was initiated which included creation of new mangrove substrate areas in the coastal inlet downstream of the dam.

A formal objection to the project was lodged with the Government by FOE, under the Hong Kong Sea Beds and Foreshore Ordinance. This ordinance requires any works planned for sea fronts to be gazetted, allowing for affected parties to register objections subject to review and ruling by the Executive Council (EXCO) of the Hong Kong Government. The FOE objection effectively prevented any construction work being commenced on the dam, pending a ruling by EXCO on the objection, and it threatened the viability of the whole project. EXCO overruled the objection and endorsed the construction of the dam following a review of the Environmental Impact Assessment and the proposed environmental mitigation works. The objection resulted in a two month delay to the dam construction programme which proved critical to the subsequent programme for full impoundment of the reservoir during the 1995 wet season.

Kau Sai Chau Island was previously undeveloped apart from some areas of diverse but poorly developed scrub/woodland and minor wetland areas in the coastal gullies. The existing vegetation within the project area was a sparse cover of grass and low scrub and this vegetation had been limited by regular occurrence of bush fires and the effects of explosives associated with the use of the area for artillery practice.

The project is designed to compliment the existing landscape and utilise natural features as far as possible. Existing streams and wetland areas are maintained wherever possible and integrated with the golf course drainage system to direct surface runoff from the golf course to the reservoir. Earthworks for the golf course, the construction of the main dam, ancillary cofferdams etc., are designed on a balanced cut and fill approach to avoid the need to import fill materials to the island or dispose of excavated materials outside the project area.

The design concept for the dam and reservoir is to provide a partly self sustaining system to cater for the irrigation of the golf course, the requirements of which are estimated to be about 360,000 m^3 per annum. The irrigation and drainage system, together with the golf course lakes, reservoir, and associated catchments are designed as a closed water management system to optimise re-cycling of irrigation runoff within the project area. Treated wastewater and sewage effluent from the clubhouse and maintenance areas discharge into the drainage system feeding the reservoir.

3 PLANNING AND DESIGN

3.1 Reservoir Site Selection

The only practical location for a reservoir of the size required based on engineering and hydrological studies was in the Kwat Tau Tam inlet which is a steep sided narrow tidal estuary on the northern side of the island. The dam site was initially selected to be short distance upstream of an old low dam which had been breached. This dam site provided a reservoir of 420,000 cubic metres active storage and 150,000 cubic metres of dead storage, with a reservoir full supply level of +9.0 mPD requiring a dam of about 11 m height above the silt bed level of the inlet. The maximum depth of silt below bed level in this area was about 10 m.

Baseline environmental studies identified the Kwat Tau Tam inlet as an important and diverse estuarine habitat due to the presence of mangals on silt deposit upstream of the old dam. The draft EIA recommended relocation of the dam further upstream, to minimise overall loss of mangroves and to retain an ecologically important mangal area just upstream of the original location proposed for the dam. The EIA also recommended protection of the retained mangal areas during dam construction and establishment of new mangals to compensate for losses.

The dam site was relocated about 150 m upstream to comply with the EIA and AFD recommendations and provision was made for additional mangrove planting areas in the tidal inlet downstream of the dam.

3.2 Works Programme

Hong Kong is subject to a wet season normally extending from late May to early October, with major rainfalls generally in the three month period from mid-June to mid-September. The original construction programme was to complete the dam and reservoir before June 1995, to impound during the 1995 wet season, and to complete the overall works by December 1995.

There was a delay of about two months in commencement of dam construction due to an objection lodged by FOE, and its review by EXCO. This meant that completion of dam construction to the original design was unlikely before the start of the wet season in June 1995. Consequently, as described below, revisions were made to the design of the dam and construction programme to facilitate later stages of embankment construction under wet season conditions and to permit commencement of impoundment before full completion of dam construction.

3.3 Dam and Reservoir

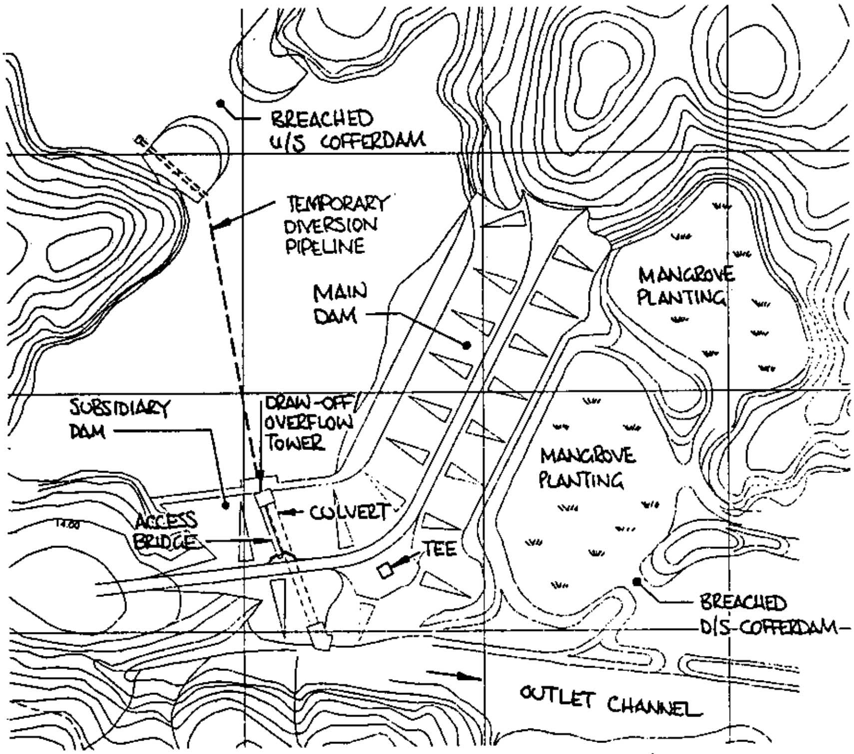
Design changes were made to the dam and appurtenant works resulting from the relocation of the dam site. They included raising the reservoir full supply level by 3 m to +12.0 mPD to maintain an equivalent storage capacity and relocation of the spillway and draw-off works from the left to the right dam abutment. The revised layout plan and typical cross-sections of the dam and appurtenant works are shown on Figure 2.

The zoning of the upper part of the embankment above silt bed level was also changed from earthfill to rockfill to facilitate embankment construction under wet season conditions if required, and to permit impoundment before full completion of dam construction. The rockfill construction to the foundation of the main dam replacing the estuarine silt above bedrock was maintained. The change from earthfill to rockfill to the upper part allowed steeper slopes to be adopted for both the upstream and downstream faces of the dam, and made available more "soft" fill material for landscaping of golf course works. Provision for early impoundment also required modifications to be made to the draw-off/overflow works.

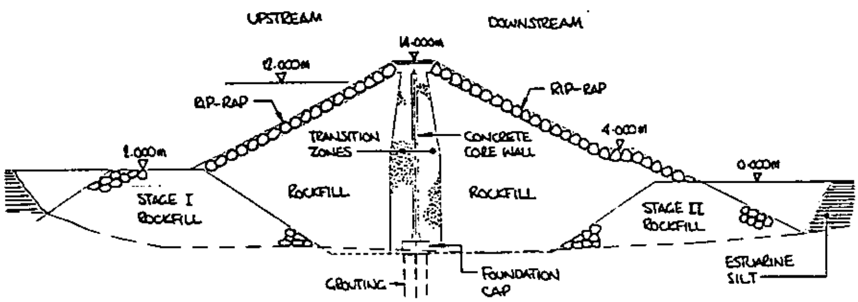
Results of the laboratory tests on possible core fill material available from other excavations for the project indicated that after compaction there would still remain a significant volume of air voids within the fill. A reinforced concrete core wall was, therefore, selected as the impermeable membrane within the main dam extending from a concrete grout cap set into bedrock to just below crest level. The core wall was designed as an elastic beam on an elastic foundation, and a minimum thickness of 600 mm was adopted to minimise stresses and potential cracking with vertical joints at 7.5 m centres.

A clay core was adopted for the short section of dam necessary to infill the open cut excavation above the diversion/overflow culvert. It was also intended to have a reinforced concrete core wall in the latter area but this was deleted on account of ground conditions encountered during excavation for the diversion culvert.

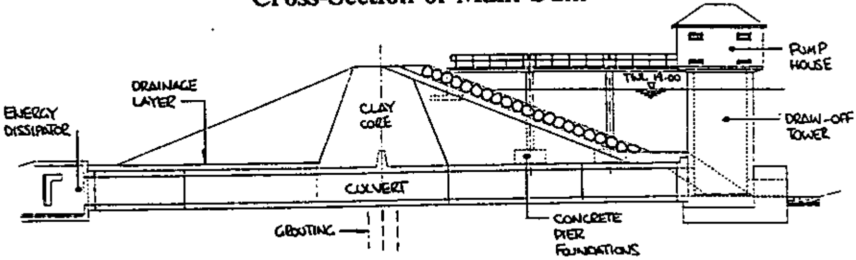
Load cases considered for dam design were end of construction, steady state conditions, rapid drawdown, and seismic loading. Permanent deformation of the dam from earthquake loading was also considered using the method developed by Sarma (1979), where the response of the dam to shaking in the underlying rock foundation is assessed from the elastic fundamental period of the dam and generalised summary curves of measured earthquakes.



Layout of Dam and Appurtenant Works



Cross-Section of Main Dam



Longitudinal Section Draw-off/Overflow Works

Figure 2

3.4 Diversion and Overflow Works

The original diversion works comprised the construction of upstream and downstream cofferdams with twin 1.5 m temporary pipelines connecting the outlet works at the upstream cofferdam to the diversion/overflow culvert. Provision was made in the design of the outlet works to the upstream cofferdam to allow for temporary impoundment for irrigation during construction of the main dam. Revisions were made to the upstream cofferdam including outlet arrangements at the request of the Contractor during the construction phase to reduce costs. The diversion works were designed to pass the routed outflow from a 1 in 10 year return period flood on the basis of a construction period of 1 year and a 10% chance of being exceeded during that period.

The reservoir presents a very low risk to the safety of persons downstream of the dam as, in the event of a breach of the dam, discharge would be made directly into the Kwat Tau Tam inlet with negligible foreseeable loss of life. The dam and reservoir is required to sustain the overall golf course development and, therefore, the overflow works have been designed to pass the routed outflow from a 0.5 Probable Maximum Flood (PMF) with provision for concurrent wave surcharge and some additional freeboard. The estimated still water level of the routed PMF is about 0.4 m below the minimum crest level of the embankment.

4 CONSTRUCTION

4.1 Cofferdams and Ancillary Embankments

Construction of the dam in the tidal estuary required excavation and removal of deep deposits of soft saturated alluvial and marine silt over the foundation area of the dam. There was also a requirement to maintain a tidal flow into a mangrove area designated for preservation which lay just downstream of the dam at the left abutment and upstream of the downstream cofferdam.

Temporary cofferdams were constructed upstream and downstream of the dam site, together with connecting diversion works to allow the dam foundation works to be constructed in the dry. The upstream cofferdam and diversion works were also designed to provide a temporary reservoir of fresh water for irrigation during construction of the golf course. The downstream cofferdam height provided protection for a height of 1.2 m above normal high tide level and 0.7 m above the 10 year maximum high tide level.

The upstream and downstream cofferdams were 6 m high and 85 m long, and 3 m high and 140 m long respectively, and both were constructed directly on the soft estuarine silt deposits of depths of about 7 and 9 m deep respectively. The upstream cofferdam was raised in height by 1 m during construction of the dam to allow one of the two diversion pipelines to be omitted by providing additional hydraulic head for discharge of the design construction flood.

Both cofferdams comprised compacted earthfill embankments and, as it was not possible to use heavy construction equipment directly on the soft silt, a layer of woven geofabric was placed at the foundation contact for purposes of material separation and load distribution. Additional interbedded geogrid reinforcement was also provided to the lowermost fill layers of both embankments to enable subsequent trafficking by construction equipment and to improve overall embankment stability. Both cofferdams were provided with temporary central steel sheet pile cut-offs penetrating through the embankment and the underlying silt deposits to in-situ weathered bedrock to minimise seepage through the embankments and the erodible foundation materials.

A supplementary cofferdam was constructed as an extension to the downstream cofferdam to close off a subsidiary area containing mangroves downstream of the left abutment of the main dam. Cyclic tidal flushing flows to this area were maintained during construction of the main dam by a buried steel pipe passing through the embankment at its junction with the downstream cofferdam. This cofferdam comprised a 3 m high x 90 m long embankment similar in construction to the two main cofferdams but without a sheet pile cut-off.

A sediment settlement/retention pond was also formed on the seaward side of the downstream cofferdam by the construction of a semi-pervious dumped rockfill embankment founded on the old disused dam. This rockfill embankment sustained a through flow of water in both directions under tidal fluctuations, and it acted as a filter to run-off from the construction site to assist with meeting EPD's requirements with regard to pollution.

4.2 Foundation Excavation

Foundation excavation for the dam required all overburden and soft materials to be removed to competent in-situ bedrock over the full area of the foundation. This entailed excavation of about 70,000 cubic metres of material ranging from a few metres depth of residual soil and in-situ weathered rock on the abutments, up to 9 m depth of marine silt deposits in the central estuary area.

Disposal of the excavated marine silt within the main project area presented difficulties as it was not suitable for golf course landfill due to its soft, saturated, and highly saline nature. However, this silt proved to be an excellent substrate material for mangroves, and the major proportion was used for compensatory mangrove planting areas. Residual soil and weathered rock materials from the dam foundation excavation were used for golf course landfills under the programme of balanced cut and fill.

Stabilisation of the dam foundation excavation below the estuary silt bed levels local to the cofferdams was achieved by progressive dewatering and placement of rockfill concurrent with excavation. This rockfill was subsequently incorporated in the outer flanks of the lower buried portion of the dam embankment. The above approach enabled the central part of the foundation area to be dewatered and cleaned to sound bedrock for construction of the mass concrete foundation cut-off, foundation grouting, and starter bays for the central concrete core wall.

A mass concrete cut-off plug about 3.2 m wide and a minimum depth of 1.5 m into bedrock and with vertical joints containing waterbars at 10 m centres was formed to provide a platform for grouting of the bedrock and a foundation for the reinforced concrete core wall. It was necessary on the left abutment of the dam to deepen the plug to take account of bedrock conditions. Early difficulties in providing a satisfactory joint detail and surface for the movement layer between the plug and the core wall on the sloping abutments were overcome after the initial two pours. A continuous waterbar was provided between the foundation plug and the core wall.

Two lines of blanket grout holes were drilled to a depth of about 7.5 to 9.5 m below the top of the grout cap over the main section of the dam at about 4 m centres. The largest grout takes were on the left shoulder where it has been necessary to deepen the cut-off plug and there were also significant grout takes at two other areas on the right shoulder. The smallest grout takes were in the central section of the dam.

Curtain grouting was carried out from a single line of grout holes between the two lines of blanket grout holes. Primary, secondary and tertiary holes were drilled and grouted over the full width of the main section of the dam. The final spacing of the holes was about 3 m with depths of about 16 and 11 m in the central area and abutments of the main dam. Grouting was also carried out on the dam centreline around the diversion/overflow culvert. The largest grout takes were again found on the left shoulder of the dam and the maximum was about 86 kg/m in an area where the dam is founded on a narrow rib of rock. There was also a significant grout take on the right abutment where the corresponding amount of blanket grouting was higher than average. Vibrating wire piezometers are installed in the bedrock across three sections of the dam to monitor cut-off performance.

4.3 Embankment Construction

Concrete core wall construction and placement of embankment rockfill were carried out concurrently to maintain support to the core wall which has a movement joint at its junction with the cut-off plug. The 0.6 m thick core wall was constructed in horizontal lifts with a typical height of 4 m, each lift comprising a series of 10 m wide bays formed with partial contraction joints containing central waterbar and polysulphide surface sealants. The embankment rockfill was constructed in 0.6 m layers, each layer being compacted with a minimum of six passes of a 10 tonne vibrating flat roller.

A transition zone of smaller rock was provided adjacent to both sides of the core wall.

Larger rock from the embankment fill operation was placed towards the upstream and downstream slopes to provide surface protection. Additional riprap was placed on the upstream face of the dam and this was extended along the shore at both ends of the dam from the reservoir floor to above top water level, with the shore slope being flattened as required to provide stability. The riprap protection to the upstream face at the right abutment extended from the main dam to beyond the infill embankment above the diversion/overflow culvert.

The core wall lifts and embankment construction were planned such that initial impoundment of the reservoir could be achieved when the core wall reached a level of +6.5 mPD with a corresponding embankment level of +6.0 mPD. Protection against overtopping of the partially completed dam was provided by modifying the diversion pipe inlet chamber at the outlet culvert to act as a temporary overflow spillway to maintain the temporary reservoir level at +4.5 mPD, allowing a 1.5 m freeboard for discharge of an increased design diversion flood (1 in 50 year return period). Completion of the dam to initial impoundment level was achieved in late June 1995.

Planting was introduced on the downstream shoulder of the dam in about six areas, using native species of ferns and shrubs which are indigenous to the island, to provide a more natural appearance to merge with the adjacent ground. The downstream cofferdam was also lowered to 3 mPD and breached to allow tidal inflow to new areas planted with mangroves. The piped inlet to the downstream mangrove area at the left abutment has also been replaced by an open channel for tidal inflows. The above provisions have satisfied the ASD and the EPD with regard to visual impact of the dam and compensatory mangrove planting.

The dam was completed to the final embankment crest level of +14.0 mPD in September 1995 and full impoundment commenced in October 1995. The reservoir will not likely fill until the next wet season which commences in June 1996.

5 ENVIRONMENTAL MONITORING

Environmental monitoring and audit schedules recommended in the EIA were implemented in the construction phase and subject to fortnightly inspections by the EPD.

Surface water on the golf course will be monitored to record the potential impacts of chemical applications in the post construction operational phase. Samples will also be taken on a monthly basis from surface streams, drainage catch pits, lakes, the reservoir, and the marine environment and a range of parameters will be tested including pH, dissolved oxygen, chlorophyll a, nitrate, orthophosphate and conductivity. The resulting information will be interpreted together with soil test data to adjust fertilizer applications where appropriate, and to mitigate the potential for excessive algal growth in the reservoir.

Monitoring of mangrove survival and growth will be conducted in fixed plots randomly located across the created mudflats for three years following dam completion. Additional planting will be undertaken as needed to replace seedlings which fail to establish. Monitoring of the planting on the downstream face of the dam will take place on a quarterly basis for three years following completion of the dam.

Wildlife use of the dam and reservoir will be monitored quarterly for three years following dam completion, as will bird use of the reservoir to determine the extent to which the new habitat provided by the reservoir contributes to biodiversity on the island.

6 CONCLUSIONS

The stringent environmental criteria imposed by the Hong Kong Government and to overcome the objections of the non-government lobby groups were overcome by the adoption of an interactive approach to environmental aspects in planning, design, and construction of the dam and appurtenant works.

Design and construction of the dam were carried out in close consultation and cooperation between the dam engineers, environmental scientists, project management, and government departments. This has resulted in the successful creation of a large freshwater reservoir, an environmentally integrated dam, and an area of newly created wetland habitat, which have preserved the ecological value of the tidal inlet in the dam area, as well as enhancing the ecological environment of the island as a whole.

REFERENCES

AXIS Environmental Consultants Ltd, Kau Sai Chau Development, Environmental Impact Assessment, Final Report, March 1994.

Environmental Aspects of Dam Construction, Kau Sai Chau Dam, Hong Kong, P N Seyler, J D Ridley and T D Dahmer (MWA International Conference on Dam Engineering - August 1995).

ACKNOWLEDGEMENTS

The authors wish to thank the Royal Hong Kong Jockey Club for permission to prepare this paper.

Walshaw Dean Lower - Embankment behaviour during prolonged reservoir drawdown.

I R HOLTON, P TEDD and J A CHARLES, Building Research Establishment.

SYNOPSIS. After nearly 90 years use as a compensation reservoir, the crest of Walshaw Dean Lower dam is still settling at an average rate of 8mm per year. Detailed observations have confirmed that the stress changes in the central clay core due to reservoir drawdown are similar to those observed in other clay core dams, as is the pattern of embankment deformations. Negligible movements have occurred in the top of the clay filled cut-off trench, although the stress changes here were greater than in the core.

INTRODUCTION.

Walshaw Dean Lower is located 5 miles northwest of Hebden Bridge in West Yorkshire. It is the lowest in a chain of three embankment dams constructed between 1901 and 1907 for Halifax Corporation Waterworks and now owned by Yorkshire Water plc. The engineer was G H Hill. The dam has a maximum height above foundation level of 22m with an upstream slope of 1 in 3 and downstream slope of 1 in 2. The central clay core has a top width of 2.6m and batters of 12 in 1 to foundation level, below which is a 3m wide puddle clay filled cut-off trench which finishes 40m below crest level in a concrete shoe 1.8m deep (Fig.1). The fill close to the core comprises a silty sandy clay with sandstone and mudstone fragments.

The dam has had a long history of problems with leakage and settlement (Barnes, 1927, Charles & Watts, 1987) and it has been estimated that the crest has settled more than a metre since the end of construction. It was considered that this settlement might be associated with internal erosion of the clay in the cut-off trench into the fissured sandstone foundation. Following an investigation in 1980, the cut-off trench and foundation rock immediately downstream were grouted (Charles & Watts, 1987), however, the crest of the dam has continued to settle at a rate of approximately 8mm per year.

Detailed deformation observations have been made at the dam by the Building Research Establishment (BRE) since 1990 as part of its study of the effects of reservoir drawdown on embankment deformations (Tedd et al, 1996). Walshaw Dean is the only dam in the study with a deep puddle clay filled cut-off trench. With the reservoir due to be emptied during 1993-94 whilst an auxiliary spillway was constructed (Harrison & Drabble, 1996), additional instrumentation was installed to gain information on the deformations and pressure changes due to prolonged drawdown. Observations from this most recent study together with those made in earlier years will be presented and discussed.

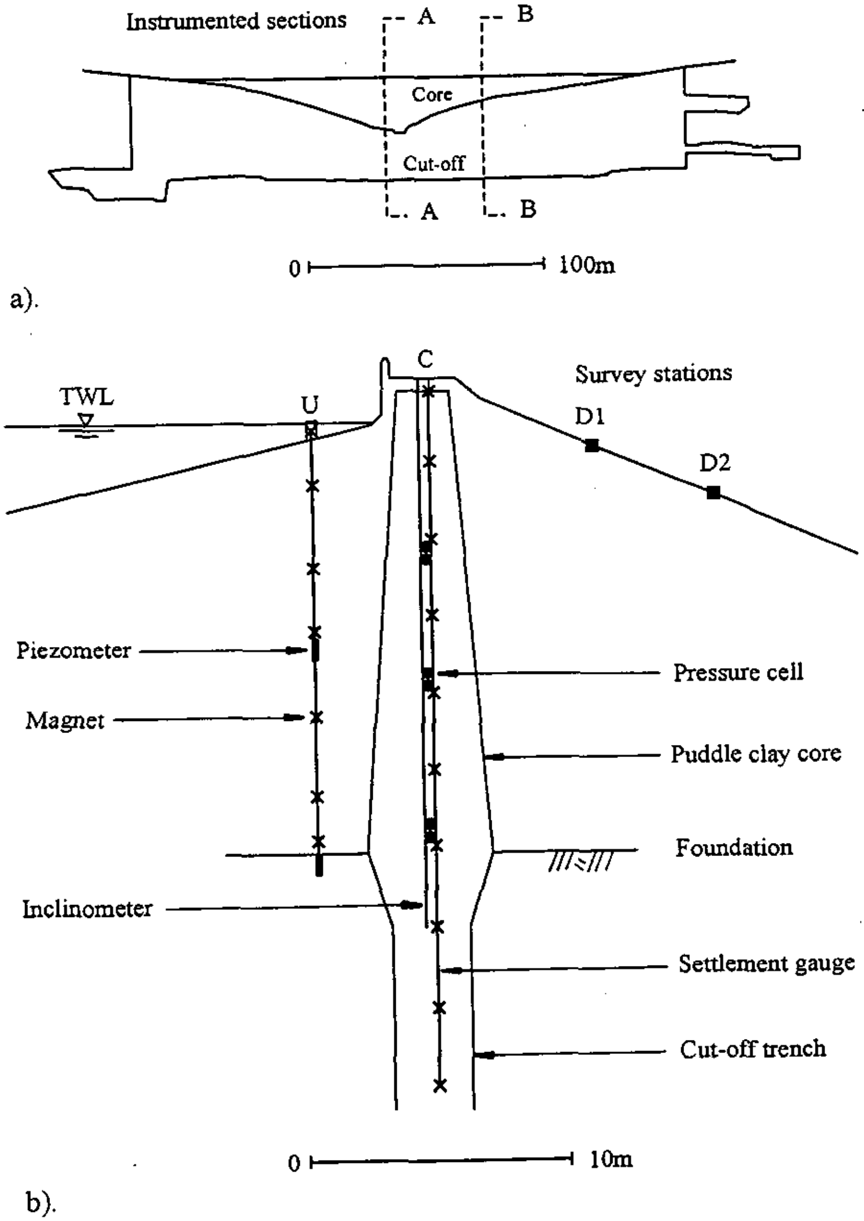


Fig.1. (a). Longitudinal section showing extent of the cut-off trench.
(b). Transverse section A-A showing instrumentation installed.

INSTRUMENTATION.

The dam was instrumented in 1989 to study surface deformations of the embankment and settlements within the clay core and cut-off trench at the two sections shown in Fig.1. In 1992, further instrumentation was installed at section A to measure stress changes in the core and deformations in the upstream fill. Figure 1 shows the instrumentation installed at section A. Magnet settlement gauges have been used to monitor vertical deformations in the core, the upper part of the cut-off trench and the upstream fill close to the core. Horizontal (upstream-downstream) deformation of the core has been measured using an inclinometer. Horizontal and vertical surface deformations have been measured at points on the crest and upstream and downstream shoulders using precise surveying from a fixed pillar remote from the dam; the tops of the settlement gauges correspond to survey points. A push-in earth pressure cell and piezometer, and five BRE miniature pressure cells (Watts & Charles, 1988) were installed to measure horizontal and vertical total stress at three depths in the core. Piezometric levels have been measured in the upstream fill and foundation using standpipe piezometers.

OBSERVATIONS.

Description of the observations will concentrate on section A where the dam core is close to its maximum height. During the period 1990-94 the reservoir underwent four major drawdowns as shown in Fig.2. The first three were typical of the normal operation of the reservoir for compensation supply, being of 6 to 10 months duration. The fourth drawdown was required primarily to allow construction of an additional overflow and at 23 months was of considerably longer duration than the others. Although the reservoir level appears to have fluctuated considerably during this period, the water level was 5m below crest level or lower for over 70% of the time. The reservoir was also effectively empty for two weeks during the drawdown.

Surface movements.

Figure 2 shows crest settlement with time over the complete 5 year period of observations. The maximum (s_m) and permanent (s_p) settlements due to each drawdown are summarised in Table 1.

Crest settlement commenced immediately reservoir drawdown began, the rate of settlement being related to the rate of drawdown. There was a delay however, between commencement of refilling and heave of the crest, particularly on the deepest drawdown, in which further settlement took place. Movement generally stopped once the reservoir had returned to TWL. The net result of each drawdown cycle was permanent settlement of the dam crest. The greater depth and duration of the 1993-94 drawdown resulted in both larger total movements and a higher proportion of permanent settlement than the other drawdowns, although the annual rate of permanent settlement averaged over the five years was still approximately 8mm per year.

Table 1. Summary of drawdowns and corresponding crest settlements.

Reservoir drawdown			Crest settlement (mm)		$\frac{s_p}{s_m}$
Year	Duration (months)	Max depth (m)	Maximum s_m	Permanent s_p	
1990	10	10.5	12	7	58%
1991	9	13.0	14	8	57%
1992	6	11.0	12	8	67%
1993-94	23	16.5	21	16	76%

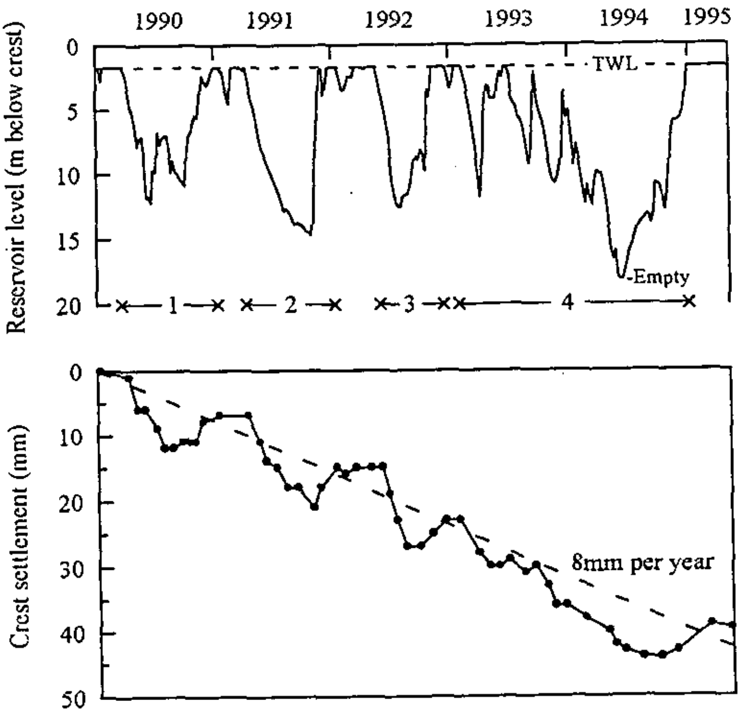


Fig.2. Reservoir level and crest settlement during the five years of observations.

Details of the horizontal and vertical deformations measured on the survey points during the 1993-94 drawdown are shown in Fig.3 as a function of time and reservoir level and as movement vectors on a section of the dam. Due to the long period of observation and the fluctuations in reservoir level, the movement vectors are quite complex and have therefore been simplified by dividing the drawdown into four stages (A-D) and drawing the vectors to represent the movement measured at the end of each stage.

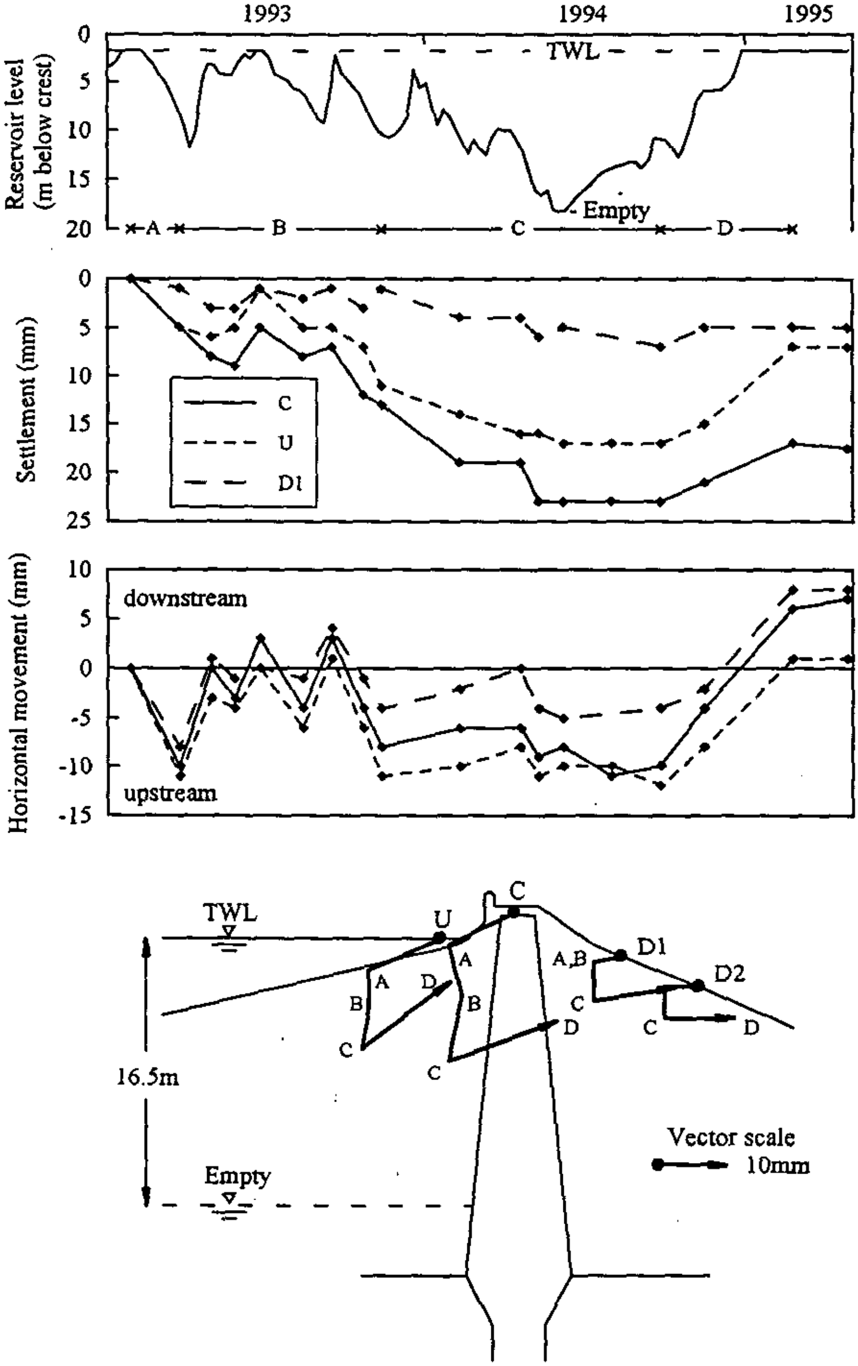


Fig.3. Development of horizontal and vertical surface deformations during the 1993-94 drawdown.

The initial drawdown, stage A, resulted in upstream movement and settlement of the crest and upstream survey points with horizontal movements approximately twice the vertical ones. Movement of the downstream points was smaller and primarily upstream. The overall effect of the fluctuations in reservoir level that occurred during stage B was further settlement of the crest and upstream points but no change in the positions of the downstream ones. During stage C the reservoir was effectively emptied resulting in settlement of all points with limited upstream movement. The greatest settlement occurred on the crest, ie above the clay core. Settlements ceased with commencement of refilling, heave and downstream movement then occurring during stage D when the majority of impounding took place. The overall effect of the drawdown cycle was settlement of the upstream fill, and settlement and downstream movement of the crest and downstream fill. The same pattern of movements was observed during the other drawdown cycles.

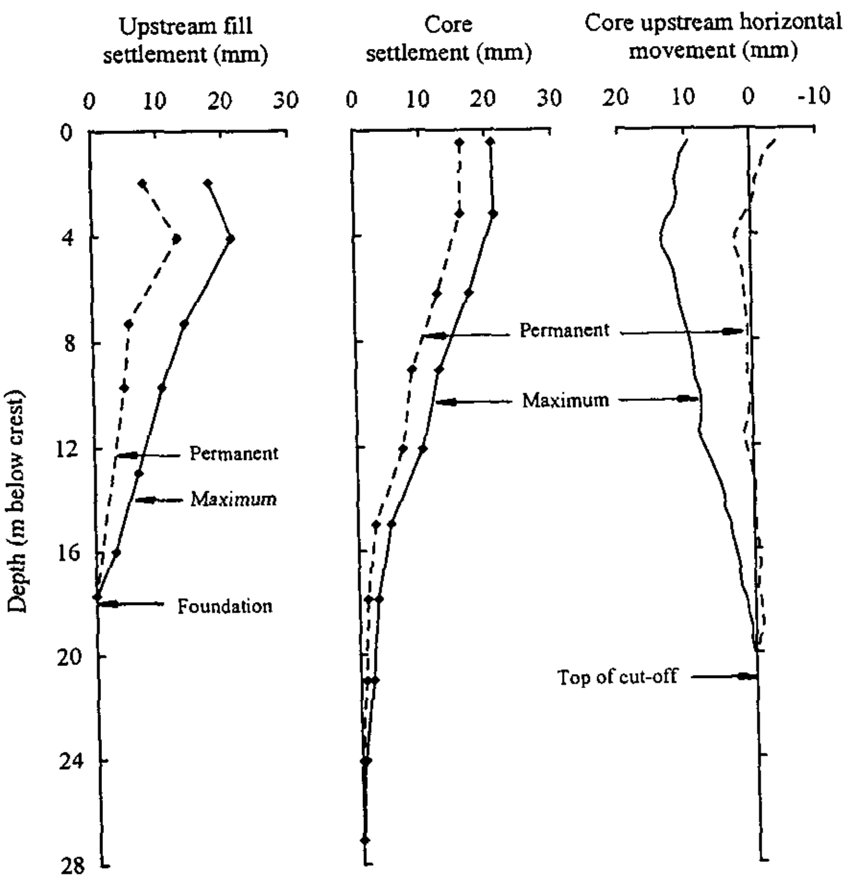


Fig.4. Profiles of maximum and permanent movement with depth in the core and upstream fill due to the 1993-94 drawdown.

Sub-surface movements.

Figure 4 shows profiles of maximum and permanent settlement in the core and upstream fill close to the core and horizontal movement in the core measured during the 1993-94 drawdown. Movements due to reservoir drawdown and refilling occurred throughout the full depth of the upstream fill and core. Negligible movement occurred at the top of the cut-off trench. The largest movements occurred at 4m below crest level in both the core and upstream fill. In the upstream fill this is at the boundary of the fill and pitching. The upstream horizontal movement of the core was recovered on refilling except in the top few metres where there was a net downstream movement. Permanent settlement occurred throughout the core and upstream fill.

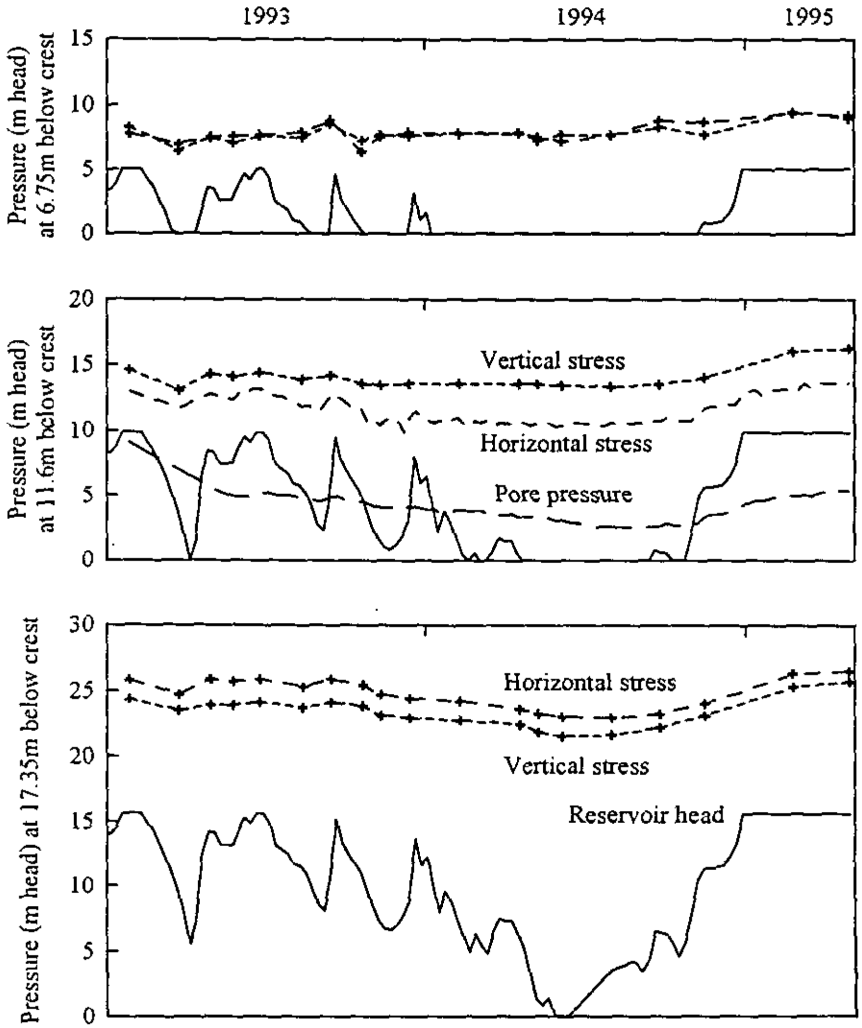


Fig.5. Pressures measured in the clay core during the 1993-94 drawdown.

Stresses in the clay core.
Figure 5 shows the total horizontal and vertical stresses measured in the core on each pair of pressure cells during the 1993-94 drawdown together with the calculated reservoir head at each depth. The pore pressure measured at 11.6m below crest level is also shown.

Total vertical and horizontal stresses were approximately equal at each depth and greater than reservoir head, the difference increasing with depth. Drawdown caused a reduction in the measured total stress on each pressure cell, the amount increasing with depth up to a maximum of 3m at 17.35m depth, but in all cases by much less than the corresponding changes in reservoir head. Similarly refilling caused an increase in total stress. Total stresses on completion of the drawdown cycle were all greater than at the start of the cycle; this may have been due to the pressure cells not having reached equilibrium following installation before the drawdown commenced. Readings on the combined pressure cell and piezometer were taken at weekly intervals; changes in total stress appeared to follow changes in reservoir head with little time delay, whilst the pore pressure showed a more gradual response.

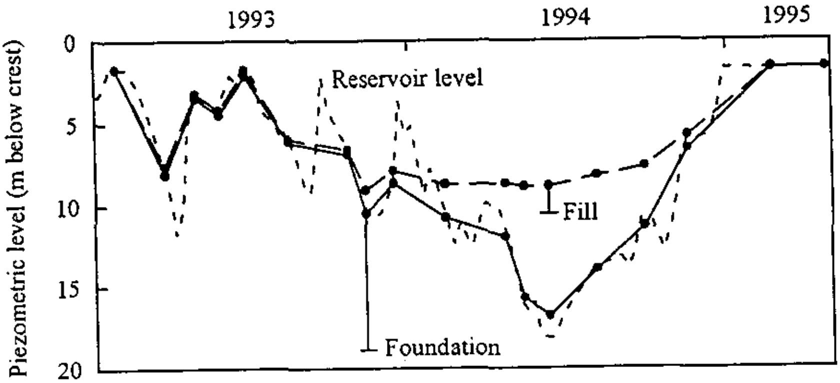


Fig.6. Piezometric levels recorded in the upstream fill and foundation during the 1993-94 drawdown.

Piezometric pressures in the upstream fill.
Figure 6 shows the piezometric levels in the upstream fill and foundation close to the core during the 1993-94 drawdown. Allowing for fluctuations in reservoir level between readings, the piezometric level in the foundation appears to follow reservoir level with little time lag. Permeability tests have shown the foundation to have a permeability in the region of $1x10^{-7}$m/s. The piezometer was however, installed close to the boundary between the foundation and fill, and may be connected to the fill. With the reservoir empty, a piezometric head of 1m is shown in the piezometer. The piezometric level in the fill also follows reservoir level closely until the level drops below 9m below crest level, when

a 1.5m head remains in the piezometer. Permeability tests in the fill have given values in the region of $1x10^{-6}$m/s. Given the clayey nature of the fill, a localised perched water table is therefore likely at this location in the fill.

DISCUSSION.

The pattern of surface and sub-surface deformations observed during reservoir drawdown and refilling at Walshaw Dean Lower was similar to that measured at Ramsden dam (Tedd et al, 1990) and at other dams with central puddle clay cores (Robertshaw & Dyke, 1990). Reservoir drawdown caused settlement and upstream horizontal movement of the dam, whereas refilling caused heave and downstream movement. The net result of each drawdown cycle was settlement and downstream movement. The largest movements were confined to the crest and upstream fill close to the core. Movements on the upstream slope were more recoverable than those on the crest, indicating that the upstream fill was more elastic than the clay core. The mechanisms causing embankment deformations due to changes in reservoir level have been described by Tedd et al (1994).

Only partial recovery of the crest settlement induced by reservoir drawdown occurred despite the history of repeated drawdown cycles during nearly 90 years use as a compensation reservoir. The relatively prolonged drawdown during 1993-94 has had little effect on the average annual rate of settlement of 8mm per year. There does not appear to be any sign of the rate of settlement changing. It seems likely that the annual rate of settlement would only increase if more frequent deep drawdowns were to occur.

The reservoir drawdown induced settlements occurred throughout the full depth of the core, but little settlement was recorded in the cut-off trench. This may be due to arching of the clay across the narrow trench resulting in stress transfer between the clay and the relatively incompressible foundation rock.

Figure 7 shows the total stresses observed with depth on the pressure cells in the core under reservoir full conditions, together with measurements from an earlier investigation in which pressure cells had been installed in the cut-off trench (Charles & Watts, 1987). The pressure cells in the core were installed at a deeper section of the dam to those in the cut-off trench. All the earth pressures measured in the core were significantly larger than reservoir head, whereas those measured in the cut-off trench were lower, indicating that whilst the core is unlikely to be susceptible to hydraulic fracture, the clay in the cut-off trench will be more vulnerable. The lower stressses in the cut-off trench can be attributed to the clay arching across the narrow trench.

The ratio of the change in total stress in the core to change in reservoir head ($\Delta\sigma_h/\gamma_w\Delta h_w$) was approximately 0.3 and is similar to those measured in the cores of other dams (Charles & Watts, 1987). Figure 8 shows the total

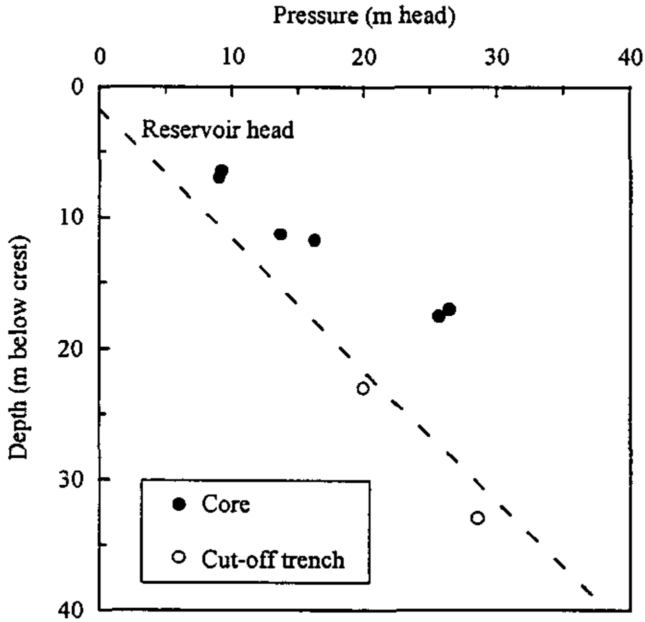


Fig.7. Total earth pressure with depth in the core and cut-off trench under reservoir full conditions.

horizontal stress measured by Charles & Watts (1987) at 33m depth in the cut-off trench for a two year period during which two reservoir drawdowns occurred. These results show that much larger pressure changes occur in the cut-off trench as a result of changes in reservoir level than occur in the core. The ratio $(\Delta\sigma_h/\gamma_w\Delta h_w)$ measured in the cut-off trench during the earlier investigation was between 0.5 and 0.9.

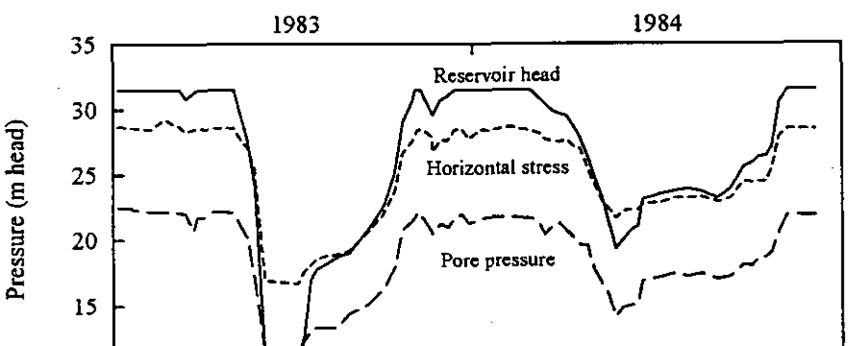


Fig.8. Stresses measured at 33m depth in the cut-off trench over two reservoir drawdowns (Charles & Watts, 1987).

Piezometric pressures in the upstream fill close to the core generally followed the reservoir level down to particular levels, indicating small perched water tables in the fill. Permeability measurements in the fill confirmed it to be sufficiently permeable to allow free drainage to occur at the drawdown rates which have been experienced.

CONCLUSIONS.

The pattern of surface and sub-surface deformations of the dam due to reservoir drawdown and refilling are similar to those observed in other studies on central clay core dams, although the magnitudes differ.

Despite undergoing repeated drawdown cycles over nearly 90 years, crest settlement of approximately 8mm per year still occurs.

The prolonged drawdown does not appear to have had any adverse effect on the embankment in terms of deformations or permanent reductions of earth pressure within the core.

Little settlement was observed in the clay filled cut-off trench, despite large changes in earth pressure.

ACKNOWLEDGEMENTS.

The main client for this work is the Water Directorate of the Department of the Environment. The co-operation of Yorkshire Water is gratefully appreciated.

REFERENCES.

Barnes AA (1927). Cementation of strata below reservoir embankments. Trans. of Institution of Water Engineers, vol 32, pp 42-68.

Charles JA & Watts KS (1987). The measurement and significance of horizontal earth pressures in the puddle clay cores of old earth dams. Proc. of Institution of Civil Engineers, Part 1, vol 82, February, pp 123-152.

Harrison PC & Drabble J (1996). Walshaw Dean Reservoirs - Spillway Improvements. Proc. of British Dam Society Conference.

Robertshaw AC & Dyke TN (1990). The routine monitoring of embankment dam behaviour. Proc. of British Dam Society Conf. on The Embankment Dam, Nottingham, pp 177-183. Thomas Telford, London.

Tedd P, Claydon JR & Charles JA (1990). Deformation of Ramsden dam during reservoir drawdown and refilling. Proc. of British Dam Society Conf. on The Embankment Dam, Nottingham, pp 171-176. Thomas Telford, London.

Tedd P, Charles JA, Holton IR & Robertshaw, AC (1994). Deformation of embankment dams due to changes in reservoir level. Proc. of 13th Int. Conf. Soil Mechanics & Foundation Engineering, New Delhi, vol 3, pp 951-954.

Tedd P, Charles JA, Holton IR & Robertshaw, AC (1996). The effect of reservoir drawdown and long term consolidation on the deformation of old embankment dams. Accepted for publication in Geotechnique.

Watts KS & Charles JA (1988). In situ measurement of vertical and horizontal stress from a vertical borehole. Geotechnique, vol 38, no 4, pp 619-626.

Cow Green Dam: Interpretation of earth pressure measurements 25 years after construction

J F PRENTICE, Northumbrian Water, UK
M FLETCHER, Rofe Kennard and Lapworth, UK
P TEDD, Building Research Establishment, UK

SYNOPSIS. It is generally accepted that long term field observations have an important role to play in assessing the safety of embankment dams. The continuing reduction in earth pressures measured at the interface between the clay core and the concrete of Cow Green dam since the end of construction in 1970 raised concern that hydraulic fracture and ultimately leakage could develop at the interface. The paper illustrates the roles and responsibilities of the statutory undertaker, the Inspecting Engineer and specialist organisations in assessing matters in the interest of safety under The Reservoirs Act 1975.

INTRODUCTION.

Cow Green dam is 570m long with a maximum height of 25 m high and was built on the River Tees just above Cauldron Snout between 1967 and 1970. It is a composite structure consisting of a concrete gravity dam founded on the Whin Sill, a hard dolerite and an embankment dam with a rolled clay core founded on a clay filled channel (Kennard & Reader, 1975, Vaughan et al 1975). The detail of the junction between the embankment and the concrete is shown in Fig. 1. The junction was formed partly by butting the core against the enlarged end face of the concrete section and by extending the core to overlap the upstream face of the concrete dam. Instruments were installed during construction of the dam to monitor earth pressures and porewater pressures at the interface. These have been monitored since construction at frequencies specified by qualified civil engineers appointed under the Reservoirs (Safety Provisions) Act 1930 and the Reservoirs Act 1975. If the risk of hydraulic fracture and possibly leakage at the interface are to be avoided then the total horizontal earth pressure of the clay against the vertical concrete face must remain above the hydrostatic pressure due to reservoir head. Instruments at the interface have shown the measured horizontal earth pressures to be decreasing towards reservoir head.

A statutory inspection under the Reservoirs Act 1975 was carried out in 1990 and included the following recommendation in the interests of safety:

> *"the continuing falls in the readings of the pressure cells at the core/concrete interface are referred to the designers of the dam"*

The reservoir as an asset. Thomas Telford, London, 1996

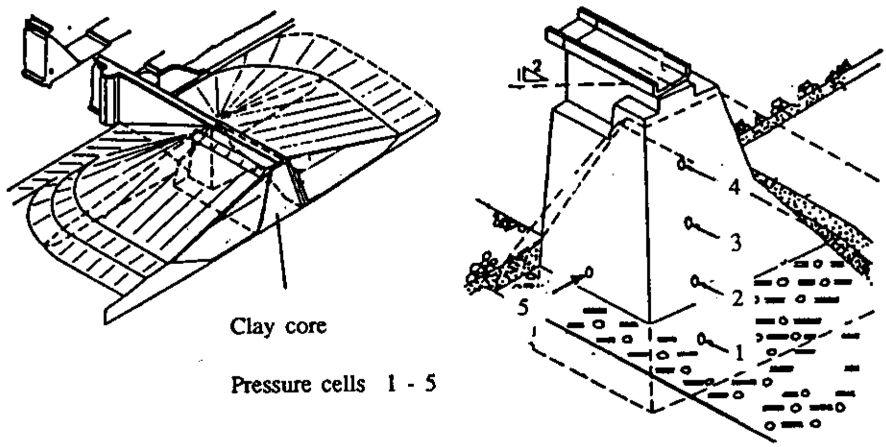


Fig. 1. Arrangement of junction between concrete and earth dams, and location of instruments at the interface of Cow Green dam (after Kennard & Reader,1975)

The designers of the dam were Sandeman, Kennard & Partners. In 1970 the firm amalgamated with Rofe & Raffety and Herbert Lapworth & Partners to form Rofe, Kennard & Lapworth. Both Mr Kennard, Partner responsible for the design and construction of the dam under the Reservoirs (Safety Provisions) Act, 1930 and Professor P R Vaughan, specialist consultant during the design and construction stages, were consulted as part of the assessment of the observations described in this paper. Advice was also sought from the Building Research Establishment where the earth pressure cells had been designed and alternative methods of assessing the risk of hydraulic fracture at the interface were proposed.

Problems with embankment dams are often associated with the interface between the embankment and ancillary structures. The interface between a clay core and a more rigid structure whether it be culvert or abutment, can lead to differential settlement, stress redistribution with often unfavourably low stresses, and ultimately the possibility of hydraulic fracture. Very rarely have the stresses at these junctions been measured. This paper describes the instrumentation, the observations over 25 years and the investigations undertaken to independently assess the risk of hydraulic fracture at the interface under all conditions (including Probable Maximum Flood). The reliability of long term pressure observations are discussed in the context of assessing the safety of a major dam under the Reservoirs Act 1975.

INSTRUMENTATION

The design of the junction, the instrumentation and the observations from the instruments during construction and 14 months after the end of construction were described in detail by Vaughan & Kennard (1972) and Kennard & Reader (1975). Both total earth pressures and pore pressures were measured.

The earth pressures were measured using BRS (Building Research Station, now BRE) type vibrating-wire earth pressure cell (Thomas & Ward, 1969). Details of the installation are shown in Fig. 2. Four cells were placed at the end face and one cell on the upstream face of the concrete dam. The cells are 280mm diameter and record pressure over a central 152mm diameter. In assessing the observations it is important to understand the construction of the pressure cells and the possible mechanisms that could lead to erroneous results. The cell consists of two similar metal discs each recessed to form a diaphragm across which is fixed a vibrating-wire strain gauge. The two halves of a cell prior to being bolted together are shown in Fig. 3. Deflection of the diaphragm due to soil pressure is measured by the vibrating-wire gauges. The cells were mounted in synthetic rubber in a cast iron backing piece to enable readings to be taken on the inner as well as the outer face. Readings have been taken on the gauges fixed to the faces of each cell, P_{CA} (against the clay) and P_{CB} (against the concrete), which have been averaged to give the lateral earth pressure P_C acting on the interface.

The vibrating-wire gauge has long term stability, robustness and freedom from electrical troubles. To avoid corrosion of the components of the gauge dry nitrogen is circulated through the cell. The pressure of the nitrogen applied to the cell is taken into account when calculating the measured pressure. Corrosion of the vibrating-wire will lead to an increase in thickness of the wire and a decrease in frequency and hence an apparent decrease in pressure. When the corrosion products fall off an increase in frequency will occur. Creep of the vibrating wire and at fixing points could also lead to an apparent decrease in pressure.

The hydraulic piezometers consist of 100mm diameter high air entry ceramic discs, set in PVC blocks. The pressures were measured with mercury manometers.

LONG TERM OBSERVATIONS

Earth pressures and porewater pressures increased uniformly with increase in overburden as the dam was raised. All the earth pressure cells performed satisfactorily until the end of construction. Cell No. 5 on the upstream slope ceased to work shortly after the end of construction.

At the end of construction the total earth pressures and the porewater pressures measured at the interface were substantially greater than the maximum reservoir pressure. From the end of construction Vaughan & Kennard (1972) reported that both total pressures and porewater pressures were decreasing but concluded that even if these trends continued, total earth pressures greater than maximum reservoir level should be maintained when porewater pressures reached their likely ultimate value.

Figure 4 shows the fluctuation in reservoir level and the observations of earth pressure and pore pressure at two locations; the top position No 4 (6.7m below crest level), and the lowest position No 1 (21.4m below crest level). Table 1 summarises the earth pressures and porewater pressures shortly after construction in 1971 and in 1992 when investigations were being undertaken. The readings from the two sides of the pressure cell, P_{CA} and P_{CB}, are presented. Both earth pressure and porewater pressure have continued to decrease since the end of construction for 25 years albeit at a gradually decreasing rate. By 1990 there was concern that the earth pressures measured at the interface would continue to decrease and that the pressure would eventually become less than reservoir head.

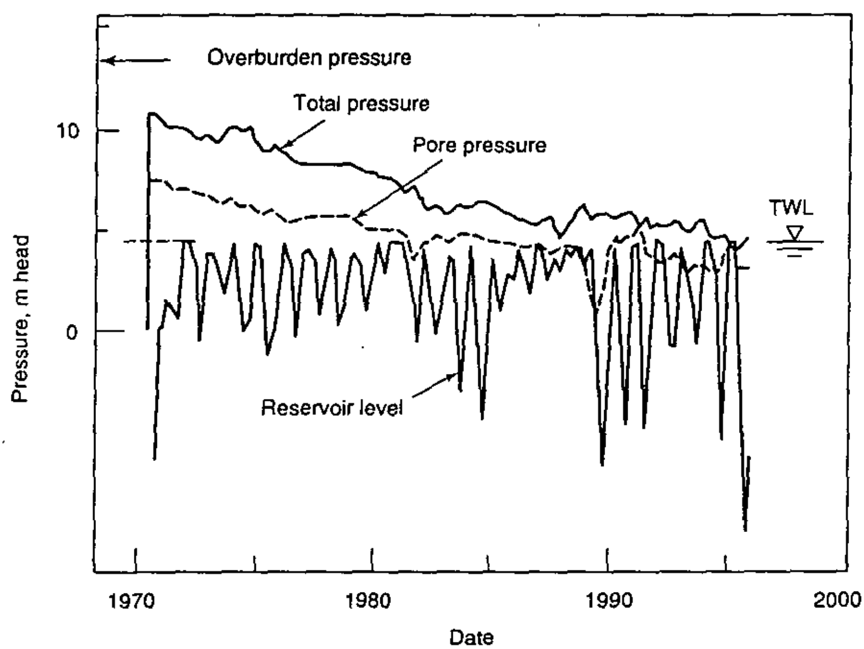


Fig.4a. Changes in earth pressure and pore water pressure measured at the interface at position No.4, 6.7m below crest level.

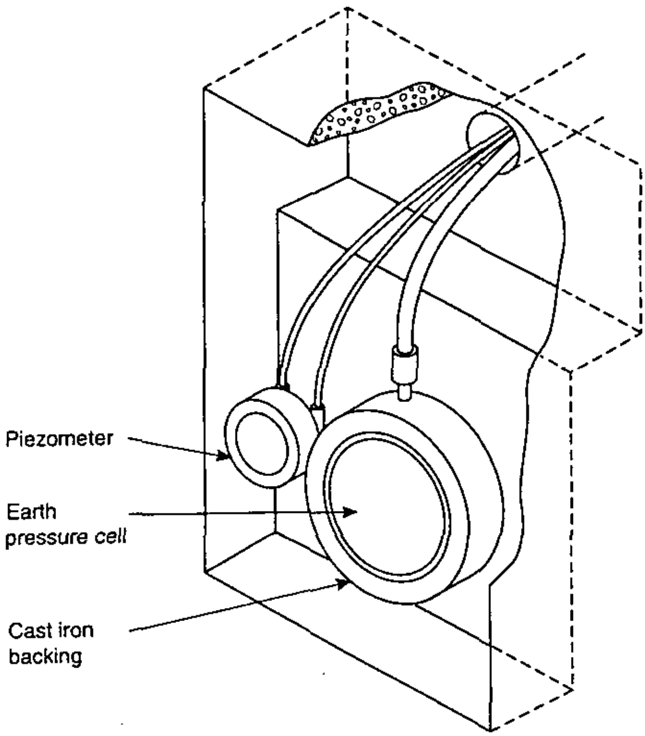


Fig. 2. Arrangement of pressure cell and piezometer on the interface

Fig. 3. The BRS vibrating-wire earth pressure cell prior to assembly

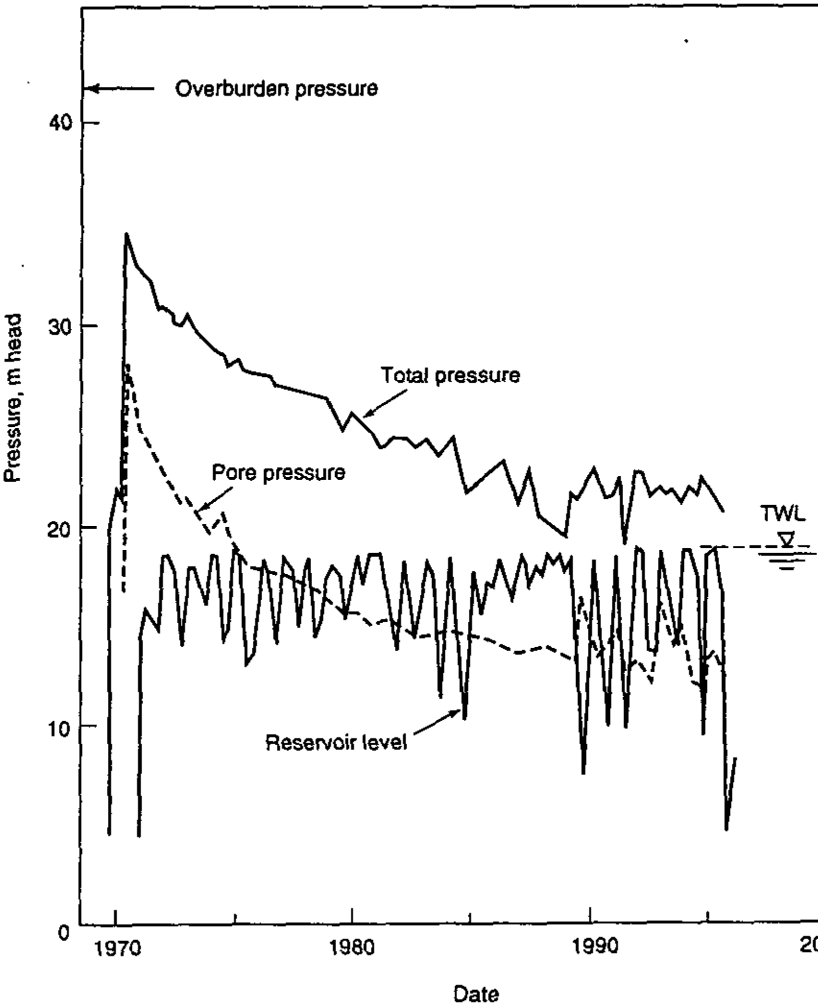


Fig.4b. Changes in earth pressure and pore water pressure measured at the interface at position No.1, 21.4m below crest level.

Since 1990, the rate of decrease has declined. Figure 5 shows the change in earth pressure from the end of construction in 1970 to 1994. No changes in earth pressures or pore pressures were observed in response to first impounding of the reservoir. The effects of subsequent reservoir fluctuations on the observations were probably masked by the relatively large dissipation of construction pore pressures. From 1990, some of the fluctuations in measured earth pressure appear to be related to changes in reservoir level while others do not. Similarly with the fluctuations in pore pressure, some can be related to reservoir level changes while others cannot. The most recent reservoir drawdown in 1995, the largest since construction, has had a significant effect on the instrument readings.

Table 1. Summary of earth and porewater pressures measured at the interface in 1971 and 1992

Cell	Pressures in 1971				Pressures in 1992				
	P_{CA}	P_{CB}	P_C	u	P_{CA}	P_{CB}	P_C	u	AP
1	32.6	27.6	30.1	22.0	20.8	22.1	21.5	11.8	29.4
2	23.6	19.1	21.3	15.5	14.3	15.2	14.2	9.4	18.4
3	17.2	17.2	17.2	10.4	11.1	10.7	10.9	6.0	16.2
4	8.6	10.6	9.6	6.6	3.4	5.2	4.3	3.6	10.6

Notes ;
Pressures measured in metres head of water
AP - Pressure applied to hydraulic piezometer
u - Porewater pressure

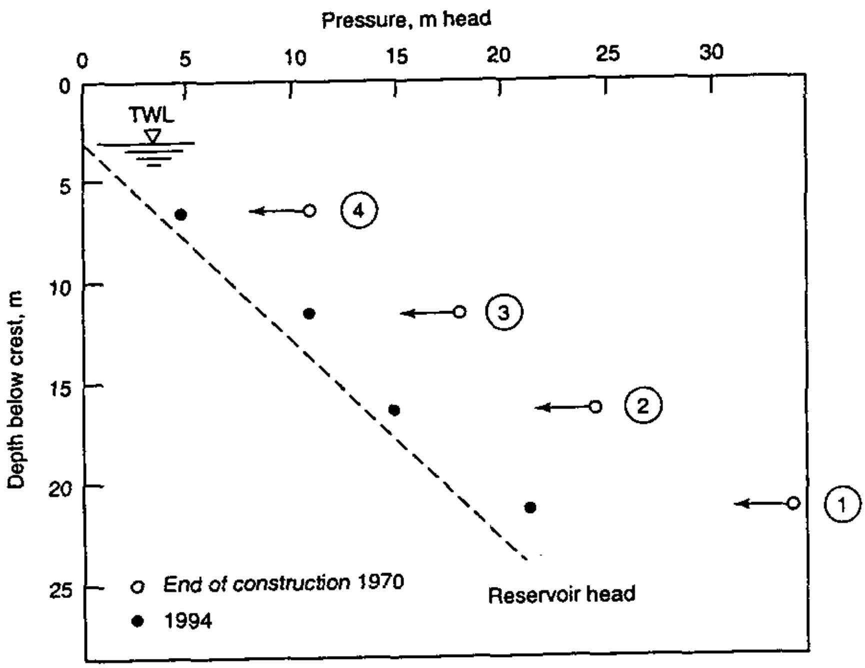


Fig.5. Decrease in measured earth pressure at interface, 1970 to 1994.

EARTH PRESSURES MEASURED WITH HYDRAULIC PIEZOMETERS
Concern about the reduction of measured earth pressures in 1990 led to a proposal by BRE to measure the earth pressures using critical pressure tests in the hydraulic piezometers. The method (Penman, 1995) involves applying a pressure to the piezometer to cause localised fracture. By observing the flow rate against pressure, the critical pressure at which the flowrate substantially reduces to a value related to the permeability of the clay can be determined. Extensive testing was carried out on the piezometers in the core during construction and it was shown that the in-situ permeability of the clay was not affected by critical pressure tests.

Initially the pressures at the interface piezometers were increased to a value equivalent to the hydrostatic pressure that would be exerted by the reservoir at the PMF level and then to a pressure (AP in Table 1) significantly above that measured by the earth pressure cells. In each case the flow out of the piezometer was very small, less than 10ml per minute and there was no indication of hydraulic fracture.

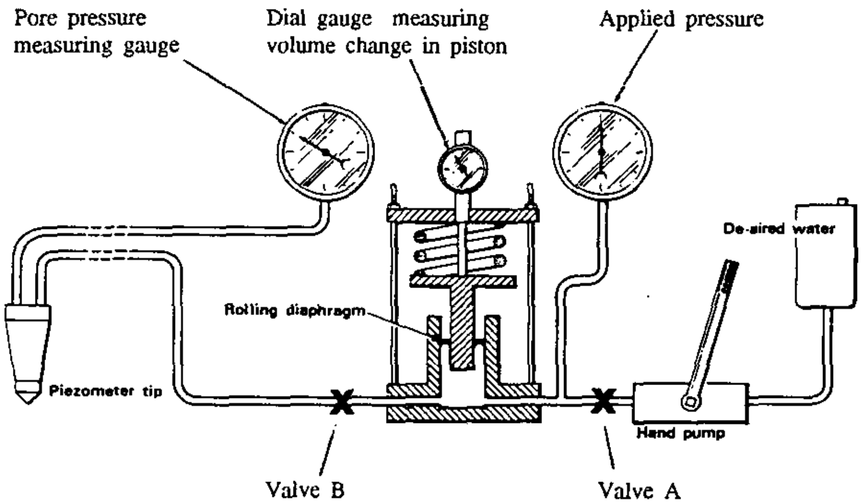


Fig. 6. Apparatus used for critical pressure tests (after Penman 1975)

Further tests were carried to measure the critical pressure using the equipment designed by Penman (1975), shown in Fig. 6. The movement of a piston sealed by a rolling diaphragm is used to measure the flow of water to the piezometer tip. The load on the piston is provided by a spring reacting against three tie rods. The load can be varied by using springs of different stiffness and adjusting the length of the tie rods. With valve A

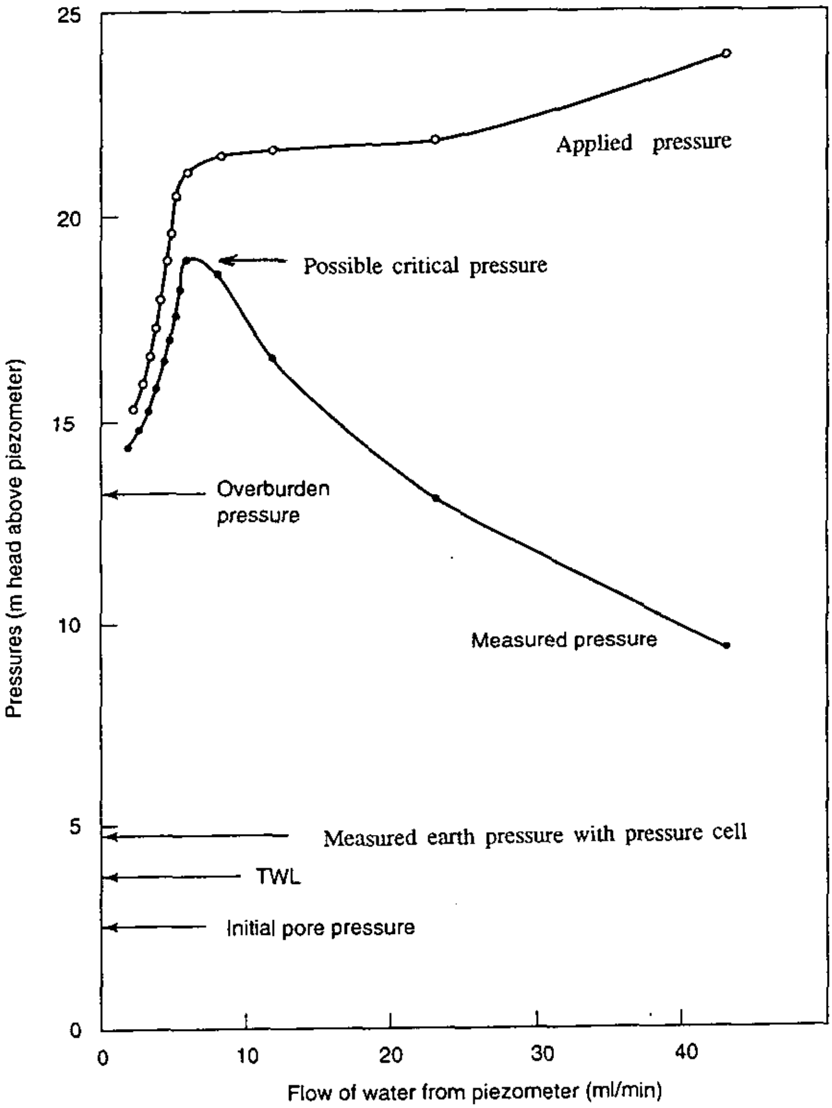


Fig.7. Plot of flow against pressure for critical pressure tests, piezometer No. 4

open and valve B closed, water is pumped using the hand pump to raise the piston against the springs and develop a pressure to be applied to the piezometer. To start the test, valve A is closed, valve B is opened. The flow of water from the piston to the tip is measured by the dial gauge and the pressure developed at the piezometer is measured through the second change in applied pressure and the pressure measured at the tip are recorded during the test. A fixed volume of water no larger than 130 ml is pushed out of the piezometer. The results of the tests were plotted in terms of flow and pressure at the piezometer tip.

Figure 7 shows the relationship between flow and pressure for one of the tests. The pressure from both pressure gauges is shown. To obtain a significant flow and therefore an indication of fracture, the initial applied pressure had to be much larger than the calculated overburden pressure. As the flow rate decreased, the pressure measured at the tip built up while the applied pressure decreased. The difference between the two values was due to the head loss from the apparatus to the piezometer tip. When the applied pressure was maintained for a few seconds after the start of the test, the two pressures were similar. At a flow rate of about 10ml/min, it can be seen that there is a distinct change in slope of the plots. This change of slope probably indicates the closing up of the fracture caused by test. The pressure at which this close up takes place, the critical pressure, is difficult to determine, but the value indicated in Fig. 7 is significantly larger than the calculated overburden pressure and the earth pressure measured by the pressure cells.

DISCUSSION

The long term trend of the earth pressure measurements at the interface of the concrete dam with the clay core at Cow Green dam raised concerns about the possibility of hydraulic fracture occurring and leading to erosion. Interface pressures are difficult to measure reliably even in the short term and it is remarkable that observations have been maintained on the pressure cells for more than 26 years. Some doubt over their long term absolute value must be cast. A decrease in total lateral pressure on the interface could occur as result of settlement of the core due to consolidation adjacent to the relatively incompressible concrete causing hangup or arching between the clay and the concrete part of the dam. It appears from pore pressure observations that equilibrium conditions have been reached although the recent fluctuations make it difficult to be certain.

In assessing the long term observations and their implications on safety a number of aspects need to be considered. Long term drift of the earth pressure cells associated with corrosion of the vibrating wire and fixing points could account for some of the decrease in earth pressure. There is evidence of long term drift of this type of cell (Brackley & Sanders, 1992) when circulation of the dry nitrogen has not been maintained. Creep of the backing material and movement of the cell leading to arching across the cell and hence a reduction in pressure on the cell which is not representative of the pressure acting on the interface.

The tests on the interface piezometers were not entirely satisfactory in terms of producing a well defined critical pressures. This may have been due to the disc type of piezometer. More satisfactory tests were obtained from the Bishop type piezometers in the main part of the core. Each test carried out on the interface piezometers indicated that the critical pressure was larger than that measured by the earth pressure cell. Penman (1975) found that critical pressures were much higher than pressures measured by pressure cells.

If all the pressure cells had ceased to work shortly after construction the present concern over the pressures at the interface would not exist. There would be uncertainty about the pressures, but it is unlikely that an investigation would have been carried out unless there was there had been evidence of distress.

ACKNOWLEDGEMENTS
The authors are grateful to Northumbrian Water Services Ltd for permission to publish this paper. Valuable comments on the paper have been received from Dr J A Charles of BRE and Dr A K Hughes of Rofe, Kennard & Lapworth.

REFERENCES

Brackley I J A & Sanders P J (1992). In situ measurement of total natural horizontal stresses in an expansive clay. Geotechnique Vol.42, No 2, pp443-451.

Kennard M F & Reader R A (1975). Cow Green dam and Reservoir. Proc.Instn Civ.Engnrs,Part 1, vol 58,pp147-175.

Penman A D M (1975). Earth pressures measured with hydraulic piezometers. Proc. *ASCE Speciality Conf. On insitu Measurement* of Soil Properties. North Carolina State University, vol 2, pp361-381.

Thomas H S H & Ward W H (1969). The Design, Construction and Performance of a Vibrating-wire Earth Pressure Cell. Geotechnique, vol.19, No.1, pp 39-51.

Vaughan P R & Kennard M F (1972). Earth pressures at the junction between an embankment dam and a concrete dam. Proc. 5th European Conf. Soil Mech.and Foundation Engineering,Madrid, vol 1,pp215-221.

Vaughan P R, Lovenbury H T and Horswill P (1975). The design, construction and performance of Cow Green embankment dam. *Geotechnique, vol* 25,No 3, pp555-580

Itezhi-tezhi dam: warm spring phenomena.

I L PHIRI, Zambia Electricity Supply Corporation Ltd, ZAMBIA

SYNOPSIS. The Itezhi-tezhi dam 230 kilometres upstream of the Kafue Gorge Power Plant in Zambia was built in the early seventies to increase generating capacity from 600 to 900 megawatts. It is an embankment dam with a slightly inclined central core of clay. Initial impounding of water in the reservoir soon resulted in the emergence of warm springs in an area downstream of the dam. Investigations to try and establish the reasons for this were quickly instituted and resulted in the installation of a pressure relief system that has been closely monitored over the years to ensure non-exceedence of pre-determined underground water levels that may threaten dam stability.

INTRODUCTION.

The Itezhi-tezhi dam 230 kilometres upstream of the Kafue Gorge Power Plant in the Republic of Zambia was built in the early seventies to increase generating capacity from 600 to 900 megawatts. It is a 65m high earth and rockfill embankment dam, curved in plan, with a slightly inclined central clay core (Figs.1 and 2). It is founded partly on mudstone and partly on granite. The foundation between the left flood plain and river channel sections consists of mudstone overlying basement granite. The maximum depth of the mudstone beneath the dam is nearly 100m close to the southern granite contact and decreases gradually towards the north abutment where the dam is founded on granite. The right abutment is also founded on granite. At full supply level (1029.5 masl) the reservoir holds some 5.6 billion cubic metres of water. The impounding of water in the reservoir started in May 1976 raising the reservoir from elevation 990m to 1002m and soon resulted in the occurrence of warm springs in an area downstream of the dam.

The first unusual phenomenon was a white efflorescence at the bottom of the spillway outlet channel noticed in August 1976. Later, springs emerged further downstream on the floodplain and it was noted that the water was warm. Several signs of instability were observed on the floodplain between the spillway outlet channel and the ferry landing area which is about 1.8 km downstream of the dam. Auger drilling of the affected areas commenced in September 1976 and diamond drilling in December the same year. In 1977, test wells and permanent relief wells were subsequently drilled in the same areas.

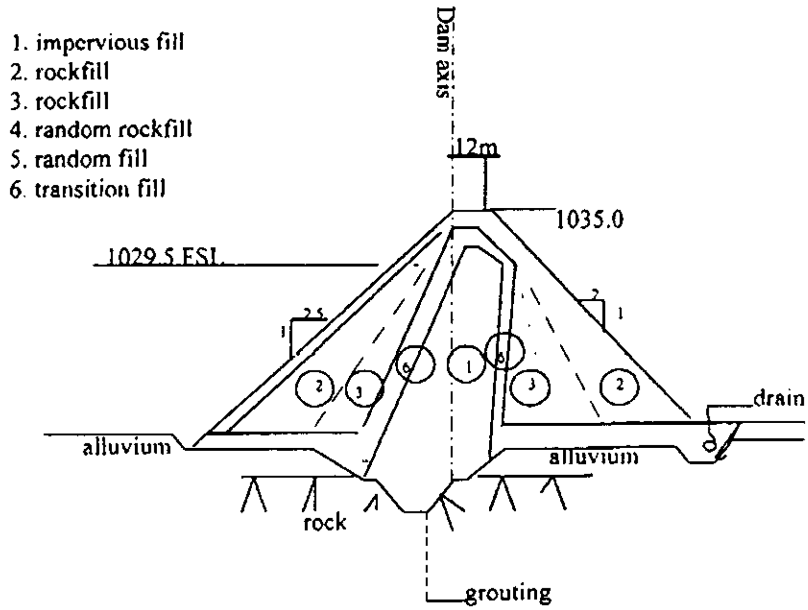


Fig.1. Dam cross-section on left hill slope.

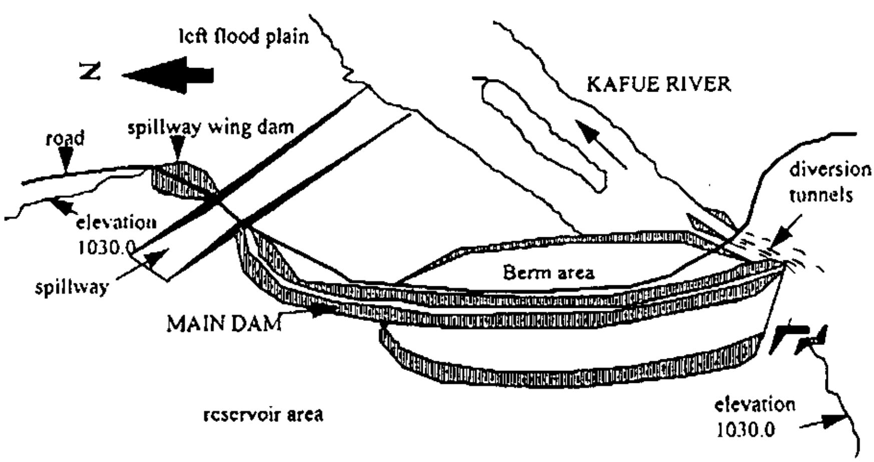


Fig.2. Sketch plan of the dam showing part of the investigated area.

The end result of the investigation was the installation of a pressure relief system, using two compressors, one at the berm and the other at the left flood plain, and a pipe network in to and out of the wells. The underground water levels have been closely monitored ever since, to ensure that they do not exceed a certain pre-determined value that may threaten dam stability. If the underground water level rises to the value that threatens dam stability, the relief system can be activated to lower it to safe levels by increasing discharge from the wells. The relief wells on the left flood plain are 70m deep, of which 50m is in granite and 20m is in overburden, and are encased in steel. A concrete ring 1.35m diameter and approximately 1.0m deep protects the well heads. The relief wells at the toe of the dam are 170m deep, of which 50m is in granite, 100m in mudstone and 20m in overburden and are similarly protected.

SITE INVESTIGATIONS.

Site investigations at Itezhi-tezhi during the pre-construction phase comprised such procedures as are normally applied for large dam projects. The extent of the investigation work can also be said to have been normal. Original investigations at the dam site were mainly core drilling, both vertical and inclined, soil sampling, test pits, test wells and seismic surveys. Investigations were also extended to, inter-alia, borrow areas. None of these tests was able to give an indication of the presence of the deep seated aquifer that lay below.

Zambia has an abundance of warm springs throughout the country, and twenty such springs are located within a radius of 150 kilometres from Itezhi-tezhi in areas of similar geology. A warm mineralized spring arises 7 km south east of the dam and has been in existence for a long time.

Most of the springs at the project site were located in a narrow band approximately a kilometre downstream of the dam, none were found closer than this. It was established that the outflow from the springs, which was measurable from a number of springs, varied with the reservoir water level; at elevation 1005m, the discharge was some 100 litres per second whilst at elevation 1011m, the discharge was found to be some 150 litres per second. An estimate of the total outflow from the springs made during a temporary closing of all gates indicated that the outflow was at least twice the above values.

A chemical analysis of the water from the springs indicated essential differences from that of river and reservoir water, suggesting that the source of the spring water was different.

Extensive site investigations of the geology and hydrogeology of the dam site and its surroundings were commissioned extending some 20 km upstream and 10 km downstream of the dam consequent to the above findings. The investigations centred on the following explorations:-

a). Detailed geological mapping upstream and downstream of the dam as part of the general geological survey of Zambia.

b). An extended seismic refraction survey covering an area some 500m to 1000m upstream of the dam and some 1500m downstream thereof.

c). A marine geophysical survey in the reservoir and in the river downstream of the dam covering a suspected fault line along which leakage could occur, up to 10 km upstream and 2.5 km downstream of the dam.

d). A thermal scanning survey, mainly covering an area up to 3 km downstream of the dam, but also extending to the known warm spring 7 km to the south east, for possible connection paths to the dam..

e). An infrared aerial photography of the reservoir area and surroundings and the area downstream of the dam up to a distance of 10 km.

f). Drilling of some 130 boreholes with core recovery, mainly along the fault line downstream of the dam suspected to carry leakage water, but also in other areas downstream as well as upstream of the dam.

g). Installation of sealed piezometers in a number of core drilled holes, mainly downstream of the dam, but also in some holes upstream. Daily recordings of water pressure, in the early stages and later by recording instruments.

h). Recordings of leakage amounts from springs and boreholes.

I). Test pumping from boreholes.

j). Inspection of areas upstream and downstream of the dam mainly along the fault line suspected to carry leakage water. The inspections extended some 20 km upstream and 10 km downstream of the dam and were aimed at discovering springs, sinkholes, settlements, salt precipitates and gas outlets from the ground.

k). Study of satellite infra-red photographs of the surroundings of the dam site.

l). A further detailed geological mapping of the near surroundings of the dam site based on the investigations indicated above and with emphasis on the engineering aspects such as fault lines, fissures, voids and other water carrying strata.

m). Chemical analyses of leakage water and river water at frequent intervals including determination of ^{18}O, Deuterium and Tritium as well as ^{14}C dating of the water.

n). Recording of temperature, conductivity and turbidity of leakage water.

RESULTS AND SOLUTIONS.

One of the major findings of the above investigations was that the surroundings of the dam site have been subjected to strong tectonic activity in several different periods including heating, melting, and re-crystallization of older sedimentary rock as well as intrusion of granites. A regional fault or rift zone developed passing through the vicinity of the dam site with a trend east-northeast. The fault is some 10 km wide and was mapped as extending hundreds of kilometres to the east of the dam site and was traced on satellite imagery 50 km to 100 km southwest of the site. Deep weathering along the fault lines and in carbonate rich rocks has continued up to the present time and has resulted in a modified karstic rock, particularly pronounced where there has been a high carbonate content in the granite.

A deep regional aquifer flow system has developed in the solution openings. The water flows from a higher terrain southwest of the dam site along the upper reaches of a tributary to the Kafue, towards their confluence. The aquifer is estimated to cover some 10 km in width. A flow model was established to simulate aquifer flow based on the above and other findings.

Using the model it was established that the artesian pressure of the aquifer acting below the main section of the dam could have unfavourable effects on dam stability. It was further established that uncontrolled leakage from the aquifer downstream of the dam could give rise to erosion in alluvium, mudstone or weathered rock which in the long run would affect the supporting fill of the dam causing settlements and slides.

The stability of the dam would come into question at reservoir elevation 1005m, which is the highest permissible pressure. However, for operational purposes the pressure was limited to elevation 998m. To achieve this limit, the outflow from the relief wells was calculated to be equal to or greater than $0.74 m^3/s$ at full supply level of the reservoir and the tailwater at elevation 991m. Twenty six wells each of 300mm diameter met this condition. It was also established that above elevation 989m, additional relief was required downstream of the dam. This was achieved by pumping in existing high discharge wells. A simple and reliable pumping system was obtained by installation of air lifts in the high discharging wells. By blowing out compressed air in a riser pipe at some considerable depth, the well discharge increases substantially thereby lowering underground water levels. Two compressors, one on the berm and the other on the left flood plain, were installed for the purpose.

It is worth mentioning that the above extensive investigations did not result in any detailed knowledge of the aquifer or its source. However, a general concept of the geological factors contributing to the existing conditions was developed. This may give some clues as to what factors deserve attention when dealing with dam site investigation. It may be stated that two or more of the following should lead to a closer investigation of the possibility that a big aquifer exists at depth. These factors are: a broad fault zone crossing the dam site, calcium content in igneous rock, precipitation of rock crystals or amorphous quartz in the joint system and warm springs in the neighbourhood of the site.

MONITORING.

Over the years dedicated staff at Itezhi-Itezhi under the supervision of a Resident Engineer have, among other duties, closely monitored the Pressure Relief System to ensure that the system is serviceable and operational. A minimum of two runs per year are required as a check that the system works. Underground water levels are collected daily using automatic level recorders and once a week manually. Since installation of the system, the underground water level has remained far below target (except for a short period between June and July 1978 when the artesian pressure in the aquifer suddenly rose by as much as 1.5m) and indications are that the situation will continue to be so for many years hence.

ACKNOWLEDGEMENTS.

The author is grateful to the following staff at Itezhi-tezhi for making available their time and help during the data collection phase for this paper:- M Simainga (Civil Engineer) and M Mulambe (Surveyor).

Refurbishment of Flood Gates at Torr Achilty

N M Sandilands, Scottish Hydro-Electric plc, UK
M Seaton, Scottish Hydro-Electric plc, UK

SYNOPSIS. Scottish Hydro-Electric plc own and operate seventy six reservoirs registered under the Reservoir Act incorporating eighty four dams. There are over two hundred major hydraulic gates associated with these structures. The average age of the dams is now approaching forty years and maintenance and refurbishment of gates is now a major task. This paper describes the refurbishment in 1995 of the three free roller flood gates at Torr Achilty Dam.

INTRODUCTION. Torr Achilty Dam is located at the downstream end of the Conon Scheme and has a total catchment area of 1000 sq km. The dam is a concrete gravity structure 17.7m high with a length of 246m. The power station is built into the dam and comprises two 7.5 MW Kaplan turbines. Construction was completed in 1955.

Due to its situation at the downstream end of a cascade system on one of Scotland's larger rivers, the dam requires to pass significant floods. The calculated PMF is 2435 m^3/s. The dam is equipped with three direct lift undershot flood gates. The centre gate is 3.1m wide x 4.1m high and the two outer gates are 3.7m wide x 5.5m high. The gates are of the free rolling type with separate roller trains connected by chain pulleys. With the three gates fully open a flood of 935 m^3/s can be passed without raising the level of the reservoir.

The PMF of 2435 m^3/s calculated in accordance with the ICE's "Floods and reservoir safety: an engineering guide" (1989) is much greater than the original design flood. The factor of safety against overturning for the dam was found to be unsatisfactory under PMF conditions. As a consequence of this post-tensioned tendons were installed in the dam in 1989/90. (Peacock & Sandilands, 1993)

REFURBISHMENT POLICY. Hydro-Electric has a policy of refurbishing major gates and valves at 25 year intervals. (Johnston & Sandilands, 1992)
Currently a number of gates and valves are in excess of 25 years old and await their first refurbishment. An accelerated refurbishment programme was commenced in 1995 to deal with the backlog. The Torr Achilty Flood Gates, now forty years old, were included in the 1995 works programme.

OPERATIONAL HISTORY. The operation of these gates was satisfactory and they had no adverse history except for problems with drive shafts in the early eighties which resulted in a refurbishment of the operating gear in 1983. This included replacement of a number of twisted drive shafts. At this time the general condition of the gates was judged to be good and a full refurbishment was not deemed to be necessary.

ISOLATION OF GATES. The gates are supported by concrete piers and have a common sill level of 15.25m OD. The arrangement is shown in Figure 1. Upstream of each flood gate are grooves for stop log gates. The stop log gates are fixed roller gates with rubber seals which are installed in three sections using a lifting gantry. These gates were refurbished in 1991 to allow for future flood gate refurbishment and are in excellent condition. There is only one set of stop log gates and it was therefore necessary to refurbish the gates in a phased programme. Seven weeks were allowed to remove each gate from site, carry out works refurbishment and refit and commission the gate, giving an overall programme of twenty one weeks.

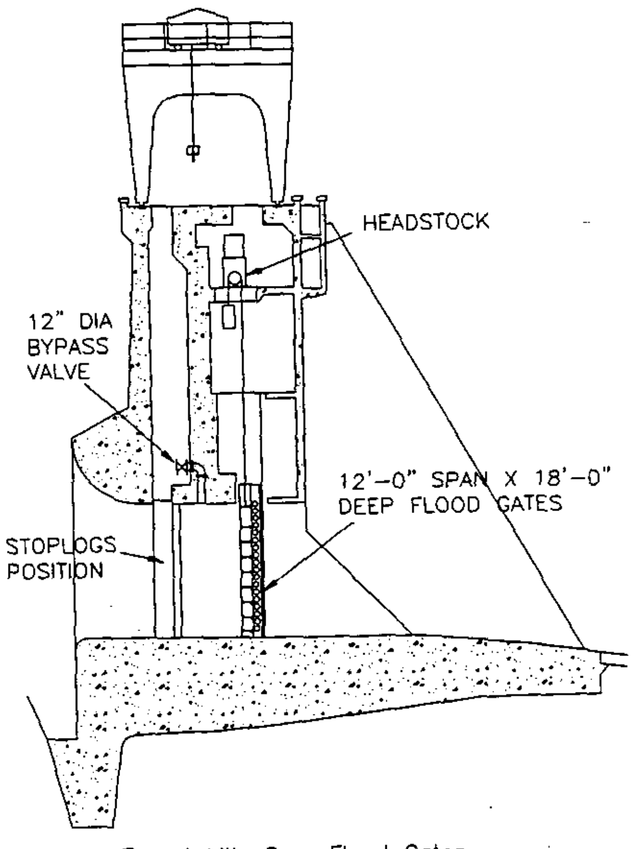


Fig. 1. Torr Achilty Dam Flood Gates Sectional Elevation

PRACTICAL PROBLEMS. Before starting the work programme a long standing site problem had to be resolved. A colony of pigeons, comprising over sixty birds, had established the gate shafts as a home. The presence of the birds was a safety risk to anyone inspecting or working in the shaft and also caused potential health risks. A pest control consultant was commissioned to advise on the steps necessary to remove the birds safely and to avoid future infestation. A pest control company removed the birds prior to starting work in each shaft. The shafts were then thoroughly cleaned to provide a suitable working environment. Temporary netting was erected to prevent the pigeons re-entering the shaft during the refurbishment, and on completion permanent galvanised mesh pigeon barriers were installed across the shaft openings and between the beams on the back of the gate to prevent access to the beam flanges.

To carry out work on the gate guides, access was required to the downstream concrete sill. The concrete sill can become submerged if an adjacent flood gate opens or if the station is generating. To ensure the safety of operatives a Code of Procedure was developed to control downstream water levels. This required that generation was restricted to 0.5 MW and that the level of Loch Achonachie was maintained below 29.9m OD. An emergency klaxon was used to warn the operatives of a turbine trip which would cause an adjacent floodgate to open to maintain compensation flows.

GATE REFURBISHMENT. The larger gates weighed 11.5 tonnes with fittings and due to their size had to be split into two to be lifted from the guides. The centre gate was able to be lifted in one piece and this constituted the heaviest single lift at 6.2 tonnes stripped of all fittings. The gates were lifted from the guides using the permanent travelling gantry and a 40 tonne crane situated in the station yard, then lifted them on to lorries for transport to works.

The gates are of bolted construction with mild steel cross beams and end posts fixed to a stainless steel skin plate. The stainless steel skin plate is an unusual feature and was in very good condition. The mild steel components had suffered significant corrosion, as would be expected after 40 years with no maintenance. The gates, seal plates and lifting rods were all shot blasted thoroughly to ISO 8501-1 SA 2.5. Three coats of Sigmacover Pitakote, a high build solventless epoxy coating were applied. It is anticipated that this coating will provide a twenty five year service life. This coating has been specified since 1993 in preference to the previously adopted Colturiet Pervon which had an extremely good proven performance, but has a solvent content of 50% and is isocyanate cured and is likely to become illegal in the near future. The fixed frames in the gate shaft were prepared by needle gun and coated with an aluminium epoxy primer and Pitakote. The bottom sections of the guides which could not be dried were coated with an underwater epoxy.

A high proportion of the bolts on the gates were found to be heavily corroded. All of the end post to cross beam fasteners had to be replaced. Around 35% of the skin plate bolts and all of the sill seal retaining bolts were also replaced. The original bolts were described on the drawings as "rustless". In practice their performance was no better than ordinary mild steel bolts.

The original roller trains comprised meehanite cast iron rollers with mild steel side plates and chains. A section through the roller train and groove is shown in Figure 2. The centre gate has 13 rollers and the two outer gates 18 rollers on each roller train. The original bearing arrangement comprised mild steel axles with high tensile tube sleeves and gunmetal bushes. Experience indicates that rollers to this specification have a tendency to stick after a number of years service and there is a risk that the whole assembly will start to slide rather than roll greatly increasing the friction and the risk of failure of components. Complete new roller trains were specified in stainless steel to grade BS 970 Part 1 : 1990 316/S11. Low friction self lubricating bushes were specified to minimise friction and no sleeves were necessary.

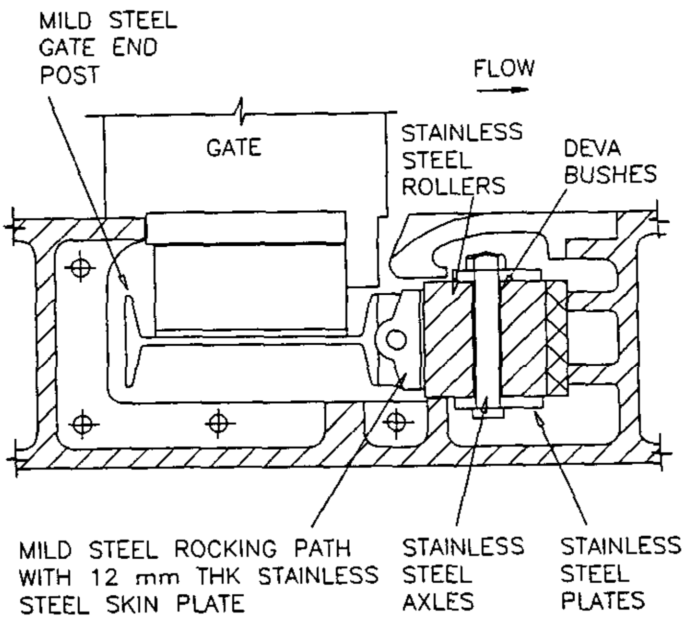


Fig. 2. Section Through the Roller Train and Groove

The original rollers were found to be in reasonable condition with moderate pitting. The chains were heavily corroded and one chain on the centre gate had failed. This would have eventually resulted in operational failure.

The rocking paths were found to be badly bowed with maximum deformation of 13mm over a 2.75m length. Due to the brittleness of cast iron attempting to straighten them was considered inadvisable. New rocking paths were fabricated from mild steel with a 12mm thick stainless steel plate pinned to the bearing surface to provide a longer service life.

The rubber seals were all found to have deteriorated significantly and all were replaced. Since rubber seals invariably require replacement an annual contract is placed to purchase the rubber seals for all gates in the current year's refurbishment programme.

Site work was carried out between early May and the end of September. Replacement of the damaged drive shaft on the north gate resulted in a return to site and the commissioning of the north gate was completed in early November. The cost of the refurbishment of the three gates was £170,000.

CONCLUSIONS. The gates at forty years old were some way past the twenty five year period adopted as policy for refurbishment of gates and this was reflected in the condition of some of the elements. The Company policy of refurbishing gates at twenty five year intervals was first proposed around fifteen years ago. Over that period around 115 gates have been refurbished.

Experience has shown that where gates are more than thirty years old at first refurbishment the cost of the works tends to increase. The risk of operational failure also increases as demonstrated by increasing failure of components and very occasional failures during service.

FUTURE POLICY. Due to the adoption of stainless steel and other high quality corrosion resistant materials for rollers and other key components, it is envisaged that future refurbishment will be simpler and cheaper. Rubber seals and paint coatings, however, will remain vulnerable elements. Due to the fairly significant degree of corrosion at first refurbishment it is important that paint coatings are maintained in good condition in the future to avoid further loss of metal. It is therefore not envisaged that the return period for refurbishment will be extended significantly beyond twenty five years in the future.

REFERENCES

The Institution of Civil Engineers (1989). Floods and reservoir safety an engineering guide. Thomas Telford, London

Peacock A R & Sandilands N M (1993). The post tensioning of Hydro-Electric mass concrete and masonry dams. GFGB Technical Symposium Maintenance of Older Dams, Chambery. Proceedings Volume 2 pp167-185.

Johnston C K & Sandilands N M (1992). Resume of maintenance contracts on Hydro-Electric reservoirs, conference proceedings, British Dam Society, Water Resources and Reservoir Engineering, Stirling. pp339-351.

Wala, Mujib and Tannur Dams, Jordan

L J S ATTEWILL, Howard Humphreys, UK

SYNOPSIS. The three dams will be situated in the semi arid area to the south of Amman and are intended to impound flood flows that would otherwise discharge into the Dead Sea. The yield of the reservoirs is intended to support municipal, industrial and agricultural water demands, mostly of the area to the south of the Dead Sea. All three dams will be founded in the limestone of deeply incised valleys and for all three sites a roller compacted concrete gravity dam has been selected.

THE NEED FOR WATER.
The present availability of water for Jordan, which has a population of 4.14 million is 860 mcm/year, equivalent to 570 litres/person/day. This per capita consumption compares with a World Bank recommended minimum for the Middle East of 2800 l/c/day.

The existing water resources comprise surface water to the north of Amman and groundwater which is exploited (and over exploited) locally throughout Jordan. Several schemes, including the Karameh dam, the raising of the Kaffrein dam and the development of the Disi aquifer, are under construction or being planned to provide additional water supplies to support both population and per capita consumption growth, which together require some 20 mcm of extra water each year.

DESCRIPTION.
The three dams will be situated on the Wala, Mujib and Hasa wadis as shown in Fig. 1. The dams will support the planned development of the Southern Ghors, an area to the south of the Dead Sea where water is required for the development of chemical industries, for irrigation and for the associated municipal water supplies. Their yield will be optimised if they are operated conjunctively with baseflows and groundwater resources. In addition, water from the Wala reservoir will augment groundwater supplies being pumped to Amman. The principal statistics of the dams are summarised in Table 1.

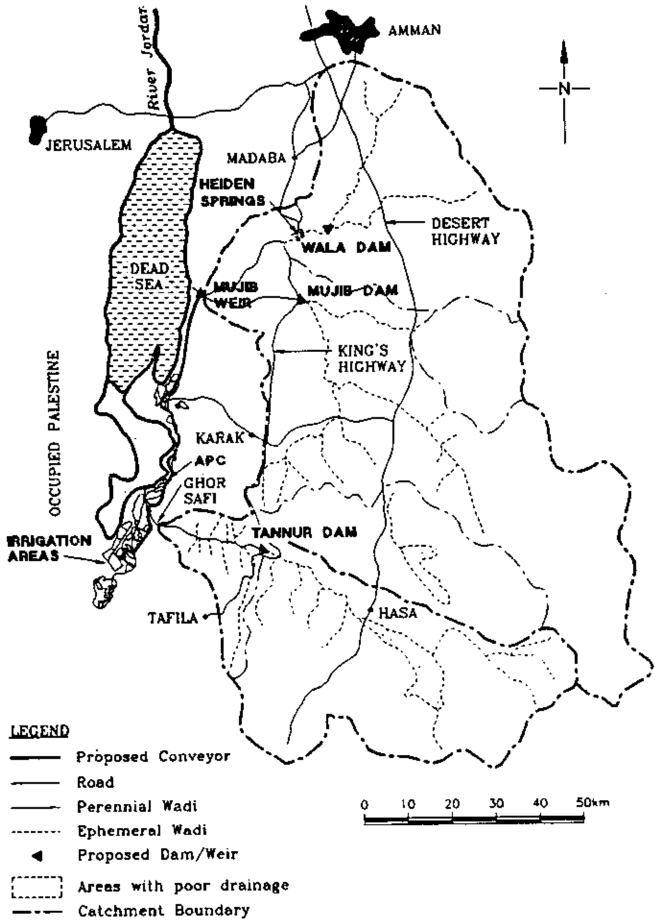


Fig. 1: Project Area

Table 1. Principal statistics of the dams

	Wala	Mujib	Tannur
Dam height above foundations (m)	45	51	62
Dam volume (cu m)	162 000	750 000	240 000
Reservoir volume (mcm)	9.3	35	16.8
Crest length (m)	180	735	350
Spillway capacity (m^3/s)	2020	5530	3080
Estimated construction cost ($ million)	30.50	70.80	28.2

The Wala dam site is situated 40 km south of Amman at an elevation of 500m. The reservoir and dam foundations will be formed in the Tafilah limestones of the Wadi Gudrun formation which are moderately permeable ($k = 10^{-6}$ m/s). Although a positive cut-off will be provided under the dam, leakage prevention measures in the reservoir area, consisting of either blanketing or grouting, would be prohibitively costly. The rate of seepage from the reservoir is likely to be sufficiently high as to preclude a reliable yield by direct abstraction. The reservoir will therefore be used to support abstractions of water for Amman that are being made from the Heiden Springs by means of the artificial recharge of the underlying Wadi As Sir Aquifer. However, because of this rather unconventional design approach, it has been decided to construct the dam in two stages, an initial stage of the minimum height followed by a second stage if the first proves to be satisfactory.

The Mujib dam site is situated 10 km further south from Wala in the bottom of a 500 m deep gorge formed by the wadi Mujib - the biblical river Arnon - and in the relatively low permeability limestones of the Naur formation. The Mujib reservoir will act as a conventional reservoir in that releases of water will be made to support the baseflow according to the demand pattern. The unusual feature of the dam is that it will carry on its crest the Kings Highway, an ancient road linking Amman with Aqaba. The main engineering challenge has been that of the stabilisation of the reservoir rim.

The Tannur dam, situated 60 km south of Mujib on the wadi Hasa is at the lowest elevation of all three dams and should present the least problems in terms of permeability. Like the Mujib reservoir, Tannur will be a conventional river regulating reservoir. The seismic implications of the adjacent El Hasa regional fault provide the defining design criteria.

HYDROLOGY.

The three dams will contribute 29.4 mcm of additional water in a part of the country where water resources are particularly scarce. The dams will be situated on wadis that drain large catchments of semi arid and desert land to the east of the Dead Sea. Annual rainfall varies from 50 to 250 mm and occurs as intense storms in the winter months. The dams will impound this runoff, but because the flows are so unreliable relatively large storage volumes are needed to provide a significant reliable yield. Furthermore the floods carry very high sediment loads and large volumes of dead storage are required as well as the provision of check dams in the catchment to control erosion.

The hydrological characteristics of the catchments are summarised in Table 2

Table 2. Summary of hydrological characteristics

Characteristic	Catchment		
	Wala	Mujib	Hasa
Area (km)	1770	4380	2160
Average rainfall (mm)	250	150	90
Average flood flow (mcm/year)	18	23	8
Yield (mcm/year)	5	16.6	7.8
Flood flows (m^3/s)			
- 1 in 50 years	600	1700	450
- PMF	2020	5530	3080
Sediment yield (t/km/year)	180	60	50
Sediment load (t/year)	330000	175000	106000

SELECTION OF DAM TYPE.

At each site various dam types were considered and at each site the final choice lay between a concrete faced rockfill dam (CFRD) and a roller compacted concrete (RCC) dam, earthfill dams being eliminated because of the scarcity of suitable core material. The substantial wadi diversion works and the separate spillway implicit with embankment dams gave a clear cost advantage to RCC, which in addition will be quicker to build.

SEISMIC LOADING.

The geology of central Jordan comprises a 600 m thickness of predominantly calcareous sediments of Cretaceous age uncomformably overlying clastic sediments of a Precambrian basement. The Cretaceous sediments are composed of interbedded limestones, chalks and marls. The three dam sites lie to the east of the main Dead Sea Rift, which, together with its associated perpendicular flexural faults, is the principal earthquake generator. Historic earthquake data from 1904 to 1994 in a 400 km radius were collected and analysed, giving the return period/magnitude relationships presented in Table 2.

Table 3. Magnitude / return period relationships

Magnitude	Return Period (years)
4.0	1
4.5	3
5.0	7
5.5	17
6.0	38
6.5	87
7.0	200

Peak accelerations at each dam site were derived from attenuation relationships linking the acceleration with the distance from the source, the magnitude, and the focal depth: three such relationships were considered, those of Ambraseys and Bommer (1991) and Joyner and Boore (1988) being adopted.

The magnitude of the Maximum Credible Earthquake (MCE) was derived from the empirical relationship of Barazangi (1983) which gives magnitudes of 7.9 and 6.6 for earthquakes generated by the Dead Sea fault and the El Hasa faults respectively. Thus for the Tannur dam site, situated just 0.5 km from the El Hasa fault, the peak ground acceleration for the MCE is predicted to be 0.5g, while for the Wala and Mujib dam sites, for which the nearest active fault is the Dead Sea fault, the acceleration is predicted to be 0.228g. The peak acceleration associated with the Operating Basis Earthquake (OBE) is 0.2g for all three dams.

STRESS ANALYSIS.

The dams were designed so that no significant damage, that is no damage that would prevent the continued operation of the dam, would be sustained in the event of an OBE. In practical terms, this criterion means that the tensile stresses resulting from an OBE are less than the tensile strength of the concrete by a factor of 1.5. In the event of an MCE the dams have been designed so that there would be no catastrophic release of water from the reservoir resulting from damage sustained by the dam.

The dams were analysed by the spectral response method, with the typical dam profile shown in Fig. 2 and the response spectra shown in Fig. 3. The material properties used in the analysis are presented in Table 4.

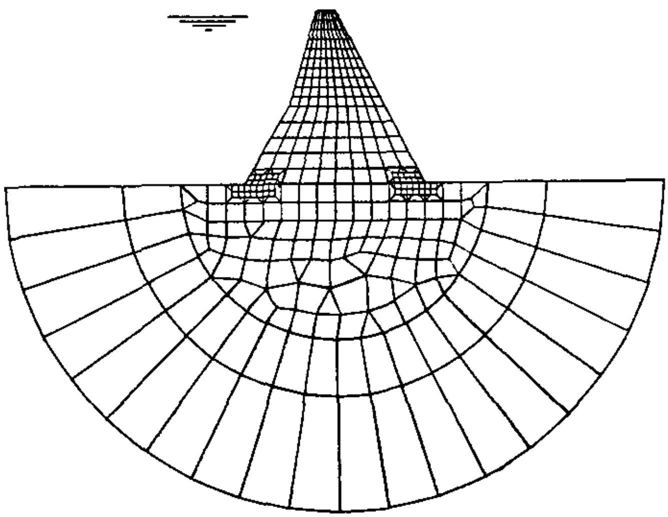


Fig. 2: Typical Dam Profile

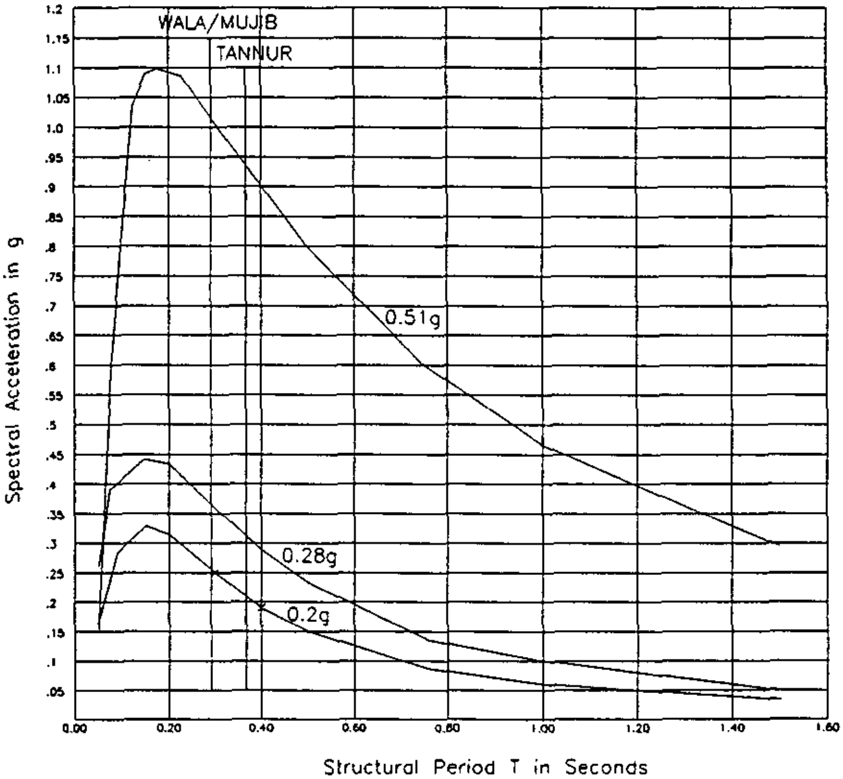


Fig. 3: Response Spectra

Table 4. Material Properties

Property	Concrete	Foundation		
		Wala	Mujib	Tannur
Compressive strength (MPa)	20	9	12	15
Dynamic modulus (GPa)	24	7.5	13.8	11
Poisson's ratio	0.2	0.2	0.2	0.3

The dam profile is unusual for a gravity dam in that it is nearly symmetrical about its axis. This feature reflects the dominating influence of the seismic loading in the stress distribution, and the relatively small influence of the water load.

RCC DESIGN.

The broad objectives of the RCC design are to produce a concrete that possesses the most appropriate combination of density, impermeability, strength, low heat of hydration and handle ability, all at minimum cost. The strength and impermeability objectives both imply the adoption of a high paste content, while the requirement for a low heat of hydration implies the use of a high proportion of pozzolan within the cementitious content.

The materials available for RCC manufacturer are as follows:

cement:
- OPC produced in the south of Jordan
- OPC with variable content (15-20%) natural pozzolan produced to the north of Amman

pozzolan:
- flyash, imported from either South Africa or Australia through the port of Aqaba
- natural pozzolan available 100 km to the east of Amman

aggregates:
- Wala dam - crushed limestone and crushed wadi bed gravel
- Mujib dam - crushed basalt and crushed wadi bed gravel
- Tannur dam - crushed limestone

Tests have been carried out with both flyash and the indigenous natural pozzolan and both are satisfactory. However, there are no facilities at present in Jordan for the milling of pozzolan and Tenderers for the construction contract will be given the choice between the two materials. At Wala dam the limestone is relatively weak and difficult to quarry and it is proposed to use the wadi bed gravel, despite its high silicon content. ASR problems will be avoided by the use of the pozzolan which keeps the alkalinity of the total mix below the critical level. At Mujib Tenderers will be given the choice of aggregate source.

A test programme was carried out on a wide range of material combinations at Kirton Concrete Services in the UK. A second phase of testing will be necessary early in the construction contract to optimise the mix design. The results of the principal tests on the tender design mixes are presented in Table 5.

Table 5. Concrete test results

Dam	Aggregate	Strength (MPa)		Modulus (GPa)
		Compressive	Tensile	
Wala	Limestone / pozzolan	16.5	0.89	18.9
Mujib	Basalt / flyash	19.0	1.23	24.5
Tannur	Limestone / flyash	18.7	1.32	36.5

In all the above mixes the cementitious content was 200 kg/m^3 with 50% OPC cement and 50% flyash. The target VeBe time was in the range 12-20 seconds. Substitution of the flyash by pozzolan reduced early strengths by 20% but it is expected that the long term strength will be similar. Further development and testing of concrete mixes is required for Wala dam in order to achieve the target tensile strengths.

THERMAL ANALYSIS.

From the thermal analyses the following provisions are necessary to prevent uncontrolled thermal cracking:

- limitation of concrete placing temperatures to 20°C, which implies concreting in the winter months (October to April) and at night
- placing vertical joints at 15 m centres

With these provisions, with a concrete in which the cement content is limited to 100 kg per m^3 it is estimated that the maximum temperature of the concrete in the centre of the dams at the end of placing will reach 40°C, and the joint opening will be less than 3-4 mm.

SPILLWAY.

For each dam the spillways are designed to pass the probable maximum flood (PMF), with optimum hydraulic performance set at the 1:10,000 year flood. At each spillway the specific discharge is limited to 20 m^3/s/m to achieve maximum energy dissipation by means of the stepped spillway, thus enabling the stilling basin length, and cost, to be minimised.

OUTLETS.

Each dam will be provided with high capacity bottom outlets controlled by roller gates to minimise the accumulation of silt, at least in the vicinity of the demand outlets. Operational outlets are provided at several levels and comprise piped intakes strapped to the upstream face of the dam - no separate structure is required.

ACKNOWLEDGEMENTS.

The author would like to thank the Director General of the Jordan Valley Authority for permission to publish this paper and to his staff, especially Mr Ziad Shehade for their co-operation and support throughout the period of study and design. Thanks are also due to the author's colleagues and to Dr Malcolm Dunstan, who together did most of the work.

REFERENCES.

Ambraseys, N.N. & Bommer, J (1991) "The Attenuation of ground accelerations in Europe", Journ. Earthq. Eng. Struct. dyn., Imperial College of Sci. London.

Barazangi, M. (1983). "A Summary of the Seismotectonics of the Arab Region". Assessment and of Earthquake risk in the Arab Region, Ed. Cidlinsky, K and M. Rouhan. Pages 43-77 UNESCO.

Joyner W.B. & Bloore D. M. (1981). "Peak horizontal acceleration and velocity from strongmotion records including records from the 1979 Imperial Valley, California, earthquake", bull. Seism. Soc. Am. vol. 71, 2011-2038

'Muela Dam : Design Aspects of Construction

M AIREY, Mott MacDonald Ltd, UK
R L PATTERSON, Mott MacDonald Ltd, UK

SYNOPSIS. 'Muela dam is a 55m high concrete arch dam that is currently under construction and due for completion in late 1996. During the construction phase, various design aspects have been considered in the light of actual conditions at site and taking into account the materials to be used in construction. These issues have impinged upon the construction activities and have included the development of a satisfactory concrete mix, the design and implementation of a post cooling system and the refinement of the arch geometry for the upper right abutment to suit geological conditions.

INTRODUCTION. The Lesotho Highlands Water Project is one of the largest water transfer projects in the world and construction of the first phase, involving dams and 80 kilometres of rock tunnelling, is now well advanced. Much has been written elsewhere about various aspects of the project including Katse dam which will impound the main storage reservoir and will be the highest dam in Africa. However the first phase also includes a second dam, situated at 'Muela, some 50 km north of Katse, where the hydropower component of the project is located. 'Muela reservoir forms the tailpond for the hydropower station and acts as a "break-pressure" point between the Transfer Tunnel which brings the water from Katse, and the Delivery Tunnel which passes it on to South Africa. It also provides the necessary back pressure on the turbines and serves as a regulating pond to balance the water supply requirements with the variable flow needed for power generation.

GENERAL DESCRIPTION OF THE DAM

'Muela dam is situated on the Nqoe river and is located where the river valley narrows to form a gorge. At this point the valley section is V shaped and has a width to height ratio of approximately 3 to 1. The dam is a double curvature concrete arch with both the intrados and extrados surfaces of the dam wall described by a series of horizontal parabolae. The overall layout of the dam and the crown cantilever section are shown in Figs. 1 and 2 respectively.

The valley profile is asymmetric with the right abutment being considerably steeper than the left. This has led to the adoption of a medium thickness arch with a significant degree of curvature in plan as illustrated in Fig. 1. The block width is nominally 12 m and provision is made for contraction joint grouting between the blocks so as to form a monolithic arch. The central blocks of the dam serve as a free overflow spillweir which discharges via a series of cascades to the stilling basin apron.

The reservoir as an asset. Thomas Telford, London, 1996

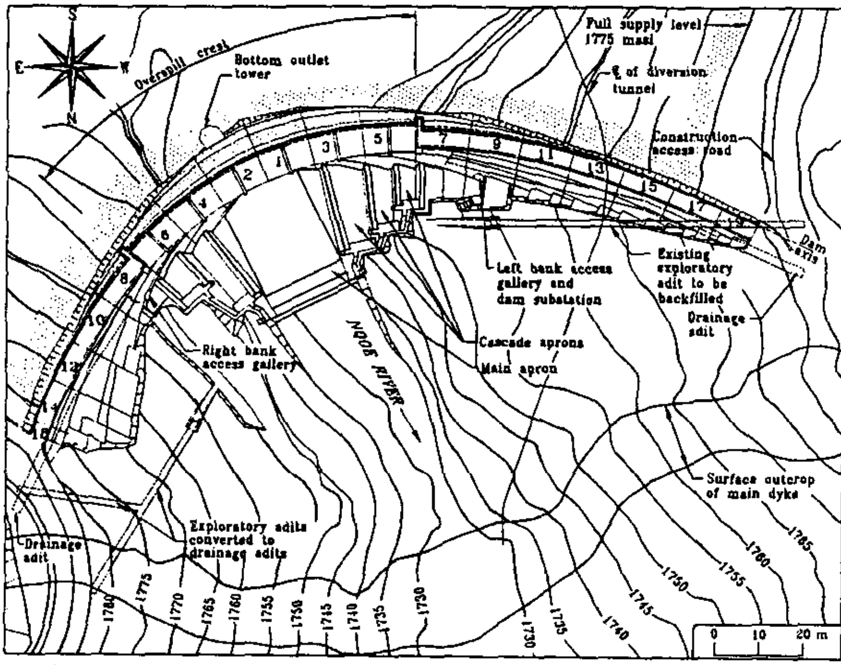


Fig. 1. General Arrangement of 'Muela Dam

The dam includes a perimeter gallery over the full length of the arch/foundation contact, as well as a mid height gallery for instrumentation monitoring. The gallery system also connects with abutment drainage adits. The bottom outlet works comprise a low level outlet culvert and compensation water release pipework. The low level outlet capacity is limited to 8 m³/sec which is sufficient to draw down the reservoir against natural inflow. The principal statistics of the dam and reservoir are given in Table 1.

Table 1. Principal Statistics of 'Muela Dam

Dam Type	Double Curvature Concrete Arch
Maximum Height above foundation	55 m
Base Thickness	15 m
Crest Length	200 m
Dam Crest Level	1777.5 masl
Volume of Concrete in Arch	63,000 m^3
Volume of Reservoir	6 Mm^3
Reservoir Area	35 ha
Probable Maximum Flood	584 m^3/sec
Spillweir Crest Length	72 m

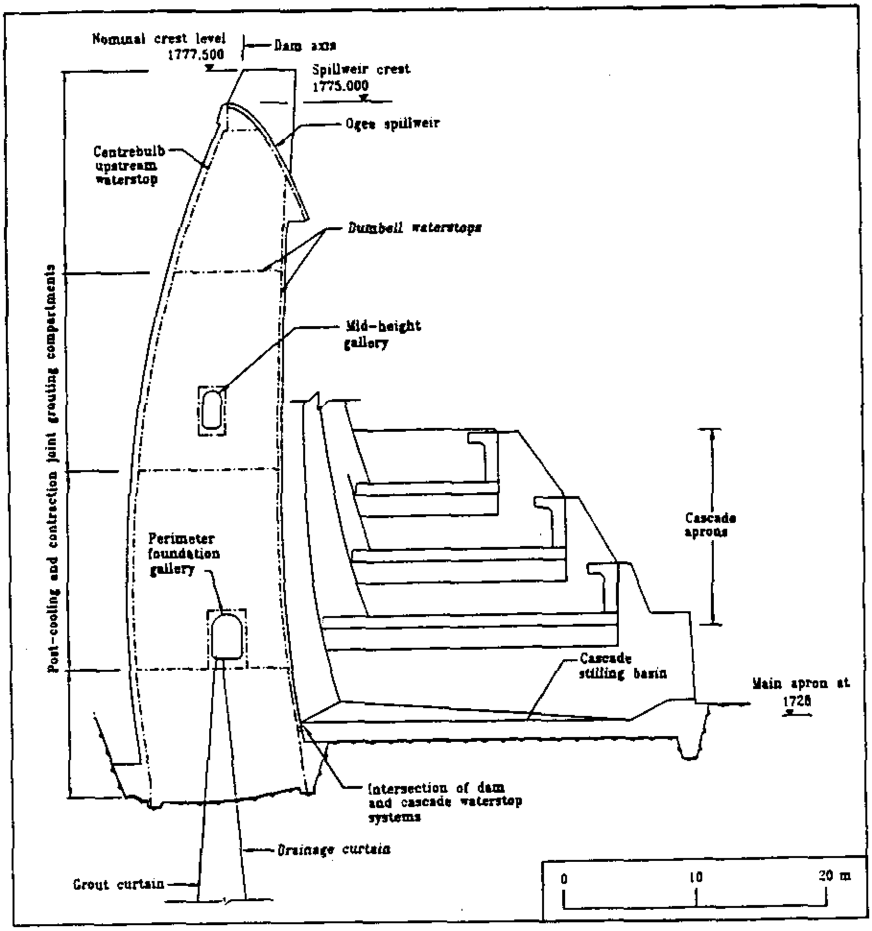


Fig. 2. Section through Crown Cantilever and Stilling Basin

The dam is founded on massive, sound sandstone of the Clarens formation. The sandstone extends below the valley floor to a depth of around 35 m, where there is some siltstone/mudstone bedding overlying further sandstone. The Clarens Sandstone is an intercalation of dune sandstone and aquatic sandstone without discontinuities between the layers. It has a narrow gradation typically between 0.10 and 0.30 mm and the rock mass exhibits few discontinuities. Unconfined compressive strengths of the rock, as determined from point load tests, range between 25 and 60 MPa. The permeability is generally low, varying between 10^{-7} and 10^{-9} m/sec and is governed by the porosity of the rock rather than by any distinct joint pattern.

Taking into account the overall geology, a single line grout curtain has been adopted which is inclined marginally upstream. Where isolated tectonic joints traverse the dam footprint, the grout holes are specifically aligned to intersect the joint. Consolidation grouting is undertaken over the full width of the dam footprint on a nominal grid of holes at 3 m spacing to a depth of 5 m. All grouting is carried out in accordance with the GIN (Grouting Intensity Number) method as proposed by Lombardi & Deere (1993). For curtain grouting a GIN value of 1500 bar.litres/m was adopted with a maximum pressure of 30 bars, whilst for consolidation grouting the GIN limit was 1000 bar.litres/m with a 10 bar maximum pressure. Except at tectonic joint locations and in the upper abutments, grout takes were generally low with the maximum pressure rather than GIN value or volume being the limiting factor.

Apart from tectonic joints, there existed stress relief joints running more or less parallel with the original valley profile. These were formed due to the erosion during geological times of up to 3 km depth of basalt lava flows. Near to the rock surface these joints were usually open, but at depth both the opening of these joints and their persistence reduced significantly. With the excavation of the foundation, all open stress relief joints were removed and those short stress relief joints that were still evident at the rock/arch contact were shown to be completely tight.

CONSTRUCTION PHASE

During the construction phase certain design issues needed to be considered in the light of actual conditions encountered, details are given below.

<u>Stability of Upper Right Abutment</u>

The geology of the upper right abutment is somewhat more complex than that of the dam site as a whole. An important feature is a principal bedding plane at 1754 masl, approximately 20 m below dam crest level. In addition, a major dyke crosses the valley just downstream of the site and several doleritic intrusions are associated with the dyke. One sill in particular has disturbed the abutment significantly through intrusive action along the horizontal bedding plane. It was investigated by exploratory adits and found to be highly weathered near to the surface and more significantly, a clay layer arising from the degradation of the dolerite was identified at the upper interface between the sill and the sandstone. Stabilisation measures for the upper right abutment were required to prevent any sliding along this thin clay layer and two options were considered.

In the first instance a smooth arch profile was designed to be founded on infill concrete which in turn was founded below the sill as shown in Figs. 3 and 4. Stabilisation measures were designed as a grid of shear keys formed by modifying and extending the exploratory adits. General consolidation grouting of the abutment would be carried out to ensure the integral action of the rock mass above the clay layer. However, full exposure of the abutment showed that the zone above the sill was highly fractured and that clay infill of the joints meant that effective grouting of the rock to transmit the arch forces to the shear keys could not be guaranteed. It was decided instead to take the arch itself down below the sill to be founded on the competent sandstone below the bedding plane. This increased the height of the cantilevers at this section and necessitated a modification to the geometry of the downstream face of the arch which achieves a near 'gravity' section towards the end of the abutment.The widening of the profile required further excavation downstream of Blocks 10,12 and 14. The extent of the additional excavation is indicated on the downstream elevation included as Fig.3. A comparison between the original arch section and the gravity section that was adopted is shown on Fig.4, whilst Fig. 5 shows the resulting layout of the whole right abutment area.

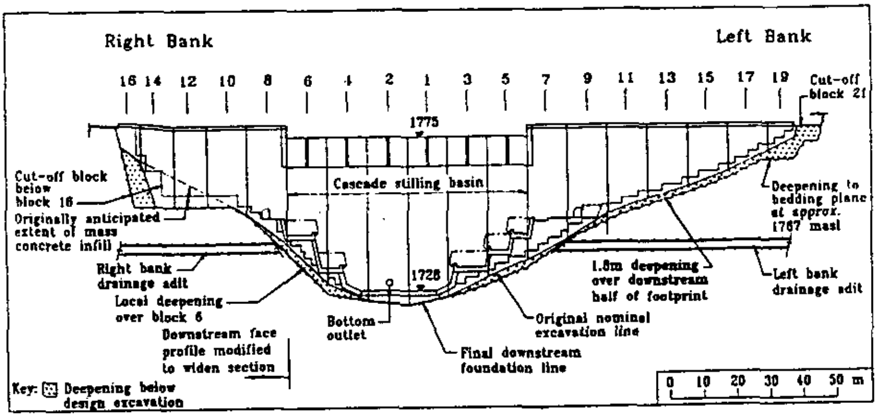


Fig. 3. Downstream Elevation

The effect of the change was to re-direct the abutment thrusts downwards and into the solid rock with no reliance on the rock mass downstream. This was confirmed by a final stress analysis that was undertaken to reassess the performance of the arch, taking account of the "as excavated" foundation profile and the revised geometry of the right abutment . The analysis was done using the LUSAS finite element system and incorporated revised material properties to reflect the "as excavated" conditions for the rock and the actual concrete properties derived from laboratory trials. With this solution there was no longer a need to backfill the exploratory adits to form shear keys. Instead, the adits were retained to provide additional drainage.

The additional excavation was carried out by drill and blast methods. Due to the fractured nature of the rock, single blasts were restricted to 1.5 m high benches with low charge rates, typically around 0.5 kg/m^3. Smooth perimeter blasting was also employed to preserve the integrity of the rock faces. Isolated rock reinforcement measures were instructed as the work progressed. These included mesh reinforced shotcrete as protection to exposed faces together with local rock bolting of any overhanging slabs above open stress relief joints. One large rock wedge situated above the stilling basin was secured by a series of 9 m long rock anchors each with a working load of 280 kN.

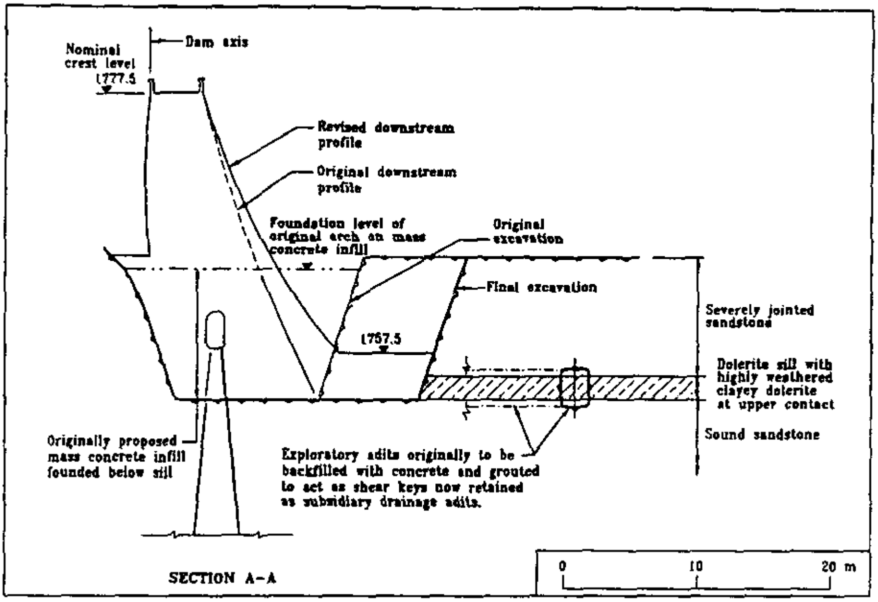


Fig. 4. Modified Right Abutment Section

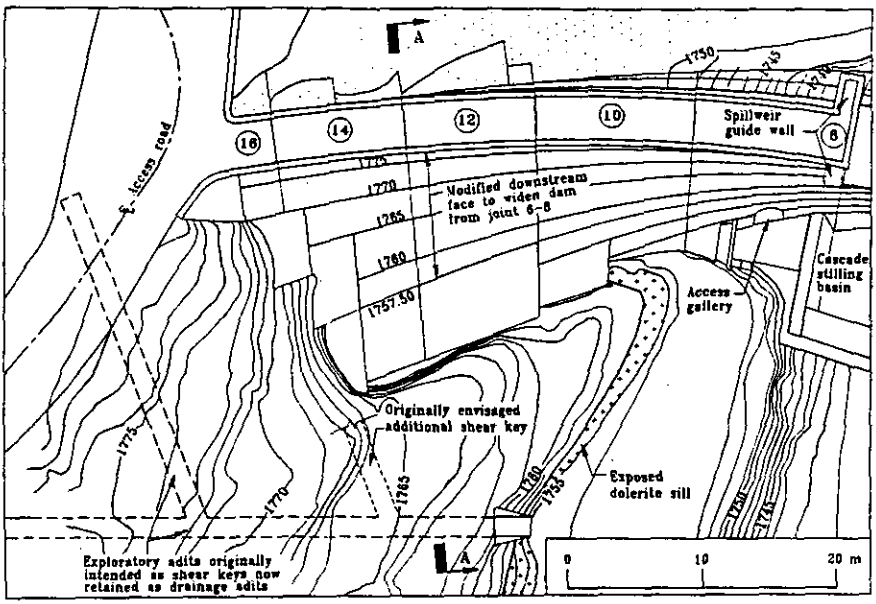


Fig. 5. Plan of Right Abutment

Concrete Mix Design

A Class 25/75 concrete was specified for the mass concrete of the dam; that is a characteristic strength of 25 MPa and a maximum aggregate size of 75mm. A comprehensive trial mix programme was initiated to produce a mix to meet the objectives of long term strength and durability, minimum heat of hydration and adequate workability. There was no particular need for early strength and thus the characteristic strength was specified at 90 days rather than the usual 28 days. Because of the large aggregates used the compressive strength was determined from 300 mm cubes.

A key factor in any mix design is the aggregate source. For 'Muela the aggregates were to be taken from a dedicated quarry situated approximately 1 km from the dam where an intrusive dolerite body rises out of the surrounding sandstone. Investigations indicated a homogeneous mass of fresh dolerite that was ideal for aggregate production. An extensive quarrying, crushing and screening operation was established to produce 5 to 12 mm and 12 to 25 mm graded aggregates together with 40 mm and 75 mm single size aggregates. Two crushed sands were also produced, one referred to as 'tailings sand' was a by-product of aggregate screening whereas the second, referred to as 'pump sand', was a finer, milled sand produced by a sand plant.

In the first phase of mix design a large number of mixes were produced in the laboratory and the following basic parameters were established :-

- Initial cementitious content = 240 kg/m^3
- OPC to PFA = 60 : 40
- Water cement ratio < 0.50
- Target air content = 4% ± 0.5%
- Slump = 40 to 60 mm

Using these parameters trial mixes were prepared to determine the ideal sand content; that is sufficient sand to give mechanical lubrication but not so much that the mix is "oversanded" which impairs workability. An ideal sand content of around 25% was determined. The aggregates were then proportioned in accordance with the method proposed by Fontoura et al , so that the combined grading curve was as close as possible to the geometric grading. This is a continuous granulometric grading curve where the percentage retained on successive sieves follows a geometric progression. Such gradings are considered to be ideal as there are no excesses of any size that may cause particle interference. For a geometric grading of 70% it was found that the fineness modulus of the total aggregate should be around 7.0 with that of the sand fraction less than 3.0 to avoid segregation and bleeding.

The final stage of mix development involved full scale trials with the batching plant, where the most satisfactory mixes were considered further. At this stage the emphasis was on reducing the total cementitious content as much as possible without adversely affecting the strength or more significantly the workability. It was found that if a blend of tailings sand and pump sand were used, a workable mix could be retained at lower cement contents without increasing the water/cement ratio. Details of the final mix adopted for the majority of concreting are given in Table 2.

Table 2. Concrete Mix Details

Description		kg/m^3
Ordinary Portland Cement	(60%)	132
Fly Ash	(40%)	88
75 mm aggregate		603
40 mm aggregate		460
12/25 mm aggregate		396
5/12 mm aggregate		281
Tailings Sand	(40%)	225
Sand Plant Sand	(60%)	350
Water		110 litres
Air Entrainer (AER 380)		1.5 litres
Water Reducing Agent (WRA P211)		0.77 litres

Post Cooling and Contraction Joint Grouting

Artificial cooling of the dam concrete was required so that a contraction joint grouting temperature of 11°C could be achieved within the overall construction programme. Thus a post cooling system was introduced, the design of which considered a number of factors including concrete thermal properties, seasonal temperature variations, the concreting programme and the timing of contraction joint grouting. Contraction joint grouting of each joint was to be carried out in four separate 15 m high compartments as indicated on Fig. 2.

The completion of post cooling of all blocks within a compartment was required before contraction joint grouting of the compartment could go ahead. To achieve the overall completion date it was essential that cooling and grouting of the lower compartments should run concurrently with the concreting of the upper part of the dam. Taking into account previous experience and the practical implications of installation, the following parameters were fixed :-

- One number cooling coil per concrete lift (2.5 m vertical spacing)
- Nominal coil spacing of 1.5 m
- Cooling coils to be formed from thin walled steel pipes (22 mm dia)
- Chilled water cooling temperature = 5°C

Based on these factors a study was carried out to investigate the post cooling response of concrete lifts and to predict time - temperature histories. From the results, design charts were developed to show anticipated cooling down periods for different sized blocks with different placing temperatures taking due account of any natural cooling. A nominal cooling water flow rate of 0.3cm³/sec/m³ of concrete was adopted which gave a maximum cooling rate of around 0.5°C per day. Total cooling periods for a particular block were between 3 and 5 months depending upon the size of the block and the time of year.

The cooling system itself consisted of a separate riser pipe for a group of coils within each grouting compartment of each block. Typically a riser pipe fed six coils with return lines provided for each coil. Both the feed and return pipes exited the dam at box outs on the downstream face. Control and monitoring of the system was carried out from downstream walkways provided at each compartment level. A single chiller unit was installed to supply the system,with separate feed and return circuits serving each compartment. The schematic arrangement for a typical compartment is shown on Fig 6.

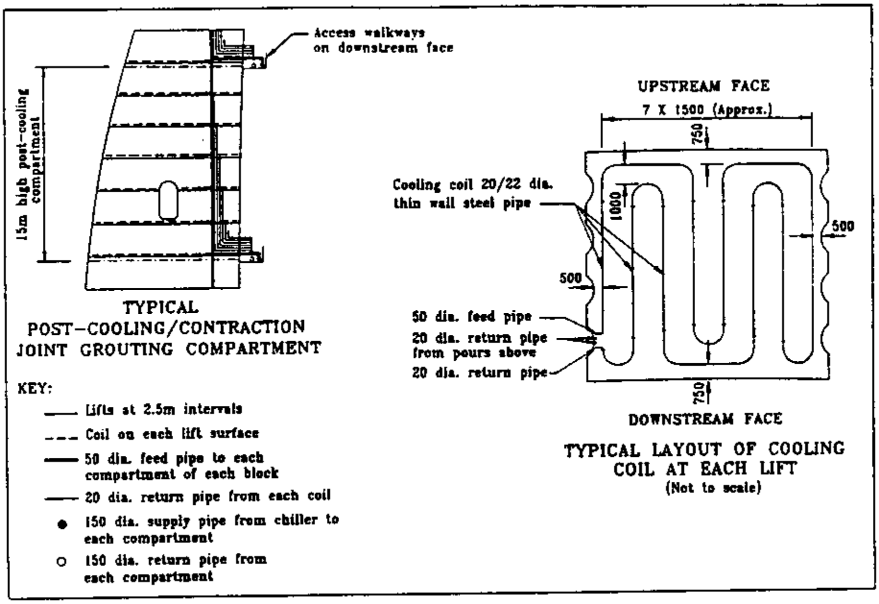


Fig.6. Schematic Arrangement of Post Cooling Coils

The strategy of the post cooling implementation was to limit temperature differentials and so ensure a crack free concrete. In particular the following criteria were applied :-

- Control of the temperature differential between the existing lift and the maximum temperature of the next pour.
- Temperature differentials within an individual lift not to exceed 20°C.
- Temperature difference between concrete and cooling water not to exceed 30°C.

An important aspect of post cooling was the operation and monitoring of the system. To check the actual performance and to assess cooling rates, a comprehensive monitoring schedule was developed. This included regular measurement of flows from each coil and adjustment of flows as necessary together with the measurement of exit water temperatures. In addition, concrete temperatures were checked throughout the cooling period. These were measured directly by embedded thermocouple sensors and also by means of the 'stagnant water' temperature within the coil. The embedded thermocouples formed part of the overall instrumentation system and will be used to monitor the long term thermal behaviour of the dam.

Contraction grouting of each compartment was one of the final construction activities. A grouting system based on reinjectable valves was adopted with a valve spacing of 1 per 10m² of joint.The valves were positioned on the joint surface of 'leading' blocks prior to concreting the 'infill' blocks. Generally the valves were at mid height of each lift with all grouting pipework routed either to the galleries or to the dam crest thus providing access for subsequent re-grouting should it ever be necessary. A separate grout flushing circuit was included at the top of each panel.

CONCLUSIONS

The design issues referred to have been addressed at various times throughout the construction period. This has required close liaison between the Engineer's site staff and the Contractor to ensure the development of technically viable solutions whilst minimising any adverse effects on the construction programme. The dam is scheduled for completion in late 1996.

REFERENCES

Fontoura J T F, Monteiro P J, Bittencourt R M & Pacelli W. A Mix Design Method for Mass Concrete based on the Fineness Modulus and Geometrical Gradation. American Concrete Institute,Materials Journal.

Lombardi G. & Deere D. (1993). Grouting Design and Control using the GIN principle. Water Power and Dam Construction, June 1993, pp 15 - 22.

Meriden No 2 Reservoir - Renovation Works

G FEAKES, Sir Alexander Gibb & Partners Ltd, UK
D N WILLIAMS, Severn Trent Water Ltd, UK

SYNOPSIS. Meriden No 2 Service Reservoir was built in 1949 at a time of national steel shortage thus maximising the use of mass concrete in its construction. During inspection in 1979 the structure was found to exhibit severe cracking to the external and division walls. This paper describes the investigations and design audit that ensued and the remedial works undertaken to renovate and partially reconstruct the reservoir.

INTRODUCTION. The reservoir of capacity 45 Ml has overall plan dimensions of 77 m x 81 m. It was constructed primarily from mass concrete, the only locations where reinforcement had been used was in the cantilevered groins and tie beams supporting a barrel vaulted roof.

A tapered division wall, supported on each side with triangular buttresses, formed two equal compartments. External walls parallel to the division wall had a thickness of 1.830 m at the base tapering from mid height to 0.6 m at the top. The upper 2.0 m of the 8.2 m high wall formed an edge beam, which connected to the column groins with tie beams. The walls perpendicular to the division wall were of identical size to the parallel walls except they were buttressed internally to full height.

Columns were of mass concrete 0.71 m square. The asphalt covered floor was 0.375 m thick and unreinforced.

Fig. 1 shows a plan of the reservoir.

INVESTIGATIONS AND DESIGN AUDIT.
By 1987, cracking to the external, division walls and columns had substantially worsened. This coupled with observed leakage prompted further investigation.

Lateral earth pressures and stability of the supporting earth embankment was investigated. Alkali Silica Reaction (ASR) was known to be active. Cores were taken to try to establish the contribution that ASR had made in the development of the cracking and its potential for further expansion. A

The reservoir as an asset. Thomas Telford, London, 1996

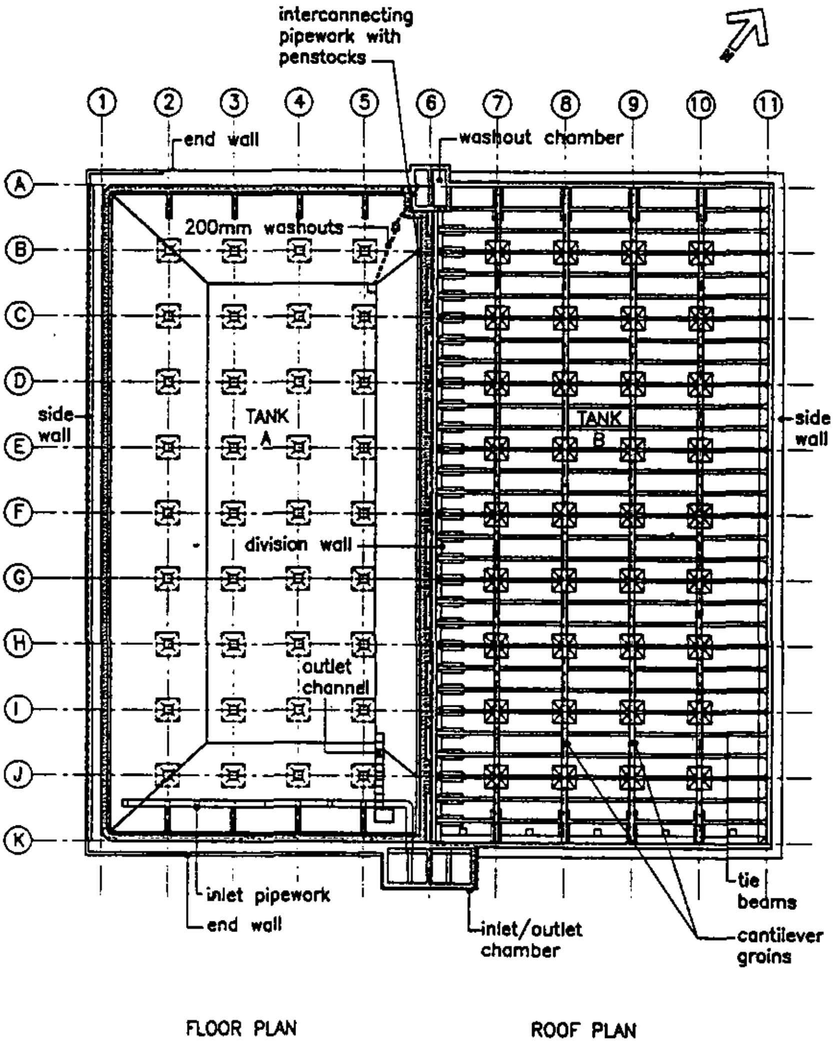


Fig. 1. RESERVOIR PLAN

design audit provided a check on the stability and structural adequacy of the concrete elements.

FINDINGS

For conditions representing the worst case that could be anticipated, the embankments were considered to be approaching limiting equilibrium. However, it was concluded that as no obvious signs of instability have become apparent that some effective cohesion may be at work within the soil and the assumed ground water conditions may not occur in practice. A geotechnical report gave effective stress parameters which when applied to an assessment of the stability of the wall showed that theoretical tensile stresses exceeded the permissible by approximately a factor of 2.

The stability of the division wall, with full water pressure on one side relied on the integrity of the concrete. With extensive cracking in the buttressing the factor of safety against overturning reduced to a value below 2. There was also a potential for shear failure across the horizontal cracks.

Trial pits in the embankment showed the cracks to extend through the body of the wall with leakage observed. Saturation of the clay was inevitable whilst reverse flow provided a means of contamination of the stored water.

The structural integrity of the floor, roof, tie beams and cantilever groins were found to be satisfactory.

CAUSE OF CRACKING

The investigations showed that the cause of the cracking was more complex than first envisaged. Contributing factors include:

i) Early thermal shrinkage.
ii) Thermal expansion and contraction induced by the roof, tie beams, cantilever groins and the walls as no provision was made to accommodate such movement. This was borne out by the concentration of cracking at the corners where complex stress patterns were set up.
iii) The increase in external soil loading due to saturation of the clay.
iv) Alkali Silica Reaction. The contribution ASR had in the development of cracking was difficult to estimate. It was known however that sufficient alkali and reactive aggregate existed for the reaction to continue.

The position was therefore reached where it was considered that the external walls were so extensively cracked that the concrete had little effective tensile properties. The buttresses no longer acted as an integral part of the structure and the potential existed for the external loads on the walls to cause displacement when the reservoir was empty and particularly after rapid drawdown.

A further key factor in the decision to carry out remedial works was the need to protect the water quality from contamination.

The investigative work undertaken resulted in the submission in June 1988 of alternative solutions to reinstate the reservoir. The proposals addressed the options of either renovation or total rebuild.

OPTIONS.

The renovation option was based on the retention of the existing structure in its entirety with the construction of new internal reinforced concrete retaining walls. This would enable the roof and its supporting structures to be retained without the necessity of major temporary propping and shoring works. The new reinforced concrete walls would utilise the existing mass walls as permanent shuttering but residual strength of the mass walls would be ignored. The division wall, which for many years had acted only as a baffle was required to be reinstated to its true purpose and therefore a similar approach was adopted with the introduction of new reinforced concrete retaining walls on either side of the existing division wall.

The option of renovation would allow the reservoir to be brought back into service in the least possible time and at the least capital cost. An advantage was that the majority of the construction works could be carried out in the "dry". However a disadvantage was that the life of the renovated reservoir was likely to be considerably less than an 80 year life expectancy from a totally rebuilt reservoir.

Even though total rebuild would provide a greater asset life the renovation option was adopted because of lower cost and shorter construction time.

RENOVATION WORKS.

A detailed design development submission was made in November 1988 which outlined the full scope of the renovation works. In addition to the construction of the new reinforced concrete walls and division wall it was considered that the following works would be necessary to meet the statutory requirements prior to bringing the reservoir into service. The existing columns, plinths and buttresses were to be rehabilitated by the application of mesh reinforcement and an additional concrete protection. The cantilevered groins and tie beams were to be repaired locally as necessary and coated with a non-toxic water tolerant epoxy coating.

The design of the new reinforced concrete retaining walls was dictated by the retention of the existing mass walls. The new walls therefore had to take an "L" shaped profile with an extended base as shown on Figs. 2, 3 and 4. Consequently a large area of the floor would be reinforced by the horizontal bases of the new walls, the remainder of the floor would therefore be overlaid with a 0.3 m thick reinforced concrete slab, constructed in panels with the joints waterproofed with rearguard waterstops and surface sealant. Additionally inclined steel dowels were set in the existing floor in order to provide bond between the new construction and the retained floor, thereby resisting the hydrostatic pressures which would be exerted when the reservoir was drained for maintenance and inspection.

Previous investigations had indicated the problem of potential water leakage through the valleys of the vaulted roof. It was therefore necessary to remove the overburden of soil and the existing drainage system, prepare the exposed concrete surface, apply a waterproof membrane, reinstate the drainage system and re-soil and sow with grass seed.

The construction of substantial retaining walls, bases and floors within the reservoir would reduce the available storage volume by some 10 per cent; however this was considered acceptable in meeting service requirements.

DETAILED DESIGN.

The detailed structural designs were carried out in accordance with the British Standards current at the time and in particular BS 8110 Structural use of Concrete and BS 8007 Design of Concrete Structures for Retaining Aqueous Liquids. The design of all new structural elements was carried out in accordance with the requirements of the control limit states with further checks for the serviceability limit states for cracking and deflection. The design crack width was limited to 0.2 mm. To ensure durability and a life expectancy for the new structural elements of the order of 60 to 80 years as defined in BS 8007 the characteristic strength of the concrete was specified as 35 N/sq.mm with a minimum cement content of 325 kg/cubic metre of concrete and the requirement of a nominal cover to all reinforcement of 40 mm. The reinforcement was specified as type 2 high yield deformed bars with a design strength of 460 N/sq.mm.

The structure was checked against flotation for a groundwater pressure of 2 metres head of water exerted on the underside of the existing floor slab.

Factors of safety for overturning and sliding were to be not less than 2.0 and 1.25 respectively under any possible combination of adverse loading.

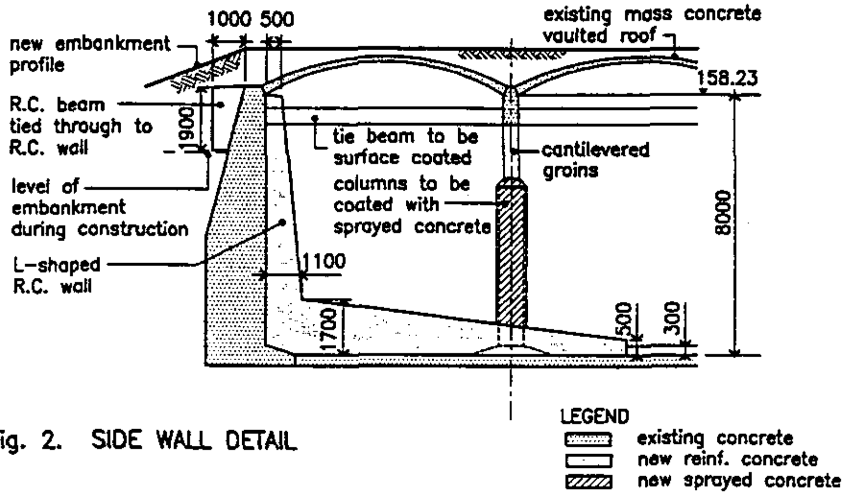


Fig. 2. SIDE WALL DETAIL

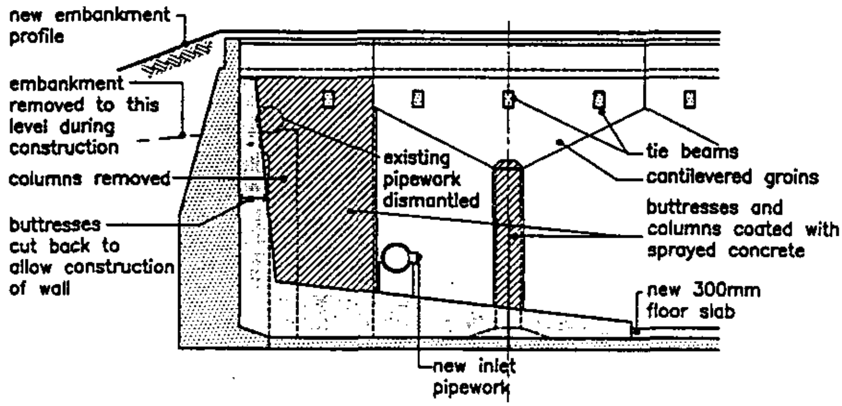


Fig. 3. END WALL DETAIL

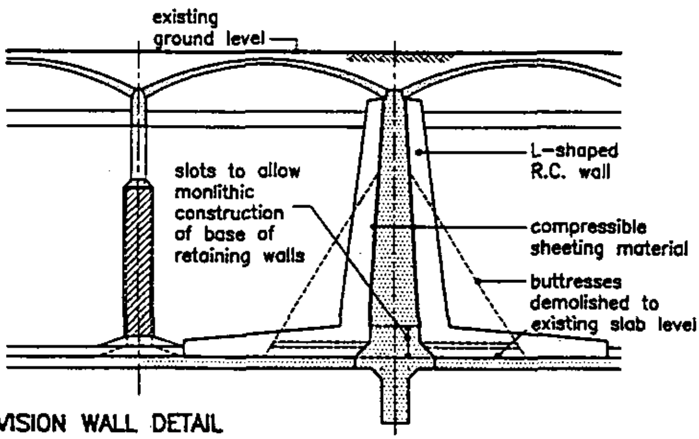


Fig. 4. DIVISION WALL DETAIL

In addition the design had to consider the interaction between the retained and the new reinforced concrete structure. It was assumed that the existing walls and slabs were able to transmit external soil and water pressures to the new internal elements but any load carrying capacity of the existing walls was ignored. It was however considered that the existing floor slab could safely transmit vertical loads to the substrata. For the reservoir walls it was assumed that any expansive forces generated due to the alkali silica reaction in the existing mass walls would be dispersed towards the earth side and no horizontal forces due to this action would be considered as acting on the new reinforced concrete walls. As the proposed refurbishment of the division wall comprised the construction of new walls on either side, provision to accommodate potential expansive forces from the alkali silica reaction was required. A separation membrane comprising a compressible filler board was applied full height to both sides of the division wall prior to construction of the new reinforced concrete walls. See Fig. 4. The height of the concrete pours to the new walls were restricted to approximately one metre such that the deformation of the compressible filler board was limited to some 15 per cent of its original thickness for each lift of concrete poured. Any expansive forces from the existing division wall would therefore be absorbed by further compression of the filler board and not transferred to the new construction.

As the central wall was now required to act as a division wall it was necessary, to meet the required factors of safety, to ensure that the two new walls acted as one unit. This was achieved by forming slots through the existing wall at floor level and casting the new reinforced concrete retaining wall bases monolithically as shown in Fig. 4.

As the design of the new internal walls assumed that the existing mass concrete had no residual design strength it was therefore necessary to investigate the conditions at the tops of the side walls where the tie beams resisting the outward thrusts from the vaulted roof connected to the top of the existing mass walls. It was decided to construct externally, at the level of the springing of the vaulted roof and the tie beams, a reinforced concrete edge beam tied through the mass walls to the new internal reinforced concrete walls thus ensuring all roof horizontal and vertical loads and forces were effectively transmitted to the new structural elements. The edge beam is illustrated in Fig. 2.

ANCILLARY STRUCTURES AND PIPEWORK.

Two control buildings were located on Site being the inlet/outlet valve house and the overflow/washout valve house. Both structures were single storey comprising a reinforced concrete roof slab supported on load bearing concrete block walls. Whilst the structures were found to be in reasonably sound condition both buildings showed signs of leakage through the roof.

To bring these buildings up to an acceptable standard waterproofing systems to the flat roofs were removed and a new membrane installed.

During the renovation works it was necessary to remove the existing inlet pipework at high level adjacent to the end wall. To improve operational conditions it was proposed that the new inlet pipework would be installed on plinths at low level adjacent to the end wall but clear of the line of the retained buttresses as indicated in Fig. 3. All the existing large diameter pipework was dismantled, refurbished and reused as necessary to form the new inlet. The two existing bellmouth pieces forming the outlet were extended upwards by 0.3 m so that they were at the level of the new floor. The existing pipework formed the unsatisfactory configuration of the inlet and outlet being adjacent to each other. But under normal operation water would be supplied into 'Tank A' through the new low level pipework and delivered into supply through the outlet culvert in 'Tank B'. Thus circulation would be achieved in the tanks with flow occurring through the interconnecting pipework in the division wall. However, it was considered operationally acceptable that with one tank empty and for short periods only, circulation would be limited.

The two existing 0.3 m diameter washout pipes were extended horizontally and upwards by 0.3 m. Additionally sumps were required in the new reinforced concrete bases with 0.2 m diameter pipelines installed within the new construction to connect the low areas of the tanks to the new sumps. To maintain operational water levels to those previously in use the existing weir overflows were modified to accommodate the new walls which were constructed against the washout chamber walls. Due to the works to the division wall the existing interconnecting pipework was redundant. A new 0.8 m diameter interconnecting pipe with spindle operated penstocks on each end was installed.

To improve access to each tank the existing access manholes were enlarged and access afforded by new ladders serving an intermediate level platform.

CONSTRUCTION.

Following initial mobilisation excavation works progressed to reduce the external ground levels around the perimeter of the reservoir to the construction levels shown on Figs. 2 and 3. Access to the interior of the existing structure was effected by further excavation at the end wall to form ramps down to the reservoir floor level. Access was then made by breaking out openings 3.5 m x 7 m in the end wall between buttresses at two locations to gain entry to each compartment. The programme duration was 26 weeks and was such that the construction period would occur during the winter months. Most of the major new works were undertaken under cover of the existing retained roof. Therefore a critical operation was to complete all external works and to waterproof the roof and resoil at the earliest opportunity. Due to the restrictions imposed on the type and operational loading of plant and equipment on the existing roof the excavation and removal of the soil covering was undertaken with small earth removing equipment that would not have been out of place in a suburban back garden. This operation proved to be painstakingly slow and totally subjected to the prevailing weather conditions.

The works had been planned so that there was a minimum amount of demolition and breaking out of the retained structure. This enabled the Contractor to open up work on several fronts simultaneously. The design of the retaining walls and the arrangement and location of joints had been standardised allowing the Contractor to obtain maximum reuse of shutters. Concrete was delivered by pump and construction of the new reinforced concrete retaining walls at all locations progressed rapidly. However, environmental conditions within the reservoir deteriorated during the operations of breaking out the slots in the division wall and drilling for the necessary dowels to floor, columns and buttresses to receive the additional protective and structural concrete.

At this time initial tests were underway on the application of the sprayed concrete protection to the columns and buttresses, an operation which further aggravated conditions within the reservoir and restricted production.

A detailed inspection of all the tie beams had been carried out once access was available. The survey showed that the extent of repair works was more than originally anticipated and that the refurbishment of these tie beams was becoming critical as the application of the necessary treatment and sealer could not be progressed at the required rate due to the dusty conditions within the reservoir. Therefore a forced air ventilation system was installed which considerably improved conditions and allowed operations to progress at an increased rate necessary to meet the tight programme.

The application of the sprayed concrete to the columns caused particular difficulties. Rebound was approaching unacceptable levels of over 40 per cent and the decision was made to revert to conventionally vibrated concrete placed within shutters, but retaining the dowel connectors and mesh reinforcement. To achieve acceptable workability, placement and compaction the concrete mix was redesigned using 12 mm maximum aggregate for the thin concrete sections in place of the 20 mm aggregate specified and used for the main structural concrete.

Following resolution of these initial teething problems construction works progressed satisfactorily leading to testing and commissioning during June 1989.

CONCLUSION.

Adopting the renovation option retained sound structural elements whilst returning the reservoir to service in the shortest possible time.

The design development demonstrated that for construction works to be successful as much detailed information as possible needs to be provided at Tender Stage. Construction restraints were identified which were drawn to the Contractors attention and full consideration and allowance was made in buildability, pricing and programming.

The renovation works were therefore completed within budget, in a period of only 26 weeks and at a cost of £2 million.

ACKNOWLEDGEMENTS.

The authors gratefully acknowledge the permission of both Severn Trent Water Ltd and Sir Alexander Gibb & Partners Ltd to publish this paper.

REFERENCES.

Meriden No 2 Service Reservoir - Report on Inspection and Remedial Works - February 1988 - Sir Herbert Humphries & McDonald.

Report of an Inspection under Section 10 of the Reservoirs Act 1975 - Mr J R Dowell, June 1988.

Meriden No 2 Service Reservoir, Renovation/Reconstruction Works - Design Stage I Report, June 1988 - Sir Alexander Gibb & Partners.

Meriden No 2 Service Reservoir, Renovation/Reconstruction Works - Design Stage II Report, November 1988 - Sir Alexander Gibb & Partners.

Evolution of Service Reservoir Design and Construction in the Loch Lomond Water Supply Scheme.

R.B.BINNIE, Crouch Hogg Waterman Ltd., UK
I.C.MACDONALD, Crouch Hogg Waterman Ltd., UK
M.C.SWEENEY, Crouch Hogg Waterman Ltd., UK

SYNOPSIS
The design of service reservoirs constructed for the Loch Lomond Water Supply Scheme over a twenty five year period is reviewed. Refinements resulting from changes in design standards, materials development, improved construction techniques and operational and maintenance requirements are identified.

INTRODUCTION

Over the past ten years the Central Scotland Water Development Board has commissioned the second generation of raw and treated water service reservoirs for the Loch Lomond Water Supply Scheme.

The design of service reservoirs has evolved quite markedly since the construction of the first phase of the Loch Lomond Scheme in the late 1960's. We have had the opportunity to compare recent construction with structures virtually identical in layout and capacity but designed to the standards current at the time of their construction. The reservoirs are roofed, have a central partition wall, are generally 5 m deep and range in capacity from 22.7 to 45.5 Ml and in value from £2 to 4 million at current prices.

Advances have been made in the codes of practice for design, including adoption of serviceability limit state considerations in CP110/BS5337 and later in BS8110/BS8007. Design details have also been influenced by CIRIA, BRE and C & CA guidelines, new methods and materials, and perhaps most significantly, operational, safety and maintenance considerations, particularly with regard to access for maintenance.

Some of the materials adopted in recent years include galvanised and epoxy coated reinforcement, protective coatings to internal reservoir walls and columns, new forms of joint sealant and water bar, baffle curtains, lightweight access fabrications and new forms of perimeter and underfloor drainage.

The reservoir as an asset. Thomas Telford, London, 1996

Site specific considerations affecting design include ground conditions (both in terms of aggressive soil characteristics and in permissible bearing capacity), topography and the proximity of existing services. The design of the next generation of service reservoirs at Balmore Treatment Works is now complete with construction presently underway. This will allow a comparison of three generations of reservoir at one site.

BACKGROUND

The Loch Lomond Water Supply Scheme was officially opened by Her Majesty The Queen on 29th June, 1971. The design ultimate yield to supply is 455 megalitres per day (100mgd). The principal installations are the River Leven barrage at Balloch, the intake works and pumping station at Ross Priory, the main treatment works, pumping station and operational headquarters at Balmore, the treatment works at Blairlinnans, and some 103 kilometres (64 miles) of large diameter trunk mains with storage reservoirs and pumping stations.

Over the past twenty five years the phased development of the scheme has continued with the duplication of pipelines, augmentation of surge suppression equipment and construction of additional service reservoirs and pumping stations in order to meet the increasing water supply demands of the central belt of Scotland.

Presently, new treatment facilities and service reservoirs are being constructed at Balmore and Blairlinnans as the final stage of their development. A further major pipeline duplication and additional pump installations will be required to bring the distribution system up to the full design capacity.

The design of the Scheme has incorporated a high degree of automation and data acquisition from the outset. The second generation control system incorporating full SCADA facilities with microwave communication links and UHF outstation scanning was commissioned in 1988. The system will be further enhanced by the incorporation of the SCADA system for the new treatment facilities.

In addition, there is a proposal to increase the authorised abstraction rate from Loch Lomond from 455 megalitres per day (100mgd) to 568 megalitres per day (125 mgd).

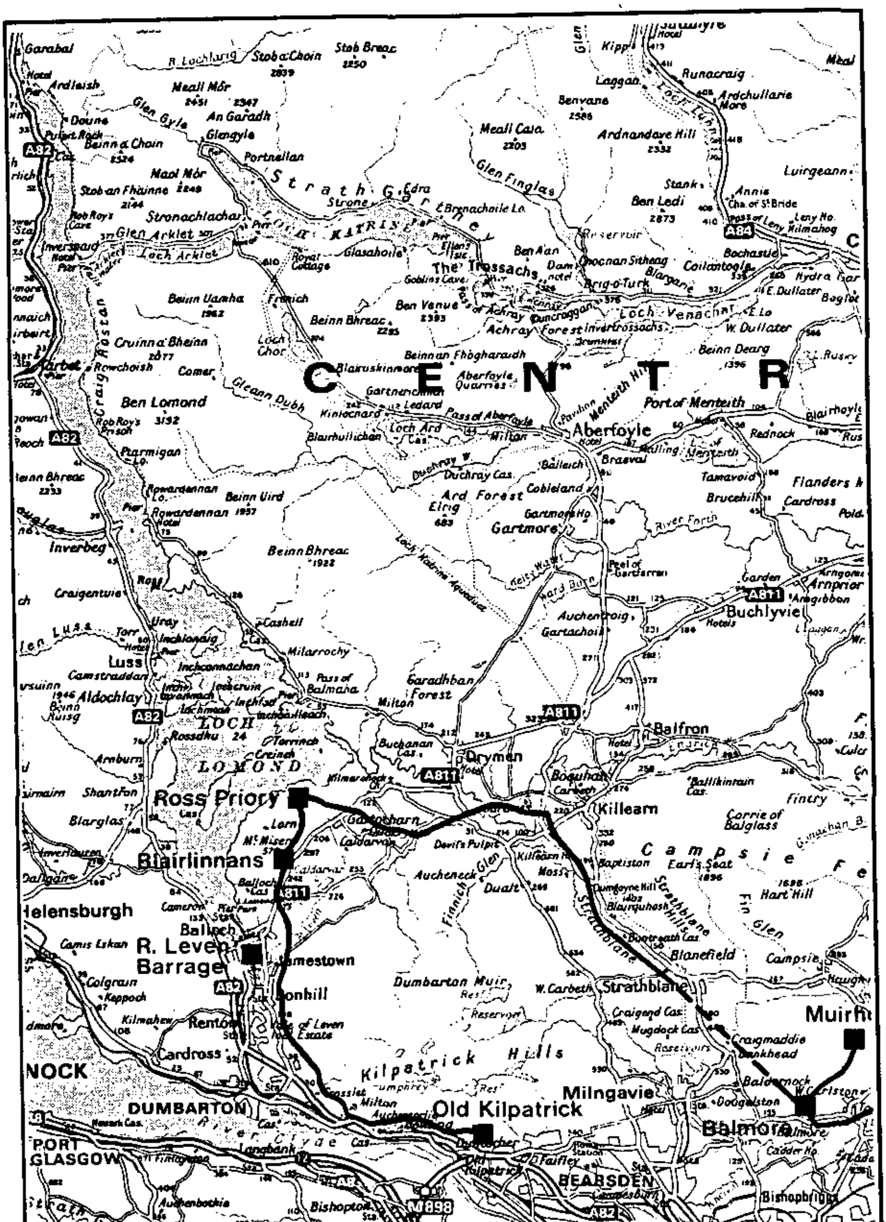


Figure 1 – Location Plan

LEGISLATION

The reservoir legislation in force at the time of construction of the first generation of service reservoirs was the Reservoirs (Safety Provisions) Act 1930. Although the initial storage capacities at Blairlinnans were less than 5 million gallons, the Board took the view that future duplication of the reservoirs would result in combined capacities in excess of that volume and that all of the service reservoirs in the Loch Lomond Scheme should be regarded as coming under the Act. A Panel 1 Engineer was appointed to oversee the design and construction of all of the first phase reservoirs.

The service reservoirs constructed in the last ten years have all been large raised reservoirs under the Reservoir Act 1975, with a Construction Engineer appointed in accordance with Section 6 of the Act.

Other recent legislation which influences the design of service reservoirs includes the CDM Regulations 1995 which place a statutory requirement on the designer to give specific consideration to safety aspects of the construction, operation and maintenance of the reservoir.

SERVICE RESERVOIRS

The earliest covered reservoirs were built in the middle of the nineteenth century as a result of growing concern over public health and the quality of water stored in open reservoirs. Many of the early reservoirs were of brick or mass concrete construction.

Technical knowledge, materials and construction techniques have changed dramatically since those early days. Today lessons are still being learnt with continuing research and development resulting in further refinements in all aspects of the design and construction of service reservoirs. To illustrate the advances that have been made in the more recent past, comparisons have been drawn between the three generations of reinforced concrete service reservoirs at Balmore Treatment Works.

Good design practice utilises current standards combined with practical experience to ensure that the construction and maintenance of a reservoir is made as straightforward as possible. This paper describes the many elements that comprise a service reservoir by explaining the philosophy behind the first generation design and construction followed by the changes which have been introduced in the detailing of these elements as a result of updated codes of practice, new materials and improved construction techniques. The quality of the reservoir is measured in terms of fitness for purpose, economy of construction and economy and ease of maintenance during its service life.

DESIGN

Clearly the most important feature of a service reservoir is its ability to contain water. This is an obvious and simple statement but in practice requires considerable thought. For reinforced concrete service reservoirs the most important element is the concrete which is an ideal material for this type of construction, providing both structural integrity and impermeability. However, careful attention to detail is needed in the design process to accommodate thermal and shrinkage movement of the structure.

Water retaining reinforced concrete design therefore has to be a compromise between the considerations of durability and restriction of shrinkage cracking. Durability is normally associated with high strength, high cement content concrete but, for low shrinkage, low strength, low cement content is more appropriate.

In the first phase construction the design codes used were CP114 with CP2007. The second generation service reservoirs were designed initially using BS5337:1976 in conjunction with the general code of practice on reinforced concrete CP110 and later using BS8007:1987 in conjunction with BS8110:1985. These design standards adopt completely different design philosophies, moving from an allowable stress consideration to a limit state analysis. The main departures from the original design philosophy are :-

(1) Crack width calculations have to be carried out to meet exposure and serviceability criteria.
(2) Minimum 50mm cover for durability.
(3) The distribution or shrinkage reinforcement is kept nearest to the concrete surfaces. This leads to heavier main reinforcement due to a reduction in lever arm for a given cross-section.
(4) 25% increase in anchorage and lap lengths.
(5) Greater guidance given on joint type and location.

The design of the structural elements and the calculation of reinforcement is based on the serviceability limit state. Other limit states, including the ultimate limit state, are checked. In many instances the shrinkage reinforcement requirement far exceeds the necessary reinforcement for structural integrity.

In the design of the third generation service reservoirs one particular area where further research has brought about a change is in the anchorage and lap lengths for epoxy coated reinforcing bars. BS 8007 states that no bond strength is lost when using High Yield Type 2 rebar which is now questioned by Task Group VII/8 of Permanent Commission VII : "Reinforcement : Technology and Quality Control for the Comite Euro-International du Beton, 1992." A further 20% allowance in lap length is recommended to overcome this problem.

The cover to reinforcement is continually under review. The first generation of service reservoirs had generally 40 mm cover to reinforcement. This was increased to 50 mm for the walls, columns and floor of the second generation reservoirs. 50 mm has been adopted as the minimum cover throughout the third generation reservoirs having given serious consideration to increasing the cover for walls and columns.

CIRIA Report 138 : 1995 offers useful guidance on the design and maintenance of underground service reservoirs.

IMPROVEMENTS IN CONSTRUCTION TECHNIQUES
It has been the practice to incorporate drainage below the floors of service reservoirs to offer relief from potential uplift pressures. A useful secondary function of these drains is monitoring and detection of leaks immediately after construction and during service, although this is hampered to a degree by the presence of a separating membrane below the structural concrete.

Application of a modest back pressure to the underfloor system, taking care not to cause uplift in the floor, with only a shallow depth of water, will identify leakage paths with bubbles rising at the leakage points. A herringbone pattern for the underfloor drains is now no longer adopted. Flat drain runs bisecting column centres avoids the potential for undermining column bases.

Floor panel pours have been simplified greatly by the exclusion of starter bar reinforcement for insitu columns. Clear flat slab floors are now poured in large panels and are vibrated with the use of a razor back screed providing a high quality surface finish. Column plinths are then cast on top of the floor. Pre-cast columns are presented to a vertical dowel bar drilled and grouted into the column plinth. The materials and method of construction have proved superior in every respect to insitu column construction.

The durability of the columns built in the first generation construction has been a source of concern over the past ten years because of loss of surface material and exposure of reinforcement. This is due to attack of the concrete by the naturally acidic raw water and the chlorine added to the water. Routine cleaning of the reservoirs is carried out using high powered jet sprays. This has contributed in some cases to a stripping of the fines from the concrete surface.

Three improvements have been made to combat this problem, namely :-

1) increased cover to the reinforcement
2) protective coating of the concrete surface of the column
3) the use of epoxy coated reinforcement.

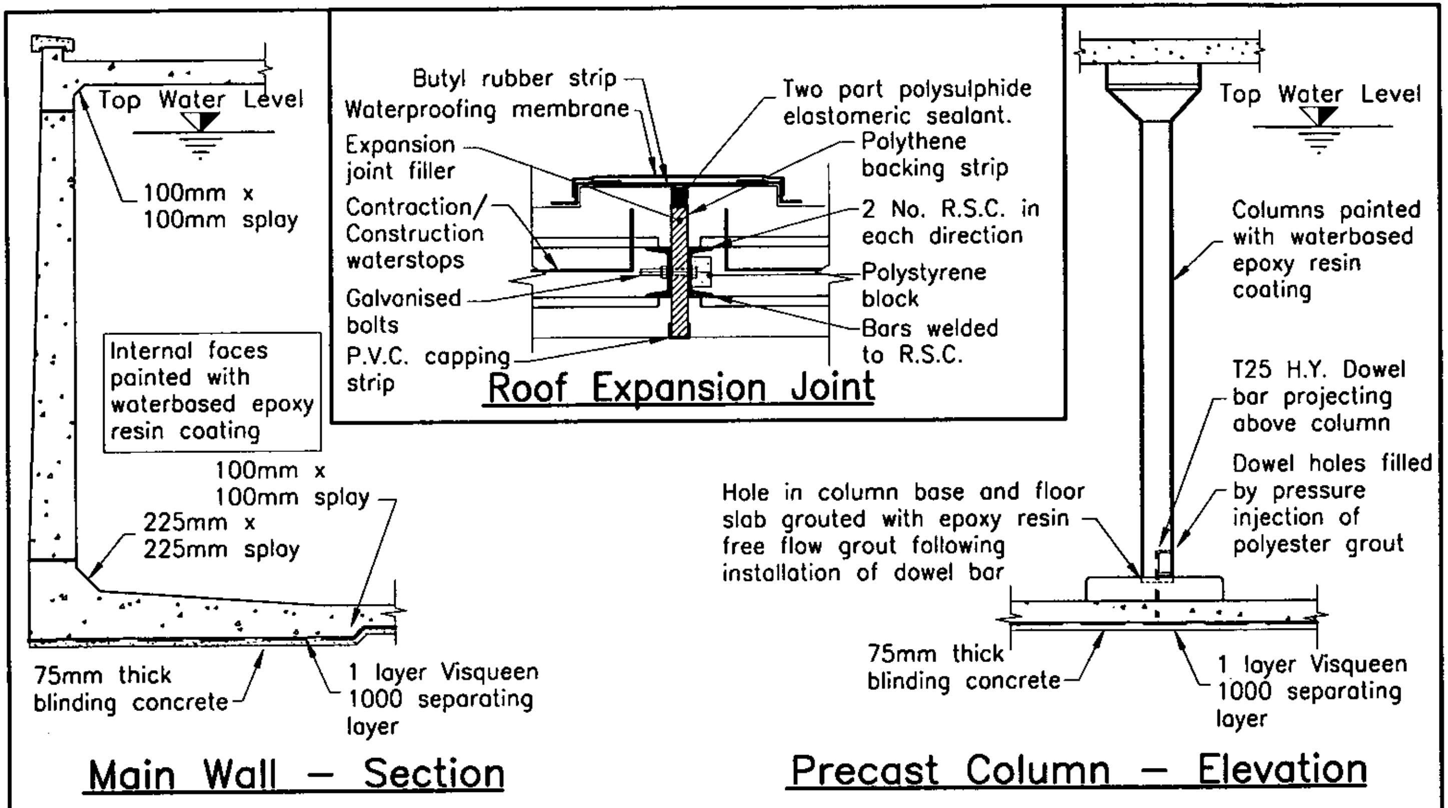


Figure 2. Construction Details

Increased cover and protective coating of the concrete extends also to the internal faces of the walls of the reservoirs.

IMPROVEMENTS IN THE SPECIFICATION

Site batching of concrete is now specified to ensure quality control and security of supply by having a dedicated batching plant to achieve consistent durable concrete product. In this connection, the specification for all materials is now much more detailed, requiring a comprehensive testing schedule for the aggregate, cement and other materials including alkali content, alkali silica reaction, flakiness index, chloride content etc. High Alumina Cement has never been incorporated in reservoir construction in this Scheme and is now specifically excluded by the specification.

Cement replacement in the form of Pulverised Fuel Ash has been adopted in some recent service reservoir construction due to its beneficial reduction of the heat of hydration during the curing process, limiting the potential for shrinkage cracking without unduly compromising the target strength of the concrete.

The quality of concrete finish achieved on surfaces which come into contact with the water is of paramount importance in ensuring satisfactory durability and ease of maintenance. Plywood faced shuttering is specified rather than metal mould systems which have not been found capable of producing the high quality finish to the concrete surface required for reservoir work.

The reinforcement used in the first generation construction was plain round mild steel bars whereas now High Yield Type 2 ribbed bars are used. Powder epoxy resin coated bars are now used across contraction joints in the roof of the structure and in the columns. At the roof expansion joint a special detail has been evolved over the period. This incorporates galvanised steel channels and anchor bars with movement dowels.

MATERIALS

Scottish Office, Welsh Office or Department of the Environment approval is now required for products used in the construction of reservoirs that will come into contact with potable water, these include curing membranes, admixtures, and protective coatings to walls, columns and internal fittings.

A variety of protective coatings have been applied to concrete faces in contact with potable water. Difficulties with some systems include contamination of the water by solvents and poor drying on wet concrete. Water based epoxy coatings are now adopted for internal wall and column protection.

Seasonal temperature variations must be taken into account in designing joint types and location for both contraction and expansion in these very large structures. Service reservoirs in this Scheme have been buried only to top water level. The roof is covered by a layer of stone chips to offer some insulation, however, temperature differences can be appreciable in high summer given that the internal temperature of the tank remains virtually constant.

Contraction joints generally do not present a difficulty as these are simply a discontinuity in the concrete with continuity of at least one layer of steel. However, an expansion joint presents a problem as it is necessary to transmit tension across the joint and the conventional system in a structure normally calls for discontinuity of both concrete and steel. Theoretically, the roof of the largest reservoir can expand by as much as 20mm in an annual cycle. The detail adopted for expansion joints is described earlier. Aluminium foil was used to cap the expansion joints previously whereas butyl rubber is now adopted.

Careful consideration was given to the location of movement joints to meet concrete shrinkage requirements and to reflect practical construction considerations.

Contraction and expansion joints are made watertight by using an appropriate pattern of PVC water bar. Although the principles behind jointing for construction purposes may not have changed, the materials used in forming the joints have improved. The width of the water bar used has increased from 7.25 inches (185 mm) to 240 mm and 9.5 inches (240 mm) to 250 mm in the wall stems and wall bases respectively. The underseal used in the roof is a preformed rubber compression seal.

One change to the principles of jointing is at the wall base / wall stem interface where a metal strip was included in the construction of the first generation service reservoirs. Water bar is now considered to be a disadvantage at this particular joint interface with honeycombing being the main source of concern. Water bar is not used now and the security of the joint is dependent on a full height wall stem pour taking place within three days of the base pour thus reducing differential shrinkage to a practical minimum.

Hot applied bitumen based sealants are used in the internal joints as other compounds such as polysulphides have been found to deteriorate due to the chlorine in the water. Joint Sealants are applied at ambient temperature during construction. This can result in considerable strain on movement joint sealants on first filling, particularly at wall stems.

Baffle curtains are used to encourage circulation of the water around the tank and were previously made of prefabricated asbestos cement panels. These have now been replaced for obvious reasons with the adoption of Hypalon curtains.

Galvanised mesh flooring although robust and durable is heavy whereas the use of GRP flooring is favoured now since it is corrosion resistant and lightweight.

PROGRAMMING

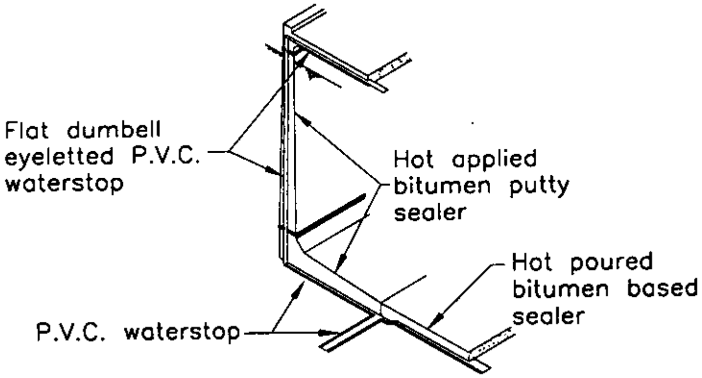
There have been many improvements in the sequence of construction as the contractor has continually sought more efficient ways of building the structure. In the first generation construction, insitu concrete was placed in the floor and wall bases first, followed by the construction of the wall stems and columns. Lastly, scaffolding was erected over the reservoir floor to allow the roof element to be poured.

The main problem encountered with this approach was that the wall suffered from cracking due to its slender section curing on top of a wall base that might have been several months old and which had completed its initial shrinkage. To overcome this difficulty, the specification now sets out that full height wall stems should be poured within 3 days of the wall base being cast to minimise differential shrinkage. Minimum wall thicknesses were increased in the second generation reservoirs to ensure adequate compaction and ease of concrete placement.

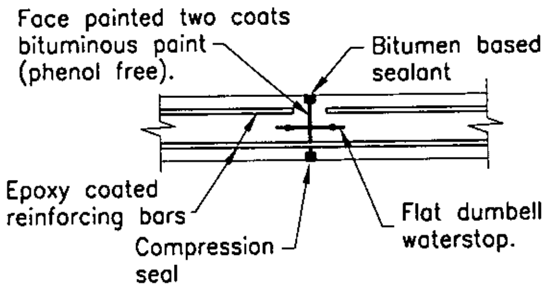
Initial resistance was met from contractors to this apparently tight pouring sequence. Operational experience led to the contractors recognising advantages by adopting a rolling programme of activity with wall base, wall stem then roof elements all advancing in sequence across the shorter length of the structure. Proprietary soffit shuttering systems were used to pour a row of roof panels across the structure adopting a hit and miss principle which improved the rate of progress for the contractor. If the contractor ensured that non-repetitive inlet and outlet chambers of the reservoirs did not become a critical path activity, then the main priority was to keep the roof advancing steadily.

OPERATIONAL, SAFETY AND MAINTENANCE CONSIDERATIONS.

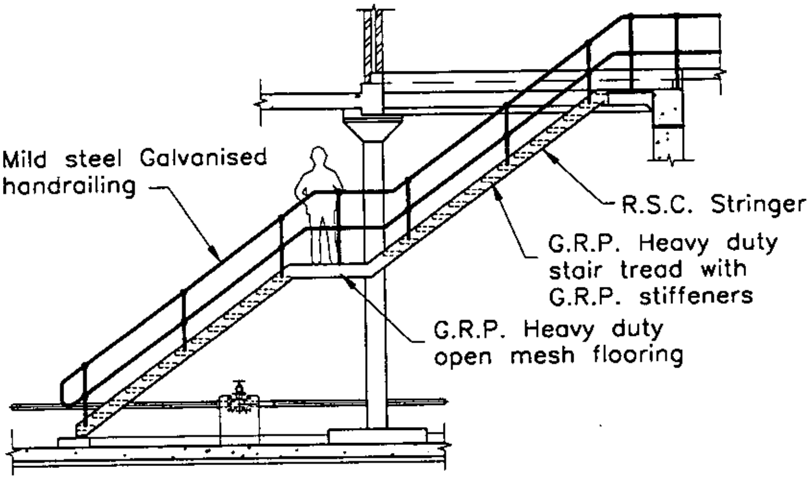
Access to service reservoirs has improved with the introduction of stairways into each reservoir compartment. This allows walk-in access with equipment and materials for routine maintenance. A number of other access points are provided in the roof of the reservoir. These are now larger and include such details as lightweight, high security access covers and fixed internal ladders with extending handholds.



Isometric



Roof Contraction Joint



Walk-in Access Stairway

Figure 3 – Access and Joint Details

An internal ring hydrant main is used for cleaning operations with a portable pump to pressurise the system. Collecting channels were laid to a fall in the first generation reservoirs. These are now designed with a level invert which results in easier detailing and construction and reduced operational hazard.

The second generation of reservoirs have also been designed with steel baffle arrangements over the outlet sump. This has hydraulic benefits in controlling tendencies to vortex formation and safety advantages.

TESTING THE TANK FOR WATERTIGHTNESS

The early specification for watertightness of the reservoir stated that the testing operation had to yield no loss of water, i.e. perfectly watertight. The practical requirement adopted for testing the reservoirs was a drop less than 0.5 " in 7 days. This was incorporated in subsequent specifications. Current practice adopts the procedural recommendations given in BS 8007 which are that the drop in level should not exceed 10 mm over a 7 day period, which represents a loss of 25 m^3 in a compartment of a reservoir of surface area 50 m x 50 m. Invariably leaks do occur, some more obvious than others. Wall joints and cracks represent frequent leakage paths.

The detection of leaks in the floor, however, is more difficult to locate despite the application of a modest back pressure to the underfloor drainage system. Time can act as a sealant with autogenous healing taking place but is normally associated only with small damp areas at construction joints drying out.

In general, leaks identified in tests have been sealed effectively using epoxy resin injection techniques.

CONCLUSIONS

The design details in the successive generations of service reservoirs in the Loch Lomond Water Supply Scheme have evolved reflecting changing design codes, construction practices, materials development and a better appreciation of operational and safety considerations.

Continuing review and refinement of these matters has led to the design of service reservoirs which are simpler and more economical to construct and easier to maintain and which should provide enhanced service life. The development of design will continue.

ACKNOWLEDGEMENTS

The authors are grateful to the Central Scotland Water Development Board for their permission and encouragement in the publication of this paper.

Service Reservoirs in Hong Kong

D GALLACHER, Senior Partner, Robert H Cuthbertson & Partners, UK
R J MANN, Senior Engineer, Robert H Cuthbertson & Partners, UK
K L CHAN, Senior Engineer, Water Supplies Department, Hong Kong

SYNOPSIS
A review is made of fresh water service reservoirs in Hong Kong operated by the Water Supplies Department which have been subject to Formal Independent Inspection. Various forms of construction of the structural elements including sealing arrangements are illustrated. Conditions found during inspection are described with reference to recurrent problems and performance. Requirements and proposals for remedial works are also described, including partial reconstruction following severe leakage at Gardens Service Reservoir and proposals for repair of extensive wide cracking in walls at two other service reservoirs.

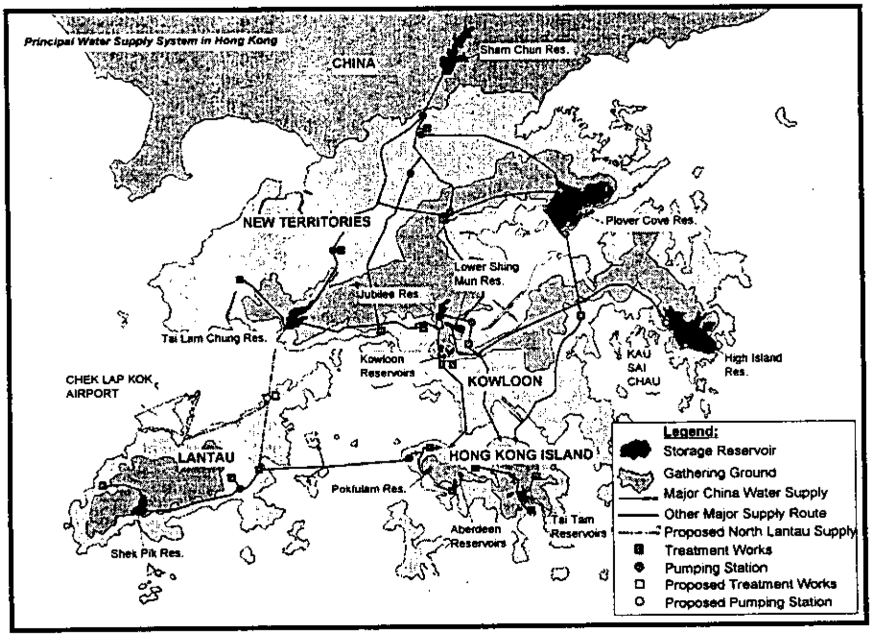


Fig. 1

1 INTRODUCTION

The earliest parts of Hong Kong's water supply system date from the 1860s. The Water Supplies Department (WSD) are responsible for the water supply system, which has been developed to cover all parts of Hong Kong Island,

The reservoir as an asset. Thomas Telford, London, 1996

Kowloon and the New Territories; Fig. 1 is a plan of the major supply schemes. Service reservoirs in use date from the nineteenth century to the present time, and forty of these have capacity greater than 25,000 m^3, the largest being Mount Davis Service Reservoir with a capacity of 142,843 m^3. The Government of Hong Kong broadly follows the UK reservoirs legislation although reservoir safety does not have the force of law in Hong Kong. A full-time Reservoir Safety (R. S.) Section has been established within WSD for about twenty years which is responsible for supervision of impounding and service reservoirs. The Fourth Formal Independent Inspection of 19 impounding and 25 service reservoirs commenced in December 1992 and it is due for completion in late 1996.

2 FORMS OF CONSTRUCTION

Nearly all of the service reservoirs are of concrete construction. Up to about 1980, designs were generally of mass concrete, and later reservoirs have been of reinforced concrete cantilever design. Designs have mostly been carried out by consultants or by WSD's own design section.

2.1 Topography, geology and foundations

The topography of Hong Kong Island and the New Territories is generally hilly. Most service reservoir sites are on sloping ground with steep cut slopes forming a platform for the reservoir site, and fill slopes formed against the walls on the downslope side. Typical geology is of igneous bedrock at depth, with degrees of decomposition to residual soil near the surface, and foundations vary from solid rock to residual soil. Some major service reservoirs, such as the Lion Rock group with capacities of between 77,000 and 102,000 m^3, are founded partly on residual soil and partly on rock.

2.2 Early designs

Early designs of concrete service reservoirs have floors of concrete slab construction with no reinforcement, walls of gravity section or lining section, and reinforced concrete columns and roof. Drainage is by a grid of sub-soil drains beneath the floor, and a wall-back drain at the outer toe connected to a porous zone against the back of the wall. Columns are founded beneath the floor. Floor joint sealants are typically of bitumen in a chase at the surface, and wall joints are sealed with a central bitumen plug and a bitumen or oakum surface sealant in a chase. Roofs are covered with soil on a grid of drains. Some roofs are surfaced for use as all-weather playing pitches.

Later designs up to about 1980, Fig. 2, have floor slabs in two layers with joints staggered; wall joints and wall/floor joints incorporate proprietary rubber or PVC waterbars and a sealant in a chase at the surface of bituminous putty in the walls and hot poured rubber bitumen in the floor. Roofs are formed in panels separated by expansion joints, supported on edge beams and walls with a bearing layer of bituminous felt.

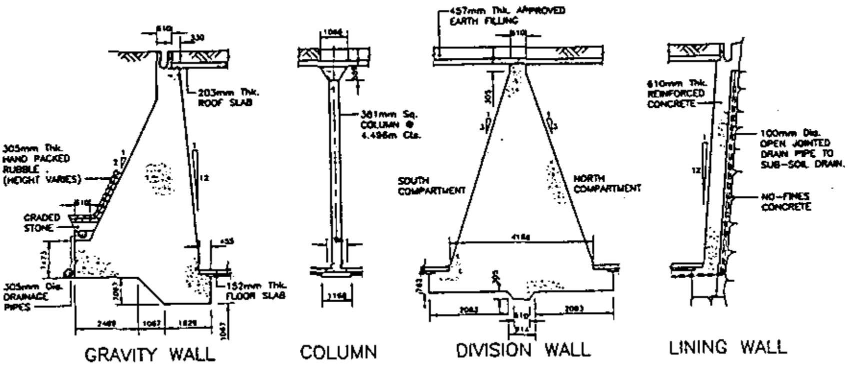


Fig.2

2.3 Recent designs

Recent designs since about 1980 are of reinforced concrete cantilever section, Fig. 3, with single layer reinforced floor in larger panels extending over several column spacings. Design is to contemporary British Standards for water-retaining structures. Joints have proprietary PVC waterbars and polysulphide surface sealants. A rubber bearing strip has been used at the roof/wall joint, with a waterbar or eaves detail to prevent infiltration. Drainage manholes are generally provided at outlets and at intervals along wall back drains to provide access for rodding.

No service reservoirs in Hong Kong are built in continuous construction.

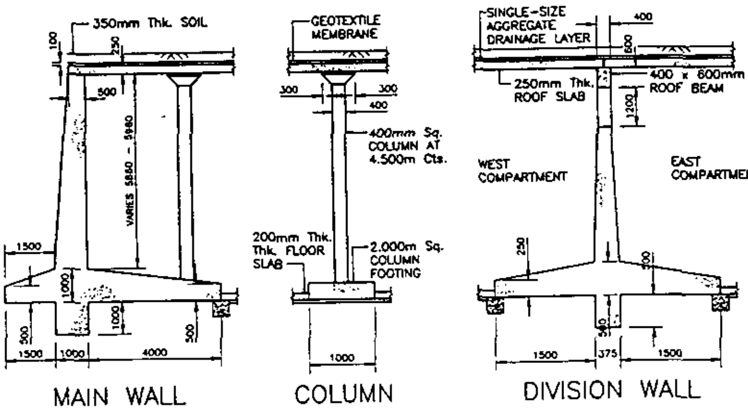


Fig. 3

3 CONDITIONS FOUND

3.1 Formal Inspections

Formal Inspections cover internal and external inspection of the service reservoirs and ancillary works. Key aspects covered include watertightness, stability, and adequacy of overflows, to determine whether measures are to be recommended in the interests of reservoir safety. In general, the standard of design, construction and maintenance of the service reservoirs is good.

3.2 Watertightness

Watertightness is assessed by the Reservoir Safety (R S) Section who monitor drop tests and drainage discharges regularly. Access to drainage outlets is often difficult particularly in older reservoirs. Monitoring results for recent years have been stored on a computer and plots are produced for assessment. Generally watertightness has been good, but problems requiring remedial works have arisen generally with floor joint sealants, and particular problems have arisen at the following reservoirs:

- Gardens S/R - leakage requiring emptying and partial reconstruction; also tree roots penetrating sub-soil drains
- Hatton Road No. 2 S/R - leaks at wall joints with a central core waterstop and failure of "Hypalon" sealant repairs
- To Shek S/R and Chai Wan North S/R - severe early thermal cracking

Remedial works recommended for the above reservoirs are described below.

3.3 Stability

Stability of slopes at service reservoirs is also monitored by the R S Section. Some of the slopes, which might have a significant risk-to-life or economic consequence, have been included in a programme of appraisal and stabilisation of slopes undertaken by the Hong Kong Geotechnical Engineering Office (GEO). Slope stability can be greatly influenced by rainfall. Stabilisation work involves drainage, surface protection, anchoring, and instrumentation to monitor continuing performance.

Stability of structural elements of the service reservoirs are checked for reservoir full and empty conditions, for seismic load, and for serviceability of reinforced concrete elements. Structures generally have adequate stability but a number of matters have arisen requiring safety recommendations:

- Gardens S/R - The leakage incident referred to above was related to movement of the perimeter wall
- Lai Chi Kok S/R - There is movement of a section of the mass concrete perimeter wall, which depends on backfill for its stability. Monitoring is being carried out to review movement during emptying and filling. Also corrosion has occurred to the roof soffit in one compartment due to lack of cover.
- Lion Rock High Level No. 1 S/R - the gravity perimeter walls are high and they are founded on residual soil. There was evidence of wall movement although the wall section was adequate. Close monitoring of the roof/wall joint indicated that the joint opens and closes in a seasonal cycle. It was concluded that cyclical movement of the roof was being partially transferred to the walls and accommodated in the foundation. A recommendation has been made to continue monitoring on a reduced scale.
- Tsing Yi East S/R - cracking of gravity perimeter walls has resulted from forces from thermal movement of the roof being transferred into the walls due to poor construction of the perimeter movement joint.

Investigation and remedial works to this reservoir are described below.

- Pun Chun Yuen S/R - Extensive minor stress cracking had occurred to cantilever wall panels, and fine vertical cracks were found in columns. A check of the reinforcement in columns found insufficient cover. Monitoring of wall movement has been established to identify trends and to confirm satisfactory performance.

3.4 Adequacy of overflow works

Service Reservoirs in Hong Kong generally have a high throughput, and many have overflows which discharge into urban storm drains. Records of inflow rates were prepared and in many cases either the overflow itself or the drainage system had insufficient capacity to discharge the excess of maximum inflow over minimum outflow. High level alarms and motorised inlet control valves have been installed at most service reservoirs, together with upgrading of the telemetry system, giving indication and remote control of inflow at continuously manned stations to reduce the possibility of overflows occurring. Measures recommended at the Lion Rock group of primary service reservoirs, where throughput is particularly high and overflows highly inadequate, are described below.

4 REMEDIAL WORKS - GENERAL

4.1 Routine inspections and repairs

Monthly inspections are made by Inspectors, and annual inspections by the Engineer/Reservoir Safety, who prepare reports including recommendations for repairs. Routine repairs are carried out by the Headworks Section of WSD. Any matters requiring further inspection or advice, or measures recommended in the interests of safety in an Independent Inspection, are normally referred to the Independent Inspecting Engineer to supervise the design and implementation of remedial measures.

4.2 Defects to floor joint sealants

Widespread deterioration of hot poured bituminous floor joint sealants has occurred to give swellings, in some cases producing grotesque protrusions, and the material has become spongey and fairly brittle. This has not generally given rise to significant leakage, except at North Point West S/R where fairly persistent leakage has occurred. Routine remedial works have been carried out in some instances using replacement hot poured sealant, and in other cases using polysulphide sealant. The deterioration of the original hot poured sealant is probably a result of traces of moisture in the chase when placing, and it is difficult to avoid this in remedial work. Replacement with polysulphide has been unsatisfactory as polysulphide is incompatible with traces of the original bituminous primer, and it has degraded and lost bond. Replacement of the polysulphide sealant repairs with bituminous putty has been recommended at several reservoirs.

5 REMEDIAL WORKS TO HATTON ROAD No. 2 SERVICE RESERVOIR

Hatton Road No. 2 Service Reservoir is of mass concrete construction, built in 1948 on a steep slope above a heavily developed area. There are no waterbars, and wall joints are sealed by a central bitumen plug. A drainage system was put in and the slope monitored by GEO, and WSD were concerned about leakage into the slope. Inspection by Mr W P McLeish the previous Inspecting Engineer revealed that settlement was probably occurring at one corner and a recommendation was made to seal the joints at this corner with "Hypalon" membrane sealant if leakage increased. Repairs with "Hypalon" were carried out to all the joints.

The reservoir was inspected again during the current programme and it was found that the Hypalon membranes had stretched, and some had ruptured, due to back pressure from within the joint. The original central bitumen plug joint sealants were located, and the sealant was found to be low in two joints, one of which was at the settled corner. A recommendation was made to top up the bitumen sealant, and to water test the reservoir.

6 REMEDIAL WORKS TO GARDENS SERVICE RESERVOIR

6.1 Inspection and investigation

Gardens Service Reservoir, constructed in 1938, has concrete gravity walls founded on the downhill (north) side on decomposed volcanic material, backed with a fill slope with mature trees. Part of the Botanic Gardens is located on reservoir roof. The reservoir had a long history of leakage, which had increased in recent years in spite of repairs to sealants.

A sudden increase in leakage occurred on 9 March 1992, with weepholes and sub-soil drains discharging strong flows which were observed on the inspector's routine visit. The reservoir was emptied promptly by WSD who requested Mr W P McLeish, then Advisor to WSD, to inspect and investigate the problem, and make recommendations for repairs. The north wall had moved slightly outwards. The fill slope behind the north wall had low stability for surface slips, but was adequate for deep slips on the assumption of a low water table as indicated by weepholes and piezometers in the slope. There was no evidence of slope movement when the large leakage flows were observed.

Some unsatisfactory aspects were identified in the course of inspection and investigation, as follows:

- The joints between floor panels, floor/columns, and floor/walls, had no waterbars and could not accommodate much movement without leaking.
- The foundation material of the north wall appeared to be fairly soft and had possibly deteriorated since an earlier investigation in 1983.
- Tree roots were blocking the sub-soil drain outlet from the West Compartment. This prevented free drainage and may have increased uplift.

6.2 Recommended remedial works

A solution was proposed to provide flexibility, reduce the bearing pressure, and control ground water levels, as follows:

(a) Construction of a new under-drainage system with modified outlets at original locations, with a cut-off key wall parallel to the north wall.
(b) Construction of a new L-section cantilever wall along the north wall and the north part of the east and west walls, with a connection to the existing division wall. The wall was tied to the inner face of the existing wall, with a footing cast on the original floor slab. This design provided some flexibility and lower bearing pressures than the original gravity section.
(c) Construction of a new floor slab on top of the existing floor
(d) Provision of joints with waterbars in the new construction at the location of all joints in the original structure, and additional waterbars set in a new concrete strips tied to the existing structure at all joints where not covered with new construction, all to accommodate minor movements.

Typical details are shown on Fig. 4.

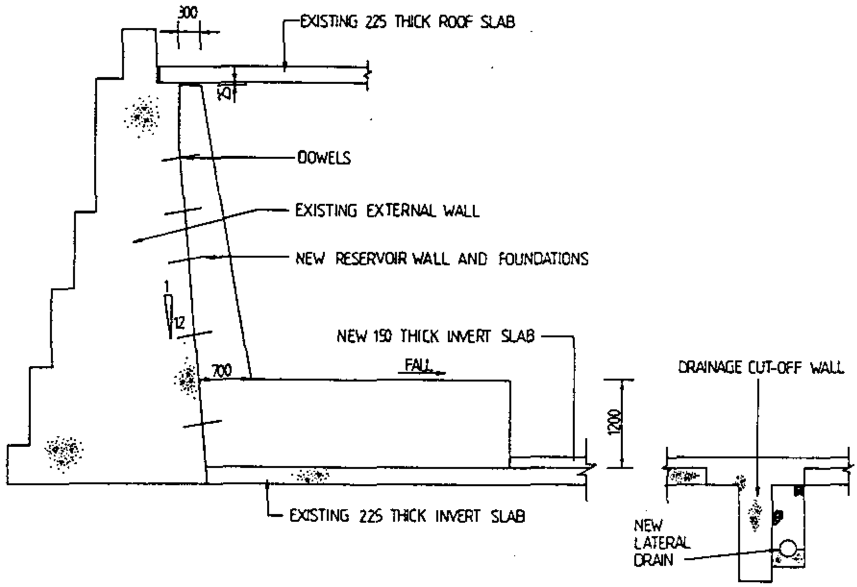


Fig. 4

Further improvement works were undertaken as follows:

(e) Demolition of a mass concrete wall sealing off the north-west corner which had been put out of use following leakage and wartime bombing, and reinstatement of the original structure together with the new cantilever wall and footing, to maintain capacity.
(f) Provision of new sumps connected to the original outlet pipework
(g) Modified outlet pipework, a new overflow and a high level alarm.

(h) Epoxy coating to columns and existing wall surfaces for long term protection, as rust marks were evident on columns, the concrete strength appeared to be fairly low, and carbonation was evident.

6.3 Design and construction

Detail design of the remedial works scheme was carried out by WSD Design Section, and construction was carried out in 1994-95 by Excel Engineering Co. Ltd under the supervision of WSD Construction Division. The reservoir site is very restricted, the only access for plant into the reservoir being by a temporary ramp at the north-east corner where the roof was reinstated at the end of construction. Inspection of trenches dug for underdrains revealed that the underlying material was fairly firm with no evidence of voids, softening or inadequacy. The site was very well organised, and the quality of construction was good. The reservoir is now fully back in service.

7 TO SHEK SERVICE RESERVOIR

7.1 Inspections

To Shek Fresh Water (F/W) Service Reservoir, completed in 1982, is of concrete gravity construction in two compartments, with a Salt Water (S/W) Reservoir constructed against the east perimeter wall. In 1983, cracks were observed in the perimeter walls during routine inspection by WSD. The Salt Water Reservoir was kept empty, and an Independent Inspection was made by Mr W P McLeish in 1990, following an initial inspection in 1985.

There was extensive wide cracking predominantly in a near horizontal direction, on all walls, as shown in Fig. 5 for the common wall with the S/W Reservoir.

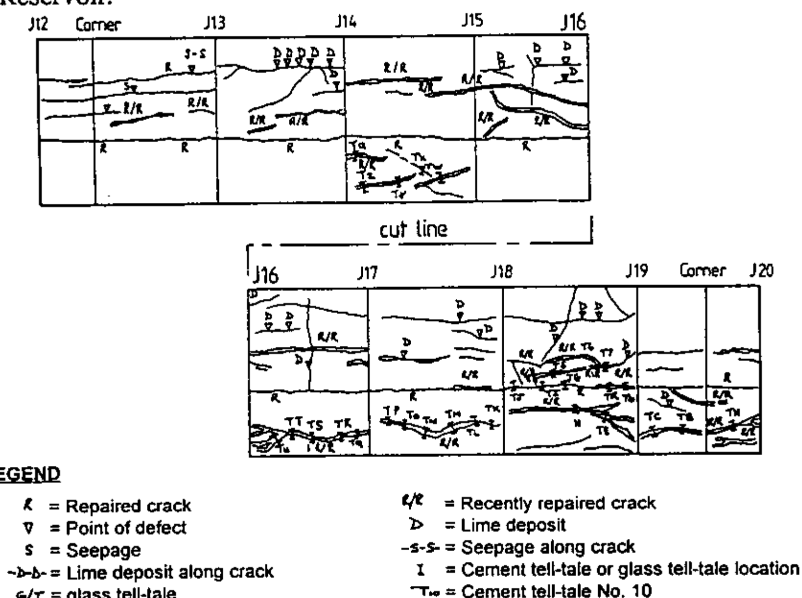


Fig. 5

The major cracks had been repaired by non-shrink grout, and leakage through the common wall into the S/W Reservoir was reduced to minor seepage. The stability of the wall section was adequate. It was concluded that the cause of cracking was shrinkage of the concrete, probably caused by an inappropriate concrete mix. Measures relating to the cracking recommended by Mr McLeish following inspection in 1990 were as follows:

- If bringing the S/W reservoir into use, construct a separate wall with a drained accessible air gap at the common wall on the S/W reservoir side, and seal all cracks in the S/W reservoir walls
- Seal cracks in the water face of the walls of the F/W reservoir by pressure grouting using non-shrink grout, and repeat repairs as necessary.

Design by WSD of remedial works to bring the S/W reservoir into use was in hand in 1995, by which time a further Independent Inspection of the F/W reservoir had become due.

The 1995 Inspection found that tell-tale readings and cracking were unchanged from that described in the 1990 inspection. Further study of the pattern of cracks indicated that they were predominantly horizontal but not confined to lift joints, and they tended to continue across vertical wall joints. This indicated that early thermal effects and end restraint, together with poor concrete mix design, were the likely causes of cracking. A water test with the S/W Reservoir full and the East Compartment empty revealed considerable leakage into the reservoir through the common wall. It was considered that cracking affected the structural integrity of the walls.

7.2 Proposals for remedial works

Since cracking was not developing, an alternative repair scheme was proposed for grouting the common wall between the F/W and S/W reservoirs to restore its integrity and prevent infiltration, without constructing a separate wall with an air gap as recommended previously. A contract for repair works, by grouting cracks to a depth of 200 mm with epoxy grout and a surface seal formed in a chase, was prepared by WSD Design Division and has been completed recently. Water testing with the S/W Reservoir full and the East Compartment empty has indicated no leakage through the wall.

8 TSING YI EAST SERVICE RESERVOIR

The service reservoir is in two compartments with mass concrete gravity walls and a reinforced concrete roof in a single panel for each compartment. The East Compartment was constructed in 1980, and the West Compartment in 1982 adjacent to the East Compartment, converting the west wall of the latter into a common wall. The roof is surfaced for recreational use.

8.1 Findings from inspection and investigations

There are horizontal cracks in the perimeter walls of the West Compartment in many locations at all elevations with a maximum width of about 4 mm. Cracks are generally at lift joints but some have a more random direction generally parallel.to lift joints. There is spalling of the top arris of the wall where the concrete appears to run up against the roof with no provision for movement at the roof/wall joint. Cracking on the earlier common wall and the East Compartment walls is only minor.

The condition of the roof/wall bearing and expansion gap was investigated by breaking out the wall top to expose the bearing. The rubber bearing strip is sound, but the gap on each side of the strip, and the expansion gap at the perimeter of the roof, are infilled with concrete instead of being a void as designed. In several locations the roof soffit is cast lower than the base of the bearing strip and extends hard against the wall top. These features of poor construction of the roof/wall joint in the West Compartment prevent accommodation of thermal expansion of the roof, resulting in the cracking of walls. The roof/wall bearing in the earlier East Compartment is on bituminous felt which appears to accommodate some relative movement.

8.2 Proposals for remedial works

A recommendation has been made for remedial works to be carried out at the roof/wall movement joint to the perimeter walls of the West Compartment to permit movement in accordance with the original design. It has also been recommended that horizontal cracks be repaired to restore the structural integrity of the perimeter wall sections. Measures to reinstate provision for movement of the roof/wall top have been prepared. They involve cutting out the top of the wall behind the expansion slot, and replacing the wall top to incorporate an expansion gap and a replacement waterbar arrangement, as shown on Fig. 6.

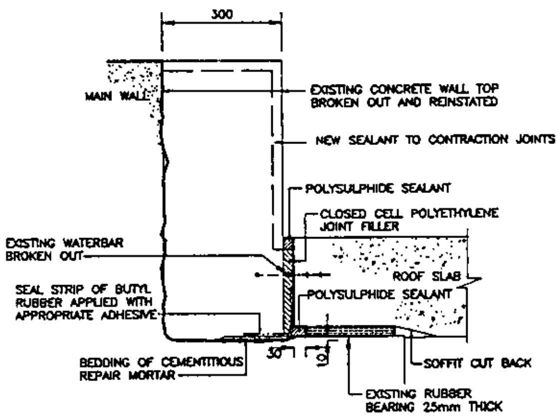


Fig. 6

It has also been recommended that horizontal cracks be repaired to restore the structural integrity of the perimeter wall sections. This involves epoxy grouting and provision of a surface sealant similar to those described above for To Shek Service Reservoir.

9 CHAI WAN NORTH SERVICE RESERVOIR

Chai Wan North Service Reservoir, completed in 1980, is of similar construction to To Shek, i.e. concrete gravity construction in two compartments, and there are extensive cracks in the perimeter and division walls. An inspection was requested to assess conditions and propose repairs. The crack pattern appeared to relate primarily to early thermal effects, although roof expansion forces may have been a factor. Recommended crack repairs are broadly similar to those for To Shek Service Reservoir.

10 LION ROCK GROUP OF SERVICE RESERVOIRS

The Lion Rock group of Service Reservoirs are three key primary service reservoirs supplying a large number of secondary service reservoirs. Supply is by pumping mains and the inflow rate to each reservoir is high (about 12 m^3/s in total).

During the previous Independent Inspection by Mr W P McLeish, the overflows were found to be inadequate, and the scope for improving overflow capacity was limited by the capacity of the nearby storm drainage systems. Safety recommendations were made to provide improved controls on inflow including an automated back-up to the present manual control on pumps, and duplication of key elements of the telemetry system. Controls will shut down selected pumps if the water level rises to set stages above alarm level. Implementation of these measures was commenced by WSD.

During the current inspection, studies were made of inflow and outflow records to determine typical maximum inflow and minimum outflow rates, and the capacity of nearby drainage systems was reviewed. The safety recommendations made in the previous inspection were broadly confirmed, and implementation of the safety measures by WSD is continuing, with advice being given by the Inspecting Engineer.

11 ACKNOWLEDGEMENTS

The authors wish to thank the Water Supplies Department of the Hong Kong Government, and Acer Consultants (Far East) Ltd, for permission and assistance to prepare this paper.

European dam safety regulations from a British perspective

J A CHARLES, Building Research Establishment, UK
C E WRIGHT, Department of the Environment, UK

SYNOPSIS. The well tried and tested system of reservoir safety which has been developed in Great Britain is based on principles which it would be advantageous to keep. At some stage developments in European harmonisation of working practices and regulations may embrace reservoir safety and, in order to assess the possible implications of this for British reservoir safety practice, the regulative frameworks for reservoir safety in other European countries have been examined.

INTRODUCTION

Reservoir safety in Great Britain relies to a major extent on the competence of individual independent qualified civil engineers who are appointed under the Reservoirs Act, 1975, and who carry a large degree of responsibility. There is a marked absence of direct control by central government. This approach, which was first embodied in the Reservoirs (Safety Provisions) Act 1930 and then in the Reservoirs Act 1975 which superseded the 1930 Act, has served Britain well and no deaths have been reported due to dam failures since the 1930 Act came into force (Wright, 1994). This well tried and tested system is based on principles which it would be advantageous to keep, although minor improvements could be made as described in the joint consultation paper on proposed amendments to the Reservoirs Act 1975 (Department of the Environment, 1993).

At some stage developments in European harmonisation of working practices and regulations may embrace reservoir safety and, in order to assess the possible implications of this for British reservoir safety practice, the regulative frameworks for reservoir safety in other European countries have been examined. This study has been carried out principally from published papers, mostly in the English language which means that the accounts of national regulations may not be as precise or as comprehensive as they would be if all the source national regulatory documents had been translated into English, but it is considered that this method has given results which are sufficiently accurate for the present purposes.

LARGE DAMS IN WESTERN EUROPE

The differences in approach to safety regulations in the various European countries reflect not only differences in legislative background but also differences in the national populations of dams. Statistics on large dams can be obtained from the World Register of Dams (ICOLD, 1984 and 1988). The conditions for inclusion in the World Register are, firstly, all dams with a height of 15 m or more and, secondly, dams between 10 m and 15 m, provided that they comply with at least one of certain specified features such as a crest length not less than 500 m or a reservoir capacity of not less than $1x10^6$ m^3. Entries are predominantly in the first category and it can be assumed that the World Register provides comprehensive information on dams with a height of 15 m or more.

This criterion for a large dam does not generally coincide with the criterion for inclusion within the ambit of national safety regulations which may include many smaller dams. In Great Britain only about 20% of the dams which come within the scope of safety regulations are large dams as defined in the World Register. It should be noted that although the British section of the World Register is labelled "Great Britain" it includes all large dams in the United Kingdom. In contrast, the Reservoirs Act 1975 applies only to Great Britain. However, this discrepancy is not significant for the present study.

Some basic statistics for large dams in western Europe are presented in Table 1. For the countries with 50 or more large dams, an analysis has been made of the type, age and size of the dams.

(a) Type. There are large differences in the composition of the dam populations; Austria, France, Italy, Portugal, Spain, and Switzerland have a relatively low proportion of embankment dams whereas in Great Britain, Germany and the Nordic countries the majority of dams are embankment dams.

(b) Age. The age distribution of the large dam population is significantly different with Great Britain unique among these European countries in having over 70% of its large dams built before 1950. In none of the other countries is the proportion as high as 50%. Another significant difference is that programmes of dam construction continue in Italy and Spain whereas few, if any, new large dams are likely to be built over the next 20 years in Great Britain. Of the other European Community (EC) countries only in Greece does it seem likely that a substantial number of new dams may be built in the next few decades.

(c) Size. The proportion of large dams which are less than 30m in height are listed in Table 1. The proportion in Great Britain is relatively high. The proportion is low in Austria, Italy, Portugal, Spain and Switzerland

NATIONAL REGULATIONS FOR DAM SAFETY

The safety regulations of all the western European countries with a substantial population of large dams have been examined. A minimum of 50 large dams was selected as an appropriate criterion. At the commencement of this study there were six countries in the EC with a substantial number of large dams; France, Germany, Italy, Portugal, Spain and the United Kingdom. There were five other western European countries with a substantial number of large dams; Austria, Finland, Norway, Sweden and Switzerland. During the course of the work it became clear that whereas Austria, Finland and Sweden would soon join the EC, Norway and Switzerland would not. Nevertheless they have all been included in the study.

Table 1. Large dams in western Europe

Country	Total (a)	TE+ER (b)	$\frac{(b)}{(a)}$	Up to 1950 (c)	$\frac{(c)}{(a)}$	<30m (d)	$\frac{(d)}{(a)}$
Austria	123	24	20%	30	24%	57	46%
Belgium	15						
Denmark	6						
Finland	50	37	74%	10	20%	43	86%
France	468	158	34%	164	35%	301	64%
Germany	261	174	67%	91	35%	173	66%
Great Britain	535	413	77%	378	71%	405	76%
Greece	13						
Ireland	15						
Italy	440	105	24%	199	45%	166	38%
Luxemburg	3						
Netherlands	10						
Norway	245	132	54%	46	19%	156	64%
Portugal	81	30	37%	12	15%	29	36%
Spain	737	151	20%	205	28%	356	48%
Sweden	141	96	68%	31	22%	107	76%
Switzerland	144	28	19%	40	28%	60	42%

Notes:

(a) Total number of dams listed in World Register of Dams (ICOLD, 1984 & 1988).

(b) TE signifies earthfill dams, ER signifies rockfill dams, thus TE+ER represents the total number of embankment dams.

(c) Dams built before 1950.

(d) Dams with a height of less than 30 m.

Of the six western European countries which were not studied, Belgium, Denmark, Luxemburg and the Netherlands have very few large dams (Table 1) and are never likely to have many. Although Greece has few large dams at present, there is a significant programme of dam construction. The Republic of Ireland and Northern Ireland operate in practice in a manner similar to Great Britain, although the Reservoirs Act 1975 does not have legal force there.

The amount of information that is readily accessible is very variable. For example, in Germany, safety regulations are associated with state water laws and information is not easy to obtain from a literature survey; the criteria in Table 2 refer only to the state of North Rhine Westphalia. For France there is much published information, but, with a long history of national regulations, the situation is quite complex. Finland, Norway and Portugal have relatively recent legislation which is available in English translations.

Table 2. Criteria for applicability of national safety regulations[(1)]

Country	Dam height H (m)	Reservoir capacity V ($10^3 m^3$)	Public safety	References[(2)]
Austria	15	500		Pircher (1991), Heigerth et al (1994), Konig & Schmidt (1991)
Finland	3	---	yes	Kuusiniemi et al (1994), Loukola (1988), NBWE (1987)
France	20	---	yes	Carlier (1973), Goubet (1993)
Germany[(3)]	5	100	yes	Czychowski (1990), Spillner (1990), Deutsche Norm (1986)
Great Britain	---	25		HMSO (1975), Charles et al (1994), ICE (1995)
Italy[(4)]	15	1000		Dolcetta & Paolina (1993), Morris (1992), Visentini (1992)
Norway	4	500		Nicolaisen (1990), NVE (1986)
Portugal	15	100	yes	LNEC (1990)
Spain	15	1000	yes	Franco and Cordova (1988), Gil & Del Hoyo (1993)
Sweden[(5)]	5	50		Hultin et al (1982), Hultin & Norstedt (1988)
Switzerland	10	50	yes	Biedermann (1985), Schnitter (1985).

Notes:

(1) The limitations of the table should be recognised. It is not possible to convey all the details of national safety criteria in such a table and, also, changes in regulations may occur at any time.

(2) In addition to the references quoted for particular countries, UNESCO (1967), Goubet (1992), Le Moigne (1994), and Le Delliou (1995) provide information on the legislation of many countries.

(3) Criteria are those applicable in North Rhine Westphalia.

(4) Revised criteria follow a decree of 8 August 1994 (Le Delliou, 1995). The criteria previously were H = 10 m and V = 100 000 m^3. Smaller dams are subject to controls from regional authorities.

(5) There are no legal criteria, those quoted have been adopted for a national inventory of dams.

Reservoir safety management is primarily the responsibility of the owner or the operator in all the western European countries, but, in other respects, the approaches adopted to ensure reservoir safety show considerable differences. These differences can be attributed not only to factors such as the national approach to regulation, particularly the extent of direct government control, and the current dam population including type and size, but also the history of dam building and the occurrence of dam failures, the presence of centres of population downstream of dams, and the pattern of dam ownership, particularly whether or not one company owns most of the large dams and hence has a major interest in reservoir safety.

The differences in approach between the countries can be grouped into three broad areas; the criteria for applicability of the regulations, the nature of the regulations, and the implementation and enforcement of regulations.

CRITERIA FOR APPLICABILITY OF SAFETY REGULATIONS

The criteria governing the applicability of safety regulations are summarised in a simplified way in Table 2. Three criteria are used:

(a) height of dam (H),
(b) capacity of reservoir (V),
(c) public safety.

In most countries the safety regulations apply if any of the specified criteria are met. For example, in Portugal the regulations apply to dams less than 15 m high if the reservoir has a capacity of over 100 000 m^3.

While Table 2 gives a good idea of the European situation at a glance, it is not possible to convey all the complexity and details of national safety criteria in such a table. Details such as the manner in which dam height is specified varies from country to country. In some countries there may also be regulations for smaller dams; for example, in Great Britain some smaller reservoirs come under other legislation such as the Health and Safety at Work etc Act 1974. There may be additional requirements for very large dams; for example, in France there is a requirement for emergency plans which applies to dams over 20 m high and with more than $15x10^6$ m^3 reservoir capacity.

It might be expected that the scope of national safety regulations would be strongly influenced by both the size of the hazard and the public perception of that hazard. The size of the hazard is related to the number and size of large dams and their position in relation to centres of population. The perception of the hazard is likely to be affected by the history of disastrous failures. The statistical study of the population of large dams in European countries has shown that the number of large dams in Great Britain is comparable to the numbers of large dams in most other major EC countries (Table 1). However, as Great Britain is more densely populated than some EC countries, there could be a greater potential threat to public safety.

Regulations in many countries specify emergency plans.

The differences in the criteria for applicability of national regulations can be summarised as follows.

(a) Height criteria vary between 3 m and 20 m with the UK unusual in having no height criterion (although the height does have to be recorded in inspection reports).

(b) Most countries have a storage capacity criterion which varies between the UK with 25 000 m^3 and Italy with $1x10^6 m^3$; a typical criterion is 100 000 m^3. (The current proposals to amend British legislation would, if passed, bring it nearer to other European countries by removing the need for continuous supervision for very low hazard reservoirs containing less than 100 000 m^3; this value was recommended partly because of its use in reservoir legislation elsewhere in Europe.)

(c) Many countries have some additional criterion to ensure that all dams, however small, which affect public safety are subject to regulation.

NATURE OF THE REGULATIONS

In Great Britain the Reservoirs Act 1975 provides a framework within which reservoir safety can be regulated and enforced. Inspection and supervision is in the hands of independent qualified engineers. Standards for stability, floods, earthquakes etc are not included in the legislation but are given in engineering guides which are not mandatory. For example, in the floods guide (ICE, 1978) it is stated that:

"The recommendations made here are not in any way mandatory and it is recognised that there will always be the occasional dam that has to be treated as an exceptional case. However, the working party suggests that where the engineer feels it is right to depart from its recommendations the fact should be recorded in his inspection report."

In some southern European countries there are detailed mandatory technical requirements. Dolcetta and Paolina (1993) commented as follows:

"The standards in Italian regulations, unlike those in many other countries, are not recommendations but veritable requirements under the terms of the law. They deal with the various aspects of design (including static analysis), construction and operation of dams in detail."

In Spain a return period for the design flood is prescribed by law. However, although Cifres Gimenez and Abad Moreno (1992) refer to the 500 year return period flood of the Spanish Dam Project, Construction and Exploitation legislation, they go on to discuss the use of design criteria based on the probable maximum flood in situations where overtopping is likely to cause failure (eg at many embankment dams) and where such catastrophic failure would mean the loss of human life.

Although there is a much greater direct involvement of government engineers in the control of dam safety in France than there is in Britain, there is some similarity to the British approach to reservoir safety in that detailed technical requirements are not enshrined in legislation.

New dam construction is still a major activity in Spain and Italy and consequently safety in relation to design and construction will continue to be an important issue in those countries. Elsewhere, the emphasis will be primarily on the evaluation of the safety of old dams and a crucial question concerns whether the safety standards for existing dams should be identical with the standards for new construction. It is difficult to justify any answer other than "yes" when public safety is involved. However, it takes time to bring old dams up to modern standards and priorities need to be established. In North Rhine Westphalia, the dam adaptation law of 1989 requires that dams are adapted to the current engineering standards set by the responsible state ministry. ENEL in Italy has a similar programme. Great Britain is well placed in this area as many overflow works were upgraded following publication of the ICE (1978) floods guide. The engineering guides published in the UK generally assume that safety standards for existing old dams should be identical with those for new dams.

IMPLEMENTATION AND ENFORCEMENT OF REGULATIONS

There are major differences between the UK and other European countries in the implementation and enforcement of regulations. In most other EC countries there is much greater emphasis on centralised control with the involvement of government control services, whereas in Great Britain reliance is placed on the expertise of an independent engineer, who has been assessed for competence by the Institution of Civil Engineers, and the enforcement duties, which are carried out by local authorities (currently under review), are mainly of an administrative nature.

In Italy, the Dams Service supervises dams in operation through twice yearly site inspections (Dolcetta and Paolina, 1993). It can require additional studies and can impose limits on reservoir water levels in the interests of safety.

In Spain a government service was set up in 1959 which approves operational procedures and educates operators concerning dam safety. Inspections are carried out by the owner and the surveillance section of the Ministry of Public Works. In Portugal the National Civil Engineering Laboratory (LNEC) reviews the monitoring plans of the owner and controls their implementation.

In France three ministries, Transport, Industry and Environment respectively, are in charge of the inspection of dams. Control Services have been established in each of three ministries which are responsible for reservoirs constructed for different purposes. The Control services are required to visit each reservoir at least annually and carry out full inspections every ten years.

The situation in Norway is not too dissimilar from that in Britain in that each owner has to employ an engineer, who has been approved as a qualified engineer, to be responsible for the safety of the dam. In Sweden the county councils carry out inspections with the support, if necessary, of experts paid for by the proprietor.

EMERGENCY PLANS

There is no legislative requirement for emergency plans in Great Britain although some owners of large dams have carried out dam break studies (Claydon et al, 1994). In many other European countries such plans are mandatory. For example, where dam failure could endanger life or property in Finland, the owner is obliged to prepare emergency plans in cooperation with the fire service authorities. In Italy the owner or operator is required to calculate the flood wave and warning sirens have to be installed. Public warning sytems have to be installed at all dams in Switzerland where failure would endanger human life.

In France, where a dam is 20 m or more in height and the reservoir has a capacity of $15x10^6$ m^3, special action plans are required based on risk analysis and dam break studies. In the zone reached by the flood wave within quarter of an hour, the population is alerted by sirens.

CURRENT DEVELOPMENTS

There are a number of developments which could encourage a movement towards harmonistion of practices and standards.

(a) Harmonisation of general working practices and the regulations governing those practices has been given a high priority in the EC and a series of Structural Eurocodes are being published to provide common unified design rules for different types of load bearing structures in different materials. At present it appears unlikely that these Eurocodes will have a direct bearing on reservoir safety as, for example, the section on embankments and slopes in Eurocode 7 (Geotechnical design - Part 1: general rules: DD ENV 1997-1; 1995) specifically excludes dams and dykes.

(b) The technical committees of ICOLD produce bulletins on a wide variety of topics many of which are relevant to reservoir safety. These bulletins are likely to increasingly influence national regulations and practice.

(c) The European Technical Symposium on Maintenance of Older dams which was held at Chambery in 1993 demonstrated that there was a growing awareness of the significance for dam safety of the political and economic developments taking place in Europe. The symposium initiated closer collaboration in research areas through a number of working groups. Of particular importance is the working group dealing with legislation (Le Delliou, 1995).

CONCLUSIONS

In the western European countries with significant numbers of large dams, reservoir safety is perceived to be a major concern and some countries have recently introduced, or are currently introducing, new regulations which embody higher safety standards. Reservoir safety management is primarily the responsibility of the owner or the operator in all the western European countries, but, in other respects, the approaches adopted to ensure safety show considerable differences which make harmonisation of safety regulations relatively difficult.

There do not appear to be any clear benefits from full harmonisation in terms of safety or economy. Differences in approach to reservoir safety need not present a barrier to trade or to professionals working in other European countries, although it should be recognised that the British system is inherently more open than the government inspection system which is operative in many other European countries.

Great Britain has a sound regulatory system, high safety standards and continues to benefit from a comprehensive research programme that has provided an up to date suite of engineering guides. In all these areas, it compares well with other European countries. It is important that the advantages of the distinctive British approach to reservoir safety, which include the independent qualified civil engineer and the engineering guides, are widely recognised. In view of the demonstrably well regulated system in Britain, the case for subsidiarity and preserving the current approach to reservoir safety needs to be understood. However, the fact that there is much that is worth preserving in the British system should not mean that no thought is given to what can be learned from other approaches to reservoir safety regulation.

Other European countries have larger reservoir capacity criteria and, under the heading of deregulation, the possibility of increasing the British reservoir capacity criteria could be examined. It is currently proposed to remove the requirement for a supervising engineer to be appointed where a reservoir has less than 100 000 m^3 capacity and where there is little or no downstream development that would be affected by dam failure (Department of the Environment, 1993).

Special safety measures might be considered for reservoirs assessed as presenting the greatest potential hazard including a formal requirement for mapping of the areas which would be inundated should the dam fail. The BRE Dams Database can provide valuable assistance in the evaluation of some of the effects of changes in regulations. Planned future work on risk assessment using information on the BRE Dams Database will also be relevant.

In the longer term there might be advantage in the development of acceptable European standards and qualifications for engineers who design, supervise and inspect the safety of the larger high hazard dams, especially those dams where failure would have trans-border effects.

REFERENCES

Biedermann R (1985). Dam safety in Switzerland. Swiss Dams: monitoring and maintenance, pp 24-33. Swiss National Committee on Large Dams, Zurich.

Carlier M (1973). French laws and regulations regarding dam supervision and inspection. Inspection, Maintenance and Rehabilitation of Old Dams. Proceedings of Conference held in Pacific Grove, California, pp 262-271. ASCE, New York.

Charles J A, Tedd P and Wright C E (1994). Safety assessment and legislation for reservoirs in Great Britain. Transactions of 18th International Congress on Large Dams, Durban, vol 1, pp 1223-1235.

Cifres Gimenez E C and Abad Moreno P A (1992). Assessing the PMF for its use in the Algar dam project (Valencia, Spain). Dams and Extreme Floods. Proceedings of International Symposium held in Granada, vol 1, pp 91-100.

Claydon J R, Walker R A and Bulmer A J (1994). Contingency planning for dam failure. Reservoir Safety and the Environment. Proceedings of 8th Conference of the British Dam Society held in Exeter, pp 224-235. Thomas Telford, London.

Czychowski M (1990). Die wasserrechtliche pflicht zur anpassung der vorhandenen talsperren an die heutigen anforderungen (The duty according to the water laws to make existing dams suitable for present day requirements). Stauanlagen im Wandel der Anforderungen, edited by G Rouve, pp 11-24. 20th International Hydraulic Engineering Seminar on Adapting reservoirs and dams to changing requirements, held at Aachen University of Technology.

Department of the Environment (1993). Proposed amendments to the Water Resources Act 1991 and the Reservoirs Act 1975: joint consultation paper. Issued jointly by the Water Directorate, Department of the Environment and the Environment Division, Welsh Office, October 1993.

Deutsche Norm (1986). DIN 19700 Part 11 Dam plants: dams, January 1986. (Translated into English.)

Dolcetta M and Paolina R (1993). Management of dam safety in Italy. Maintenance of Older dams: accidents, safety assessment, repairs. Proceedings of European Technical Symposium held in Chambery, pp 199-210. Comite Francais des Grands Barrages.

Franco M A and Cordova J Y (1988). The management of dam safety in

Spain: general perspective. Seminar on Dam Safety held in Rovaniemi, August 1988, pp 295-302. National Board of Waters and Environment, Helsinki, Finland, 1990.

Gil A and Del Hoyo R (1993). Safety for concrete dams. Maintenance of Older Dams: accidents, safety assessment, repairs. Proceedings of European Technical Symposium held in Chambery, vol 1, pp 285-305. Comite Francais des Grands Barrages.

Goubet A (1992). Safety. Dams in Europe and USSR: a geographical approach, pp 92-94. ICOLD, Paris.

Goubet A (1993). Regulations concerning dam safety in France. Unpublished notes distributed at Chambery symposium.

Heigerth G, Melbinger R, Obernhuber P and Tschernutter P (1994). Assessing and improving the safety of existing dams in Austria. Transactions of 18th International Congress on Large Dams, Durban, vol 1, pp 991-1026.

HMSO (1930). Reservoirs (Safety Provisions) Act, 1930. Her Majesty's Stationery Office, London.

HMSO (1975). Reservoirs Act 1975. Her Majesty's Stationery Office, London.

Hultin S A, Sjoborg K A, Bostrom H and Persson T (1982). Safety of dams in operation in Sweden. Transactions of 14th International Congress on Large Dams, Rio de Janeiro, vol 1, pp 447-457.

Hultin S A and Norstedt U (1988). Swedish embankment dams. Unpublished lecture to BNCOLD meeting on 7 March. Swedish State Power Board.

ICE (1978). Floods and reservoir safety: an engineering guide (reissued 1989). Institution of Civil Engineers, London.

ICE (1995). Information for Reservoir Panel Engineers. Institution of Civil Engineers, London.

ICOLD (1984). World Register of Dams; full edition. International Commission on Large Dams, Paris.

ICOLD (1988). World Register of Dams: updating. International Commission on Large Dams, Paris.

Konig F and Schmidt E (1991). Construction of dams in Austria: authorisation procedure. Dams in Austria, pp 53-63. Austrian National Committee on Large Dams, Vienna.

Kuusiniemi R, Loukola E and Maijala T (1994). Dam safety measures in Finland. Transactions of 18th International Congress on Large Dams, Durban, vol 1, pp 1251-1259.

LNEC (1990). Portuguese regulations for safety of dams. Laboratorio Nacional de Engenharia Civil, Lisbon.

Le Delliou P (1995). Comparaison de réglementations européennes relatives aux barrages. Proceedings of Conference on Research and Development in the Field of Dams held in Crans-Montana, Switzerland, pp 595-606.

Le Moigne G (1994). Supervision of dams by governmental authorities world-wide (2nd updating). World Bank, Washington DC.

Loukola E (1988). Some features of the role of different parties involved in dam safety. Seminar on Dam Safety held in Rovaniemi, August 1988, pp 252-258. National Board of Waters and Environment, Helsinki, 1990.

Morris D (1992). Surveillance and monitoring methods for Italian dams. Water Resources and Reservoir Engineering. Proceedings of 7th British Dam Society Conference held in Stirling, pp 361-372. Thomas Telford, London.

NBWE (1987). Dam safety code of practice. National Board of Waters and Environment, Helsinki, Finland.

Nicolaisen B (1990). Dam supervision, re-evaluation and rehabilitation in Norway. Stauanlagen im Wandel der Anforderungen, edited by G Rouve, pp 311-325. 20th International Hydraulic Engineering Seminar on Adapting reservoirs and dams to changing requirements, held at Aachen University of Technology.

NVE (1986). The Norwegian regulations for planning, construction and operation of dams (in English; Norwegian edition was published in 1981). Norwegian Water Resources and Energy Administration, Norwegian University Press.

NVE (1992). The Norwegian Dam Safety Project: Main Report 1992. (There are 7 subsidiary reports). Norwegian Water Resources and Energy Administration.

Pircher W (1991). The history of dam constuction in Austria. Dams in Austria, pp 29-44. Austrian National Committee on Large Dams, Vienna.

Schnitter N J (1985). The development of dam engineering in Switzerland. Swiss Dams: monitoring and maintenance, pp 11-23. Swiss National Committee on Large Dams, Zurich.

Spillner A (1990). Aufgaben und probleme des talsperrenwesens in Nordrhein-Westfalen. (Tasks and problems of dams in NRW). Stauanlagen im Wandel der Anforderungen, edited by G Rouve, pp 25-43. 20th International Hydraulic Engineering Seminar on Adapting reservoirs and dams to changing requirements, held at Aachen University of Technology.

UNESCO (1967). Recommendations concerning reservoirs. United Nations Educational Scientific and Cultural Organization, France.

Visentini G C (1992). Morning glory spillways and Italian regulations: a case history. Dams and Extreme Floods. Proceedings of International Symposium held in Granada, vol 1, pp 135-143.

Wright C E (1994). UK reservoir failures and safety legislation. Dams and Reservoirs, vol 4, no 3, pp 20-21.

Reservoir Inspections - Time for Change?

J HAY, Rofe Kennard & Lapworth, UK

SYNOPSIS
The Reservoirs Act 1975, like its predecessor, the Reservoirs (Safety Provisions) Act 1930, embodies the principle that an independent qualified civil engineer is solely responsible for the inspection of a large raised reservoir in the United Kingdom. This contrasts with some European countries where the responsibility for dam safety may lie with a panel of experts backed by a central authority. It may be that UK reservoirs legislation on dam safety will eventually be brought into line with European legislation. It is essential that the best features of the current UK system of reservoir inspections are retained in any new legislation.

INTRODUCTION
Concern for dam safety has always been at the heart of UK reservoirs legislation. The Reservoirs (Safety Provisions) Act 1930 [the 1930 Act] followed the collapse of dams at Skelmorlie (Ayrshire) and Coedty (N Wales) in 1925, with the loss of 21 lives. Failures in the 19th century, most notably at Bilberry (1845) and Dale Dike (1864), had resulted in the loss of over 400 lives (Charles, 1993). The 1930 Act laid down, in the interests of safety, procedures to be followed in the design, construction and inspection of reservoirs. The Reservoirs Act 1975 [the 1975 Act] introduced continuous supervision between statutory inspections; and enforcement, registration and reporting by local authorities.

The safety record of UK dams has improved since 1925, subsequent failures involving no loss of life (Wright, 1994). Many attribute this improvement to two main features of the legislation; the unique panel system, whereby engineers must be appointed to a panel before being permitted to perform specific functions; and the principle that an independent qualified civil engineer carries sole responsibility for the inspection of a large raised reservoir. The panel system ensures that the design, inspection and supervision of reservoirs can only be undertaken by well-qualified, experienced and competent professional engineers.

Social, political and economic pressures can compromise the ability of the inspecting engineer to act independently. Society has undergone huge changes since reservoirs legislation was first introduced; social attitudes have changed; the status of the engineer is lower; market forces now dictate policy; and safety laws are tighter. Statutory bodies such as the National Rivers Authority and various environmental pressure groups have been formed. The reorganisation of the water industry in England and Wales in 1974 led to the transfer of ownership of many dams from small water boards to large water authorities. This profoundly altered the relationship between the dam owner and the inspecting engineer, as did the later formation of the major water companies under privatisation.

Financial constraints have led to a lack of investment in reservoirs, cuts in maintenance, and, amongst the larger organisations, reductions in staff responsible for reservoirs. The safety recommendations of an inspecting engineer, despite being mandatory, are sometimes deferred, or not implemented at all, because of a lack of resources. Enforcement has generally not proved effective.

The paper is based on examples that the author has come across in the course of his work and in discussions with panel engineers. The views expressed are personal, and derive from a fundamental concern that the UK system of reservoir inspections has changed in recent years; that current reservoir legislation does not adequately reflect these changes; and that the long-term implications for dam safety remain uncertain. European safety legislation may be standardised at some future date, and this seems an opportune time for dam engineers to debate these issues.

THE ROLE OF THE INSPECTING ENGINEER

What the law states

The 1975 Act places the responsibility for inspecting a large raised reservoir in the United Kingdom on a single individual – the independent qualified civil engineer. He is appointed by the Secretary of State, following consultation with the Institution of Civil Engineers, to a particular panel depending on the type of reservoir being inspected – impounding, non-impounding or service reservoir. His is an individual appointment.

Under Section 10 of the 1975 Act, the independent qualified civil engineer ("the inspecting engineer") is appointed by the undertaker specifically to carry out the periodical inspection of a large raised reservoir. As soon as practicable after the inspection, he makes a report of the result of the inspection which includes any recommendations he sees fit to make as to the time of the next inspection; any measures that should be taken in the interests of safety; and a note of matters to be watched by the supervising engineer. The inspecting engineer also gives a certificate stating whether his report does or does not include safety recommendations and, if appropriate, the period within which the next inspection should be made.

The inspecting engineer has duties and responsibilities under the Health and Safety at Work Act, 1974. He is the technical expert and carries professional and legal responsibility. His duty is to exercise reasonable care and skill. If his inspection report did not give warning of any fairly obvious hazard, he could be liable for criminal prosecution, and open to action from any injured parties under civil legislation in the event of a failure or serious incident. It is the view of the Health and Safety Executive that a person can only be responsible for something that he has been given authority to change. This does not mean that an inspecting engineer does not have a responsibility to advise if a change is needed; only that he will not be blamed if others fail to follow his advice.

What happens in practice

An individual inspecting engineer will fulfil his responsibilities in a manner which reflects his age, knowledge, experience, attitude and personality. Some may adopt a legalistic, conservative, uncompromising approach; others may be more flexible and pragmatic. This can lead to a lack of consistency both in the manner of carrying out inspections and in the preparation of inspection reports.

Particular problems can arise when a new inspecting engineer adopts different criteria from those of his predecessor. For example, there can be significant cost implications for the undertaker when an inspecting engineer places a reservoir in a different category, and adopts a different design flood from his predecessor. "Floods and reservoir safety: an engineering guide" (ICE, 1978) recommends that reservoirs be placed in one of four categories depending on the size of community at risk downstream. It is understood that the revision of the guide to be published in 1996 (ICE, 1996), which will supersede the earlier guide, retains the same four categories, and the return period and PMF flood standards. Guides are not mandatory, but an inspecting engineer who chose to depart from the recommended standards would be expected to explain in his report why he had done so.

Despite efforts by the Reservoirs Committee of the Institution of Civil Engineers to clarify the meaning of safety in the context of the 1975 Act (ICE, 1987), it is known that inspecting engineers continue to interpret it in different ways; some believe that the integrity of the dam itself has to be considered, others that public safety is the criterion. There is at least one example of a category "A" reservoir lying immediately upstream of a category "B" reservoir with no community between the two.

The overall picture is confusing, perhaps because hazard and risk assessment is not part of the normal civil engineering prospectus and few engineers will have received adequate training in the subject. There are those who feel that the selection of the reservoir category is too important to be left to the subjective judgement of one individual but should be carried out by specialists using more rigorous methods of hazard and risk assessment.

Despite the fact that the inspecting engineer fulfils his obligations with the submission of his report, many undertakers have come to expect him to have an on-going involvement in a reservoir and to be available to offer further advice as and when it is needed. However, the on-going commitment of the inspecting engineer is rarely covered by a separate agreement under which he can be paid, and the context of any further advice which he gives is unclear. To clarify matters, and safeguard themselves, some inspecting engineers are now only prepared to offer further advice if they are formally retained to do so.

Some inspecting engineers are known to feel exposed under current legislation. Understandably, they adopt a conservative approach to inspections. Evidence of this may be seen in the contents of inspection reports; the wording of safety recommendations; and in restrictions placed on consultations between the inspecting engineer and supervising engineer.

<u>Inspection Reports</u> Some inspecting engineers now include a caveat in their inspection reports drawing attention to the status of the inspection and the report. They set out clearly the scope of the inspection, stating that such matters as the overall stability of the structure, the condition of the foundations, the forces on the foundations, and the capacity of the overflow works have not been assessed in detail, adding that the report is not an absolute assurance or guarantee of total reservoir safety in the future. Other inspecting engineers may choose to analyse embankment stability, assess the magnitude of floods and calculate the overflow capacity, before issuing their inspection report. The implication from Statutory Instrument 1986 No. 468 (HMSO, 1986) which states that the inspection report should contain the engineer's findings as to "the adequacy and condition of the waste weir or overflow ....", is that a flood assessment is necessary before the report is submitted.

As a result of a more legalistic approach by inspecting engineers, perhaps compounded by the pressures of fee competition, inspection reports nowadays tend to contain more "loose ends" and are often conditional on further work. A recent inspection report recommended in the interests of safety (and therefore mandatory) a stability analysis of the dam; a flood study; a check on the capacity of the overflow works; a check on the hydraulic capacity of the outlet pipe - all as additional work. The report would be of limited use to the undertaker until such time as the results of the additional analyses were available; only then could a decision be taken as to whether any remedial measures were needed.

Safety recommendations Safety recommendations may sometimes be used to demonstrate that the inspecting engineer has considered a particular matter and cannot be accused later of failing to point out a hazard. One inspecting engineer acknowledged that he used such recommendations simply to "cover his back". These recommendations can often be difficult, if not impossible, to implement satisfactorily. An example is that of a reservoir with no bottom draw-off where the inspecting engineer recommends the setting up of an emergency procedure requiring the installation of a large number of temporary pumps to lower the water level quickly. Whilst the inspecting engineer may have addressed the hazard posed by the reservoir, the undertaker may have been left with a safety recommendation that is almost impossible to implement wholly satisfactorily to cover every conceivable eventuality.

Communication between inspecting engineer and supervising engineer One firm of consulting engineers has instructed that any internal discussions that take place between their own inspecting engineer and supervising engineer for a particular reservoir (after the inspection report has been submitted) should not be formally recorded in the supervising engineer's annual statements because of fears that the inspecting engineer could be compromised. If a safety incident were to occur at a later date as a consequence of an action which had been brought to the inspecting engineer's attention, he might be deemed to have acquiesced to what had been done. The firm's actions are in response to the current legislation. Their concerns must be recognised, particularly since the inspecting engineer's responsibilities are fulfilled with the submission of his report, and the 1975 Act does not define the context of any further advice which he may give.

The above examples are an indictment of the current system of inspections rather than of any individual. Reservoir safety depends on effective communication between the responsible parties but it seems that certain aspects of the legislation discourage this. A legalistic approach will almost inevitably lead to a breakdown of communication. Inspecting engineers will be reluctant to share ideas and freely offer advice if they can be held liable at a later date for doing so.

<u>How the role of the inspecting engineer is perceived</u>

There is a commonly-held view, often propounded by panel engineers themselves, that the inspecting engineer has "undivided responsibility" for the safety of a reservoir. The basis for this is not clear; the Act places no duty or responsibility on the inspecting engineer other than those outlined above, and the case of Rylands v. Fletcher (1868) established that an undertaker has ultimate responsibility for ensuring the safety of his reservoir. The idea may have arisen under the 1930 Act when, under normal circumstances, the inspecting engineer would be the only technical expert responsible for a reservoir; even then his strict legal obligations related only to the inspection. The supervising engineer and enforcement authority (both introduced under the 1975 Act), with longer term roles for supervising and overseeing a reservoir respectively, must presumably also carry some responsibility for ensuring its safety.

Some undertakers still regard the inspecting engineer's report and certificate as guaranteeing that the reservoir may safely be used to store water until the next inspection; many believe that if they receive a "satisfactory" report for, say, 0.3 PMF, then the reservoir is safe for all eventualities. However, unlike the construction engineer issuing the final certificate for a reservoir which he has deemed satisfactory, the inspecting engineer is not required to state in his certificate that the reservoir is sound and satisfactory and may safely be used for the storage of water, this only being implied in the absence of any recommendations in the interests of safety. The situation is comparable to that of the MOT certificate for a car. Such a certificate does not guarantee safety for the ensuing year as the condition of the vehicle can change. Similarly, the condition of a dam and reservoir can change.

HOW THE SYSTEM OF RESERVOIR INSPECTIONS HAS CHANGED

<u>Changes in Legislation</u>

Although the defined role of the inspecting engineer has remained essentially the same, the system of reservoir inspections has undergone significant change since it was first introduced in 1930. Improvements have been made in response to pressure from undertakers and engineers in the light of experience. The 1975 Act introduced continuous supervision of reservoirs between statutory inspections and procedures for enforcement. A series of statutory instruments set out prescribed information such as the content of enforcement authority registers, certificates and reports from panel engineers, etc. Legislation is being enacted at the present time which will transfer responsibility for enforcement from local authorities to a new Environment Agency.

Reservoirs Committee Advice

In 1986, responding to pressure from the Country Landowners Association on behalf of owners of small private reservoirs, the then Minister for Housing, Urban Affairs and Construction, expressed the hope that inspecting engineers making recommendations in the interests of safety would exercise their discretion in such a way as to keep expenditure to a scale justified by the risk. In 1987, the ICE Reservoirs Committee (ICE, 1987) issued advice to inspecting engineers on what constitutes a safety recommendation, pointing out at the same time that "safety" was not defined in the Act and what constituted "safety" could only be decided by a ruling of the Court. The Reservoirs Committee's most recent publication "Information for Reservoir Panel Engineers" (ICE, 1995) contains helpful background information and more advice, including suggested check lists for inspection reports and supervising engineers' annual statements.

Relationship between owner and inspecting engineer

In 1930, most reservoirs were owned by small private owners or by local authorities and waterworks companies, which were usually small by today's standards. The inspecting engineer could expect to form a working relationship with the owner or the Engineer and Manager of the water company. Nowadays, this seldom happens in the large modern water companies. The senior executive post will often be occupied by a lawyer or accountant, and the inspecting engineer will deal with less senior members of staff, often in a specialist reservoir department. The inspecting engineer is more remote from decision-making; dam safety matters may be given a lower priority than he considers appropriate.

Technical Advances

Increasingly rapid advances in scientific knowledge and computer technology continue to revolutionise engineering design. More and more the inspecting engineer finds himself having to turn to specialists for advice. He may be expected to keep up to date with the latest technical advances in dam engineering through membership of learned societies, attendance at conferences and technical meetings. The informal Panel AR forum serves the specialist interests of inspecting engineers. Technical guidance for undertakers and engineers is contained in a series of engineering guides, in published papers and in reports of research studies, many under the DOE Reservoir Safety Research Programme.

PRESSURES FOR FURTHER CHANGE

The Need for Further Change

Since its introduction in 1930, the system of reservoir inspections has significantly improved the safety record of dams in this country. Almost all large raised reservoirs in the UK now receive a statutory inspection in compliance with the 1975 Act. Is there a need for further change? There have been a number of well-documented emergencies and failures since the system was introduced, and there is no room for complacency. Dam safety is not being taken seriously in some quarters; there is a lack of public awareness; there are instances of safety recommendations not being implemented and enforcement authorities not taking action. Other recent developments which have affected the system of reservoir inspections are:

Shortage of panel engineers

The slump in dam building in the UK has meant fewer opportunities for civil engineers to obtain relevant experience in the design and supervision of construction of new large raised reservoirs. It had been feared that this might lead to difficulties in finding suitable candidates for appointment to Panel AR, and it was one of the main reasons behind the recent move to revise the panel structure. A contrasting view is that there are enough overseas opportunities to enable engineers to obtain the necessary experience in dam building. Changes to the panel structure have been deferred because there is no shortage at the present time; but the situation will have to be kept under review. A shortage of engineers of the right calibre would place undue strain on the system and on those panel engineers already registered.

Prior consultation

Some undertakers now insist on being consulted in advance on the proposed content of inspection reports. This may seem perfectly reasonable, especially where (mandatory) safety recommendations can lead to expensive remedial measures. It also gives an undertaker the opportunity to check that a report is factually correct and to comment on certain types of safety recommendations which may cause difficulty (Hay, 1994). Consultation can however lead to compromise; some inspecting engineers, wishing to remain completely independent, have refused to consult with a dam owner, or even to submit a draft of their inspection report in advance, for this reason. Any inspecting engineer who does not accede to a dam owner's request to be consulted is unlikely to be reappointed for a subsequent inspection. However, if he does compromise after consulting the owner, existing legislation does not recognise any sharing of responsibility. The inspecting engineer still carries full responsibility for the advice which he gives.

Loss of reservoir staff

Re-organisation, principally amongst the larger undertakers, has led to a reduction in manpower. Many experienced engineers and reservoir keepers have opted for early retirement with the inevitable loss of valuable background knowledge and operational experience, as well as the presence of a full-time on-site observer. Ironically, these cuts in manpower have taken place since the 1975 Act sought to increase the level of supervision of reservoirs by introducing continuous supervision between inspections by the supervising engineer. He was intended to supplement, not replace, the routine surveillance provided by the undertaker's staff. At the time of his inspection, the inspecting engineer may well be presented with less relevant up-to-date local information than has been the custom in the past. Of even greater concern, early warning of a serious development affecting safety may not be noticed until too late.

Short termism

The inspecting engineer must assess the condition of a dam's pipework system; the use of the wrong type of valve, or equipment with a lower design life, introduces another uncertainty, and may be unsafe. There are many examples of engineering design being compromised by cost considerations. The more substantial waterworks pattern valves, traditionally used in the past on water projects, are now rarely specified on grounds of cost. Butterfly valves, although unsuitable for controlling flows except at relatively low heads, are often used to replace defunct control valves simply because they are much cheaper. It is recognised that these replacement valves are not capable of withstanding the service conditions for long and will themselves have to be replaced again within a few years.

Fee competition

The introduction of fixed fees and fee competition for reservoir inspections seriously threatens the independence of inspecting engineers and their ability to exercise care and skill impartially. Reservoir inspections cannot be defined precisely in advance. The inspecting engineer alone determines the extent of the inspection; how long it should take; what analyses he deems necessary; what further information he requires; and the contents of his report. He may decide that he needs specialist advice. How should he respond if that advice is denied him either because the fee has been exceeded or the need for it was not foreseen, and allowed for, at the outset?

A distinction has to be made between inspections carried out by an inspecting engineer who is a private individual, or an independent consultant, or employed by a firm of consulting engineers or other commercial organisation. The private individual, with fewer overheads, may quote a lower fee for an inspection. He may not however, have the same ready access to the specialist "in-house" expertise of the consulting engineer, nor the same level of professional indemnity cover.

Reconciling one's individual commitment to dam safety with the interests of a commercial organisation can be difficult; more so when the inspection is the outcome of fee competition. It is known that some inspecting engineers in private firms have had to undertake inspections in their spare time and have had the use of support staff denied to them because the fee has been exceeded. If this situation were to recur frequently, clearly there may come a time when the individual or the firm would decline reservoir inspection work, further adding to the pressure on existing panel engineers.

At meetings of the panel AR forum and at the BDS Exeter Conference, inspecting engineers have wholeheartedly endorsed the principle that reservoir inspection work should be time-based at an appropriate hourly rate, especially since the amount of time involved often cannot be known before an inspection and a study of the available drawings and other relevant information. In spite of this, fee competition is becoming more widespread – apparently at the insistence of some of the major undertakers. The risks must be recognised; inspections undertaken at unrealistically low fees can compromise the inspecting engineer and endanger dam safety. Fee competition is not appropriate for reservoir inspections. The reality is – the undertaker gets what he pays for.

Loss of continuity

A change of inspecting engineer can also mean loss of background information and detailed knowledge of a reservoir. The previous inspecting engineer, or his firm, will have collated prescribed records, earlier inspection reports, construction drawings, surveillance records. He is likely to be familiar with the reservoir and aware of its idiosyncrasies. A new inspecting engineer cannot be expected to assimilate the information to the same degree, particularly if constrained by a tight budget.

A fresh appraisal may however be beneficial at some stage; an inspecting engineer should not necessarily expect to be automatically re-appointed to inspect a particular reservoir. Nevertheless the benefits of maintaining continuity must be recognised; changing the inspecting engineer at each inspection should perhaps be the exception rather than the norm.

Confidentiality

Some undertakers have prevented the publication of technical papers, either because they wished certain information to remain confidential or they wished to maintain what they regarded as some form of commercial advantage. Such a policy is alien to engineers who have always willingly shared technical expertise, recognising the benefit to themselves and others of having their work scrutinised by their fellow professionals.

INTRODUCING CHANGE

In the current climate of deregulation, it seems unlikely that any further significant changes in UK reservoirs legislation will be forthcoming in the foreseeable future unless a major safety incident occurs, raising public awareness. Reservoir inspections however are not simply defined by legislation. They also reflect current engineering practice and what is to be expected of an inspecting engineer by his peers – an important aspect if the inspecting engineer's advice was to come under scrutiny at a later date. Dam engineers, who understand the technical issues involved, are best suited to define current dam engineering practice and what should be contained in an inspection.

It is suggested that the dam engineering community might be consulted on the issues raised in the paper, with the aim of reaching a consensus on bringing about some measure of consistency in the carrying out and reporting of inspections. Following the consultation process, advice could be issued by the Reservoirs Committee, thus avoiding the need for prescriptive legislation. This would hopefully lead to a better understanding of the role of the inspecting engineer and the external pressures which he now faces with tighter safety laws, financial pressures, and fee competition. Areas of uncertainty could be clarified, such as the categorising of reservoirs; the inspecting engineer's relationship with other parties; the inspecting engineer's continuing involvement with a reservoir, and the context of any further advice which he gives, after his inspection report has been submitted.

Harmonisation with European legislation

A delegation of UK dam specialists visited seven European countries in 1993 under the auspices of the Department of Trade and Industry's Overseas Science and Technology Expert Mission Scheme (OSTEMS). The aim was to promote greater awareness amongst dam engineers in the UK of the European approach to dam safety. They suggested several areas where Europeans may feel the Reservoirs Act 1975 should be strengthened: regulation of design and construction; hazard assessment; involvement of local authorities; emergency planning. They reported that reservoir inspections in Europe are usually carried out by a panel, with the backing of a central authority, the "Dam Safety Office" or "Dam Commission". None of the seven European countries visited allow inspection of large dams by an individual engineer. A Dam Data Book, including statistical data, operating and maintenance instructions, and an emergency action plan is a common requirement in Europe.

The delegation reached the worrying conclusion that "dam safety in Britain requires more attention than it is receiving". This despite the fact that few countries had dam safety legislation that was as widely adhered to as was the legislation in Britain.

There is a clear need for the important issues raised by the delegation to be fully investigated as a matter of urgency. At some point, we will have to reconcile the state–controlled prescriptive approach, which sets design standards, and procedures for construction inspection and surveillance, with the typical UK system, where responsibility is devolved to individuals and agencies. A major advantage of the UK system is that only registered panel engineers can be responsible for the inspection and supervision of dams.

CONCLUSIONS

- The UK system of reservoir inspections has improved dam safety since its introduction in 1930.
- The system has been modified at intervals in the light of experience.
- The system could be further improved to ensure an ongoing commitment to dam safety and to address known problems such as a lack of consistency in carrying out inspections and the effect of legal and financial pressures on inspecting engineers.

- The system should enable inspecting engineers to communicate freely and effectively without being compromised.
- Further guidance is needed from the Reservoirs Committee to clarify matters such as the uncertainty surrounding the categorising of reservoirs; the sharing of responsibility; whether an inspecting engineer should have an on–going involvement with a reservoir after his inspection report has been submitted; and if so, whether it should be on the basis of a formal appointment.
- The UK registration system for panel engineers should be retained in any pan–European legislation.

It is hoped that these and other issues will be constructively debated by all those responsible for dams, so that a clear policy towards dam safety can be developed to assist those formulating future legislation in the UK and Europe.

ACKNOWLEDGEMENT

The author wishes to thank friends and colleagues who are inspecting engineers, and who have willingly shared their experiences of carrying out inspections under the current legislative framework.

REFERENCES

Charles, J A (1993). Embankment dams and their foundations: safety evaluations for static loading. Dam Safety Evaluation. Proceedings of International Workshop held at Grindelwald, Vol 4, pp 47–75. Sutton. Dam Engineering.

Department of Trade and Industry. Overseas Scientific and Technical Expert Mission Scheme (OSTEMS), Dam Safety in Europe (1993). London. CIRIA.

Hay, J (1994). Implementing Safety Recommendations on UK Dams – A Supervising Engineer's View. Proc. 9th Conf. British Dam Society, Exeter, pp 247 – 259. London. Thomas Telford.

HMSO (1986) Statutory Instrument 1986 No. 468. The Reservoirs Act 1975 (Certificates, Reports, and Prescribed Information) Regulations, 1986.

Institution of Civil Engineers (1978). Floods and reservoir safety: an engineering guide (re–issued 1989). London. ICE.

Institution of Civil Engineers, Reservoirs Committee (1987)
Circular dated 16th February 1987 to all Engineers appointed to Panels AR, NIR and SR. London. ICE.

Institution of Civil Engineers (1995). Information for Reservoir Panel Engineers. London. ICE.

Institution of Civil Engineers (1996). Floods and reservoir safety: an engineering guide. 3rd edition. London. ICE.

Rylands v Fletcher (1868) LR3 HL 330.

Wright, C E (1994). UK reservoir failures and safety legislation. Dams and Reservoirs, Vol. 4, No. 3, October, pp 20–21. London. BDS.

Management of Reservoir Safety

P MILNE, Severn Trent Water Ltd, UK

SYNOPSIS. Reservoirs represent a major asset (or liability) to owners. Effective management is essential to ensure continued reservoir safety, and without the need for undue or unreasonable levels of expenditure. This paper describes the management systems used by Severn Trent Water Ltd to help achieve this, and includes our relationships with Panel Engineers and other professionals. Further, it describes how the Company plans for, and deals with, potential emergency situations.

INTRODUCTION

Severn Trent Water Ltd (S.T.W.) is the second largest UK water services company, the turnover for 1994/95 being £847m. This paper concentrates in particular on the Company's approach to maintaining and operating its raw water reservoirs and large service reservoirs. It outlines briefly the duties and obligations of S.T.W and how these assets are effectively managed.

Reservoirs are generally considered to be a low risk/high consequence scenario and careful management of these risks is essential.

LEGAL REQUIREMENTS

S.T.W. has to meet two legal requirements with regard to reservoirs:-

Reservoirs Act 1975

Reservoirs covered by this Act are defined as 'Large Raised Reservoirs' ie they are designed to hold, or be capable of holding, more than 25,000 cubic metres of water above the natural level of any part of the land adjoining the reservoir. Currently S.T.W. owns 67 Large Raised Reservoirs, of four different types:
28 Impounding; 11 Pumped Storage; 23 Service; 5 Surface Water.
The asset value of these reservoirs is £1billion.

Common Law

This applies to the remainder of S.T.W's reservoirs (including towers), around 800 in total, the majority of which are small service reservoir tanks on the distribution system. The total value of these 'tanks' is £500m.

MANAGEMENT OF SAFETY

Management of safety of S.T.W's reservoirs is primarily designed to meet the requirements of the Reservoirs Act 1975. This work is primarily undertaken by the Reservoirs Section, part of Severn Trent Engineering, which provides a specialist service associated with the Act and in other areas related to that expertise. Similar, but less stringent principles apply to the 'non statutory reservoirs'.

Reservoirs Section
Consists of 10 personnel based at various locations throughout the region ie. Birmingham, Leicester, Coventry, Warwick, Shrewsbury. This ensures a high quality local service to customers. The key to this service lies with the Reservoir Supervising Engineer - S.T.W. directly employs two members of the DOE/ICE Supervising Engineer Panel to 'watch' the reservoirs and to keep the Company advised of their behaviour. Each Supervising Engineer is supported by two technicians.

To achieve regional consistency on matters of safety, S.T.W. has identified criteria for the Supervising Engineers and support staff. Each dam has to be visited by "Reservoir Section personnel" at least four times a year; each service reservoir a minimum of twice a year. Internal inspections of service reservoirs are undertaken whenever the reservoir is taken out of service for maintenance purposes or as required by the Inspecting Engineer. Each visit to a reservoir will generate an Occasional Visit Report (see Appendix A). Under Section 12 of the Reservoirs Act 1975 the Supervising Engineer is required at least annually to submit to the Company a Report on those matters to be 'watched', as identified by the Inspecting Engineer. In addition, S.T.W. requires that comments on all visits to site throughout the year are included. The Report is to a regional standard (see Appendix B) and requires that the Supervising Engineer confirms (or otherwise) that the reservoir is behaving satisfactorily. The Report includes for operational requirements as well as safety items, together with an Action Plan which identifies capital/revenue works to be undertaken, and agreed timescales. The completed Report/Action Plan is to be completed, signed for by the accountable manager(s), and issued within six weeks of the Inspection date. Normally, Supervising Engineer Reports are not forwarded to the Enforcement Authority. Inspection and the Report dates are fixed annually at least twelve months in advance and are a monitoring mechanism for the Supervising Engineers. Some Reports generate capital works and, as the scheme progresses through the Company Procedure, the Supervising Engineer will act as the 'expert advisor' to ensure continued reservoir safety.

Three regional surveyors provide part time support to the Supervising Engineers. Dams generally are levelled only on an annual basis, although at a few selected sites geodetic surveys are undertaken. No regular surveys are carried out at service reservoirs.

For non statutory reservoirs, a different regime exists. Recently S.T.W. established in each of the fourteen water distribution Districts and the three Water Supply Groups, the new position of Service Reservoir Inspector whose role is to undertake inspections, and ensure cleaning and remedial works are undertaken. Internal inspections will, on average, be once in every five years, and annually for external inspections. In addition, these Service Reservoir Inspectors are able to request advice, guidance and assistance from the Supervising Engineers/technicians when appropriate.

ROLE OF INSPECTING ENGINEERS

Each large raised reservoir is to be inspected by an appropriately qualified independent DOE/ICE Panel Engineer who has to prepare a Report and Certificate of the inspection. These reservoirs are inspected within ten years of the last inspection, or at a lesser interval as determined by the last Inspecting Engineer. The Inspecting Engineer's Report will generally conform to a standard format (set by the DOE) and works required 'in the interests of safety' are mandatory on the Company. Much of the Inspection can be said to be subjective and, whilst there are several 'Guides' available for use by Inspecting Engineers and others, there are no definitive documents. This makes the choice of Inspecting Engineer all the more important - common sense and realism are as important as technical ability. Historically we have experiences where Inspecting Engineers have made recommendations in the interests of safety and which are more related to maintenance aspects, or cannot be certified as required by the Act. In addition, there have been occasions where recommended expensive construction works have subsequently proved to be unnecessary.

For the appointment of Inspecting Engineers, S.T.W. now tends to use one of the six or seven Panel Engineers who have historically provided a good value for money service and who meet our requirements. Each reservoir tends to be 'matched' to an Inspecting Engineer. Fixed fees are usually required, and occasionally fee competition between Panel Engineers is sought. Inspecting Engineers are requested to submit draft reports to enable facts to be verified and also to give S.T.W. the opportunity to make suggested amendments to the text. S.T.W. aims to ensure that the Report is factually correct and that all comments are pragmatic such that they are unlikely to create difficulties during future inspections. Panel Engineers are requested to submit Reports within the same timescale as given to S.T.W. Supervising Engineers, ie six weeks, and generally good compliance is achieved. It is the responsibility of the Supervising Engineer to ensure that an Action Plan is prepared on the Inspecting Engineer's Report, and that this Plan is complied with.

ROLE OF REVIEW PANEL

The Company continues to use the services of the Carsington Dam

Reconstruction 'Review Panel' to advise on matters principally associated with dam and geotechnical engineering but not exclusively. The Panel consists of three eminent individuals/experts, together with a specialist academic, and is under the chairmanship of Mr R E Coxon. The Review Panel has in the past six years provided real assistance and advice to S.T.W, usually within a short timescale, and has without doubt saved unwarranted and unreasonable expenditure.

MANAGEMENT SYSTEMS

With the benefit of hindsight, most dam failures could or should have been predicted and perhaps action taken to reduce the subsequent consequences. The identification of change and how this should be dealt with is therefore important, and systems/procedures need to be in place to help achieve this. The long term surveillance and monitoring of our large raised reservoirs is of prime importance but generates huge quantities of data. Most monitoring regimes are recommended by previous Inspecting Engineers, although S.T.W. believes that the observations and comments of the Supervising Engineer and the owner should also be considered.

Effective management and assessment of this data is really only possible using a computer. Currently in S.T.W. this data (which includes levels, line, piezometer details, reservoir water level, rainfall, etc etc) is stored on the PC based 'US Corps of Engineers' package. Typical long term outputs are on a time series basis. Within S.T.W it is the Supervising Engineer's responsibility to initially assess this data and comment within his annual Report. Should the Supervising Engineer be unsure or have any doubts relating to this data, he has access to the Review Panel or a Panel Engineer.

Reservoir Files

The Reservoirs Act 1975 identifies the minimum records to be kept, but from a S.T.W. viewpoint, these are considered inadequate. Further, five years ago, details on the majority of statutory reservoirs were scattered throughout our region - not very desirable. S.T.W. established the need, for each statutory reservoir, for all the relevant technical information to be collected, collated and vetted and placed in a regionally formatted system, called the Reservoir Files, (standard format given in Appendix C). This has the benefit of the Files being consistent in format but holding different technical details. For each statutory reservoir a Reservoir File is held centrally at Birmingham Headquarters, with the area offices at Leicester and Coventry holding identical copies for their area. This ensures security of information. An audit system has been established to ensure continued commonality between locations. Currently S.T.W. holds paper records of all data despite the significant volume of storage space required. The possibility of transferring this to a computer base has been appraised but

S.T.W. is not currently convinced of the technical nor economic advantages - no doubt as technology improves and costs reduce this will become a more realistic proposal.

EMERGENCIES

For the past seventy years no UK dam collapse or partial failure has resulted in loss of life. Undoubtedly this is due to a combination of effective reservoir safety management and good luck. As an owner, S.T.W. obviously has to concentrate on the first of these.

Effective management of reservoirs reduces the risk associated with dam failure; however, we should be prepared to deal with the emergency situation should/when it arises. S.T.W. has established a standard Emergency Procedure to apply to all functions including dams/reservoirs. This Procedure is not site specific but one which can be directly related to specific needs. Within the dams/reservoirs procedure, the Supervising Engineers are identified as having the responsibility of initially deciding the nature/level of the problem, course of action to be taken, assistance required, etc. Under normal circumstances, the Supervising Engineers can be at most sites within two hours of receipt of a call. Further external assistance can be urgently obtained via the Review Panel, or selected Qualified Civil Engineers.

As an integral part of the Emergency Procedure, for virtually all the raw water and the larger service reservoirs, S.T.W. has undertaken studies which give an indication of the possible loss of life and damage caused downstream, as a result of possible major structural failure. This information and detail is shared with all the appropriate and selected organisations. Various parts of S.T.W's Emergency Procedure are tested on a regular basis; at least annually at director level. The last exercise specifically involving dams/reservoirs was a 'Gold Level Exercise' involving Directors, and Chief Officers of a County Council and all the Emergency Services. The scenario was of a seismic event which resulted in a dam breach with the reservoir contents spilling into the downstream river and major conurbations. These exercises have proven value, and are of significant benefit when the Company is required to react to a real emergency, as experienced in the 'Worcester Incident'.

In addition to our Reservoir File, the Company has established the Emergency Reservoir Information Catalogue (ERIC). This consists of two manageable A4 files which contain all the relevant information for the large raised reservoirs to assist 'Panel Engineers' in the initial stage of an Emergency. Details include how to get to site, valve layout/details, contact numbers, brief statements regarding site history, potential problems etc.

CONCLUSIONS

Reservoirs provide S.T.W. with a low risk/high consequence scenario. It is essential that the management of reservoirs, particularly related to safety and the Reservoirs Act 1975, is such that, should the unexpected occur the Company is well placed to act quickly and effectively.

The day to day safety responsibilities lie with the 'in house' Supervising Engineers who are able to obtain further advice and assistance from our Review Panel or Panel Engineers.

Careful selection of the independent 'Inspecting Engineer' is essential.

With the establishment of our Reservoir Files, including storage on PC of the long term surveillance and monitoring data, and other systems, it is considered that S.T.W. is able to manage safely, effectively and economically the large raised reservoirs.

ACKNOWLEDGEMENT

The author is grateful to Severn Trent Water Ltd for permission to publish this paper.

APPENDIX A

RESERVOIRS ACT 1975 - SECTION 12

RESERVOIRS TECHNICIAN'S OCCASIONAL VISIT REPORT

AREA:	RESERVOIR NAME:
DATE OF VISIT:	WATER LEVEL:
WEATHER:	SHEET ... OF ...

REPORT* (Continue on separate sheet if necessary)

ACTION REQUIRED | RESPONSIBILITY

DATE: SIGNED: (Reservoir Technician)

AGREED:

DATE: SIGNED: (Reservoir Supervising Engineer)

* Report to include purpose of visit, points identified, photographs taken etc.

APPENDIX B

Standard Section 12 Report Format (Page No 8)
and Example Front Sheet (Page No 9)

Front Cover	-	Statement/Brief Details
Section 1	-	Introduction
Section 2	-	Statutory Inspection Details
Section 3	-	Identification of Unique Events
Section 4	-	Embankment/Dams
Section 5	-	Hydraulic Aspects
Section 6	-	Control/Designated Valves
Section 7	-	Ancillaries
Section 8	-	Investigative/Geotechnical Works
Section 9	-	Monitoring/Surveillance Records
Section 10	-	Prescribed Form of Record
Section 11	-	Health and Safety at Work
Section 12	-	Summary

Annexure

A	-	Key Plan
B	-	Specific Drawings
C	-	Designated Valve Operation Record
D	-	Agreed Action Plan

RESERVOIR FILE SECTION 10

SEVERN TRENT WATER LIMITED
RESERVOIRS ACT 1975

SECTION 12 - SUPERVISING ENGINEERS FINAL REPORT
EMBANKMENTS AND DAMS

Name of Reservoir

Grid Reference

Type of Reservoir

Total Capacity of Reservoir (cubic metres)

Date(s) of Site Inspection

Weather Conditions

Water Level

Category of Structure
(as defined by ICE Flood and Reservoir Safety Guide)

Type of Dam

Height of Dam

Design Flood

Head above Overflow at Design Flood

Wave Surcharge

Inspection Engineer Details - Section 10 Inspection

Last Inspecting Engineer/Company

Date of Last Inspection Report Certificate

Expiry of Current Certificate

Works Recommended in the Interests of Safety and dates completed

Supervising Engineer Details - Section 12 Inspection

Named Supervising Engineer

Date of Last Supervising Engineer's Report

This Report meets the obligations of the Supervising Engineer and the Company under the Reservoirs Act 1975.
The Reservoir has/has not been found to operate satisfactorily (see Action Plan attached)

Signature .... Date ....
Reservoir Supervising Engineer

APPENDIX C

RESERVOIR FILE CONTENTS FORMAT

Reservoir Name :   Date :

Section 1	-	Index
Section 2	-	Reservoir/Dam Physical Data
Section 3	-	Reservoir Book - Statutory Requirement
Section 4	-	Design Detail, Revised Analysis/Report, Hydrological Assessment, Hydraulic Details
Section 5	-	Site Investigation Details
Section 6	-	Additional Reports, Details etc
Section 7	-	Construction Works Details
Section 8	-	Construction Engineer Report/Certificates
Section 9	-	Inspecting Engineer Report/Certificates plus Action Plan
Section 10	-	Supervising Engineers Report plus Action Plan
Section 11	-	Enforcement Authority Details
Section 12	-	Reservoir Data Base
Section 13	-	Surveillance/Monitoring Details
Section 14	-	Drawing List (drawings on negative & microfiche held elsewhere)

Development of National Dam Safety Assurance Programme in Indonesia

D GALLACHER, Senior Partner, Robert H. Cuthbertson & Partners, UK
G N R LAMOND, Partner, Robert H. Cuthbertson & Partners, UK

SYNOPSIS
The World Bank funded Indonesian Dam Safety Project commenced in May 1995, and is scheduled for completion in March 2000. The Project comprises three components - Institutional Strengthening and Training, Dam Rehabilitation Programme and Basic Dam Safety Facilities Programme. The Project provides for support to the Directorate General of Water Resource Development (DGWRD) in the Ministry of Public Works (MPW) in respect of programming, budgeting and reporting on the project activities, establishment of Central and Provincial Dam Monitoring Units (DMUs) and assistance to the Dam Safety Unit (DSU) and DMUs in the execution of their duties, including design reviews for new dams and modification of existing dams, inspection, monitoring, surveillance and routine inspection of dams. The development and implementation of appropriate legislation and regulations on dam safety is a key aspect of the Institutional Strengthening component, providing the basis for the establishment of a national dam safety assurance programme in Indonesia.

1 INTRODUCTION

1.1 Initial Preparation

The Preparation Project for the Java Irrigation Improvement and Water Management Project (JIWMP) which commenced in March 1992 included a Dam Programme comprising the following sub-components:

- Dam Inventory and Safety Inspections
- Review of schemes prepared for rehabilitation of six dams in Central and West Java
- Dam Safety Assurance and Dam O & M Programme

A number of Brief and Detailed Safety Inspections of dams were carried out from late 1992 to early 1993 and recommendations were made on measures to be implemented in the interests of safety. Recommendations were also made concerning the establishment of a National Dam Safety Organization, the legislative framework for a Dam Safety Assurance Programme and outline of the function and responsibilities of a Dam Safety Organization, comprising a Dam Safety Commission, Dam Safety Unit and Dam Monitoring Units.

The reservoir as an asset. Thomas Telford, London, 1996

1.2 Detailed Preparation

Detailed Preparation for the Dam Safety Project was carried out in 1993 and 1994 by Robert H. Cuthbertson & Partners in association with DHV Consultants of the Netherlands, financed through the World Bank. A Draft Project Preparation Report was completed in September 1993, allowing the World Bank to appraise the Dam Safety Project in October/November 1993.

The primary objectives of the Dam Safety Project are:

- Formation of a Dam Safety Organization (DSO) within MPW and transfer at a later date to operate on a national basis covering all dams in Indonesia.
- Assistance for remedial work and other facilities to improve the safety of existing dams owned by MPW.

The Dam Safety Project to meet the above objectives was defined in terms of three components:

- Institutional Strengthening and Training
- Dam Rehabilitation Programme
- Basic Dam Safety Facilities Improvement

This paper refers to the Institutional Strengthening aspects of the first programme component.

2 LEGISLATION AND REGULATIONS

2.1 Existing

A review of current legislation relating to dam safety was undertaken. The principal legislation and regulations were contained in Law 11 -1974 Water Resources Development, Government Regulation (PP) 35 1991 Rivers, Decree No. 378/KPTS/1987 and Guidelines on Dam Safety SKBI 1.7.10.1987.

The legislation placed the responsibility for safety of dams with the Ministry of Public Works. PP No.35 stipulated that the construction, management and safety assurance of dams and reservoirs is the responsibility of MPW. Decree No. 378/KPTS/1987 and the Guidelines on Dam Safety SKBI 1.7.10.1987 established the principle of a Dam Safety Commission, and provided the framework for technical design considerations for the construction of new dams and inspection of existing dams.

There are about 120 referable dams in Indonesia which meet the criteria for referable dams as specified in the Guidelines on Dam Safety SKBI 1.7.10.1987 as follows:

- max height at least 15 m, with min capacity > 100,000 cu.m.
- max height less than 15 m, with min capacity > 500,000 cu.m.
- other dams excluded from the above defined by DSC.

2.3 Development

An Ad-Hoc Committee was set up within DGWRD during the preparation stage to progress the formulation of legislation for the establishment of a Dam Safety Organization (DSO). This was achieved by Ministerial Decree (KEPMEN) 98/KPTS/1993 in February 1993 issued by the Minister of Public Works which prescribes the DSO, comprising a Dam Safety Commission (DSC) and Dam Safety Unit (DSU) with jurisdiction over dams owned by MPW. The Decree established the DSC as an advisory group reporting to the Minister of Public Works with primary responsibility to oversee the activities of the DSU. The Decree established the DSU as the core executive organization of the dam safety programme reporting to the DGWRD. This Decree did not include regulations covering the design and construction of new and enlarged dams, inspection of existing dams and other such matters as abandonment of dams, emergency action plans etc.

A draft Ministerial Regulation (PERMEN) having national scope was prepared as the next stage of development of legislation in late 1994 as it is intended that the dam safety assurance programme include referable dams of all dam owners. This procedure was followed as it was considered that the new KEPMEN (Ministerial Decree) covering legislation and regulations initially required for MPW dams should be consistent with proposed national provisions. Consequently, a new draft KEPMEN was prepared to replace the existing legislation, Decree No. 98/KPTS/1993, in early 1995 to include additional clauses on certification, inspection, monitoring and surveillance etc, as proposed in the draft PERMEN.

Further review and re-drafting of the KEPMEN was undertaken between June 1995 and January 1996. The revised KEPMEN now comprises two parts - Part 1 covering the organization structure, functions and responsibilities of the DSC, DSU and DMUs, with Part 2 providing details of the Regulations. The KEPMEN is currently under final review prior to submission to the DSC and the Minister of Public Works for approval and issue.

The dam safety legislation and regulations will provide the Dam Safety Organization with the necessary statutory authority to regulate all matters which have a bearing on dam safety. This will include the authority to regulate design and construction of new dams, remedial works and improvements to existing dams, inspection of existing dams, abandonment, operation and maintenance and monitoring of all MPW dams in Indonesia covered by the legislation. Some of the above key activities are being regulated in accordance with the provisions of the draft KEPMEN.

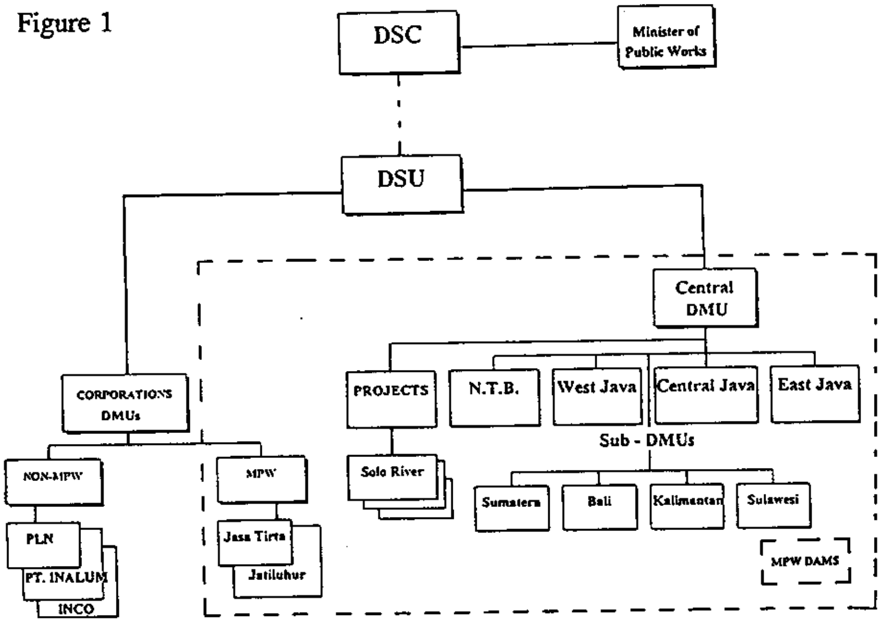
The development of the PERMEN will be implemented in accordance with a structured process of drafting, review, consultation and approval to ensure introduction of national legislation in January 1997.

3 ORGANIZATIONS

3.1 Introduction

The revision to KEPMEN No. 98/KPTS/1993 makes provision for the establishment of Dam Monitoring Units (DMUs) in addition to the DSC and DSU. This document defines the structure, function and responsibilities of the DSC, DSU and DMUs, based on a new organization structure recently established in MPW. The overall arrangement of the Dam Safety Organization is shown in Figure 1 which includes reference to MPW and other dams.

Figure 1



Appointments have been made to the DSC and DSU and both organizations are functioning. The DSC currently meets at about two monthly intervals. Work is in hand to establish the various Dam Monitoring Units (DMUs) in Jakarta (Central DMU), the Provinces and in the Project Authorities (Brantas, Solo etc.)

3.2 Dam Safety Commission

Membership of the DSC includes representatives from the major Ministries including MPW, other dam owners including the national power authority PLN, the Indonesian National Committee on Large Dams and law enforcement agencies. The DSC is responsible for overseeing the activities of the DSU, reviewing and approving dam safety guidelines, standards and instructions pursuant prepared by the DSU, reporting to the Minister of Public Works on the activities of the DSC and DSU, supervising enforcement of dam safety legislation and supervising development of a programme for dam safety awareness.

3.3 Dam Safety Unit

The DSU is the core executive organization of the DSO and guides and monitors progress on the programme. The DSU reports to the DGWRD, with the Head of DSU, as Member/Secretary of the DSC, providing regular reports to the DSC on the implementation of the programme and other relevant matters. The DSU is responsible for issuing directives to dam owners relating to dam safety, establishing a dam inventory/records, initiating and carrying out periodical (5 year) inspections of dams, preparing guidelines and standards, training and education etc.

The DSU staff presently comprises nine full-time and six part-time members of staff, compared to the planned total of 20 full-time staff. Difficulties are being experienced with the appointment of senior full-time staff experienced in dam engineering. It is proposed to shortly augment the full-time staff by less senior personnel who will obtain training under the Project.

3.4 Dam Monitoring Units

Dam Monitoring Units (DMUs) are the Dam Owners' units responsible for the routine surveillance, inspection and reporting of information on the condition of dams to the dam owner and the DSU. Preliminary proposals for the establishment of the DMUs within the MPW were formulated at the Project Preparation stage as indicated in Figure 1. DGWRD has established a Sub Project for Dam Monitoring under the Dam Safety Project to provide the leadership and guidance for the provincial DMUs and draft instructions for the provinces to establish DMUs have been prepared.

Preliminary meetings have recently been held with some provinces with a view to shortly establish provincial DMUs. This will be followed up by the establishment of DMUs under project authorities such as Jasa Tirta (Brantas River Basin), Solo River Project etc. The establishment of DMUs is currently less advanced than the DSC and DSU but it is recognized that it a key element in the development of the overall dam assurance programme.

4 CONCLUSIONS

Considerable progress has been made with the legislation/regulations and organization development parts of the dam safety assurance programme. This will allow progress to be advanced with the dam inspection programme and the consequent rehabilitation works. Inspection of an additional forty dams is included in the 1996/97 programme, and with rehabilitation works commencing on 10 to 15 dams depending on conditions found.

A parallel dam project, also funded by World Bank, has been carried out for PLN by the Consultants. A joint committee has also been established by MPW and PLN to consider certification of PLN dams which should assist with the development of the national dam safety assurance programme.

The Reservoir as an asset to Society

R.WILKINSON, Severn Trent Water Ltd, UK

SYNOPSIS. Severn Trent Water Ltd views its reservoirs as assets not only to the Company but also to the whole of society. It has, generally, set out to make the facilities they provide available to the general public. Moreover, it has set out to maximise the full use of reservoir to cover all operations whilst minimising the impact the reservoir has on the adjoining riverine environment. Procedures have also been established to cover any emergency that might occur at the reservoirs.

INTRODUCTION

Severn Trent Water Ltd (STW) owns, maintains and operates 44 large raised reservoirs and 23 large service reservoirs within its area. The size of these reservoirs varies from the largest single impoundment at Vyrnwy in North Wales, which has a capacity of 59700 Megalitres, to small 'pools' which only just meet the minimum size specified in the Reservoirs Act 1975, ie 25 Megalitres. Most of these reservoirs are deemed to be of strategic importance to the Company's Business Plan. Whilst these reservoirs are considered to be assets of the Company, and as such have an ascribed monetary value, STW takes the wider view that its reservoirs are assets to the whole of Society. In doing so it generally makes the facilities the reservoirs provide available for all to enjoy.

This paper looks at how this view has been developed in STW and its predecessor, Severn Trent Water Authority, and how facilities and amenities have been set in place to maximise the potential that each reservoir holds as an asset to Society. Figure 1 below illustrates where the Company's largest reservoirs are in relation to the major population centres.

SOCIETY'S VIEW OF RESERVOIRS

If Carsington Reservoir near Matlock in Derbyshire is to be taken as an example, then the one million visitors it attracted in the first 18 months of its existence is evidence of how Society sees reservoirs as a source of recreation and interest. This however is not a new phenomenon to STW. The visitor centre at Derwent Reservoir near Sheffield, which was opened in the late 1980s, has attracted many hundreds of thousands of hikers and holiday makers who want to enjoy the environment of the reservoir. The new visitor centre at Tittesworth Reservoir near Leek is proving just as popular. This is far removed from the days of closed reservoirs which the public were

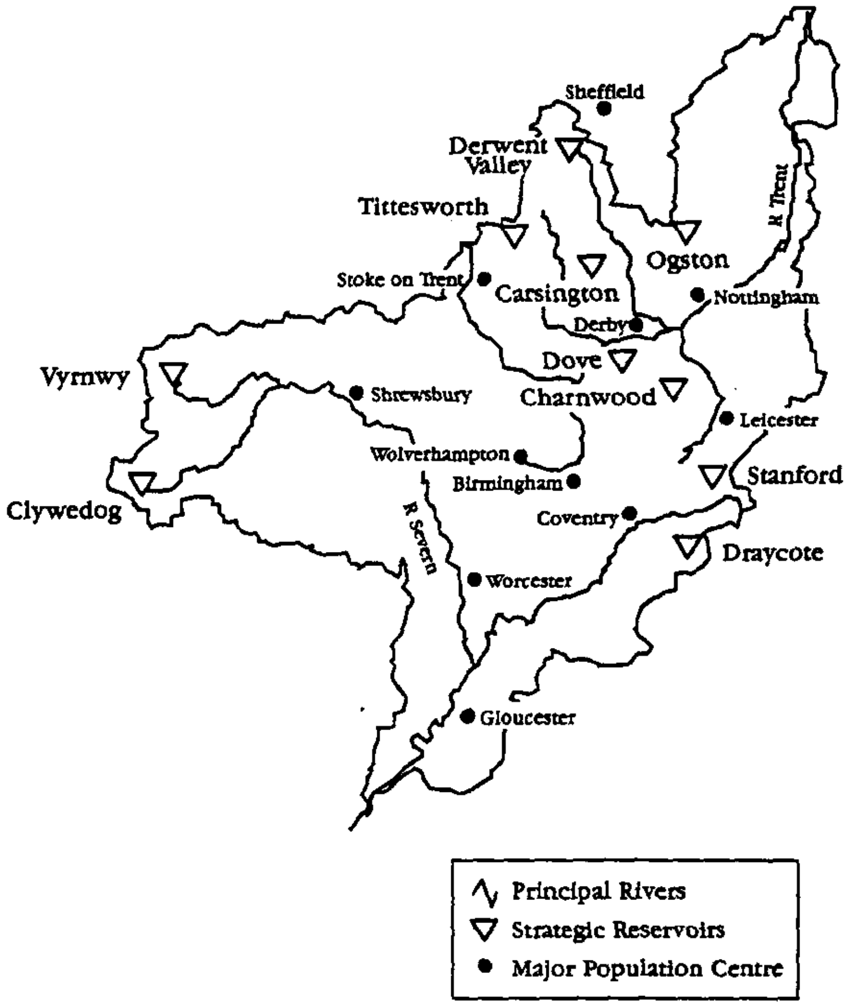


Figure 1 - The location of STW's major reservoirs

dissuaded from visiting and utilising save perhaps for a few fortunate members of local angling and sailing clubs.

THE SEVERN TRENT WATER VIEW.

Direct benefits to Society.

The interest and value that the public places in reservoirs was recognised by STW during all stages of design and construction of Carsington Reservoir. Consultations with local and national groups with interests in the recreational use of the reservoir were actively encouraged, and these parties were invited to submit their views to the design team. This ensured that the visual beauty of the reservoir and ease of accessibility to its environment were achieved. For example, top water level was designated such that an 'island' adjacent to the visitor centre was created which is accessible via a narrow neck of land, thereby adding to the visual dimension of the lake.

Of course, the Company has not lost sight of the primary purpose of its reservoirs ie., water supply. Nevertheless, consideration is given whenever possible to other users of the facility. Carsington's sister reservoir, Ogston, some ten miles to the east, is a favourite with local anglers and in the dry summer of 1990 when reduced inflow threatened to reduce the water surface to an unfamiliar level and expose vast areas of bare mud, STW went to considerable lengths to liaise with the angling clubs to establish temporary stands further and further down the normally submerged valley sides as the reservoir level dropped.

Indirect benefits to Society

To help ameliorate the effects of the other extreme of our climate, STW operates some of its major reservoirs such that they provide flood storage in the winter months. This helps reduce the risk of flooding in towns situated on the rivers below the dams. For example, the Derwent Valley Reservoirs are held at 80% of maximum capacity from October until the end of January to provide flood storage. The provision of this flood storage is formalised in the terms of agreements with the National Rivers Authority and similar agreements exist at Clywedog on the River Severn and at Lake Vyrnwy.

THE WIDER VIEW IN DAM ENGINEERING

The aesthetic appearance of the dam is a key issue with STW when undertaking engineering works at any of its dams. Unfortunately, this was not the case in the 1960s when spillway improvements were constructed on some of the smaller dams. These improvements consisted of massive concrete structures set at the end of these small dams. These remain today and form what many consider to be scars on the landscape.

Thankfully, they would probably not be designed and constructed in such a way today and evidence of this more aesthetically pleasing approach to spillway and dam maintenance works is Stanford on the Upper Avon on the Northants/Leicestershire border. A review of this dam and spillway's capability to withstand the design flood indicated shortcomings which gave the opportunity to provide safe facilities at economic cost and which were environmentally acceptable. This allowed for the protection of the downstream face of the dam to withstand overtopping. So as not to cause the dam to visually stand out from the natural rolling grasslands of this area, STW chose to reinforce the dam face with grasscrete, a material the combines resistance to erosion with aesthetic acceptability. Similarly, the reinforced concrete wave wall at Ladybower Dam was intentionally clad in Millstone blocks so that it would blend in with the surrounding landscape.

WIDER ISSUES THAT BENEFIT SOCIETY

Operationally, STW takes its responsibility further than its legal obligations. It takes cognisance of not only the local environment of the reservoir but also the wider issues downstream. For example, whilst STW conforms with the statutory obligations of compensation releases, it strives to release water that is compatible with the riverine environment below the reservoir. The quality of the water released in terms of oxygen content, temperature and turbidity is considered in the choice of draw-off to be used in making the release. Again close liaison is maintained with the NRA on this matter.

The Company also actively pursues a programme of Dam Safety Management. Most dams would eventually fail catastrophically without adequate and timely maintenance. The work of the Company's Reservoirs Supervising Engineers reduces this risk to a level which is considered infinitesimal. This aspect is covered by Milne (1996) in which the innovative and proactive aspects of the Company's approach to Dam Safety Management are discussed.

Operationally, synergies have been formed between Reservoir Supervising Engineers, operators and emergency planners, including those in the local County Councils, by means of regular emergency exercises. These exercises test the Company's multi-functional response to a serious situation at a reservoir. It is through this process of exercising and training that all STW staff know their roles should a problem occur.

SUMMARY

To summarise, STW has embraced its role as custodian of large areas of the countryside, and generally, does not see itself as a stakeholder with exclusive rights to its reservoirs. Even though raw water reservoirs impound over 1000 square kilometres of Severn and Trent catchments and affect the low flows in over 400 kilometres of river, STW has maximised the benefits a reservoir and its environment can give to itself and the public. At the same time it has ensured compliance with all its statutory and regulatory requirements in a pragmatic and effective way.

ACKNOWLEDGEMENT

The author is grateful to Mr Mike Yates, Head of the Water Supply Group for his permission to publish this paper.

REFERENCES

Milne P. (1996) Management of Reservoir Safety. Proc 10th Conf. British Dam Society. York

River Liffey Reservoirs: 50 years protecting and supplying Dublin City.

B.J.MANGAN, ESB International, Ireland.
T.A.HAYES, ESB International, Ireland.

SYNOPSIS. Three Dams on the River Liffey, constructed 50 years ago principally for the purposes of power generation, continue to provide major benefits to Dublin City. In addition to supplying approximately 75% of Dublin's potable water the impounded reservoirs provide valuable recreation and water based sports amenity resources for the people of Dublin. Pollaphuca Reservoir acts as a flood relief reservoir and has greatly reduced the scale and frequency of flooding in the Liffey valley. Ongoing monitoring and maintenance of the dams is essential to ensure that the Liffey Reservoirs continue to supply and protect Dublin City.

INTRODUCTION.

The River Liffey, which rises at about 760m O.D. in the Wicklow mountains south of Dublin City, is approximately 120km long from source to sea, draining an area of over 1300km^2 (Fig. 1). The City of Dublin was founded at the mouth of the Liffey over a thousand years ago and has expanded gradually to its existing size with a population of more than one million people. The River Liffey continues to be a major asset to the people of Dublin providing potable water, power and leisure facilities.

River Liffey Catchment.

Hydro-electric development of the River Liffey, between 1937 and 1949, resulted in the construction of three Dams namely Pollaphuca, Golden Falls and Leixlip. Pollaphuca Reservoir in the upper catchment is served by an area of 309km^2 consisting mainly of a blanket bog overlying granite and having an average annual rainfall of 1,245mm.

The reservoir as an asset. Thomas Telford, London, 1996

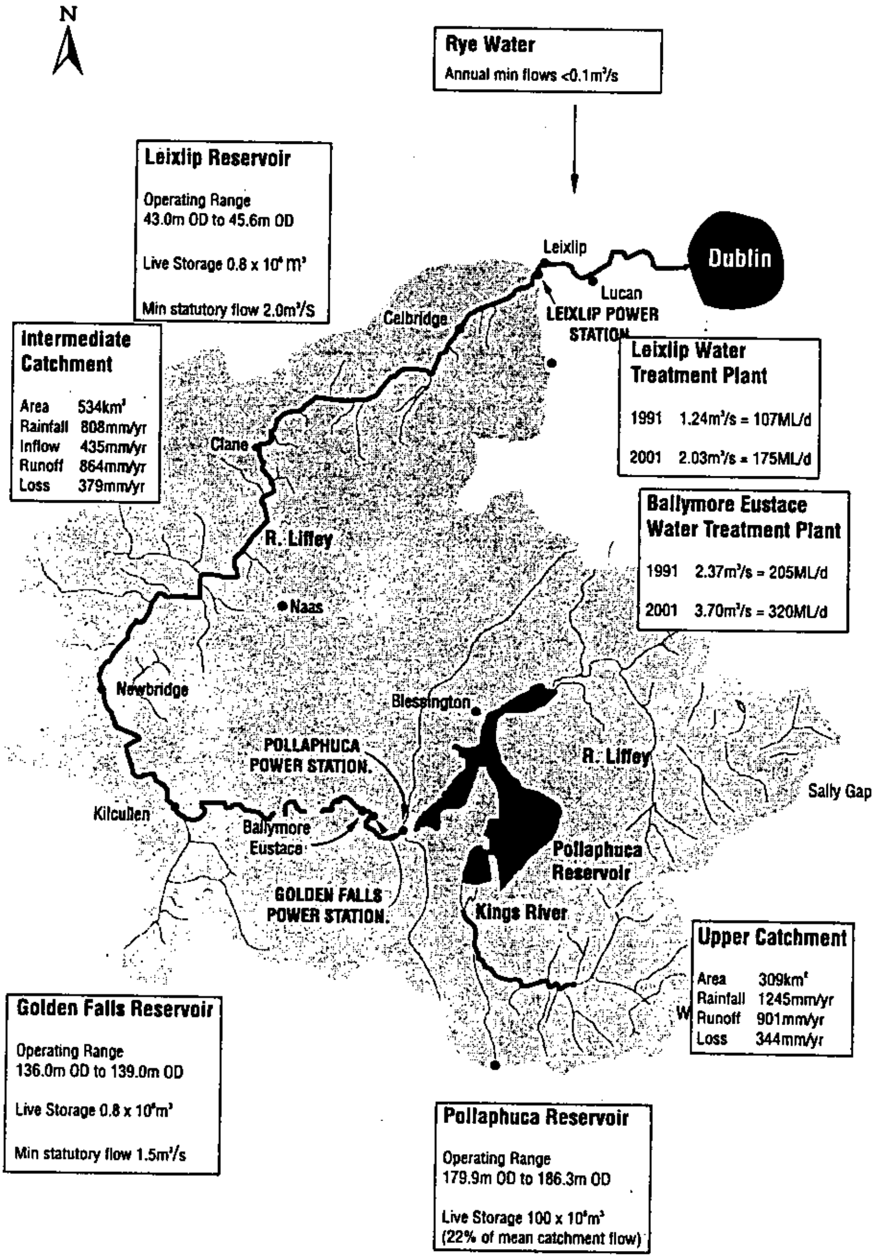


Figure 1
River Liffey Catchment

Pollaphuca Reservoir acts as a flood relief reservoir for the downstream Liffey Valley and greatly reduces the frequency and extent of flooding in the intermediate and lower Liffey valley. Golden Falls Reservoir is situated about 2km downstream of Pollaphuca and acts as a regulating reservoir for discharges from Pollaphuca. Controlled discharges from Pollaphuca and Golden Falls under normal and flood conditions are specified in the River Liffey Control Regulations. After passing through Golden Falls the Liffey flows approximately 56km through Co. Kildare to Leixlip Dam which is 20km from Dublin City. The river flows in a deep wide valley as far as Kilcullen, below which its course lies through a flat drift covered plain with deep sand and gravel deposits. There are a number of small towns and villages located on its course, including Ballymore Eustace, Kilcullen, Newbridge, Clane and Celbridge.

Leixlip Dam, the lowermost of the three dams on the River Liffey, is situated just upstream of Leixlip Bridge approximately 20 km from Dublin City. The Rye Water joins the River Liffey upstream of Leixlip Bridge approximately two hundred metres downstream of the dam. The Liffey continues its course as far as Palmerstown, near Dublin in a narrow steep-sided gorge. Below Palmerstown the river flows in a wide valley to the sea. The River Liffey is tidal up to Islandbridge Weir which is located in the western suburbs of Dublin City approximately fifteen kilometres downstream of Leixlip Dam.

LIFFEY DAMS.

When the three dams and their associated impoundments were first established their prime purpose was to provide hydro electricity. They were built some fifty years ago by the Electricity Supply Board (ESB) - Ireland's power utility. However as time progressed their importance as a source of potable water increased, with the drawoff from Pollaphuca increasing from a daily average of $0.3m^3/s$ in 1944 to $2.8m^3/s$ in 1994, and today it is ESB stated policy that water supply takes precedence over power generation.

Dam Construction.

Each of the three dams is of the concrete gravity type. Pollaphuca Dam (height 30.5m, length 79m) is located in a deep narrow gorge and is founded on very sound rock consisting of well bedded slates and greywacke. Golden Falls Dam (height 15m, length 100m) is founded on sound rock, consisting of metamorphous slate and greywacke which has been subject to complex folding. Leixlip Dam (height 21m, length 114m) is founded on a series of limestones with interbedded shale layers.

A comprehensive safety review of the three Liffey Dams was carried out in the mid-eighties. This examined the safety of the dams in the light of modern standards from the point of view of structural stability and spillway capacity. Remedial works were recommended at the time which have since been carried out. These works included the installation of rock anchors, the upgrading of monitoring systems at each dam, drainage of the foundation rock, increased storage at Pollaphuca and improved downstream protection. The original monitoring system comprised uplift and seepage/leakage measurement at the three dams. This system has been upgraded and now incorporates crest alignment, inclinometers, extensometers, jointmeters and thermocouples together with continued seepage and more extensive uplift and suspended solids monitoring. Daily and weekly readings are entered on site into a database which are transmitted via the ESB communications network to various locations. The database alerts users immediately if any values entered fall outside a predetermined range.

Dam Standards.

Since there is no specific legislation covering the design, operation, inspection or maintenance of dams in Ireland, the ESB established their Dam Safety Standards following a review of international practices. The standard adopted for the three Liffey Dams, (which would be classed as 'Category A' dams under the UK Institution of Civil Engineers Guide), is:

1. Ability to pass the 10,000 Year Flood without overtopping dam crest when all spillway gates are in operation.

and

2. Ability to pass the 1,000 Year Flood with one spillway inoperable (i.e. one gate failed in closed position) and with a freeboard allowance for wave run-up

Monthly certification is provided by ESB engineers while ESBI Consulting Engineers undertake an annual inspection. Five yearly inspections are carried out by the ESB's Chief Civil Engineer. Compliance with these standards is reviewed regularly by an External Dam Safety Committee appointed by the ESB. In addition and in keeping with widespread international practice downstream inundation studies have been undertaken for the three Liffey Dams.

BENEFITS.

The Liffey Reservoirs are at present used for power generation, as the main water supply source for Dublin City, for flood control and for a variety of leisure pursuits. In general the order of priority for the Liffey Reservoirs can be considered as:

1. Flood Alleviation
2. Water Supply
3. Power Generation
4. Leisure Interests

Flood Alleviation.

The overall safety of Pollaphuca Dam and the proper management of floods takes precedence over all other factors. The middle and lower reaches of the Liffey have in the past been subject to chronic flooding. The construction of the three Liffey Dams, particularly Pollaphuca with its large associated reservoir, has significantly reduced major flooding on the river. The largest discharge from Leixlip since the construction of the dams occurred in December 1954 when the peak Leixlip Dam discharge was 175m^3/s. This combined with an estimated peak discharge of 90m^3/s from the Rye Water, which enters the Liffey just downstream of the Dam, resulted in extensive flooding of the towns of Leixlip and Lucan. Both these towns have grown substantially in the last twenty years and can now be considered as suburbs of Dublin. Consequently, the alleviation of flooding by means of utilisation of reservoir storage capacity, particularly at Pollaphuca, is taken for granted by an ever increasing number of people.

'Hurricane Charlie'. Intense rainfall, which resulted from an offshoot from hurricane 'Charlie', on the Eastern half of Ireland gave rise to widespread flooding on the 25th and 26th August 1986. Twenty-four hour rainfall, in excess of 200mm and 100mm, was recorded at Kippure in the Dublin Mountains and in parts of South Dublin City respectively. The peak inflow to the Pollaphuca Reservoir during this event was 445m^3/s but because of the large retention storage in the reservoir there was no discharge through the Pollaphuca spillway or turbines. Consequently the peak flow into Leixlip Reservoir was only of the order of 50m^3/s and there was no flooding along the lower reaches of the Liffey in contrast to that which occurred on all other Rivers on the east coast of Ireland. A hydrological model of the Liffey catchment was used to simulate what would have occurred during this event if the reservoirs had not been constructed. The rainfall on the upper catchment was considerably more severe than that on the intermediate catchment. This model estimated that a flow of approximately 380m^3/s would have occurred at Leixlip under these circumstances and there would have been extensive flooding of Leixlip, Lucan and the Dublin suburbs. Fig. 2 compares the recorded flow hydrograph at Leixlip with that which would have occurred if there were no reservoirs on the Liffey.

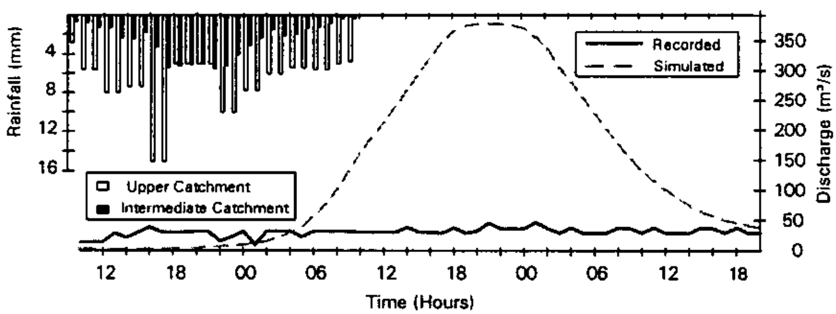


Fig 2 : August 1986 Flow Hydrograph at Leixlip (Recorded v Simulated)

Water Supply.

The average annual inflow to Pollaphuca over the forty-five year period 1950-1994 is $8.8m^3/s$ ranging from a low of $5.7m^3/s$ in 1953 to a high of $12.6m^3/s$ in 1960. The average natural catchment inflow to Leixlip over the same period was $14.6m^3/s$ ranging from a low of $9.3m^3/s$ in 1953 to a high of $21.2m^3/s$ in 1960. However, due to the abstraction at Pollaphuca ($2.8m^3/s$ in 1994) the mean outflow of the catchment at Leixlip over this period was only $13.2m^3/s$.

The effective working storage in Pollaphuca Reservoir (179.9m O.D. to 186.3 m O.D.) is 99.8×10^6 m^3 which equates to 36% of the mean annual flow of the Upper Liffey catchment to Pollaphuca or 22% of the mean annual flow of the catchment to Leixlip.

The effective working storages in Golden Falls Reservoir (136.0m O.D. to 139.0 m O.D.) and in Leixlip Reservoir (43.0m O.D. to 45.6 m O.D.) are both approximately 0.8×10^6 m^3.

The Greater Dublin Region consumes over 400 Ml/d of water and more than 75% of this comes from the River Liffey. Dublin Corporation extracts approximately 240Ml/d at Pollaphuca upstream of all power stations while Fingal Co. Council extracts approximately 105Ml/d at Leixlip.

It is proposed to increase the abstraction rate at Pollaphuca from $2.8m^3/s$ (240Ml/d) to $3.7m^3/s$ (320Ml/d) and at Leixlip from $1.2m^3/s$ (105Ml/d) to $2.0m^3/s$ (175Ml/d) by the end of the century. In addition the required minimum residual flow downstream of Leixlip Dam may have to be increased from $2.0m^3/s$ to $2.5m^3/s$ for dilution purposes. This will give a total demand of $8.2m^3/s$. While it ought to be possible, leaving power generation requirements aside, to meet these demands a co-ordinated catchment policy is required to ensure that sufficient supply is available at each location. Controlled discharges from Pollaphuca will be required to supplement inflows to Leixlip Reservoir during drought periods and this release from storage at Pollaphuca must not be allowed to hinder future supplies from this reservoir. Analysis of records for the summer of 1995 showed that there would have been a shortfall in supply at Leixlip if the proposed demand was required. This co-ordinated policy will be of even more importance if a projected ultimate extraction rate of $5.3m^3/s$ from Pollaphuca is developed. The existence of a comprehensive database of flows on the River Liffey over the last fifty years will assist the development of an efficient operating policy to meet future requirements.

Power Generation.

Pollaphuca hydro-electric generating station consists of two 15MW turbine/generators using water stored artificially in the reservoir. Operation is in accordance with the demands of the National electrical system. The water released feeds into a balancing reservoir at Golden Falls and is discharged at a reduced rate over a longer time through a 4 MW turbine into the intermediate Liffey channel. The power station at Leixlip consists of 1 No. 4MW turbine/generator set. The total annual average generation from the Liffey Stations is approximately 40,000 MWhrs, but this is diminishing and will continue to reduce as the drawoff rate from Pollaphuca and Leixlip increases. Increased drawoff from Pollaphuca results in a loss of generation potential at the three stations.

Leisure Interests.

In addition to its primary functions of hydro-electric power generation and water supply, the Liffey Reservoirs and in particular Pollaphuca Reservoir, are valuable recreation and water based sports amenity resources. Activities on the reservoirs include angling for coarse and game fish, rowing, sailing, canoeing, board sailing and water skiing. The Liffey Descent, an annual canoeing event, is made possible by controlled releases from Golden Falls and Leixlip Reservoirs. An international Ski jump festival is held annually on Golden Falls Reservoir and the level on the reservoir is controlled to facilitate the event.

RISKS.

The reduction in the extent and frequency of flooding on the Liffey has brought about demands for urban development in some lands which previously might not have been considered fit for this purpose. Fortunately, the lower Liffey valley, because of its physical characteristics, does not allow for development throughout. However, there are small flood plains where development has been possible, notably at Leixlip, Lucan, Chapelizod and around Islandbridge. The public perception often appears to be that the dams have practically eliminated the danger of natural flooding on a great part of the flood plain. While this is true in respect of floods with short return periods the flood control benefits of a relatively small reservoir such as Leixlip can become negligible when one considers larger and less frequent flood events. Unfortunately, when a natural flood causes unusually widespread inundation the public often believe that there must necessarily have been a malfunction of some sort, if not in the structure or equipment of the dam, then in the management of the particular flood. It is in this context that the ESB provide the relevant local authorities with predictions of the extent and rate of rise of inundation resulting from natural and postulated dam breach flood events downstream of its dams thus ensuring that the relevant facts are available to those charged with decision making in regard to developments on the flood plain and with emergency regional planning for large scale accidents.

Postulated Dam Breach Scenarios.

There is a very small risk of failure associated with all well maintained dams - failures, though very rare have occurred in various countries for a wide range of reasons including overtopping, foundation problems, slips and sabotage. Inundation studies on the River Liffey to examine the downstream consequences of extreme floods and possible breaches of the dams were carried out.

A computer model of the entire Liffey Valley was constructed using DAMBRK, a software package developed by the U.S. National Weather Service. This enabled the magnitude and time scale of inundation resulting from natural floods and dam breach scenarios on the River Liffey to be estimated and inundation maps and photographs clearly indicate those areas likely to be affected.

Representative cross-sections of the channel and flood plain were constructed, based on specially commissioned aerial surveys, at significant points along the Liffey valley. Inundation mapping for the catchment below Leixlip was then prepared by overlaying flood contours on the aerial photography. Colour aerial photographs were scanned and digitally rectified to provide a digital image of the affected areas at a scale of 1:5000. The flood contours were prepared on contour mapping and registered in raster form with their scanned digital imagery. Properties vulnerable to flooding from the different events can thus be easily identified.

The inundation studies confirmed the beneficial effect of Pollaphuca Reservoir which greatly reduces the frequency and extent of flooding in the Liffey Catchment. For example, while the 10,000 Year Flood peak inflow to Pollaphuca is estimated to be in excess of $1000m^3/s$, the estimated maximum discharge under current operating guidelines would only be $320m^3/s$. Postulated breaches at Golden Falls and Leixlip would cause extensive flooding in the areas immediately downstream but the breach flood wave would quickly dampen out. On the other hand the postulated breach in Pollaphuca would cause major flooding throughout the valley and into Dublin City. Because of the large impounded volume of water in Pollaphuca Reservoir a large discharge would continue over a long duration and there would be little attenuation of the dam breach flood wave. Analysis has shown that the inundation in Dublin City due to the postulated breach at Pollaphuca would be similar to that which would result during a 10,000 Year Event if there were no reservoirs on the Liffey.

THE FUTURE.

This paper has described the growing importance of the Liffey Dams to the basic infrastructure of the capital city of Ireland. 75% of all domestic, commercial and industrial water supplies in Dublin City and County are dependent on the Liffey Dams while many suburbs rely on the Dams for protection from serious flooding. It is obvious that the consequences of a dam failure, particularly at Pollaphuca, would have major long term effects on public health and commercial life in the City. It is therefore vital that the risk of failure is kept to negligible proportions by ensuring the continued surveillance and maintenance of the dams. ESB policy has been to provide the funds necessary to undertake this work. The recently completed improvement works should obviate the need for significant expenditure in the foreseeable future but obviously the cost of maintenance is likely to increase in the long term. However, the level of expenditure required is always likely to be minuscule relative to the benefits accruing to Dublin City.

ACKNOWLEDGEMENTS

The permission of the ESB to present this paper and, in particular, the help of Mr. Jack O'Keeffe, Chief Civil Engineer and of Mr. John Godden, Deputy Manager, Liffey Stations, is gratefully acknowledged.

Reservoir Sedimentation - Some Aspects of Reservoir Asset Management

J N DUDER, Tonkin & Taylor Ltd, Environmental & Engineering Consultants, Auckland, New Zealand

SYNOPSIS:
Reservoirs and lakes accumulate gravels and sands particularly in flood flows, forming extensive deltas, and reduce the live storage which is one of the economic barometers of a reservoir for water supply, irrigation or hydro generation. Effects will vary depending on reservoir size, the nature of sediment and the reservoir operation characteristics.

Finer particles may be deposited continuously even at low inflows with environmental effects in addition to depleting storage. If not settled out in the reservoir or excluded at an intake, particles can be carried through conduits and turbines or pumps inducing wear and hence reducing the value of a downstream asset.

Sedimentation of reservoirs is active in NZ because of its varied geology and the paper will give examples from a variety of sedimentation conditions and outline ways in which adverse effects can be mitigated.

INTRODUCTION.

The principal role of a reservoir or managed natural lake is to control river flow by storing excess flows (wet season, winter and possibly floods), and even night time flows, for use when more valuable e.g. for summer water supply or peak electricity generation. The extent of this cycle of storage and subsequent drawoff depends on the volume of "live" storage, as a function of the manageable range of water levels in the reservoir.

All natural lakes or developed reservoirs are destined to fill with sediment - by natural erosion and often by catchment "development" or abuse. When considering the longer term management of the asset represented by the live storage, allowance should be made for its progressive depletion and steps taken to reduce the loss of storage due to sedimentation. Allowances should also be made for the effects of removal of sediment on the downstream river regime.
This paper summarises the causes, types and effects of reservoir sedimentation and focuses on means of mitigating those effects. Examples are drawn mainly from a variety of reservoirs in New Zealand, which having a geologically young and varied topography, exhibits a range of reservoir sedimentation

The reservoir as an asset. Thomas Telford, London, 1996

characteristics.
The economic aspects of longer term asset management should include downstream effects of interception or flushing of sediments, but these are beyond the scope of this overview.

CAUSES & TYPES OF SEDIMENTATION
The natural processes of erosion and transport of sediments from hinterlands to the sea means that all rivers carry sediment of coarse and/or fine sizes which will be intercepted by reservoirs or diverted through conduits. The natural mobilisation of sediment depends on a number of factors, including geological and land instability, vegetation cover and climatic variations. It is often exacerbated by catchment development.

There may be substantial time lags before increased fluxes of sediment are felt in a downstream reservoir : removal of vegetation and slope instability may release much coarse material into adjacent water courses but successive floods may be needed to carry the shingle and gravels to the reservoir.

Conversely finer sands and silts are readily mobilised and may be encouraged to pass through the reservoir aided by low level flushing and/or timely drawdown.

The special characteristics of each catchment, river and reservoir or lake will merit individual attention to methods of sediment control, management and mitigation. Particular note should be taken of volcanic catchments, where lightweight particles, such as pumice can be transported continuously at low flows, and volcanic eruptions produce the full range of sediments from ash to lahars with boulders suspended in mud flows. While extreme cases they are not without precedent in recent years as witness Mt St. Helens, Pinatubo and last year Mt Ruapehu in New Zealand.

PREDICTION & MEASUREMENT OF SEDIMENT FLUX
Optimum prediction and management of sediment flux for a particular catchment and reservoir will require the fullest practicable understanding of sediment sources and movement.

Predictions of long term total flux are best made by observing deposition rates and characteristics (e.g. grain size) from comparable catchments and reservoirs. Measurement of suspended sediment is relatively straightforward using well established sampling methods, although accuracy of overall prediction will depend on obtaining credible readings in flood times when much of the flux occurs.

However, more difficult to predict or measure is bedload. This comprises the coarser sands and gravels which, once deposited at the head of an on-stream reservoir, require most effort to remobilise for natural flushing or removed by dredging. Bedload may typically be 5-25% of total flux, but its incidence in a reservoir will be most dependant on flood frequency and intensity.

Sediment is typically deposited in the form of a fan or delta once the velocity decreases sufficiently. Coarser materials progressively develop the topset slope by advancing the foreset face : finer sediments drop out further down the reservoir forming the bottomset slope.

Several methods are available to predict reservoir life when subject to ongoing influx, making allowance for progressively reducing trap efficiency e.g. Brune (1953) (Fig. 1). A useful commentary on storage losses from sedimentation in reservoirs is given by Graf (1984).

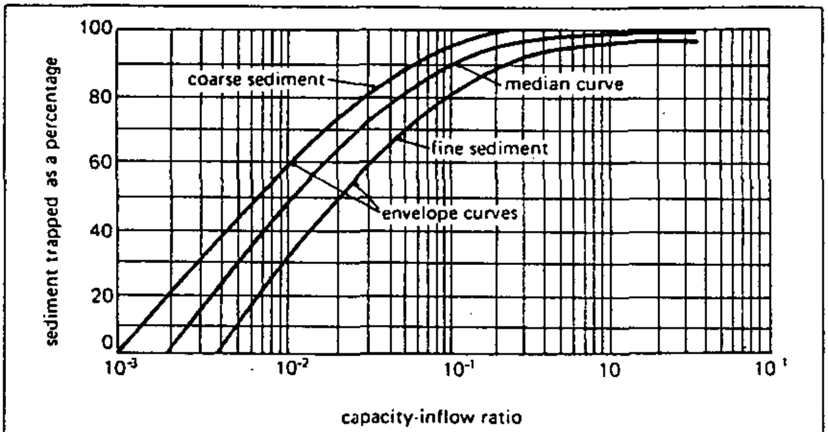
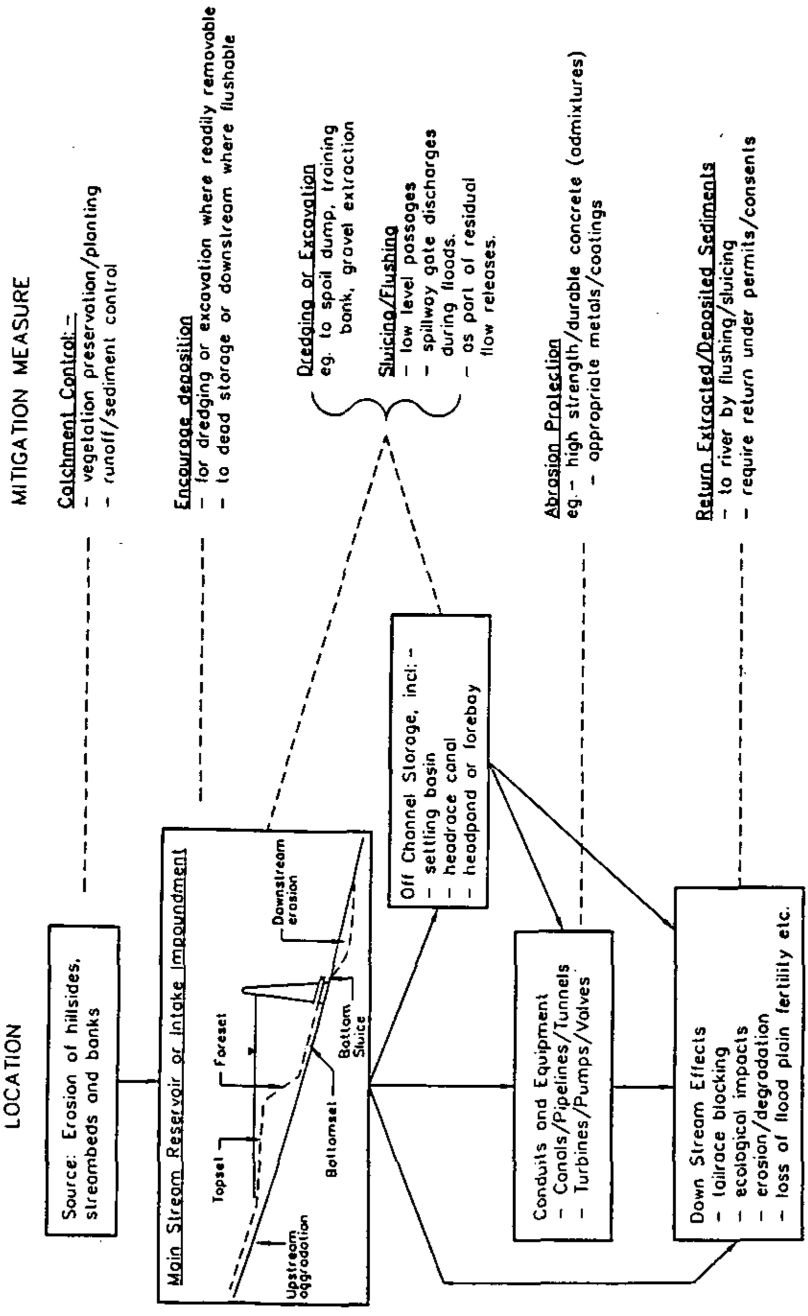


Fig. 1. Brune's trap efficiency curve[1].

Table 1
Reservoir Sedimentation - Situation, Effects and Remedies

Location	Likely Effect	Practicable Remedies
(i) Interception in main stream or reservoir of all bed load and some suspended sediment	• Loss of live storage loss of flood attenuation capacity • Upstream aggradation • Environmental detriment • Downstream erosion	• Drawdown and/or sluicing after, and preferably during floods • Dredging
(ii) Entrainment in headworks intake of sand sizes and larger	• Choking of Intake • Abrasion of concrete and conduit	• Regular flushing during floods and freshes • Careful design and positioning of intake
(iii) Deposition in canal or off channel storage pond	• Loss of flow and storage capacities • Weed growth etc.	• Optimum intake design • Bypass during floods • Scour through sluices • Dredging as last resort
(iv) Passage through penstocks and turbines	• Wear of conduit invert and/or protection system and turbine • Tailrace deposition	• Upstream interception • Abrasion resistant design of concrete, conduit linings and turbine runners

FIGURE 2 RESERVOIR SEDIMENTATION CONTROL



METHODS OF CONTROL & MITIGATION

General Comments.

Some impacts of sediment on a reservoir and associated diversion facilities are summarised in Table 1. Unless sediment can reliably be flushed through the facility, it is preferable to consider preventing or intercepting it as far upstream as practicable. The following methods of mitigation and control follow the downstream movement of sediments and are illustrated by case histories from New Zealand. Parts of NZ are geological young and either tectonically or volcanically active, and display a variety of sediment types, movement and deposition.

Fig. 2 gives a summary of movement of sediment down a river and through a reservoir or diversion facility and ways of mitigating the effects.

Catchment Management

A major cause of increases in reservoir sedimentation worldwide has been the deforestation of catchments, increasing soil instability and runoff. Conversely, the massive sedimentation problems of reservoirs and river experienced in India, China and elsewhere are being countered by reforestation projects with a suite of parallel benefits e.g. controlled timber harvesting, and further downstream, check dams to arrest sediment before it enters a major reservoir.

Much can be achieved by cooperation at both corporate and individual levels e.g. control of disturbance and runoff during forestry logging, and contour ploughing by farmers. In NZ, progress is being made with improved forestry management, particularly in the erodible pumice lands of the North Island.

Interception or Exclusion

If the reservoir to be managed is offchannel or remote from a river intake e.g. a water supply storage lake or a hydroelectric scheme headpond for daily regulation, careful attention will be needed to excluding sediment from the diversion intake. This will involve settling of coarse grain sizes either in the impoundment caused by a diversion dam or weir and subsequent flushing through low level sluices, and/or by providing a settling basin to remove typically sand sized particles, preferably for return to the parent river.

Much has been written on experience and refinement of sediment extraction systems. A useful discourse on the 3-dimensional movement of river borne sediment is given by Bucher & Krumdieck (1988) for a range of intake sizes on the outside of bends. Lysne et al (1995) gives details of the "serpent" system for progressively flushing sediment from a trapezoidal basin.

A specialised case of interception of gravels for subsequent removal is under study for the Ohau powerstation in South Island, New Zealand. The dewatered stretch of the Ohau River is subject to sporadic overflows from the diversion weir which mobilise alluvial gravels and deposit them in the powerstation tailrace, causing tailwater elevation and generation head loss. The immediate solution is to dredge the tailrace gravels : longer term the proposed remedy is to excavate a settling basin clear of the tailrace and maintain it by excavation of gravels.

Dredging of Sediment

Mechanical removal of sediment is commonplace using cutter suction or pumping equipment for finer sediments, and draglines or excavation plant for coarser gravels. High pressure jetting or even siphoning over the dam has been employed in a few instances. Gravels may be extracted for commercial use.

Several hydroelectric schemes in NZ's North Island, owned by Electricity Corporation of New Zealand (ECNZ) or local electricity supply authorities, are subject to sedimentation from volcanic material. The storage lakes of the Tongariro project have been affected by ash from the late 1995 eruption of Mt Ruapehu, despite intakes being shut off during flux of rain mobilised ash. Regular dredging has been practised over the 20 years of the project's life but sands and finer pumiceous materials passing through the feeder tunnels have caused significant abrasion of tunnel inverts. ECNZ consultants have recently modified the Whakapapa streambed intake to include a larger settling facility which intercepts coarser sands for return to the river, however dredging of fine sediments is a significant unavoidable maintenance cost.

The Aniwhenua project's reservoir has accumulated some 1.5M m^3 of predominantly lightweight pumiceous sands in 15 years of operation. Limited dredging has been considered economically justifiable by Bay of Plenty Electricity, coupled with training banks to encourage flushing of sediment into dead storage, thus preserving live storage (Fig 3). An approximate future value of peak generation from live storage at Aniwhenua is NZ\$0.50/m^3/annum, suggesting a present worth of \$5.0 - 6.0/m^3 - this is close to the cost of bulk dredging to nearby disposal areas, and was considered viable when coupled with other environmental benefits. Extension of initial training banks has encouraged sediment deposition further into the dead storage of the reservoir (Callander & Duder 1979).

Discussion on the provision and value of regulation storage for minihydro schemes is given in Palmer and Duder (1990).

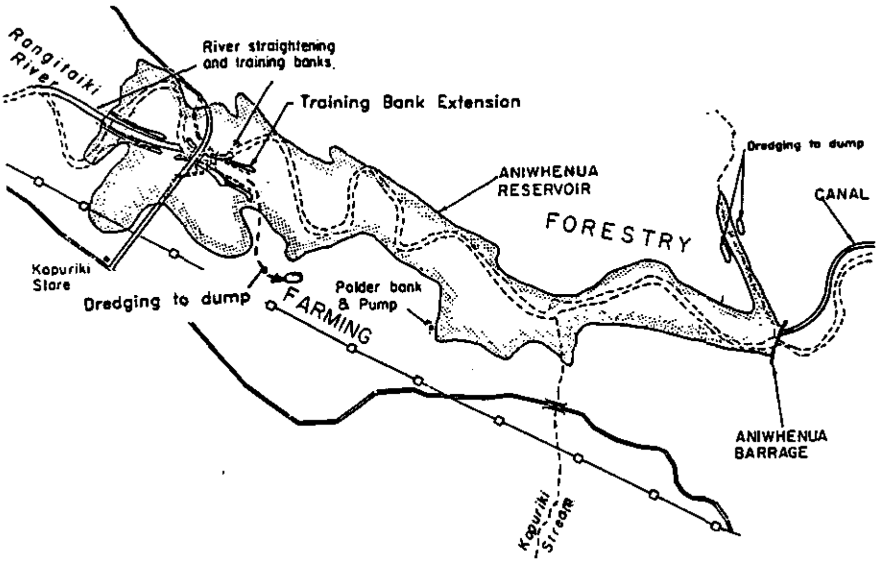


FIGURE 3: Sediment Control in Aniwhenua Reservoir

GRAPHIC SCALE m 500 0 0.5 1.0 1.5 2.0

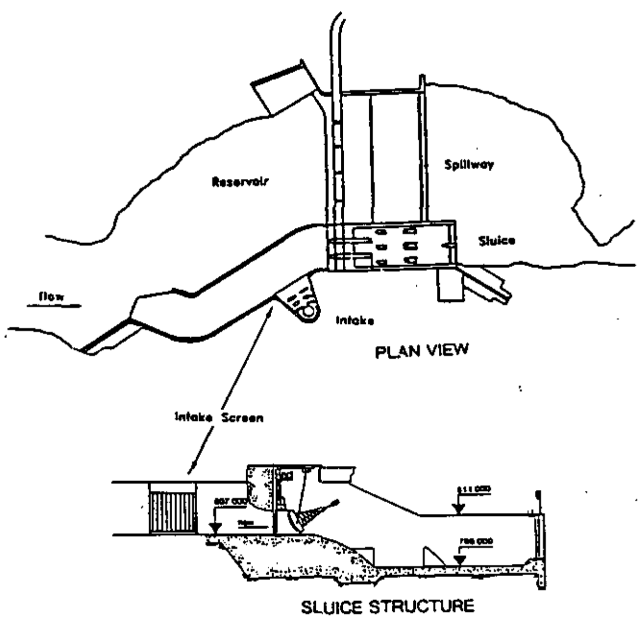


FIGURE 4: RANGIPO DAM INTAKE & SLUICE

Further down the Rangitaiki River lies ECNZ's Matahina reservoir. The dam was significantly damaged by earthquake in 1987 - during reservoir drawdown for repair works, the stability of 30 + years accumulation of sediments was a crucial factor in determining the extent of drawdown : remobilisation of these deposits was a combination of localised downcutting and limited slumping, indicating that effective flushing can be limited by flow concentrations with granular or partly cohesive soils.

Sluicing & Flushing

The common provision of low level diversion and dewatering sluices in dams may also be valuable for flushing sediment provided the intake does not become blocked. Effective if overdue sluicing at low level of fine sediments was achieved in ECNZ's Mangahao No. 2 reservoir after some 30 years accumulation. Severe storms shortly after commissioning in the 1930s caused extensive catchment erosion, and the diversion tunnel/sluice on the lower dam was effectively blocked. Some time after opening the vertical tunnel sluice gate a plug of sediment was extruded like toothpaste and a whirlpool developed in the reservoir over the diversion tunnel mouth, effectively flushing out most of the fine sediments. Subsequent flushings have kept fine sediments on the move, but by pass of gravel from the upper No. 1 Reservoir coupled with accumulation of logs and debris from the initially uncleared reservoir resulted in almost complete blockage of the No. 2 tunnel intake. After eventual dewatering the blockage was cleared by heavy plant.

For low dams main spillway gate operation may assist in passing finer sediment in floods particularly if density currents of silt laden water can be generated through the body of the lake

If raising of a dam operating level is contemplated one of the benefits will be the increased live storage, initially free of previous sediment deposits.

A example of effective interception and flushing of coarser sediments is at the Rangipo Intake of ECNZ's Tongariro project. The headpond behind the Barrage settles out sands, gravels and boulders : when the accumulation of the deposits is approaching the intake screen, low level radial gate sluices are opened to flush material downstream through a walled flume past the closed off screens (Fig. 4).

An example of possible adverse effects of longer term sediment accumulation is the effect of flood levels upstream of Lake Roxburgh on the Clutha river, South Island, New Zealand (McGregor et al 1995) (Fig 5.).

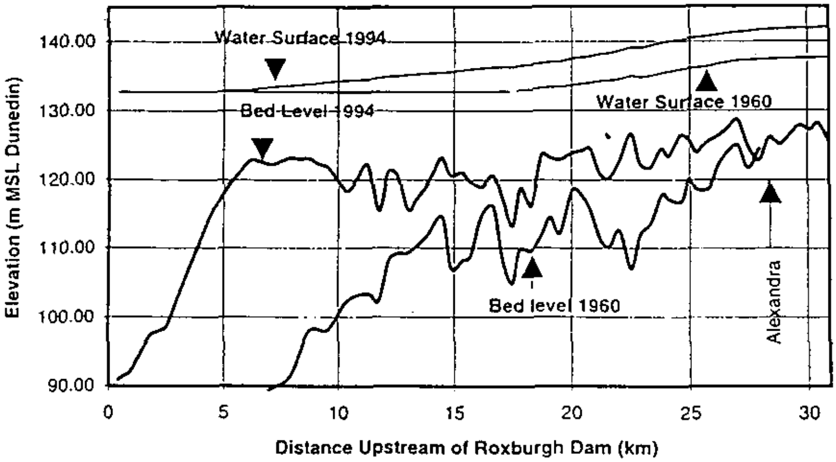


FIGURE 5: Thalweg bed level profiles and 2800m³/s flood profile
Source: McGregor et al 1995

Sediment has been depositing since 1960 with a backwater effect on flood flows entering the reservoir, and flood levels have been increasing in the township of Alexandra. A number of mitigation options were considered including stopbanking, property purchase, sediment removal and flushing. The strategy finally adopted after modelling and calibration during a flood involves sediment redistribution during floods by partial lake drawdown. Public consultation was an important part of gaining acceptance of this natural least impact solution.

DOWNSTREAM EFFECTS

While beyond the scope of this paper consideration should be given in reservoir management to the environmental implications downstream of impounding or flushing sediment. A classic example is the effect of the Aswan High dam on the downstream fertility of the Nile Valley and reported effects on Mediterranean fishery.

Downstream effects of sediment interception may include degradation and erosion of the river regime. In New Zealand, a discharge permit from the river regulatory authority may be required under our all embracing Resource Management Act 1991 before sediment is specifically flushed through sluice gates ; inevitably less controlled is the flushing of sediment during floods as part of the natural processes.

Careful consideration should also be given to the effects on aquatic flora and fauna of sediment extraction from, and preferably its retention in the river system. A case can be made for requiring downstream passage of sediment in consents or permits for diversion or storage of water for ecological benefits.

SUMMARY OF CONCLUSIONS

1. Sediment accumulation is inevitable in any impoundment on a water course. The degree depends on grain size, high flow frequency, size of impoundment, location of outlets and mode of operation.

2. An underlying principle of sediment control is to prevent initial erosion and transport by catchment care and management.

3. Sediment should preferably be kept in the main water course channel for downstream environmental benefits rather than removed by dredging : however economic benefits may accrue from soil reclamation or gravel usage.

4. Sediment accumulation in reservoirs can result in loss of live storage and flood attenuation capacity, upstream aggradation and flooding, development of wetlands, and downstream erosion. Low level outlets may become blocked unless operated regularly.

5. Entrainment of sediment in conduits and mechanical plant can reduce capacity and induce abrasion of concrete and metal surfaces.

6. Mitigation of adverse effects associated with the reservoir facility may include:
   - appropriate design of river intakes for conduits or offchannel storage to exclude or flush sediments
   - inclusion of settling basins close to the intake with assured flushing systems to return sediment to the river
   - dredging of sediments : usually an unavoidable maintenance cost but may be viable in terms of preserved live storage
   - training banks may encourage sediment deposition into dead storage
   - increased volume of live storage, initially sediment free, by raising reservoir operating levels
   - flushing of finer sediments through appropriately located and operated sluices

7. Progressive reduction of reservoir storage due to sedimentation is a natural phenomenon : while a variety of mitigation measures may be available, some allowance should be made for progressive depletion of the asset represented by reservoir storage.

ACKNOWLEDGEMENTS

The permission of ECNZ and Bay of Plenty Electricity, to refer to sedimentation aspects of their projects is acknowledged.

REFERENCES

Brune G M (1953). Trap Efficiency in Reservoirs. Trans Am. Geophys. Union. Pp 407-418

Bucher E & Krumdieck A (1988). Guidelines for the Design of Intakes & Structures for Small Hydro Schemes

Callander R A & Duder J N (1979). Reservoir Sedimentation on the Rangitaiki River. NZ Engineering (34, 9) 15.9.79.

Graf W H (1984). Storage Losses in Reservoirs. Water Power & Dam Construction. April 1984

Lysne D K, Olsen N R B, Støle M & Jacobson J (1995). Sediment Control : Recent Developments for Headworks.Hydropower & Dams. March 1995

McGregor N, Foster P & Goring D (1995). Sedimentation Effects within Lake Roxburgh & Mitigation of Flood Risk at Alexandra. Dams : The Implications of Ownership : IPENZ Proceedings of Technical Groups 21/1 (LD).

Palmer M D & Duder J N (1990). Regulation Storage for Minihydro Schemes. International Water Power & Dam Construction. Oct. 1990.

Barrow No 3 Reservoir - A Case History of Simplified Seismic Safety Evaluation

N G SWANNELL, Sir William Halcrow & Partners Ltd, UK

SYNOPSIS. Barrow No 3 is a large raised fully bunded reservoir. It overlies the Lower Lias Clay from which the embankment fill was derived. Construction commenced in 1887, but final filling was delayed some 49 years due to a succession of well recorded slips. In 1992 the reservoir was subjected to statutory inspection and a seismic safety evaluation recommended to meet the requirements of the BRE Guide to Seismic Risk to Dams. A simplified form of evaluation was considered appropriate and was successfully completed allowing a conclusion to be reached that there is negligible risk of breaching under the safety evaluation earthquake.

INTRODUCTION. Barrow No 3 Reservoir is one of a family of large raised raw water storage reservoirs situated south of Bristol and owned by Bristol Water plc. No 3 Reservoir may be seen in the foreground of Fig 1. The site is traversed by the A38 Bristol to Bridgwater road.

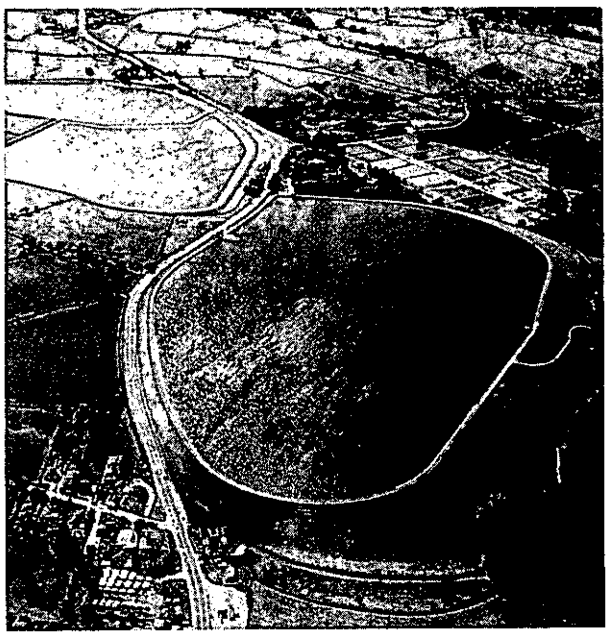


Fig 1. Aerial view of the Barrow Reservoirs

The reservoir as an asset. Thomas Telford, London, 1996

The reservoir is a non-impounding fully bunded structure which overlies the Lower Lias Clay. The embankment has a puddled clay core with clay fill shoulders, all sourced from the Lower Lias strata. The height of the embankment varies from a minimum of approximately 5 metres above original ground level adjacent to the main road to the south of the site, to a maximum of approximately 12 metres on the opposite, or northern, bank.

Construction commenced in 1887, but was only completed to full height with some difficulty, with the embankment suffering a succession of well recorded slips over a period of some 49 years. Major slips occurred on both the inner and outer embankment slopes at many locations around the reservoir perimeter both during construction and during various attempts at impounding. Slopes were progressively flattened and/or provided with rockfill toe berms. Overall slopes were typically reduced from the order of 1 in 3 to 1 in 8 with rockfill toes.

The history of the reservoir was comprehensively researched in 1981/82 by Messrs Watson Hawksley on behalf of the Owner (Watson Hawksley, 1982). The cause of the slips was identified at that time as being due to weak foundation conditions which include a solifluction sheet underlying approximately the north eastern half of the site and soft valley infill material underlaying four sections of the northern bank.

The reservoir, with its much flattened slopes, has generally performed satisfactorily since the 1930s, although leakage has been a persistent and ongoing problem requiring remedial grouting of the puddle clay core in various locations.

In 1992 the reservoir was subject to routine statutory inspection in accordance with the Reservoirs Act 1975. In view of the problematic history of the reservoir embankment and the then recently published guidance on seismic risk to dams in the United Kingdom - the BRE Guide (Charles et al 1991) - an appropriate seismic safety evaluation was considered to be appropriate and was recommended.

The recommendation was accepted by the Owner and arrangements made for an evaluation to be carried out in accordance with the BRE Guide.

The paper summarises the studies and investigations carried out to establish appropriate input data, describes the methodology adopted and presents the results of the evaluation. The general procedure has been subsequently described by Swannell (1994) for the evaluation of embankment dams of this class *ie* E_b and E_c of the BRE Guide, being those requiring 'some relatively simple analysis'.

SITE SEISMIC CHARACTERISATION. The site was characterised by the Inspecting Engineer in terms of seismic risk in accordance with the BRE Guide as follows:

		BRE Guide Reference
Dam Category	III	Table 3
Seismic Zone	B	Figure 5
Return Period for the Seismic Evaluation Earthquake (SEE)	10,000 years	Table 4
Level of Safety Evaluation	E_c	Table 5

For this level of safety evaluation (E_c) the Guide recommends at least a relatively simple form of seismic analysis. In the case of embankment dams a pseudo-static form of analysis is indicated as being appropriate.

METHODOLOGY. The methodology adopted for the evaluation has been described for general application by Swannell (1994).

In the case of Barrow No 3 the particular history of repeated embankment failure was considered to be of sufficient significance to warrant a site specific investigation to determine the relevant geotechnical data necessary to carry out pseudo-static stability analyses. A small scale investigation was therefore devised to target the particular information required.

It was recognised that in view of the history of slope failure it would be important to determine with confidence the residual strength of the Lower Lias clays.

The detailed and well maintained records of some of the pre-existing failures gave scope for back analysis to assist in this determination.

Therefore, in summary, the methodology adopted for the evaluation was as follows:

i) site specific effective stress soil strength parameters to be established from consideration of

- desk study of published results of testing of the Lower Lias clays
- back analyses of historical slope failures at the site
- ground investigation and laboratory testing

ii) groundwater conditions in the outer embankment slopes with the reservoir full *ie* the critical condition for seismic safety evaluation, to be established by installation of piezometers at the highest embankment section

iii) pseudo-static analysis of the highest embankment section outer slope to be carried out to determine the

- static factor of safety
- factor of safety with a pseudo-static acceleration applied appropriate to the SEE determined from the Guide
- critical seismic acceleration (k_c) *ie* the acceleration which would reduce the factor of safety to 1.0

iv) if the calculated pseudo-static factor of safety was found to be less than 1.0 under the SEE then k_c could be used to estimate the horizontal displacement in the downslope direction using the semi-empirical relationship of Ambrasseys and Menu (1988); alternatively if the factor of safety under the SEE was found to be greater than 1.0 there was considered to be no risk of significant displacement and the evaluation was concluded at that point

v) the vertical displacement corresponding to the estimated horizontal displacement was compared to the available freeboard below embankment crest level to assess the probability of a breach occurring under the SEE event.

DESK STUDY, BACK ANALYSIS AND SITE INVESTIGATION.
The desk study consisted of three principal components as follows:

i) geological and geomorphological review of the site

ii) study of published results on the properties of the Lower Lias clays

iii) review of the reservoir history and back analysis of the best documented historical slips

The desk study was subsequently of value in finalising the site investigation requirements.

The site investigation was targeted on the outer embankment slope at the highest section. Four cable percussion boreholes were sunk to penetrate original ground. The maximum depth of hole was approximately 19m.

Casagrande standpipe piezometers were installed in each of the boreholes and monitored for approximately six months until it could be judged that equilibrium had been achieved.

Laboratory testing was restricted to index testing, undrained triaxial tests with pore pressure measurement, direct shear strength tests, multi-reversal shear strength tests and ring shear tests.

A detailed description of the geotechnical desk studies, back analyses and site investigation is beyond the scope of this paper, but it is hoped to be able to publish this separately at a later date. The principal focus of the geotechnical work was the residual effective stress strength parameters for the Lower Lias Clay. In summary, there was found to be good agreement between published results and the site specific back analysis and laboratory test results. Most importantly, the residual strength of the Lower Lias Clay at the site was confirmed as being normal stress dependent, as shown on Fig 2.

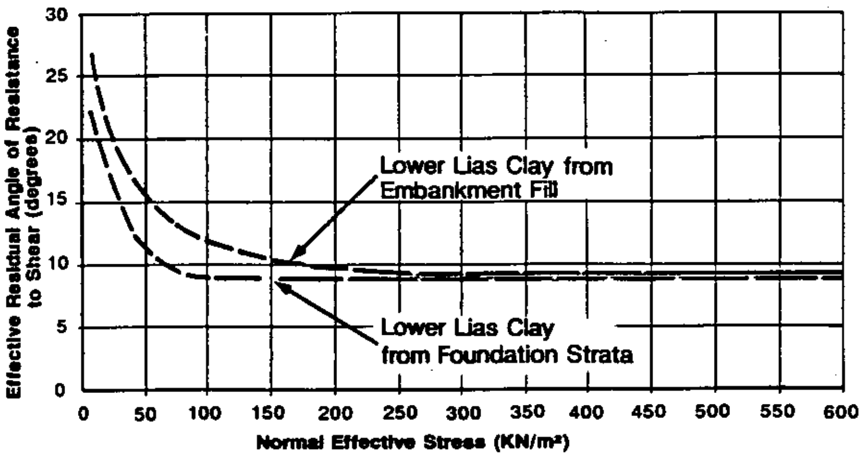


Fig 2. Variation of the angle of effective residual shear strength with effective normal stress for the Lower Lias Clay

A summary of the laboratory test results for the main soil types at the site is given in Table 1.

Table 1. Summary of Average Laboratory Test Results

Soil Type	w (%)	PL (%)	LL (%)	Bulk Density (Mg/m^3)	Peak		Residual	
					c (kN/m^2)	ϕ' (°)	c (kN/m^2)	ϕ' (°)
PC	36	25	66	-	-	-	-	-
C	37	24	62	1.84	0	33.5	0	-
F	32	25	55	1.84	8.5	24.8	0	18.7$^{(1)}$
LLC	26	25	56	1.96	8.5	24.8	0	16.1$^{(1)}$

PC - Puddle clay core
C - Colluvium
F - Fill
LLC - Lower Lias Clay

Note $^{(1)}$ - given for a normal effective stress of 30 kN/m^2, results are stress dependent (see Fig 2)

SEISMIC CRITERIA. For a Category III dam in Zone B (see Site Seismic Characterisation above) the BRE Guide in its Table 4 suggests a peak ground acceleration (PGA) of 0.2g as being appropriate for the SEE.

However, the Guide (Chapter 6, Section 6.2) further suggests that, for the purpose of pseudo-static analysis, a seismic acceleration coefficient equal to two thirds of the PGA may be satisfactory for the analysis of embankment dams. Therefore a seismic coefficient of 0.13g was adopted for the evaluation of Barrow No 3.

ANALYSIS AND RESULTS

Analysis

The seismic safety evaluation was based on an analysis of the outer embankment slope at its highest section. From the study of the history of the reservoir it was known that this section of the embankment had failed at least once during construction and that one of the infilled valleys lay below this section.

The site investigation confirmed a weak foundation layer at the base of the embankment in this area. Although no discrete shear surfaces were recorded during the investigation it was felt that this weak layer would almost certainly have formed the failure surface of the recorded slips. The slow rates of failure described for the old slips were also considered to be indicative of failure through fill rather than through undisturbed foundation strata.

It was therefore considered reasonable to assume that any further failure would occur on the existing basal failure plane and that the shear strength on this plane should be assumed to be at or very close to residual strength.

Because the residual strength failure envelope of the Lower Lias Clay had been found to be non-linear (Fig. 2) it was necessary to follow a simple iterative procedure when carrying out the stability analyses in order to ensure that the mean strength assumed on any failure plane was appropriate to the effective normal stress on that plane. Fortunately the effective normal stress on the critical planes was sufficiently uniform and within a range, with reference to the shape of Fig. 2, such that it was reasonable to assume that a mean strength value applied along the full length of the failure planes without any need for extensive subdivision.

A piezometric line for the fill and the colluvium with the reservoir full was determined from the monitoring results. Hydrostatic conditions were assumed.

The embankment section analysed and the critical failure surface are shown on Fig. 3.

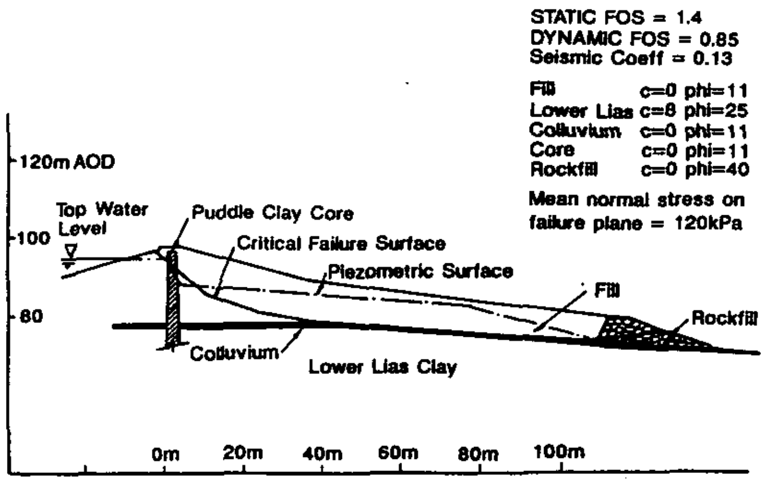


Fig 3. Results of the stability analysis

Because of the slope geometry and the presence of weak foundation material a non-circular analysis was considered appropriate. Sarma's Method was used because this gave necessary output data in addition to factors of safety. In particular, information was given on

i) the critical seismic coefficient (k_c)

ii) the distribution and mean value of effective normal stress on failure planes which allowed an iterative process to be followed to take account of the stress dependency of the residual shear strength, as noted above.

Results

The critical slip surface for both the static and pseudo-static conditions is shown on Fig. 3. It can be seen to be a basal failure plane passing through the weak foundation stratum, as was to be expected. The results were as follows:

Static FOS	1.40
Pseudo-static FOS	0.85
Critical Seismic Coefficient (k_c)	0.07

For the pseudo-static case the factor of safety was found to be less than unity and therefore some displacement is to be expected under the SEE. This was estimated using the semi-empirical predictions of Ambrasseys and Menu (1988), Fig. 4 refers, with the following results:

i) For 25% probability of exceedence :

- horizontal displacement in the downslope direction	140mm
- approximate equivalent vertical displacement	12mm

ii) For 1% probability of exceedence :

- horizontal displacement in the downslope direction	400mm
- approximate equivalent vertical displacement	35mm

The predicted displacements were therefore found to be very small. The available freeboard at Barrow No 3 is approximately 1250mm and thus it could be concluded that there is negligible risk of a breach occurring under the SEE although precautionary drawdown and remedial measures may, of course, be required as appropriate to the actual deformation arising.

These results were sufficient to allow the Inspecting Engineer to issue a Section 10(6) certificate for the reservoir in accordance with the Reservoirs Act 1975.

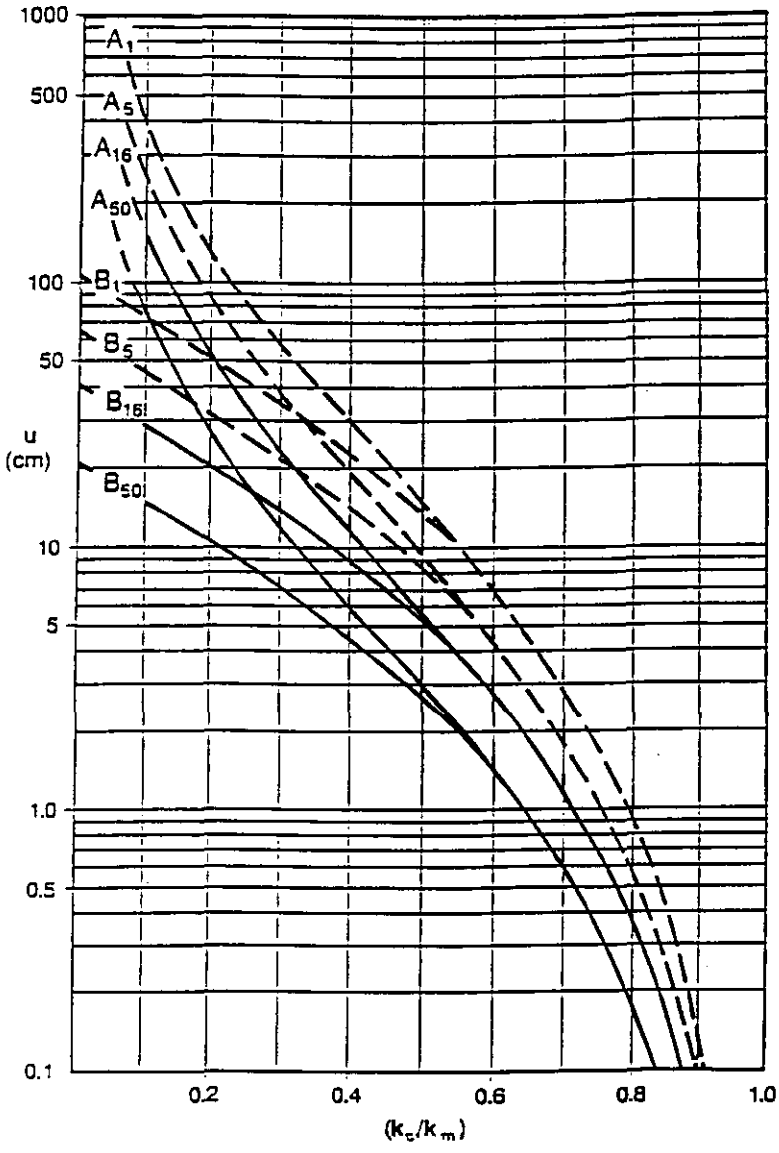


Fig 4. Earthquake induced ground displacements : predicted values of maximum unsymmetrical (A) and symmetrical (B) displacements for probabilities of exceedence of 1, 5, 16 and 50% (after Ambrasseys and Menu, 1988)

In Fig. 4 u is the horizontal displacement in the downslope direction and k_m is the maximum applied ground acceleration, assumed for this purpose to be the peak ground acceleration (PGA).

CONCLUSION. The case history of the seismic safety evaluation of Barrow No 3 Reservoir demonstrates that the requirements of the BRE Guide for a fairly typical form of UK embankment dam can be complied with in a relatively straightforward manner. The particular history of this reservoir demanded a site specific investigation to establish critical analytical input data - in particular the residual strength of the Lias Clay and its normal effective stress dependency - whereas in other situations this may not be necessary and a 'parameter study' approach would most likely give equally satisfactory and acceptable results.

ACKNOWLEDGEMENTS
The Author wishes to acknowledge the assistance of Mr P J Smith and Mr R J Evans of Sir William Halcrow & Partners Ltd in undertaking the evaluation and of Mr P J Matthews and the staff of Geotesting Services Ltd for undertaking the site investigation with great care and diligence. Thanks are due also to the Inspecting Engineer, Mr J L Beaver for his guidance and encouragement and to Bristol Water Plc for permission to publish the paper and for providing the aerial view reproduced in Fig. 1.

REFERENCES

Ambrasseys N N & Menu J M (1988). Earthquake Induced Ground Displacements. Earthquake Engineering and Structural Dynamics, vol 16, pp 985-1006

Charles J A, Abbiss C P, Gosschalk E M & Hinks J L (1991). An Engineering Guide to Seismic Risk to Dams in the United Kingdom, Building Research Establishment, Watford, UK

Hawkins A B (1973). The Geology and Slopes of the Bristol Region. Quarterly Journal of Engineering Geology, vol 6, nos 3 and 4, pp 185-206

Swannell N G (1994). Simplified Seismic Safety Evaluation of Embankment Dams. Dams and Reservoirs, vol 4, no 3, pp 17-19

Watson Hawksley (1982). Barrow No 3 Reservoir. Review of the History of the Embankment Dam with the Site Investigation of 1981. Report to the Bristol Water Works Company.

Remedial works at Rivelin and Redmires Reservoirs

J.R.CLAYDON, Yorkshire Water Services, U.K.
N.REILLY, Mott MacDonald, U.K.

SYNOPSIS
The Rivelin/Redmires system comprises six reservoirs impounded by embankment dams constructed between 1854 and 1869 to serve the City of Sheffield. The dams are earth fill with puddle clay cores typical of their time and some of the overflow facilities do not satisfy present capacity criteria. Analysis based on the 1974 Flood Studies Report was used to reassess the safety of the system in extreme conditions and it was also necessary to carry out four physical hydraulic model tests to assist with analysis and redesign. The studies also examined the seismic stability of the embankments in the light of the 1991 BRE guide. Remedial works are required at all six dams but the most significant works will be at Rivelin Upper and Lower dams and these are described. The paper concludes with a discussion on the future of old Pennine reservoirs.

INTRODUCTION.
The reservoirs were built in the valley of the Rivelin Brook, a tributary of the River Don which it joins near the centre of Sheffield. They were built between 1854 and 1869 and are earthfill dams with puddle clay cores typical of the period. Principal features of the reservoirs are given in Table 1. The location and catchment details are given in Fig. 1.

Table 1 - Principal features of the Rivelin & Redmires Reservoirs

Name	T.W.L. (m AOD)	Reservoir surface area (ha)	Reservoir volume (tcm)	Dam height (m)	Dam length (m)	Date constructed
Redmires Upper	349.97	22.7	1559	15	710	1854
Redmires Middle	336.33	19.4	852	12	825	1836
Redmires Lower	325.39	16.6	634	14	525	1849
Rivelin Upper	212.70	4.4	220	12	221	1848
Rivelin Lower	204.4	11.9	796	20	305	1848
Rivelin Depositing Pond	167.87	1.6	36	7	82	1869

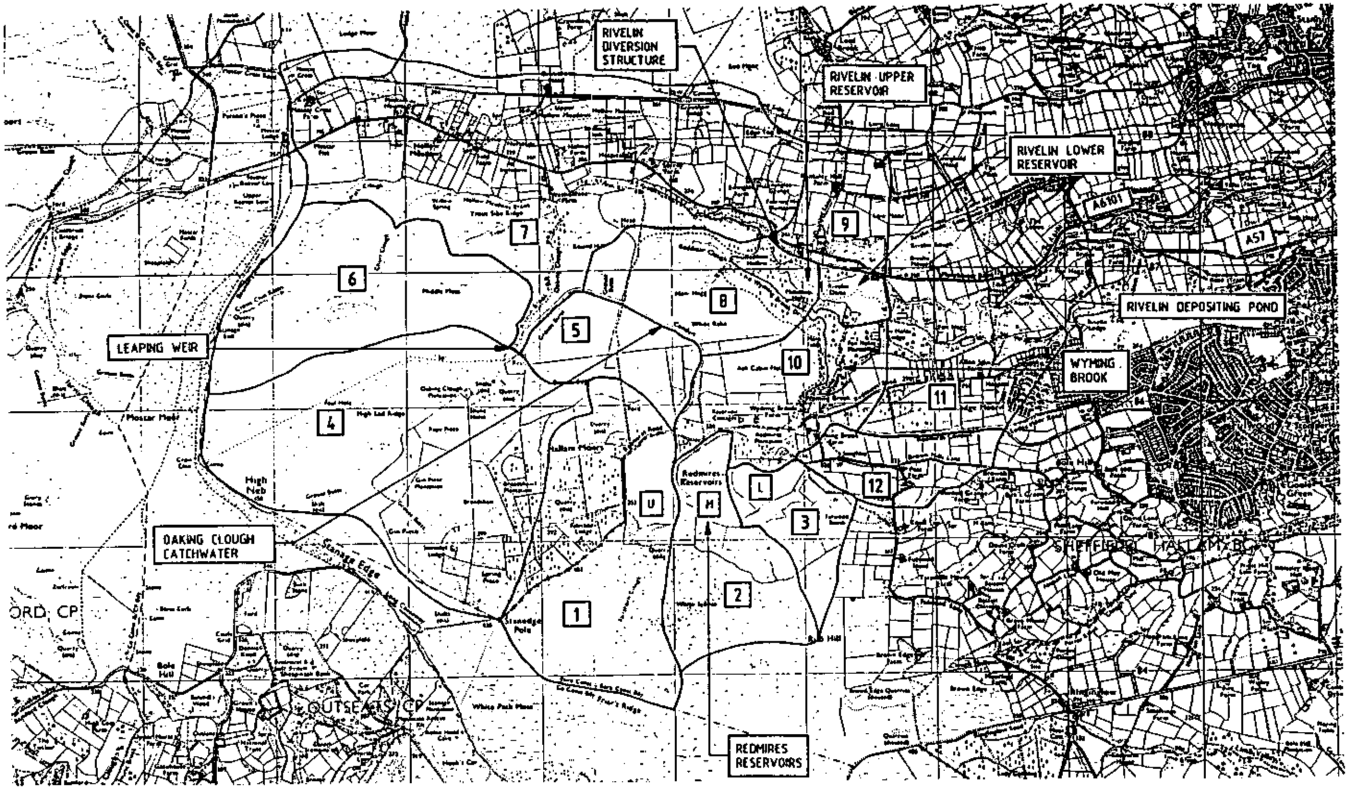


Figure 1 : Location and Catchment Details

The reservoirs, now owned by Yorkshire Water, supply water to treatment works at Rivelin and Redmires, which both use the Sirofloc process, almost unique in the UK. The Rivelin works has a capacity of 75 tcmd, much greater than can be supported by the local catchment because Rivelin Lower receives a bulk supply via a tunnel from Ladybower reservoir owned by Severn Trent Water. The reliable yield of the local catchment is estimated at 15 tcmd.

IDENTIFIED PROBLEMS AND INVESTIGATIONS.

All six reservoirs were inspected under the Reservoirs Act between 1988 and 1991 and a number of recommendations in the interests of safety were made. The main recommendation, made in respect of all the reservoirs, was to carry out definitive flood studies to assess the performance of the structures in Probable Maximum Flood (PMF) conditions. Previous flood studies had been carried out but only against estimated spillway rating characteristics. As will be seen later, some of the spillway weirs do not lend themselves to accurate assessment of rating and consequently physical models were necessary.

A second main recommendation in respect of the Redmires reservoirs (reported on in 1992 just after the publication of the BRE guide on this subject [BRE 1991])) was that seismic assessments should be carried out. Yorkshire Water decided that the comparable Rivelin Upper and Lower dams should also be assessed in the same investigations. The Rivelin Depositing Pond, having a much lower dam, was not considered to warrant an assessment.

The third key aspect that required investigation was the instability of the crest of the Rivelin Lower dam which carries a private road on fill contained between dry stone walls. A part of this had collapsed on one occasion and sections of it showed appreciable signs of distress, necessitating the imposition of a 3 tonne weight limit.

In addition to the above major issues, there were also matters relating to the levels of the clay cores, repairs to upstream revetments, valves, draw-off shaft steelwork, access bridges and other minor items. The Inspecting Engineer had also directed that a method of long term monitoring of the stone upstream slope protection be instituted.

It was clear from the outset that some major works would be required at the Rivelin reservoirs, and in order to make progress it was decided to proceed immediately with minor remedial works at Redmires while parallel investigations into flood and seismic issues were carried out for the whole group.

MINOR WORKS: PHASES I & II

For operational reasons the minor works were carried out in two phases and included the following:

- Redmires Upper: completion of repairs to the upstream face slope protection (commenced under an earlier contract but not finished due to the requirement to refill the reservoir) and raising of the clay core.
- Redmires Middle: repairs to the upstream face slope protection, repairs, repointing and stabilisation of damaged sections of the wave wall, renewal of draw-off controls and renewal of the access bridge to the valve tower.
- Redmires Lower: renewal of metalwork in the valve tower and making the siphon draw-off operational.

In addition to the above and in response to the direction of the Inspecting Engineer regarding the monitoring of the slope protection, an innovative method devised by Exeter University was adopted (Kalaugher and Grainger 1994). This has been fully described elsewhere but in essence it consists of taking oblique photographs (colour transparencies) of marked areas of pitching from accurately repeatable locations over a long period of time. On subsequent visits, a special device is used to compare the earlier pictures with the current scene through a viewer which fuses the two views. A number of areas have been surveyed in this way on the Redmires Upper and Middle reservoirs and to date there has been one return visit after a period of 10 months. The interval was too short to come to a conclusion on the efficacy of the system but it appears to have promise.

GEOTECHNICAL INVESTIGATIONS

Two ground investigation programmes were instituted. The first comprised trial pits, boreholes and dynamic probing over the whole group of reservoirs to establish depths to clay core and parameters for crest and seismic stability. The second programme was confined to Rivelin Lower and was designed to obtain data necessary for the design of remedial works.

The investigations found that the embankments consist of heterogeneous predominantly clayey fill of glacial origin containing varying proportions of silt, sand gravel, cobbles and pockets of organic matter. The fill was well graded and the clay generally of low to intermediate plasticity. The clay cores were composed of similar material but without the coarser fractions except at Redmires Middle where some layers of loose silt were found in the upper layers. The tops of the cores at all three Redmires dams and Rivelin Lower were found to be below normal TWL.

Piezometric data shows that the clay cores are only moderately effective in controlling water pressures in the downstream slopes and in the case of Redmires Middle the core has very little effect. At Rivelin Upper the pressures at depth are less than hydrostatic, indicating significant drainage into the underlying gravels. A full range of laboratory testing was carried out on soil samples and typical results are presented in Table 2.

Table 2 : Summary of Soil Test Data

INDEX TESTS	Typical Range	Mean
Moisture Content %	15 to 32	22
Plasticity Index	6 to 29	16
Liquidity Index	-0.35 to + 1.13	0.1
Clay fraction %	7 to 40	15
STRENGTH PARAMETERS	c' kN/m^2	Ø' Degrees
Puddle Clay	3	30
Embankment Fill	3	30
Glacial Clay (foundation)	3	30

Seismic stability of the dams was assessed in accordance with the recommendations of the BRE guide (BRE 1991). Apart from the Rivelin depositing pond, which is Category I, all dams in the group fall into Hazard Category III. For these categories the quasi-static method of analysis is considered adequate for initial appraisal. The results of the analyses are presented in Table 3. The stated displacements can be interpreted as vertical settlements and are sufficiently low to pose no threat of release of reservoir contents. In the case of Redmires Middle, the presence of some silty layers in the upper part of the core will require further investigation.

Table 3 - Summary of findings of seismic analyses

Reservoir	Category (Ref BRE Guide)	Static conditions	Seismic conditions (0.14g)	Predicted displacement (mm)	Comments
		Factors of Safety			
Redmires Upper	III	1.2	0.85	250	
Redmires Middle	III	1.2	0.84	250	Further investigation recommended
Redmires Lower	III	1.8	1.1	Negligible	
Rivelin Upper	III	1.1 - 1.2	<0.8	450	Effect of under drainage will increase F.O.S. & reduce displacement
Rivelin Lower	III	1.2	0.85	<250	3D effects would increase F.O.S.
Rivelin Depositing Pond	I	Not evaluated	Not evaluated	Negligible	

HYDRAULICS AND HYDROLOGY.

The catchment of the Redmires/Rivelin system comprises gently sloping grouse moor rising to an elevation of 458 m and lying to the west of the City of Sheffield. The total catchment area above Rivelin depositing pond (the lowest in the system) is 20.4 km^2. Figure 2 shows the hydraulics of the complicated interconnected system. The three Redmires reservoirs are in natural cascade above Rivelin Lower and connected to the latter by Wyming Brook. However the spillways, which are unusual in design, do not route flows through all the reservoirs in the cascade. A further complication is that low flows from the upper Rivelin catchment are diverted by a leaping weir and catchwater into Redmires upper. The Rivelin lower reservoir also receives flow from the Derbyshire Derwent catchment via the Ladybower-Rivelin tunnel at a maximum rate of 1 m^3/s.

No stream gauging data exists and it was necessary to carry out the flood assessment wholly by the method of catchment characteristics using the Flood Studies Report (FSR 1974 & supplements) and the software package MicroFSR. Standard procedures were used but the complexity of the hydraulic system rendered it necessary to break the catchment into 12 separate sub-catchments each with its own run-off characteristics.

Summation and re-routing of the hydrographs was carried out by import and export of data from MicroFSR to Lotus spreadsheet and back. This facilitated the division of flows eg between the two overflows at Redmires Middle and subsequent combining with flows from other subcatchments.

The critical storm durations varied between 4.6 and 8.0 hours and it was necessary to carry out a range of computer runs to establish the worst conditions for each reservoir.

The spillways at three of the reservoirs were sufficiently complex hydraulically to preclude accurate assessment of their rating curves by calculation and physical model tests were carried out for these.

At Redmires Upper the main spillway is a morning glory shaft located in the catchwater and discharging into a long culvert. (There is also a gated structure but as this discharges to a very narrow channel running down the left hand mitre it will be rendered inoperable). The location of the shaft asymmetrically in the catchwater leads to highly swirling flow and a severely reduced rating. The incorporation of a guide wall to reduce the swirl improved the characteristic considerably but its effect on reservoir rise was insufficient to eliminate the need for an increase in freeboard and it was decided not to adopt it. The rating without modification was therefore used in the flood routings.

The Rivelin Upper overflow is a double side channel finger spillway extending into the reservoir. The channel is shallow and the physical model test proved that the weir drowns at relatively low flows. No feasible modification would enable the structure to pass the PMF and so the rating was accepted and used in the routing to size an auxiliary spillway.

The Rivelin Lower spillway is a side channel structure which discharges via a tumble bay to a masonry chute running down the left hand mitre. The tumble bay also receives flow from the byewash channel which bypasses both the Rivelin reservoirs. The test showed that there is adequate weir capacity but the chute was much too small. The model was used to develop an enlarged chute capable of carrying the PMF flow.

The chute discharges via an 8 m high drop into a masonry lined stilling basin located at the toe of the embankment. This basin, which had fallen into considerable disrepair, was also not adequate to cope with even the 100 year event and could thereby endanger the dam itself. A further physical model was constructed to develop a more substantial stilling basin able to protect the dam from serious erosion during extreme floods.

The results from the hydrological and hydraulic investigations are presented in Table 4.

The results showed that all the reservoirs had insufficient freeboard to cope with the PMF event. In the case of Redmires Middle, the deficiency was very slight (< 80 mm) and at Rivelin Lower the deficiency disappears if the chute is enlarged so that drowning of the weir does not occur.

During the course of the investigations the ICE Guide to Floods and Reservoir Safety (ICE 1989) has been under review. One of the authors was a member of the Working Party and had access to the revised version a few weeks before the first contract was due to go out to tender. The flood calculations were immediately checked and comparisons made. The revised combinations of floods and concurrent wind speed for Category A reservoirs required slightly higher freeboard at Rivelin Upper, although this was still below the minimum of 0.6 m, while at Rivelin Lower an additional 0.18 m of freeboard would be required. The reconstructed crest of the dam incorporates a settlement allowance and is in any case highly resistant to damage by overtopping and it was decided not to change the design. If the dam does settle appreciably in the future the copings could be raised.

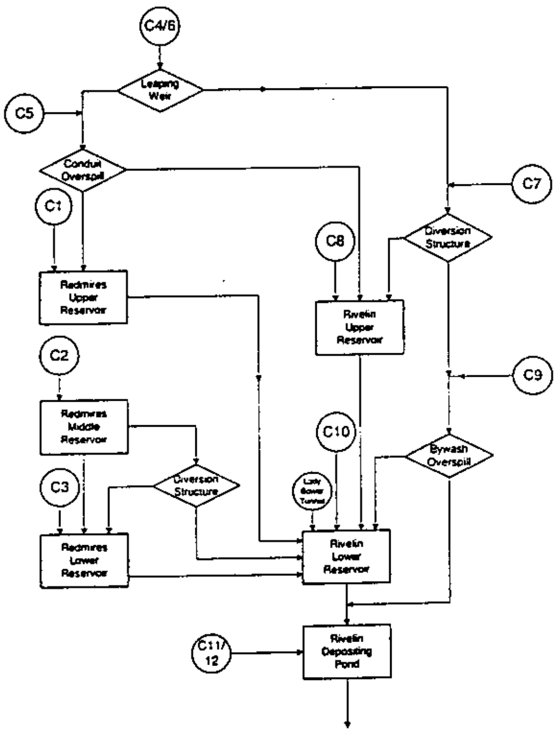


Figure 2 : Schematic Diagram of Flood Flow Routing

Table 4 - Existing Reservoir Conditions - PMF Event

Reservoir	Definitive Routed Outflow m^3/s	** Maximum Reservoir SWL m AOD	Minimum Embankment Crest Level m AOD	Deficiency w.r.t. SWL m	Required Wave Allowance m	Required Minimum Level of Wave Protection m AOD	Actual Minimum Level of Wave Defences m AOD	Deficiency w.r.t. Waves m
Redmires Upper TWL = 349.97m AOD	15.9	351.27	351.11	0.16	0.68	351.95	351.42	0.53
Redmires Middle TWL = 336.33m AOD	4.8 + 2.9	336.87	337.28	Nil	0.6	337.47	337.39	0.08
Redmires Lower TWL = 325.39m AOD	6.3	326.09	325.83	0.26	0.66	326.75	325.83	0.92
Rivelin Upper TWL = 212.70m AOD	108.2	217.21	214.11	3.10	0.6	217.81	214.52	3.29
Rivelin Lower TWL = 204.4m AOD	175.8	208.90	206.29	2.61	0.6	209.50	206.99	2.51
Rivelin Depositing Pond TWL = 167.87m AOD	182.0	172.50	169.25	3.25	0.6	173.10	169.25	3.85

** Calculation of reservoir rise does not allow uncontrolled overtopping of embankment

SOLUTIONS.

It was apparent from early work that significant improvements would be required to overflow capacity. A notional scheme had been prepared to build a flood retention dam upstream of Rivelin Upper reservoir. This was estimated to cost about £7 million and this became the benchmark against which subsequent schemes have been judged.

Once the problems had been quantified, the methods of value engineering were applied to identify the full range of solutions to be examined. In this process, the design team present the problems and an initial range of possible solutions to a group of technically knowledgeable but non specialist people who are not involved in the detail of the project. The meeting is guided by a person experienced in the procedure (the Facilitator) and in the brainstorming that follows irrational preferences are exposed and unlikely options tested.

The Value Management study team was formed of staff from a number of YWS departments (Capital Development, Operations, Reservoir Safety, and Finance) plus the Mott MacDonald Project Manager and Director. This enabled quick appraisal of options and decision taking. Four Value Management studies were held by the team, using an external facilitator. A brief outline of each study is given in Table 5. The procedure proved useful, not least in involving the operating staff in the conceptual design to ensure their full participation in the "ownership" of the solutions adopted.

Table 5 - Value Management studies

Stage	Backround	Outcome
1. Inception	Flood studies almost complete	Confirmation of design floods and model testing programme.
2. Option identification	Flood studies complete. Site investigation & model testing in progress.	Confirmation of options to be tested. Elimination of non viable and clearly non economic options, including flood retention dam
3. Risk analysis	Costing of options almost complete.	Agreement on preferred options. Risk analysis. Identification of contingencies. Overall programme.
4. Constructability study	Tender documents well advanced.	Contract issues. Impact on operations and public relations.

The main deficiencies over the whole reservoir system having been found to be hydraulic, consideration was given to the options available to rectify the situation. The range of possible solutions identified in the value engineering process included the following for the hydraulic deficiencies:

1. Raise the embankment and/or wave wall,
2. Enlarge or reconstruct the overflow,
3. Construct an auxiliary spillway,
4. Reduce normal top water level,
5. Introduce upstream storage,
6. Introduce (or increase) bywash capacity.

The last three options were rejected on the basis of brief examinations which showed them to be uneconomic or impractical. These conclusions were reached before the severe 1995 drought which served only to confirm the correctness of the decision not to reduce normal TWL. In preparing schemes for the other options, it was considered a prerequisite that the reservoirs remained in service; albeit drawn down for a period.

In the cases of Redmires Upper and Lower, the crest level deficiencies were relatively small (0.53 and 0.44 m respectively) and it was found that the cheapest solution would be to raise (or construct in the case of Lower) the wave walls. At Redmires Middle, the crest deficiency was found to be only 0.08 m close to the south abutment and it was decided not to rectify this in the current round of works.

At all three Rivelin reservoirs, the crest level deficiencies were substantial, at 3.29, 2.51, and 3.85 m respectively down the valley; these deficiencies being calculated on the basis of the excess water being retained in the reservoir. In the case of the Depositing Pond, the recommended solution is to improve the rating of the existing weir by the relocation of a footbridge and to protect the embankment, (which is only 7 m high) to enable it to overtop.

The works required at Rivelin Upper and Lower are the most substantial in the group and therefore considered to be the most urgent. Consequently these are planned for execution in 1996, and are described below.

WORKS AT THE RIVELIN RESERVOIRS.

After investigation and comparison of the options at Rivelin Upper, the solution found to be cheapest by a very substantial margin was to retain the existing finger spillway as the service overflow and to construct a new reinforced grass auxiliary spillway over the central part of the embankment. Even this would not be adequate on its own and an enhanced wave wall is included in the scheme.

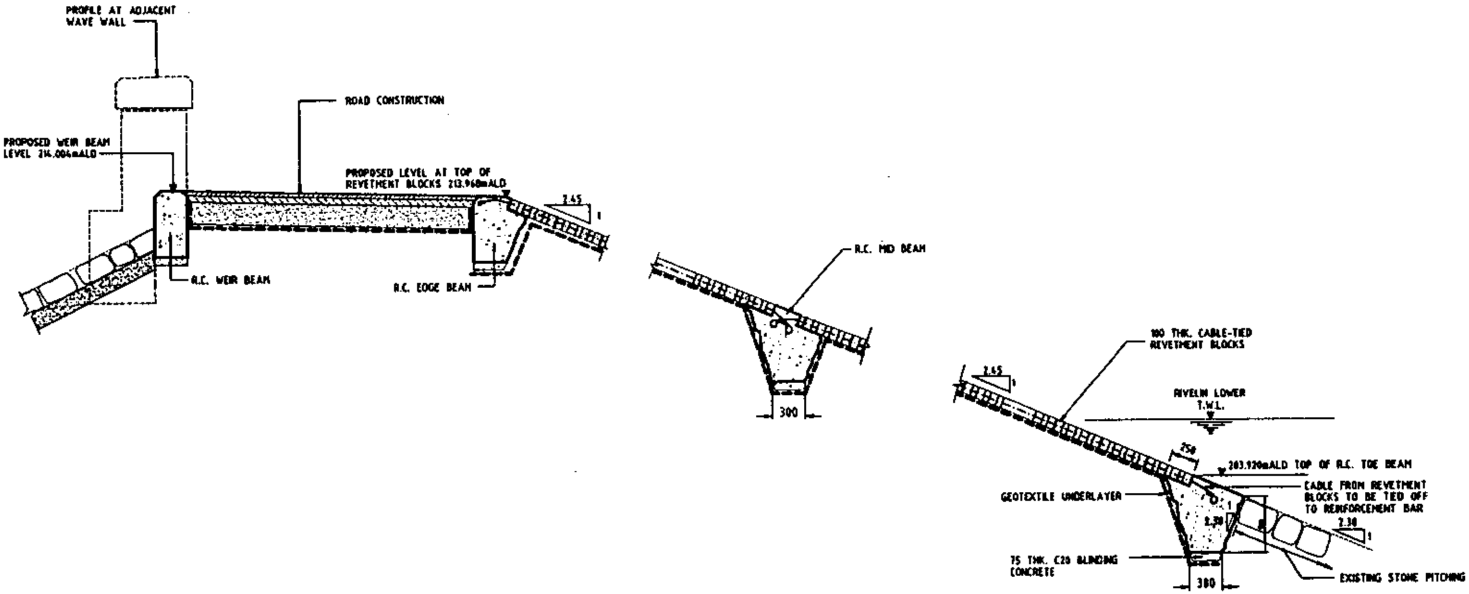


Figure 3 : Rivelin Upper Auxiliary Spillway

The auxiliary spillway, will be 70 m wide and approximately 25 m in length down the slope (see figure 3). Apart from wave slop at lesser events, it will just come into operation at an event equivalent to 0.5PMF and at full PMF will pass a maximum of 78 m^3/s over a period of about 4 hours. In these conditions the estimated flow velocity at the toe is 8.3 m/s and the depth 0.2 m. The lower reservoir forms the energy dissipator and, whatever its starting condition before the event, its capacity is such that it will be full before the auxiliary spillway comes into action. The hydraulic loading of this spillway is close to the maximum recommended by the CIRIA guide (CIRIA 1987) but the guidelines were formulated for service conditions rather than for an emergency facility that operates at very rare intervals and in circumstances when damage is expected and acceptable.

The auxiliary spillway is insufficient on its own to eliminate the crest elevation deficiency and the remaining length of the embankment will have a new reinforced concrete wave wall 1 m high. Other works include repairs and protection measures to the existing service spillway.

At Rivelin Lower the model test showed that the level deficiency arises because the constriction in the spillway chute at high flows causes the weir to become drowned. Reconstruction of the chute to accept more flow removes this constraint although the weir will still be partially drowned in the PMF situation due to the shallow apron downstream. The narrow masonry chute (3 m wide by 1 m deep) is being replaced by a reinforced concrete structure 6.5 m wide by 3 m deep.

The existing chute has a gradient of 1 in 12 and discharges over an 8 m high drop structure into a circular masonry lined stilling basin at the foot of the drop and close to the toe of the embankment. The bankfull capacity of the chute was estimated to be 20 m^3/s. The masonry of the stilling basin was in a very poor state of repair and substantial repairs were clearly necessary to enable it to fulfil its service function. However the peak discharge down the chute in the PMF event is 170 m^3/s and in these conditions it appeared that serious erosion of the toe of the embankment was likely, probably endangering the dam itself.

A mobile bed physical model of the stilling basin area was commissioned and confirmed that the toe of the dam was indeed at risk at very high flows and the model was used to investigate a solution. The aim was to devise the simplest and least cost solution which would protect the dam during a PMF but not necessarily to provide a fully effective energy dissipator to protect the bed downstream. Thus the problem is to deflect the high velocity flows (14m/s exit velocity) away from the dam and valley side towards the channel downstream. The rather unusual structure shown in Fig 4 was the outcome. It contains two deflectors, one at high level to control the falling

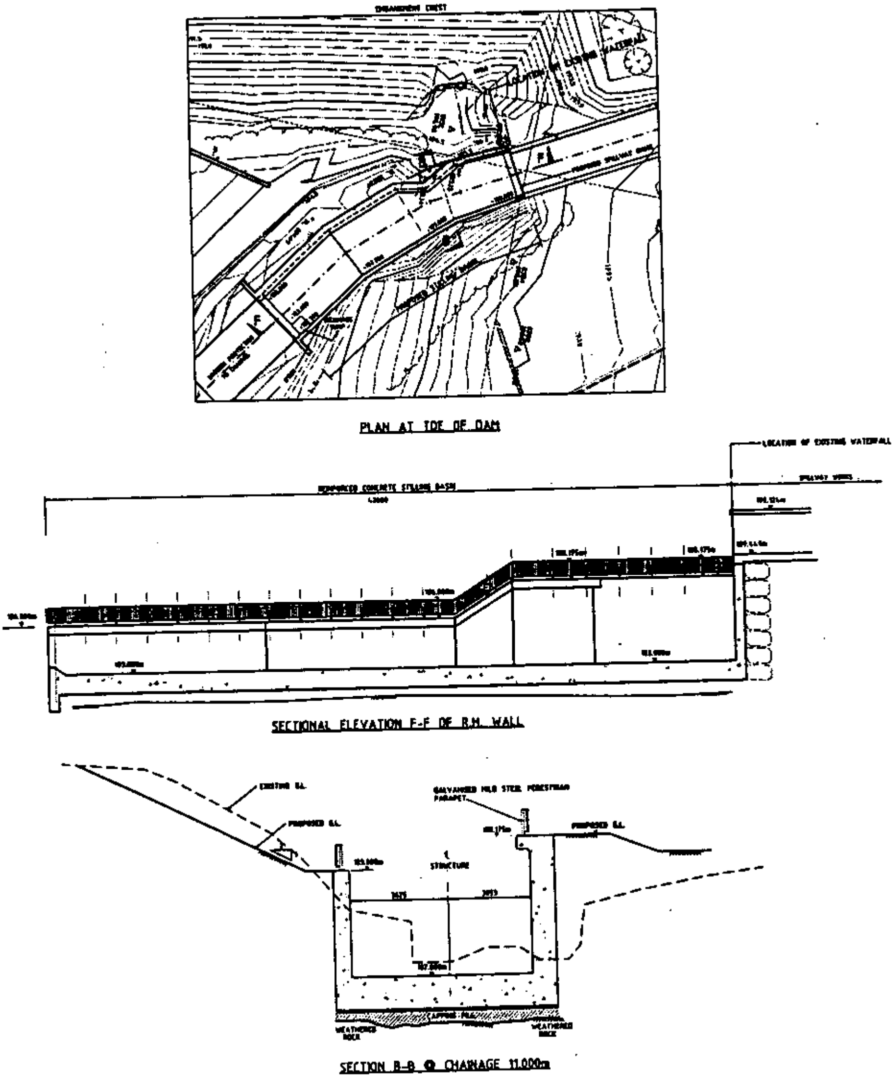


Figure 4 : Rivelin Lower Stilling Basin

nappe and the other long and low to deflect the bottom flow. The model test showed it to be effective in diverting the flow away from the dam. The overall size of the stilling basin was derived from the model test performance, recognising that 170m^3/s is not a service condition and that some damage is acceptable. However it is not an easy matter to design such a structure for damage at a certain flow threshold and it is likely that it is somewhat over designed.

In the case of Rivelin Lower, there was additionally the problem of instability of the crest of the dam. The possible solutions identified to rectify this situation included:

1. Replace existing fill by light weight fill,
2. Improve drainage,
3. Reconstruct retaining walls,
4. Reinforce existing retaining walls,
5. Reprofile the embankment crest,
6. Reduce top water level,
7. Provide external buttresses.

The third option was adopted and the solution is illustrated in Figure 5.

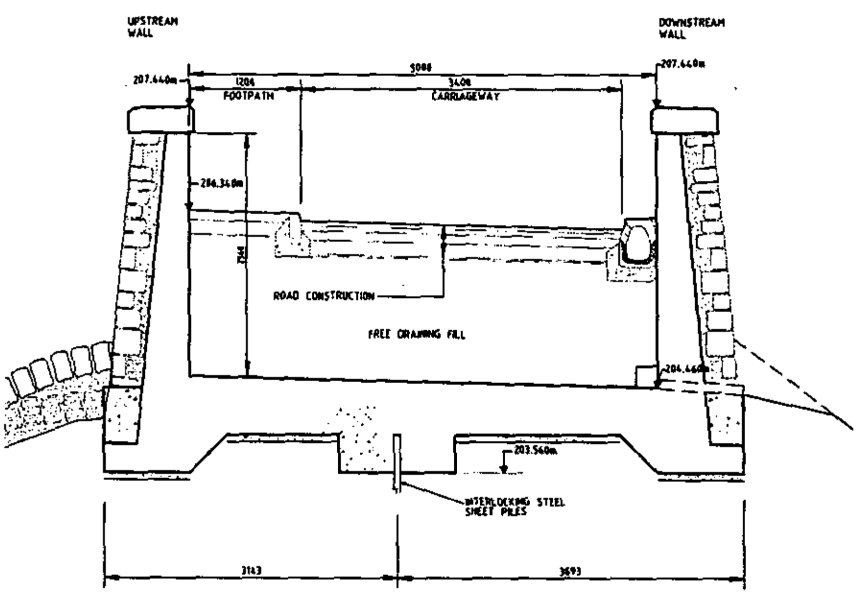


Figure 5 : Rivelin Lower Crest Reconstruction

During the execution of the remedial works there is inevitably some risk of the dams being damaged by flood waters. In order to minimise the risk, the reservoirs will be drawn down to the maximum amount consistent with maintaining water quality, the restriction being necessary due to the considerable quantities of silt, particularly in Rivelin Lower. The drawdown, combined with the maximum outflow from low level outlets, will enable the reservoirs to absorb the average annual flood without the need to spill. To protect the works against damage in greater flood events the contractor is required to work in sections such that the exposure is minimised and to have readily available means and an emergency plan to protect the working areas. This has to be backed up by a procedure to use real time rainfall data to generate flood forecasts with sufficient lead time to enable protective measures to be put in place.

PROCUREMENT

Mott MacDonald were engaged under ACE Agreement 1 to carry out the feasibility work, including flood studies, supervising ground investigations and model testing.

Ground investigations were done by Norwest Holst and Geotechnical Engineering. Hydraulic model testing was done by Hydraulic Models Ltd of Horsforth, under ACE Agreement 1.

Design and supervision of the first contract (Contract 1), for the Rivelin Upper and Lower reservoirs is being done by Mott MacDonald under the Professional Services Contract. The construction contract is being let by competitive tender under the ICE 6th Edition. Subsequent contracts may use the NEC conditions.

CONCLUSIONS.

The paper has outlined a programme of major and minor works which are in course of implementation at the six reservoirs and are designed to secure them for the next few decades of service. While not large, these reservoirs form a significant component of the water supply to Sheffield and the drought of 1995 has underlined the value of storage. Already over 100 years old, the resource will be required well beyond the foreseeable future, ie for 50 to 100 years or more; not least because of the severe public opinion constraints on the development of new sources involving the construction of dams.

Recent experience has shown the need for regular maintenance either to compensate for deterioration (eg settlement) or to meet the requirements of increasing safety criteria (eg flood capacity or seismic resistance). It is to be hoped that the development of safety criteria in these areas has reached a limit and that consequently no more corresponding major works will be required. Deterioration will undoubtedly continue and it may be that future inspecting engineers will become concerned about the lack of knowledge of the interior conditions of these old dams which lack the instrumentation of their modern equivalents and which pre-date by many decades a real understanding of the science of soil mechanics. So it seems likely that a greater emphasis on the installation of instruments will be a feature in certain cases in the future.

Another area that may receive attention is the removal of silt from reservoirs, a number of which, like Rivelin Lower, have lost a large proportion of their live storage. To do this economically has baffled operators since dams were first constructed but the need is there.

ACKNOWLEDGEMENTS

The Authors are grateful to Yorkshire Water Services Limited for permission to publish this paper.

REFERENCES

BUILDING RESEARCH ESTABLISHMENT (1991) An Engineering Guide to Seismic Risk to Dams in the United Kingdom.

CIRIA (1987) Design of Reinforced Grass Waterways, CIRIA Report 116.

INSTITUTION OF CIVIL ENGINEERS (1989) Floods and Reservoirs Safety, an Engineering Guide.

KALAUGHER & GRAINGER (1994) Photographic Monitoring in the Safety Assessment of Dams and Reservoirs. Safety of Embankment Dams 1994.

NATURAL ENVIRONMENT RESEARCH COUNCIL (1975) Flood Studies Report.

Author index