BANGOR

BDS

British Dam Society

The prospect for reservoirs in the 21st century

Proceedings of the tenth conference of the BDS held at the University of Wales, Bangor on 9–12 September 1998

Edited by Paul Tedd

Thomas Telford

Organising Committee: Ian Carter (Chairman), Ian Hay, Jim Millmore, Paul Tedd and Owen Williams

Published by Thomas Telford Publishing, Thomas Telford Ltd, 1 Heron Quay, London E14 4JD.
URL: http://www.t-telford.co.uk

Distributors for Thomas Telford books are
USA: ASCE Press, 1801 Alexander Bell Drive, Reston, VA 20191-4400, USA
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

First published 1998

Also available from Thomas Telford Books
The reservoir as an asset. British Dam Society. ISBN 0 7277 2528 9.
Reservoir safety and the environment. British Dam Society. ISBN 0 7277 2010 4
Water resources and reservoir engineering. British Dam Society. ISBN 0 7277 1692 1

A catalogue record for this book is available from the British Library

ISBN: 0 7277 2704 4

Printed in Great Britain by Hillman Printers (Frome) Limited

Preface

The tenth conference of the British Dam Society was held at the University of Wales, Bangor, in 1998. The theme of the Conference *The Prospect for Reservoirs in the 21st Century* attracted papers on a wide variety of subjects including risk assessment, flood control, environmental issues, sedimentation problems and rehabilitation. Many of the 33 papers in the proceedings relate to the management, performance and rehabilitation of the existing stock of ageing dams in the UK. Increasingly it is expected that activities which impose risks on the public should be subjected to some form of risk analysis and a number of papers relate to the use of risk analysis in the safety management and maintenance of dams.

Papers describe investigations of internal erosion. Case histories of remedial works to control leakage include traditional grouting using cement-bentonite and slurry trench cut-off walls. One paper describes the first use of a slurry trench cut-off wall with an HDPE membrane for a new flood storage reservoir, a technique common in pollution containment. Other topics include the deterioration and repair of an upstream asphaltic concrete membrane, bituminous grouting of rip-rap, ASR damage on a spillway and the abandonment of tailings lagoons as wetlands.

The 1998 Geoffrey Binnie Lecture *Lives of embankment dams: construction to old age* was given by Dr Andrew Charles of the Building Research Establishment and is published in the Society's journal *Dams & Reservoirs*.

Contents

Reservoir operation and management

Construction and rehabilitation case histories

The role of risk analysis in the safety management of embankment dams

J A CHARLES, Building Research Establishment, UK
P TEDD, Building Research Establishment, UK
H D SKINNER, Building Research Establishment, UK

SYNOPSIS. Large reservoirs can present a serious public safety hazard where downstream areas are heavily populated. Increasingly it is expected that some form of risk analysis will be carried out for activities which impose risks on the public. Reservoir owners and operators have specific responsibilities for their dams and need to formulate safety management procedures within the context of the Reservoirs Act 1975.

INTRODUCTION

A substantial number of large reservoirs in the United Kingdom are located immediately upstream of, or adjacent to, heavily populated areas and the number of casualties in a worst event could be greater in a dam failure than in most kinds of technological disaster. Many of the reservoirs are impounded by embankment dams built many years ago without the benefits of modern earthmoving plant or current understanding of soil behaviour. Long term satisfactory performance of these structures is vital for public safety and has significant financial implications for the owners. In a paper presented at the 1996 British Dam Society (BDS) Conference, Milne described the management system for reservoir safety of the second largest UK water services company and affirmed that as reservoirs are generally considered to be a low risk/high consequence scenario, careful management of these risks is essential. In managing reservoir safety two systems need to be considered (Betamio de Almeida and Viseu, 1996); the dam-reservoir system and the downstream valley system. For the dam-reservoir system the risk of a dam breach with a consequential uncontrolled release of water needs to be evaluated and reduced as far as is reasonably practicable. For the downstream valley system the effect of an uncontrolled release of water in terms of property damage and casualties should be assessed together with the need for warning systems to ensure rapid evacuation in an emergency.

QUANTITATIVE RISK ANALYSIS

Risk analysis can be qualitative or quantitative. The term quantitative, or quantified, risk assessment (QRA) refers to the technique of assessing the frequency of an unwanted event and its measurable consequences in terms such as number of fatalities or cost of damage. This type of analysis is also described as a probabilistic risk assessment.

Risk analysis techniques are advocated by many regulatory bodies to assess the safety of modern, complex process plants and their protective systems. QRA has found application in the nuclear, chemical, offshore, defence, marine and automotive industries (Health and Safety Executive, 1989). However, there are important differences between chemical and nuclear plant, for example, and reservoirs impounded by embankment dams. The former are built of a large number of components of specified reliability and the reliability of the entire system can be analysed using fault trees and event trees. The dam-reservoir system includes the embankment dam and its foundation, the reservoir and its bed, and ancillary works including overflow and outlet structures. While the pipework in the outlet works might be considered to be analogous to process plant, the embankment dam and its foundation are quite different and the stages in any malfunctioning are not amenable to fault tree or event tree analysis. The overall probability of dam failure being caused by internal erosion can be estimated as reliably as the various stages of the internal erosion process.

QRA has been applied to a typical earth embankment dam in north west England (Parr and Cullen, 1988). The method involved large numbers of fault trees and event trees concerning various hazard scenarios for which little information on the probability of occurrence was available and it was concluded that risk assessment based on the existing database could not be relied on to quantify the risk of dam failure. It was recommended that retrieval and collation of information on dam incidents should be continued in order to expand the database. The BRE dams database has been developed to meet this need (Tedd et al, 1992). A number of research projects undertaken as part of the DETR Reservoir Safety Research Programme (Wright et al, 1992) have provided assistance in carrying out risk analyses (Charles, 1997).

It can be questioned whether QRA as used in process industries involving large numbers of fault trees and event trees is appropriate for the assessment of reservoir safety, particularly where embankment dams are concerned. Nevertheless, many of the principles of QRA are applicable and need to be adapted to the reservoir safety situation in the same way that they are being adapted to the hazards posed by, for example, contaminated land (Ferguson and Denner, 1996). The need for quantitative risk assessments of reservoir safety is becoming more widely appreciated and the status of risk assessment in Australia, Austria, Canada, France, Germany, the Netherlands, Norway, Sweden, Switzerland, the United Kingdom and the United States of America was reviewed in the Workshop on Risk-Based Dam Safety Evaluations held in Trondheim (NNCOLD, 1997).

QRA generally involves the following stages:

- identification of potential hazards,
- estimation of probability of occurrence of hazards,
- estimation of consequences of hazards,
- comparison of results of analysis with acceptability criteria.

In applying QRA to reservoir safety it is necessary to implement these various stages for both the dam-reservoir system and the downstream valley system. For the dam-reservoir system there are several hazard scenarios which could lead to an uncontrolled release of water, most of which are associated with a breach in the dam but could also include failure of the outlet or overflow works. The potential causes of a breach in an embankment dam are relatively easy to identify and include the following:

- overtopping during an extreme flood,
- internal erosion,
- slope instability.

The first two of these hazards account for the majority of failures. As spillways are designed for floods based on a probable maximum flood (PMF) or a return period, it is not too difficult to estimate the probability of failure due to this cause. However, there is considerable difficulty in estimating the probability of the occurrence of internal erosion, although the BRE dams database provides helpful historical data on incidents. Slope instability often occurs during embankment construction prior to reservoir impounding. Settlement of the embankment due to consolidation of the fill and foundation will occur so slowly that it is unlikely to lead to breaching of the embankment.

For the downstream valley system, the hazard is an uncontrolled release of water. The downstream conditions are independent of the dam-reservoir system, but ideally the breach formation needs to be coupled to the analysis of the dam-reservoir system. The consequences of the occurrence are a function both of the hazard itself (eg speed at which breach develops) and the downstream conditions including topography, location and number of habitations, and emergency warning facilities. Dam break analysis can be carried out to evaluate the formation and propagation of a dam failure wave along a valley. DeKay and McClelland (1993) have related loss of life to the population at risk and warning time in dam failures in the USA.

HISTORICAL LEVEL OF RISK

Two different types of risk should be distinguished:

- risk to public safety posed by the possibility of an uncontrolled release of water,
- risk of economic loss to the dam owner or operator which can occur

simply due to the need to draw down the reservoir level as a safety measure.

It is helpful to analyse the historical data for three periods and it can be shown that there has been an improvement in safety in these successive periods:

- 1831-1930; the 100 year period prior to the Reservoirs (Safety Provisions) Act of 1930,
- 1931-1985; the 55 year period when the Reservoirs (Safety Provisions) Act of 1930 was in force,
- 1986-present; the 12 year period during which the Reservoirs Act 1975 has been in force.

In considering the risk to public safety, the historical information on failures which have caused loss of life should be reasonably complete and provide a reliable basis for evaluating trends in national risk. A failure causing one or more deaths occurred on 12 occasions during the period 1831-1930. This is shown on an F-N plot in Figure 1. During this period the number of dams where failure could cause loss of life was on average about 500 and the occurrence of a failure causing loss of life was therefore $3x10^{-4}$ per dam year. In the following 55 year period which followed the introduction of reservoir safety legislation, no failures occurred which have caused loss of life. This means that no loss of life occurred throughout some $1x10^{5}$ dam years. This figure is based on the population of reservoirs within the safety legislation which could cause loss of life if there was a failure. This suggests that the probability of a failure which would cause loss of life had been reduced by a factor of 10 or more.

The above data refer to British dam failures causing loss of life. However, serious property damage may result from an uncontrolled release of water even when there is no loss of life. A major failure in the UK today could lead to property damage of several hundred million pounds. Wright (1994) presented data for dam failures during the period 1960-1971 and listed 8 catastrophic failures in Great Britain during this 12 year period. The failures, which were mostly of small dams, corresponded to a failure rate of $3x10^{-4}$ per dam year. Wright commented that no catastrophic failures were known since the implementation of the 1975 Reservoirs Act in 1985-86. This indicated that the risk of a catastrophic failure had been reduced by an order of magnitude.

Many of the earlier failures were due to overtopping which an embankment dam can rarely withstand for a long period. New overflow works have been built at many large dams to meet new flood standards and this should greatly reduce the incidence of failures due to overtopping in extreme floods. The principal remaining hazard scenario is a rapid failure of an old embankment dam due to internal erosion such as that which occurred at Warmwithens in 1970. Internal erosion is being examined by BRE as part of the reservoir safety research programme of the DETR and there is a European Research Working Group on Internal Erosion in Embankment Dams (Charles, 1998).

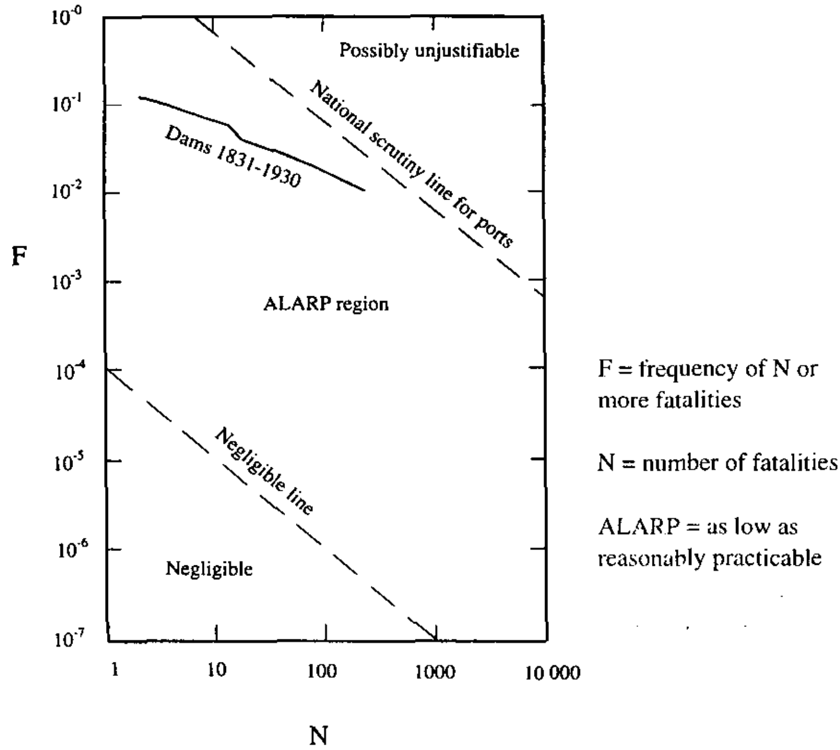


Figure 1. Comparison of British dam failures during the period 1831-1930 with risk tolerability for ports as given by Health and Safety Commission Advisory Committee on Dangerous Substances (1991).

It is not easy to quantify the risk posed by internal erosion and Salmon (1997) affirmed that piping and internal erosion present unique challenges. Information concerning incidents has been compiled in the BRE dams database (Tedd et al, 1992) and analyses of this data may give some indication of what could happen. A subset of 700 British embankment dams which are at least 10m high has been used to study the occurrence and seriousness of internal erosion. Data on reported incidents, categorised according to their severity, have been plotted in Figure 2. The information on failures and major incidents is likely to be much more complete than that for more minor, although still relatively serious, incidents. The data have been normalised by dividing the number of reported incidents during a particular period since completion of construction by the number of dams with an age greater than or equal to that period. Nearly a half of the incidents occurred in the early years during first filling of the reservoir. This is followed by a long period when the frequency of reported incidents is low, with some increase in problems in old age. This pattern of behaviour has some similarity to the "bathtub" curve which describes the variation with time of the failure rate of mechanical and electrical components. This statistical data can be used to estimate the probability of internal erosion at old British dams.

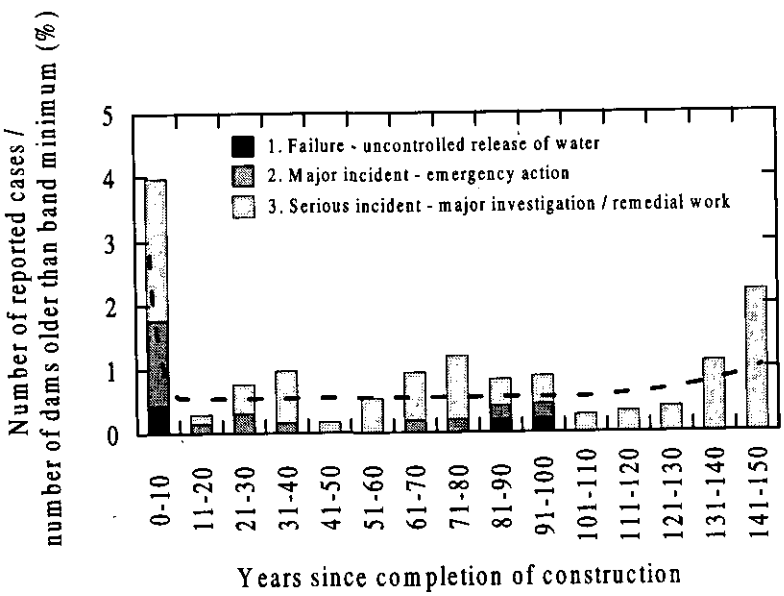


Figure 2. Internal erosion at British dams 10m or more in height categorised by seriousness and age.

A safety incident could cause serious economic loss to a reservoir operator or owner even though public safety was not imperilled. The occurrence of problems with slope stability and outlet works has been examined for the subset of 700 British embankment dams which are at least 10m high. Slope stability problems have been reported at some 9% of these dams, but there have been few failures. The failure of Carsington dam during construction in 1984 was the most notable incident and the emergency drawdown and subsequent demolition of Lambieletham dam has been the most serious in-service incident attributable to slope instability. Generally the slope stability problems have involved only small scale slips necessitating geotechnical investigation and remedial work. Some 5% of the dams have reported problems with outlet works which have necessitated remedial action; there have been structural and mechanical problems as well as siltation of the tunnels.

ACCEPTABLE RISK

As it is not feasible to entirely eliminate the risk of failure, the safety management of a reservoir should be undertaken with some view of what is an acceptable level of risk. The acceptability of risk of damage is relatively easy to assess from an economic and insurance perspective. A more difficult issue concerns loss of life. There is a substantial body of opinion that considers that it is not appropriate to put a price on a life and therefore the risk of casualties must be dealt with in other ways. A reasonable approach can be based on the principle that the hazard should not impose an involuntary risk on the downstream population which makes a significant difference to the pre-existing comparable natural risks. Regulatory bodies work on the as low as reasonably practicable (ALARP) principle. This should not cause any difficulties for new construction, but may have significant implications for existing infrastructure.

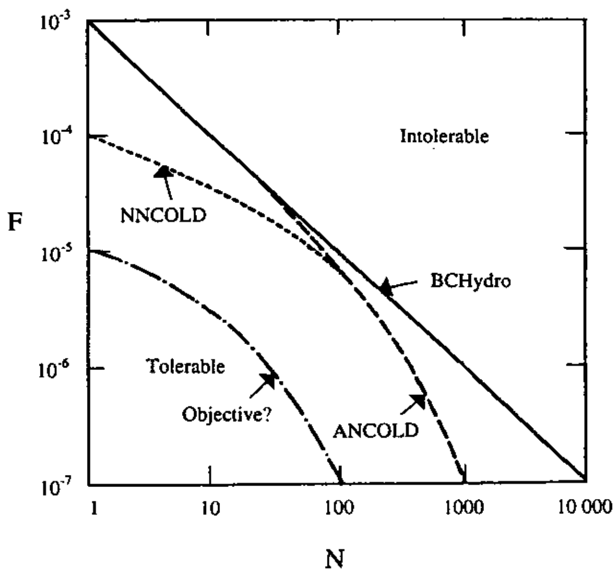
HSE has proposed a value of $1x10^{-6}$ per year as an individual level of risk that would be "broadly acceptable" to members of the public (Health and Safety Commission Advisory Committee on Dangerous Substances, 1991) and the implications of this criterion for the acceptable risk of failure of a reservoir need to be examined. It does not necessarily mean that the risk of a breach must be smaller than $1x10^{-6}$ per year as a particular individual living in a vulnerable location may not be a casualty due to absence, evacuation, or providential deliverance out of the flood and it can be argued that a risk of breaching of $1x10^{-5}$ per year is unlikely to impose a risk of greater than $1x10^{-6}$ per year on an individual.

The guide to floods and reservoir safety (Institution of Civil Engineers, 1996) recommends that category A dams (ie dams where a breach could endanger 10 or more lives) should be designed to take the PMF unless overtopping is tolerable. Although the PMF cannot be related in a simple way to an annual exceedance probability, this flood standard is likely to be broadly compatible with the requirement that the risk of breaching should not be greater than $1x10^{-5}$ per year.

Societal risk is usually expressed in terms of a relationship between the number of fatalities (N) and the probability of occurrence (F). F-N curves derived from Hoeg (1996) are shown in Figure 3; these suggest that the objective should be an annual probability of failure causing loss of life of not greater than $1x10^{-5}$. McDonald (1997) has reported that there is a view in Australia that the curve representing societal risk criteria should be truncated at 1×10^{-6} per annum as it is misleading to suggest that the much lower probabilities of failure for high prediced loss of life are achievable.

SAFETY MANAGEMENT AND SAFETY STANDARDS

The process of implementing decisions associated with the assessment, toleration and reduction of risks can be termed safety management. Reservoir owners and operators have specific responsibilities for their dams and need to formulate safety management procedures within the context of the Reservoirs Act 1975. Technical and managerial approaches should be utilised to improve safety and reduce risk. The safety management of dams has major implications for public safety as well as financial and insurance implications for dam owners.



F = annual probability of N or more fatalities due to failure of a specified dam
N = number of fatalities
ANCOLD = Australian National Committee on Large Dams
NNCOLD = Norwegian National Committee on Large Dams

Figure 3. Proposals for acceptable societal risk from failure of a dam (after Hoeg, 1996).

An increase in reservoir safety is provided at an increase in cost and a balance has to be found between dam safety and economy. Cost effective risk reduction involves defining the acceptable level of risk, reducing the risk of the dam breaching to an acceptable value and implementing emergency management procedures to endeavour to ensure that there is no loss of life should the dam breach. The approaches to risk reduction for the dam-reservoir system can include structural improvements to dam and ancillary works, improved surveillance, monitoring and maintenance. The approaches to risk reduction for the downstream valley system include the preparation of inundation maps, estimation of time of arrival of flood wave at different locations and the duration of inundation and the implementation and maintenance of emergency warning procedures and systems. The difficulty in knowing when to give warning makes the operation of emergency procedures very difficult. The rapidity with which a reservoir can be drawn down can be critical in, for example, an internal erosion incident.

For embankment dams, continuing day to day safety of the dam-reservoir system will depend on some form of observational method involving surveillance and monitoring (Charles, 1993; Charles et al, 1996). However, while the observational method is necessary, it is not sufficient as unusual operating conditions and extreme events such as floods and earthquakes must also be considered.

It would seem sensible to require higher safety standards where the risk to life and property is greater. The four categories in the guide to floods and reservoir safety (Institution of Civil Engineers, 1996) are based on this principle as are F-N criteria. However, there is no basis for accepting a greater risk of failure for an old dam than for a new dam. If this means that safety standards should be the same, there are considerable financial implications. It has been claimed that:

"No country in the world can afford to upgrade all its dams to current safety standards" (Chemaly and Nortje, 1994).

It would appear that some parts of the world have made greater efforts than others in this respect. Considerable work has been done in North Rhine Westphalia and Northern Ireland. A substantial proportion of the large dams in Germany are in North Rhine Westphalia and in 1989 this state implemented a dam adaptation law which required old dams to be rehabilitated according to current engineering standards as set by a government department; dams which did not meet these technical standards had to be modified within a given period (Rissler, 1993). In Northern Ireland a comprehensive programme of investigations and remedial works adopted when, in 1972, the Department of the Environment for Northern Ireland Water Service assumed responsibility for all public reservoirs in the Province (Cooper, 1987). Most of the 70 or so reservoirs are impounded by embankment dams. Filters with overlying rockfill stabilising berms were installed at nine locations. The capital expenditure of the entire programme was reported as £6 million at 60 sites over a 13 year period.

Even if acceptable risk should be no greater for an old dam than a new dam, this does not necessarily mean that safety factors need to be the same. A large proportion of problems occur during first filling of the reservoir and it would seem reasonable to accept somewhat lower slope stability factors of safety at old embankment dams where this most vulnerable stage is over and there is a long record of satisfactory performance. Linking safety standards to risk analysis can provide a way forward.

CONCLUDING REMARKS

Increasingly it is expected that some form of risk analysis should be carried out for activities which impose risks on the public. The dam engineering profession has a duty not only to achieve high standards of reservoir safety, but also to be able explain its actions and the uncertainties involved.

For a dam which presents a significant hazard for public safety (category A dams according to the classification in the guide to floods and reservoir safety), it is suggested that an acceptable probability for a breach with a consequential uncontrolled release of reservoir water is of the order of 1×10^{-5} per year. The PMF flood standard is likely to be broadly compatible with this for the risk of failure due to overtopping in an extreme flood, but the risk of internal erosion is much harder to assess. It is difficult to make a realistic quantitative assessment of the safety of an old embankment dam, particularly when little is known of its construction. A large proportion of failures and serious incidents occur during first filling of the reservoir.

It should be remembered that the objective is not merely to analyse and assess risks, but rather to reduce risk and improve safety. The approaches to risk reduction can include:

- structural improvements to dam and ancillary works,
- improved surveillance, monitoring and maintenance,
- improved emergency management procedures.

The United Kingdom has a good record for reservoir safety, but complacency would be out of place. Some warning notes were sounded at the 1996 BDS Conference and the following factors were cited as causes for concern (Hay, 1996); there has been a loss of reservoir staff, fee competition for reservoir inspections has been introduced, there was often a loss of continuity in supervision and inspection and, excessive confidentiality was required by some dam owners. One area in which the United Kingdom might be perceived to be weak in comparison with many other European countries is the lack of legal requirements for emergency procedures. Furthermore, while there has been major expenditure on overflow works to meet improved flood standards, there has been no comparable work on upgrading old embankments.

Quantitative risk assessment can assist in improving safety at minimum cost. The reservoir safety research programme of the DETR has included several projects of direct relevance to risk assessment and currently work is being carried out under a CIRIA contract which should lead to the development of QRA techniques which are appropriate for reservoir safety.

ACKNOWLEDGEMENTS
Research on the safety of embankment dams at the Building Research Establishment (BRE) has been supported by the Department of the Environment, Transport and the Regions (DETR). The support of DETR and their permission to publish the paper are gratefully acknowledged.

REFERENCES

Betamio de Almeida A and Viseu T (1996). Dams and valleys safety: a present and future challenge. Dams and Safety Management at Downstream Valleys. Proceedings of International NATO Workshop, Lisbon, November 1996, pp 3-25. Balkema, Rotterdam, 1997.

Charles J A (1993). Embankment dams and their foundations: safety evaluation for static loading: keynote paper. Proceedings of International Workshop on Dam Safety Evaluation, Grindelwald, April, vol 4, pp 47-75. Dam Engineering, Sutton, Surrey.

Charles J A (1997). Acceptable risk and reservoir safety. Dams & Reservoirs, vol 7, no 2, July, pp 7-12.

Charles J A (1998). Internal erosion in European embankment dams: Progress report of Working Group on Internal Erosion in Embankment Dams. To be published in Proceedings of International Symposium on New Trends and Guidelines on Dam Safety, Barcelona, June.

Charles J A, Tedd P, Hughes A K and Lovenbury H T (1996). Investigating embankment dams: a guide to the identification and repair of defects. Building Research Establishment Report BR 303. Construction Research Communocations, London.

Chemaly A G and Nortje J H (1994). Dam safety in South Africa: legislation and experience. International Journal on Hydropower and Dams, vol 1, no 6, November, pp 42-46.

Cooper G A (1987). The reservoir safety programme in Northern Ireland. Journal of Institution of Water and Environmental Management, vol 1, no 1, pp 39-51.

DeKay M L and McClelland G H (1993). Predicting loss of life in cases of dam failure and flash flood. Risk Analysis, vol 13, no 2, pp 193-205.

Ferguson C C and Denner J M (1996). Risk assessment as a key to risk management of contaminated land. Contaminated Land Conference. IBC UK Conferences Limited.

Hay J (1996). Reservoir inspections - time for a change? The Reservoir as an Asset. Proceedings of 9th Conference of British Dam Society, York, September, pp 192-205. Thomas Telford, London.

Health and Safety Executive (1989). Quantified risk assessment: its input to decision making. HMSO, London. 30pp.

Health and Safety Commission Advisory Committee on Dangerous Substances (1991). Major hazard aspects of the transport of dangerous substances. HMSO, London. 371pp.

Hoeg K (1996). Performance evaluation, safety assessment and risk analysis for dams. International Journal on Hydropower and Dams, vol 3, no 6, pp 51-58.

Institution of Civil Engineers (1996). Floods and reservoir safety: 3rd edition. Thomas Telford, London. 63pp.

McDonald L A (1997). Status of risk assessment for dams in Australia. Proceedings of Workshop on Risk-Based Dam Safety Evaluations, Trondheim, June 1997. Norwegian National Committee on Large Dams, Oslo.

Milne P (1996). The management of reservoir safety. The Reservoir as an Asset. Proceedings of 9th Conference of British Dam Society, York, September, pp 206-215. Thomas Telford, London.

NNCOLD (1997). Proceedings of Workshop on Risk-Based Dam Safety Evaluations, Trondheim, June 1997. Norwegian National Committee on Large Dams, Oslo.

Parr N M and Cullen N (1988). Risk management and reservoir maintenance. Journal of Institution of Water and Environmental Management, vol 2, no 6, pp 587-593.

Rissler P (1993). The Intze-type masonry dams in Germany: recent safety assessments, adaptations and repairs. Maintenance of older dams: accidents, safety assessments, repairs. Proceedings of Technical Symposium, Chambery, September, vol 1, pp 171-197. Comité Francais des Grands Barrages.

Salmon G M (1997). Risk assessment for dams - better tools are needed. Keynote talk. Proceedings of Workshop on Risk-Based Dam Safety Evaluations, Trondheim, June 1997. Norwegian National Committee on Large Dams, Oslo.

Tedd P, Holton I R and Charles J A (1992). The BRE dams database. Water Resources and Reservoir Engineering. Proceedings of 7th British Dam Society Conference, Stirling, pp 403-410. Thomas Telford, London.

Wright C E (1994). UK reservoir failures and safety legislation. Dams & Reservoirs, vol 4, no 3, October, pp 20-21.

Wright C E, Coats D J and Charles J A (1992). The reservoir safety research programme of the Department of the Environment. Water Resources and Reservoir Engineering. Proceedings of 7th British Dam Society Conference, Stirling, pp 411-424. Thomas Telford, London.

Risk assessment strategies for dam based hydro schemes

N M SANDILANDS, Scottish Hydro-Electric plc
M NOBLE, Scottish Hydro-Electric plc
J W FINDLAY, Babtie Group

SYNOPSIS. The paper describes the asset management approach adopted by Scottish Hydro-Electric for their civil asset base, including dams, and highlights some of the risk assessment techniques developed with the help of Babtie Group. The second part of the paper draws on these experiences to discuss the relevance of risk assessment to individual dams in the UK.

INTRODUCTION

Scottish Hydro-Electric own a large and varied civil asset base including a large number of major dams. As an informed and proactive owner the Company has always taken a wider view on the safety of dam structures. As part of the general asset management procedures, risk assessment strategies have been developed, with the assistance of Babtie Group. This has been based on Babtie Group's experience in using the Failure Mode Effect and Criticality Analysis approach to risk assessment of water industry infrastructure. The paper presents the particular circumstances of Scottish Hydro-Electric's asset management requirements and the application of risk assessment techniques to this before discussing some of the wider influences on the debate on risk assessment requirements for individual dam structures.

RISKS ASSOCIATED WITH AGEING INFRASTRUCTURE

Asset Management

Scottish Hydro-Electric is the largest generator of conventional hydro power in the UK, owning and operating 56 hydro power stations with a total installed capacity of 1100 MW. The Company also owns one 300 MW pumped storage scheme. A total of 76 reservoirs are registered under the Reservoirs Act, 1975 and these are impounded by 84 dams. 56 of these dams are listed in the ICOLD Register. Diverted catchments form 40% of the total catchment area and contribute significantly to the complexity of the civil works. There are more than 400 intakes, 200 km of aqueducts, 300 km of tunnels, 75 km of steel pipeline, 200 major gates and valves and 500 km of roads. The current cost capital value of the civil works is around £1800M. The oldest scheme is Tummel, incorporating Rannoch and Tummel Bridge, which was commissioned in 1932. The newest scheme is Strathfarrar which

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

was completed in 1963. Thus the age of the works now varies between 35 and 65 years old. The assets are therefore mature and a different strategy is now required from that which was appropriate a decade ago.

Since privatisation in 1991 the Company has reviewed the management of these assets with a view to establishing a modern system of asset management which extracts the optimum value from the assets. Prior to 1991 a system was in place which would have been considered good practice for a public utility at that time. All assets were inspected at regular fixed intervals. Inspections were generally defect based and the maintenance and refurbishment programmes were short term and aimed at addressing identified problems.

Forward planning is now based on a twenty-five year time scale. The inspection programme is more flexible with frequency of inspections related to perceived risk. At each inspection a report is produced which includes a forward work plan for twenty-five years ahead. The engineer identifies both the estimated cost of the works and the year when the work should be carried out. This is based on an assessment of operational history, past maintenance history and performance of similar assets as well as current physical condition. One of the specific aims of this approach is to align maintenance and refurbishment budgets to the needs of the asset rather than to ration the available budget. For this reason a points system approach for prioritising works has been rejected. The commitment to maintain the assets is demonstrated by an increase in the annual maintenance and refurbishment budget for hydro civil works from around £2.5M in 1991 to around £5.5M in 1997. About 40% of this expenditure relates to dams and reservoirs.

Contingency Planning

In addition to managing the condition of the assets, contingency arrangements also require to be managed. Hydro-Electric has a well-developed system for coping with emergency situations. The hydro control centres at Dingwall and Clunie act as the co-ordination centres in any emergency situation. The Operations Department has regular contact with the other organisations involved in emergency planning, the Police, SEPA and local authorities. Emergency exercises are held periodically to assess the effectiveness of the arrangements.

Even the best prepared systems will, however, have great difficulty in coping with the consequences of an event such as a major dam failure. One of the aims of the risk assessment process is to reduce the probability of a failure occurring. An additional purpose is to assist in emergency and contingency planning for these events. Although such an occurrence is extremely unlikely it cannot be ignored because of the extreme consequences.

To cope adequately with the consequences of a significant dam failure the ability to predict the likely extent of flooding will be very helpful. A project is currently underway to develop a policy on inundation mapping. Scottish Hydro-Electric have a number of reservoirs where flows in extreme natural events can be very large in comparison to the capacity of the reservoir. The policy will therefore consider the need to assess the effects of extreme natural floods as well as dam failure scenarios. A pilot study is currently being carried out on the Glenmoriston scheme.

THE DEVELOPMENT OF A RISK ASSESSMENT STRATEGY FOR HYDROPOWER INFRASTRUCTURE

The Scottish Hydro-Electric Case

Hydro Generation is a highly profitable part of Scottish Hydro-Electric's Generation Business. The business produces an annual operating profit of around £50 M, but is required to support uneconomic rural distribution and also to invest in a major programme of plant refurbishment. It is essential for the future that the income stream from hydro generation is maintained and indeed moderately enhanced by efficiency improvements. It is also essential that the cost base reduces.

The currently established budget of £5.5M for civil works maintenance and refurbishment does not include in-house engineering costs but includes engineering investigations carried out by external consultants. The Company is committed to maintaining this expenditure in the future at the level required to keep the assets in optimum condition. Business requirements, however, mean that any significant increase in this budget will be difficult to sustain. It is therefore essential that the costs of maintenance and remedial works, engineering investigations and routine surveillance are kept in an appropriate balance and that each of them produces a benefit which can be readily related to cost.

One of the key principles of Scottish Hydro-Electric's approach to asset management is the importance of ownership of knowledge. This is considered essential to maintain continuity and use asset management as a long term approach rather than a short term problem solver. To achieve this an appropriate balance must be achieved in the use of in-house and external staff resources. In-house staff will provide expertise in operations, maintenance, refurbishment and surveillance as well as knowledge of long term performance. External resources are used for both routine technical tasks, where they will be more cost effective, and to provide high level technical expertise which it is not practical to maintain in-house. In seeking to introduce risk assessment it is therefore very desirable to adopt a system which can be implemented mainly by in-house staff with the assistance of consultants to advise and provide technical expertise.

Scottish Hydro-Electric has an excellent record for the safety of the civil engineering assets. To date no major asset failures have occurred. There have, however, been a number of more minor failures and it has been evident in recent years that incidents have been more frequent with increasing age of the assets. The operation of the Reservoirs Act has led to some re-evaluation of certain design aspects of dams. In general, however, the original design and construction has been accepted as sound to a considerable extent. To ensure that the safety record is maintained in the future it is considered essential to review the safety of the assets in a more fundamental way. This has led to risk assessment being considered as an additional asset management tool.

In considering failures which have occurred elsewhere it is observed that a number of these failures appear to have come as a complete surprise to the operators even where they had highly effective and competent engineering capability. The actual nature of the failure was not anticipated and therefore it was not possible to initiate remedial action before the event. One of the very important aims in adopting risk assessment was to reduce the likelihood of unexpected failures in the future. To achieve this it is necessary to have a relatively simple structured approach which allows potential mechanisms of failure to be identified.

Application of Failure Mode Effect and Criticality Analysis (FMECA)

Understanding the problems of a system through risk assessment, to allow risks to be managed effectively, leads also to an improvement in the quality of the business supported by the system. Initially, simple techniques allow key issues to be identified and available resources to be applied cost effectively to the reduction of risk in these areas. For civil infrastructure, the effectiveness of analysis is, in most cases, limited by the lack of immediately available data. The priority in any assessment of risk is to understand the consequences of an event, in the terms of the business to which the assessment applies. Thus there has to be a means of quantifying not only loss in monetary terms, but the significance of consequences to the confidence of third parties. The solution from a management point of view is to be prepared for the likely risks and to make provision for reducing them to acceptable levels, or for dealing with them effectively with minimum loss if they should occur.

FMECA is a logical and structured method of assessing possible failure scenarios for elements within a system and enumerating the criticality of each element to the business. Criticality is defined in this context as a function of Consequence, Probability and Detectability. Values are assigned on a simple scale of 1 to 5 to the three input parameters - consequence of failure, probability of failure, and likelihood of detection - to give an index for each element of the system (and each failure mode) in the range 1 to 125. This figure is not an actual probability but gives a relative ranking of risk. The end result is a list of criticalities which can be used to prioritise management decisions. The method is based on BS 5760: Part 5: 1991 dealing with reliability of systems, equipment and components, modified by Babtie Group to suit its application to civil works assets.

The assessment can be done qualitatively and the numbers assigned are relatively coarse. Despite this simplicity the resulting criticality index has sufficient range to identify categories of potential risk. This allows findings on high risk items to be questioned and compared with judgments based on experience. By varying some of the scale indices the analyst can test the sensitivity of the decision making or check alternative outcomes. High criticality items, once identified, can be studied in more detail and reassessed indices fed into the overall analysis and used as a tool for identifying the best actions for reducing criticality.

The essential input to the method is experience of the particular type of infrastructure, backed up by operational knowledge of the system being studied. While detailed information on the performance of the elements can be useful in assigning scale values to the three categories it is not essential. Thus FMECA can be used to support engineering analysis of risk allowing owners to prioritise their decision-making on a rational basis, despite the absence of detailed data or time for rigorous quantitative assessment.

Pilot risk assessment studies

In 1996 a pilot risk assessment study using the modified FMECA approach was carried out on the 6 MW Allt na Lairige scheme (Beak et al, 1997). This is a relatively small scheme with one dam and reservoir which incorporates all the normal civil engineering elements of a hydro scheme on an easily manageable scale. A small in-house team was set up. The team included staff with experience in operations, monitoring and maintenance and refurbishment of the various structures. Babtie Group acted as facilitators and advisers and provided training for the assessment team.

The adopted procedures involved breaking the works into Locations, Elements and Components. The Location is the overall scheme and the Elements the major assets within the scheme. Each Element is then broken down into several Components. Various failure modes are considered for each component and the relative risks assessed taking into account the three factors previously described. These are combined to give an overall Criticality of the failure mode being assessed.. This enables the components which need a more detailed assessment to be determined. Forms of action to reduce the criticality are also considered

The assessment team initially carried out a desk study to identify the scheme elements and components and gather available data. A site inspection was then made by all members of the team followed by a further desk study to complete the data sheets and prepare the assessment and report.

It is notable that the three highest risk events identified in the Allt na Lairige study all related to the pipeline rather than the dam. This confirmed a view formed from experience of pipeline failures in other countries. As a consequence a Pipeline Safety Assessment programme was commenced in 1997.

The Allt na Lairige pilot study was considered a success and a programme to assess all schemes was commenced in 1997 with Glenmoriston. This is a cascade scheme with three major dams and three power stations with a total capacity of 71 MW. The approach adopted was similar to Allt na Lairige with some improvements to the data collection forms based on experience. In 1998 the 245 MW Tummel Scheme is to be assessed together with the very small Claddoch scheme on Arran. This will involve three separate assessment teams.

Flood gates were seen as a particular risk and to provide the necessary detail a separate study was undertaken in 1997. The assessment was carried out on the same basis as that for schemes, but components were sub-divided to a greater degree. The study has been completed and in every case fairly major further investigations are required. The nature of this assessment required the inclusion in the team of engineers experienced in the operation and maintenance of large gate structures. An external expert was also employed to provide specialist advice. The result of the process has been to prioritise the refurbishment programme and to identify the areas where further engineering investigations are required. These will include NDT testing, structural analysis and review of operating procedures. The assessment has indicated that braking mechanisms and electrical controls for many gates require modifications to install more modern systems.

In the work done to date dams have formed a key part of the systems analysed. In making the criticality assessment, considerable reliance has been placed on the most recent Inspecting Engineer's report and there has been no attempt to re-inspect the dam in detail as part of the risk study. This has a number of consequences, the most important of which is the assumption that all the failure modes have been identified and realistically assessed in terms of probability and consequence. Additional effort is required to consider the detectability of any potential failure mechanism and to make wider judgements on the effect of a failure on the owner's undertaking in ways other than just safety under the Act.

RISK ASSESSMENT OF INDIVIDUAL DAMS

The Scottish Hydro-Electric Perspective

The adoption of a risk assessment approach has not changed any of the reservoir safety procedures currently in place for individual dams (Beak, 1994).

Routine visits by local staff are considered to be of critical importance. Civil Plant Attendants visit dam sites regularly. These visits will generally be at least weekly for all of the significant dams. During these visits the attendants will read water levels, and any prescribed leakage measurements and will take any pendulum or uplift measurements at the intervals specified for the dam. They will also observe the general condition of the dam and will look for any significant changes. A system is in place to send all measurements to the Reservoir Safety Administrator at regular intervals and to report any observed changes in behaviour to the Supervising Engineer.

Continuity and contact with the local operations staff are considered to be of benefit to Reservoir Supervising Engineers. All appointments are therefore made from within the Company's Civil Assets department. The Department has two Reservoir Safety Engineers. Two project engineers and one manager also act as Supervising Engineers. Experience in dam and reservoir related projects and a general familiarity with hydro operations is considered to be of benefit to Supervising Engineers. Most reservoirs with significant dams are inspected twice annually while others have one annual visit.

Ownership is again highly valued. The Supervising Engineer has responsibility for recommending the maintenance requirements of the dam and for monitoring the execution of planned work as well as monitoring dam safety and compliance with the Reservoirs Act. The long term work plan for each dam is regularly updated by the Supervising Engineer based on his inspections and assessment of the dam's requirements. Supervising Engineers are supported by the Reservoir Safety Engineers who maintain a database of reservoir safety information. This includes Reservoirs Act records and details of Supervising Engineer and Inspecting Engineer visits.

Inspections by independent qualified engineers are carried out at ten year intervals in accordance with the Reservoirs Act, 1975. Engineers are selected on merit and appropriate rates are negotiated based on time. The requirements for any additional studies and the aims of the inspection are always discussed in advance with the appointed engineer. This allows requirements to be understood clearly and assists in controlling costs.

Risk due to flooding has been very adequately addressed by the provisions of the Reservoirs Act and existing reservoir safety procedures. Recent developments have also addressed the need to consider seismic risk at least for the dam structures although knowledge of the seismic response of structures such as towers, access bridges and gates is still limited. There are, however, a range of other risks which have only been addressed to a limited extent at present. These include the failure of flood gates and bottom outlets. Such failure events would release very much smaller quantities of water than dam breach events. With the much increased recreational use of rivers, however, these aspects are now very significant

There is no doubt that the implementation of the Reservoirs Act, 1975 has led to an improvement in the way in which Hydro-Electric manages the safety of its reservoirs. The increased frequency of reservoir visits and the continuity created by the appointment of Supervising Engineers are major benefits. As practices for asset and risk management have developed, however, some possible weaknesses have become apparent.

Assessing the risk of a complex reservoir requires a very wide range of engineering skills. This may include hydraulics and hydrology, soil and rock mechanics, structural analysis, seismic engineering and mechanical and electrical engineering for gates, valves and other ancillary equipment. Very few, if any, individuals will ever have this complete range of skills. In addition the Supervising Engineer and the owners' operating staff may have both significant expertise and useful knowledge of the history and behaviour of the dam and reservoir. It is Scottish Hydro-Electric's view that for more complex dams and reservoirs Inspecting Engineers should have the back up of a range of specialists. The Inspecting Engineer should also consult widely with both the Supervising Engineer and the owner's staff in preparing his report and recommendations to ensure that he has taken advantage of all the knowledge and expertise available to him. Ultimately the responsibility for the report and recommendations rest with the Inspecting Engineer and in the event of differences of opinion his view will prevail.

The Act also appears to assume that any item of work "in the interests of safety" will be identified by a Supervising Engineer or an Inspecting Engineer. Scottish Hydro-Electric favours a fully proactive approach to the management of its assets including the management of safety and risk. Safety related work will therefore be carried out on dams and reservoirs where no defect may currently be considered to exist. Where work on a dam is considered to be "in the interests of safety" but has not been identified as a defect during an inspection, a "Qualified Engineer" is always appointed to advise on and oversee the work. A similar approach is also taken with major safety or risk related studies. Studies are currently being carried out on Seismic Hazard, Inundation and a General Risk Assessment. In all three cases an AR Panel Engineer has been appointed as an advisor. These appointments are not, however, covered by the Act and the status of any certificates which may be issued is not clear. It would be preferable if the Act made due allowance for a proactive approach from owners.

Also in need of consideration is the present limit of 25,000 m^3 on reservoir capacity that defines whether a reservoir comes under the Act. Notwithstanding the common law obligations of owners most reservoir engineers can cite examples of reservoirs below the statutory capacity limit that represent a greater potential risk to public safety than many modest reservoirs that fall within the Act. In Scottish Hydro-Electric's case the small weirs at Lochan Nam Faoileann, Grunavat, Sreinge and Lundie vary in height from only 1.37 m to 1.69m but all impound reservoirs within the scope of the Act. On the other hand the 15m high arch dam at Crarae is recorded on the ICOLD register but is not covered by the Act. Some discretion by the Inspecting Engineer to use a form of simple risk assessment would allow low risk reservoirs to be excluded from the legislation or to have less onerous surveillance regimes imposed.

In developing their approach to asset and risk management Scottish Hydro-Electric has concluded that complying with the Reservoirs Act is not enough to ensure that dams and reservoirs are adequately managed. A willingness to go beyond the minimum requirements is essential for owners of major dam stocks. As part of this approach the Company has developed a pro-active asset management approach which includes a structured programme of risk assessment.

The general need in the UK

There is now a fairly widely recognised need to address the issue of risk assessment for dams and studies are underway internationally through ICOLD and nationally through CIRIA to define the needs of those concerned with dam engineering. The background and potential influence of these studies is worthy of consideration.

The UK dam safety situation is determined largely by the age and type of dams which are predominantly embankment and commonly over 100 years old. There is also well established dam safety legislation that has operated effectively for over 60 years. The main concern of the Act is with public safety, but it is recognised that owners have wider interests in relation to asset management. Current experience shows that the principal influences on dam safety in the UK are floods and structural defects, particularly internal erosion of earth fill dams. Seismic influence, although a subject currently considered important, has not led to any failures to date.

The role of the Inspecting Engineer is central to the assessment of dam safety in the UK approach. He is appointed on the basis of his ability to assess the safety of typical UK dams using his technical knowledge and experience while referring to the available guidance on key issues and, with appropriate technical support, applying sound engineering judgement. By obtaining an informed briefing on the dam and carrying out a condition assessment on site the Inspecting Engineer can make a performance assessment, identify any weaknesses and assess the safety of the dam. This is essentially a risk assessment report.

There is perhaps a belief in some quarters that we should be taking a more rigorous approach to dam risk assessment. A report was produced in 1985 which concluded that it was practical to apply probabilistic risk assessment to UK reservoirs (Safety and Reliability Directorate, 1985). An assessment of an earth embankment dam in North West England was subsequently carried out (Parr and Cullen, 1988). This study encountered some difficulties mainly due to the lack of adequate statistical data on dam failures and incidents. Although a computerised database was subsequently developed to provide information on the performance of UK dams (Tedd et al, 1992), there has been little activity in probabilistic risk assessment since. Thus having investigated the probabilistic approach, the UK has not felt a particular urgency to develop more formal techniques.

Dam owners in the UK are generally satisfied with the Reservoirs Act, 1975 approach and most informed owners believe that it serves them well from a safety perspective. It is recognised, however, that the Act does not address the wider issues of risk to which an owner may be exposed, particularly those whose commercial activities place a heavy reliance on the continuous performance of their dams. The high cost of probabilistic risk assessment is undoubtedly a factor which has deterred owners from following this route. The lack of new dam construction in the UK in recent years has also been a factor which has inhibited the development of risk assessment.

The international perspective

The situation in other countries varies considerably. An international workshop held in June 1997 prior to the Hydropower '97 conference in Norway reviewed the current position (Hydropower and Dams, 1998). The most developed use of risk assessment is in Holland. Three other countries (Canada, USA and Australia) strongly favour rigorous methods of risk analysis although the adoption of this practice is by no means uniform in these countries. Other countries (France, Switzerland, UK) are reluctant to depart from a standards based approach which has served very well.

One of the key differences in approach between these countries is priority in allocation of dam safety resources. The rigorous risk assessment approach applies considerable resources to the study of risk. Acceptability criteria will be based on the assessed probability of loss of life from the studies against the levels of societal risk which are considered acceptable. This will often allow lesser criteria to be applied than would be the case with a standards based approach. In other countries, including the UK, there is a preference to implement works which will reduce risk rather than to use resources to justify not doing the work. In a UK context this approach is generally practical. The scale of the works and the level of risk involved is undoubtedly a major factor in determining which approach is appropriate.

A strategy for assessing dam risk

There are two main reasons for being concerned with dam risk. The first is responsibility to third parties. The second is ensuring as an operator that the performance of the dam as an asset is fully understood and maintained.

It can be argued that the current UK approach to reservoir safety is not transparent and that the use of a single inspecting engineer does not guarantee that all the relevant influences have been identified and analysed appropriately. One possible approach for an Inspecting Engineer is to demonstrate in reports that all the relevant influences on the dam have been recognised in terms of design, construction and environment. This is carried through descriptive and assessment sections to lead to clear findings under the several categories. It is important that findings can be traced back through the report, if necessary, by an informed third party. This is essentially the aim of the guidelines produced by the ICE Reservoirs Committee.

A key issue is the risk of missing potential failure modes by virtue of our concentration on floods and structural failures. This should not be a problem in the UK if we continue to appoint Panel Engineers with a sound understanding of potential failure modes (and consequences) and the experience and confidence to interpret real situations. It is important that pressure on costs is not allowed to affect this situation. It may also be appropriate to concentrate research on understanding the influences on all our dams rather than methods to rigorously analyse risk on individual dams when reliable basic data may not exist to allow this to be done.

Assessment of risk depends on the definition of acceptable levels of consequences to those affected. If the objective is to prevent loss of life as a result of reservoir operation, as in the Reservoirs Act, 1975, then there may be little merit in justifying the acceptance of some lesser standard if this is likely to lead to failure with loss of life. In the case of spillway capacity in the UK it is relatively straightforward in most cases to make provision for coping with the full PMF for a Category A dam. This simplicity of choice is a function of catchment size and climate, both of which are not extreme in international terms. It is therefore likely that in many cases it would cost more to present an acceptable case for a lesser standard than to construct the full capacity spillway. Other countries do not have this luxury on many larger dams and in some cases it is patently impractical to construct a spillway for PMF discharges.

It presently seems unlikely that we will reach the situation in the UK where we will debate or analyse the need to carry out dam improvement works as opposed to simply implementing them to alleviate the risk. When relatively simple measures can be taken to improve dam safety, there seems little benefit in spending valuable resources in justifying this action in analytical or numerical terms. The key questions are whether the event being assessed will lead to failure of the dam and whether this will in turn lead to the uncontrolled release of sufficient water to put lives at risk. The specifics of the UK situation make risk removal a valid option and often militate against detailed analysis of risk in a probabilistic way. We must ask ourselves as responsible dam engineers whether this is a supportable view and how strong might any criticism be.

Approaches to the assessment of risk in engineering structures can perhaps be described in three ways - rigorous, judgmental and engineering.

Advocates of the rigorous approach are concerned about the use of dam experts' 'opinion' and believe that this is often given with insufficient data support. Proponents believe that a fully rigorous approach to safety related mechanisms of failure is the only valid one and have difficulty in seeing that cost should be a consideration. They believe that detailed risk analysis is necessary to justify major expenditure on a dam. This seems to stem from experience that highly commercial organisations are reluctant to make major investments in remedial or improvement works purely on the basis of an 'expert' dam safety inspector's opinion.

At the other extreme, the judgmental approach may be likened to the paternalistic Victorian Engineer role where decisions were handed down with little or no explanation of the rationale. It is clear that in modern businesses it is necessary for decision making to be clear and supportable and the judgmental approach is no longer viable.

The informed engineering approach might combine the merits of the other two approaches. The assessment of risk is carried out in a structured manner supported by simple systems which do not require advanced statistics or risk assessment specialists. Where hard data is not available the considered opinion of acknowledged experts is valued and no attempt is made to quantify this unless it is supported by recorded data.

The authors have great respect for the work done by practitioners of the rigorous approach such as BC Hydro and recognise the value of their work in the context in which it is applied. They have great doubt, however, about the applicability of this approach to the very different conditions existing in the UK.

The Failure Mode Effect and Criticality Analysis approach is used in many production engineering industries. It can readily be matched to the needs of individual situations. As used and developed by Babtie Group for Scottish Hydro-Electric, FMECA offers a tool that supports the judgment of the qualified and experienced practitioner who is making the assessment of risk. It offers a structured and logical framework within which to place dependable data at an appropriate level of detail and against which decisions can be confidently made, tested and where necessary audited to provide a transparent process. One of its principal values is in providing a framework to consider whether there are any potential modes of failure which have not been taken into account. As such, it could be applied to inspection reports by basing the format of the report on the assessment of potential failure modes. Ultimately, however, it is only a tool and is useless or even dangerous if not set up by those who best understand the system under analysis.

CONCLUSIONS

1) Risk assessment is appropriate for major dam owners. Civil engineers are implementing risk assessment all the time as part of their work, although the output is not always seen as such. This fact should be recognised in responding to calls for more risk assessment in reservoir safety work. Appropriate adjustments can be made in the way in which studies are addressed and reported to bring out the reasoned risk assessment decisions being made.

2) In the UK context a rigorous probabilistic approach to reservoir safety is rarely justified.

3) Risk assessment should be seen as complementary to existing reservoir safety provisions, but goes beyond the legislative framework. The information generated should be made available to panel engineers who should be seen as partners in the overall process as well as the statutory arbitrator of legislative dam safety.

4) Risk assessment is of benefit to asset management in a business and operational context as well as reservoir safety. It is therefore appropriate for dam owners to take ownership of the process. To ensure continuity, in-house staff should be involved with appropriate support and expert advice from consultants. Scottish Hydro-Electric have found a partnership approach with their consultants to be beneficial.

5) Owners should expect risk assessment to lead to further engineering investigations and should see the process as ongoing. Increasing knowledge will require re-assessment of currently perceived risk.

6) Judgement is an essential part of assessing risk. In a perfect world engineers would have all the relevant data on a structure's performance and could analyse this to give a firm numerical value for various assessment criteria. In practice both time and reality prevent this.

REFERENCES

Beak C, Findlay J W and Aikman D I, (1997). Experience of Failure Mode, Effect and Criticality Analysis on UK hydro power schemes. Hydropower '97, Trondheim.

Beak C (1994). Implementation of the reservoirs Act, 1975, and monitoring of dams. Proc. 7th Conf. British Dam Society. Stirling, pp 287-299. Thomas Telford, London

Safety and reliability Directorate, UK Atomic Energy Authority (1985). A feasibility study into probabilistic risk assessment for reservoirs. Water Research Centre External Report No. ER 188E

Parr N M and Cullen N (1988). Risk management and reservoir maintenance. Journal of Institution of Water and Environmental Management, vol. 2, no. 6, December , pp 587-593.

Tedd P, Holton I R and Charles J A (1992). The BRE dams database. Proc. 7th Conf. British Dam Society, Stirling, pp 403-410. Thomas Telford, London.

International Journal on Hydropower and Dams (1998). Risk-based dam safety evaluations. vol. 5, Issue One, pp 89-97.

A programme of risk assessments for flood gates on hydro electric reservoirs

N M SANDILANDS, Scottish Hydro-Electric plc
M NOBLE, Scottish Hydro-Electric plc

SYNOPSIS. Scottish Hydro-Electric owns and operates 76 reservoirs covered by the UK Reservoirs Act which are impounded by 84 dams. There are a total of 28 flood gates at 14 dams. These have been recognised as a significant risk and a programme of risk assessment has been initiated. This is aimed at ensuring adequate standards of safety and prioritising the future refurbishment programme. This paper describes the risk assessment process carried out and the results. The engineering investigations and planned refurbishment on the 27.5m long drum gates at Pitlochry Dam are also described to illustrate the further work generated by the process.

INTRODUCTION

The reservoirs operated by Scottish Hydro-Electric generally operate in cascade systems with the downstream reservoirs having low storage capacity and high flows. A number of these dams are equipped with gates to pass flood flows. Gates are of various types including direct lift, radial and drum gates.

A programme of refurbishment has been progressing on the total stock of over 250 major gates and valves for some years and a number of flood gates have been refurbished (Sandilands and Seaton, 1996). As part of the development of asset management policies for civil engineering works, Hydro-Electric adopted a programme of risk assessment in 1996. This programme uses a qualitative approach and is initially being carried out at a high level to assess all elements of hydro civil works (Beak et al, 1997). Two other more detailed studies were commenced in 1997 to assess structures which have been identified as higher risk. These are the safety evaluation of pipelines and a risk assessment of flood gates.

RISK ASSESSMENT APPROACH

A decision was made in 1995 to adopt a risk assessment approach for hydro civil works. Different systems were evaluated and during this process the Babtie Group were commissioned as advisors. Babtie had developed an approach using an adapted version of the FMECA method for the assessment of civil engineering infrastructure. Following a pilot study on the

Allt na Lairige scheme in 1996 this approach was adopted for all Hydro-Electric's hydro civil works.

Flood gates had been identified for some time as being a potentially high risk area which merited further study. This is particularly the case because of the very large differences between the original design floods and the currently assessed design floods for all of the barrage type dams. During 1996 a study was made of the failure of the Folsom Dam radial gate. The site was visited and the incident and subsequent investigations were discussed with the operators (Todd, 1997). At the same time Professor Jack Lewin was appointed as an adviser on gates and valves and a brief study was carried out on a number of flood gates.

A decision was made early in 1997 to carry out a risk assessment study of all flood gates. After evaluation of possible options it was decided that the FMECA method again offered the best approach. There were two main reasons for this:

> The data on probabilities of failure of components in flood gates is not readily available and is not easily assessed with any degree of accuracy.
>
> It was recognised that modes of failure which had actually occurred were not readily identifiable prior to the failures using a probabilistic approach.

Thus the key factor was the identification of the potential failure modes by an engineer with significant experience of the operation of gates.

FAILURE MODE, EFFECT AND CRITICALITY ANALYSIS (FMECA)

The system adopted is based on BS 5760: Part 5: 1991 and is intended for use with both mechanical and electrical plant systems (British Standards Institution, 1991). The system as adopted and adapted by Scottish Hydro-Electric and Babtie Group allows for the flood gates, in all their forms, to be broken down into their component parts. The method is based on a qualitative analysis that allows the company to set targets for action and provides guidance for the Engineer on priorities where a major programme of refurbishment works needs to be carried out over a period of some years .

The key to the method is the Criticality index. This Criticality derives from the product of the probability of failure, consequences failure, and the likelihood of detection. Values are attached to each between one and five, giving a resultant between 1 and 125. Although this is a simplistic approach the resultant provides a reliable ranking of significant risk for action.

Scottish Hydro Electric uses a computerised maintenance system to co-ordinator works on its plant, option reports and log plant histories. Each reservoir, gate, type, element, component and sub-components are identified by a unique number. This number will highlight at risk items that require further investigation. In this manner it is possible to identify all gates where there are common items such as e.g. dog couplings and initiate action to remedy recurring problems.

Table 1 Data Collection Sheet

							Sheet No.	0_0
Scheme	Awe							803
Location	Barrage						**Ref.**	7
Element	Radial gate No 1						**Ref.**	53
Component	Brakes						**Ref.**	5
Component Function	Provide automatic braking and holding of gate in set position.							
Failure Mode	Binding, slipping						**Ref.**	
Cause/Trigger	Corrosion , brake wear.							
Effect	Gate will either seize in the raised position or drop shut, this will cause either prolonged water release or a surge effect if the gate drops.							
Knock-on effect likely		Y	**Parts affected**		Gate structure and deck			
Multiple Failure Likely		N						
Overall Consequences	Surge of flows in event of gate drop may affect attendants							
Preventative Actions	Maintain brake as manual.							
Contingency Plans	Manual override available.							
Time to develop	1 min		**Probability of Prior Detection of Event**					M
Inspection	12 monthly		**Probability of Detection of Failure**					H
Criticality Assessment	**Probability of Failure**	4	x **Severity of Consequence**	1	x **Likelihood of Detection**	3	= **Criticality**	12
FMECA Comment	The brakes do not at present receive the same attention as the rest of the system.							

DATA COLLECTION

The process begins with the collection of all known data on the flood gate in question. This starts with plant history, inspection reports and maintenance history. Following this a site inspection is carried out and standard inspection sheets completed. The information gathered is transferred to the data collection sheets used to arrive at a criticality rating. The information is checked for reliability by questioning both local operatives and maintenance Engineers. Continuity of information is achieved using a group of people that have carried out similar tasks on other sites and can audit the information given by local staff.

A standard data collection sheet has been developed specific to gates. This is shown in Table 1. This allows for direct comparison of plant, back up systems and maintenance criteria.

PROCESS

The information gathered during the Data Collection phase is transferred into the standard sheets developed for the gates. Each gate, element and component is allotted a number and this is used within the risk assessment process to systematically identify and assess the components within the unit. The gate is broken down to the following components in the initial stages:-

Ropes
Support Structure
Motor
Bearings
Brakes
Drives
Seals
Gearbox
Gate Structure
Starters/Wiring
Control Sequence
Float Controls
Intake Screens
Control Pipework
Control Valves
Spindles.

This list is by no means comprehensive but is aimed at identifying the principal components. Each component is then given a score rating through the use of the data collection sheets.

The values arrived at for Likelihood and Severity are taken from a matrix. This has been drawn up to reflect a relative ranking of events by Scottish Hydro-Electric. It is not considered appropriate to place a monetary value on human life and therefore this is treated as a separate item. The scoring matrix adopted for consequences of failure is shown Table 2.

This matrix is unique to the particular circumstances of Scottish Hydro-Electric and is applicable only to its current priorities. These may change in the future and the index scores will be adjusted to suit.

Table 2. Consequences of Failure Matrix.

Severity	People In Normal Location	People in Chance Location	Third Party Costs	Hydro-Electric Costs	Loss of Generation	Public Image
5	Loss of Life					
4	Major Injury	Loss of Life	> £100,000	> £500,000	> 1 Year	
3	Minor Injury	Major Injury	> £50,000	> £100,000	> 6 Months	Major Adverse Publicity
2		Minor Injury	> £10,000	> £10,000	> 1 Month	Some Adverse Publicity
1	No Injury	No Injury	< £10,000	< £10,000	< 1 Month	No Significant Adverse Publicity

The detectability matrix follows the same basic format as the consequences of failure matrix and yet again reflects Scottish Hydro-Electric's own current priorities. The detectability matrix is shown in Table 3.

The probability of failure is arrived at through both engineering judgement and experience, using the data collected. Past knowledge of the plant history accumulated over more than 40 years is used to attach an index score. The three indices so arrived at are multiplied together to give a final criticality index. This criticality is used to prioritise the refurbishment programme. The actual value assigned to the probability index is not critical to the final overall ranking. A variation of one unit has been found to have little significant effect.

RESULTS OF ASSESSMENTS

As a result of the assessment process Hydro Electric have been able to produce a planned form of action to rectify highlighted problems and investigate areas where knowledge is lacking. The actions and remedial actions that are highlighted for all levels of risk i.e. low, medium and high give an insight into the problems that can be expected not only from flood gates but also intake gates and bottom outlet gates.

It was found that there was a lack of knowledge as to the original design parameters and the effect modern design philosophy and recalculated design flood levels will have on this. The risk assessment shows that for 16 gates desk study design checks will reduce the criticality index from an average of

35 to 14, thus reducing the risk category. Where design checks have already been carried out it has been found that some older gates are of very robust construction. A number of other gates have, however, produced unsatisfactory factors of safety when checked against modern design criteria and will require remedial works.

Table 3 Detectability Matrix

Time to Develop	Inspect Frequency	Probability of Detection	Category
Develop in 6 months or more	6 Months	High	2 - 3
Develop in 6 months or more	6 Months	Medium	3 - 4
Develop in 6 months or more	6 Months	Low	4 - 5
Develop in 6 months or more	1 Month	High	1
Develop in 6 months or more	1 Month	Medium	1 - 2
Develop in 6 months or more	1 Month	Low	2 - 3
Develop in 6 months or more	1 Week	High	1
Develop in 6 months or more	1 Week	Medium	1
Develop in 6 months or more	1 Week	Low	1 - 2
Develop in 1 month or more	6 Months	Any	5
Develop in 1 month or more	1 Month	High	2 - 3
Develop in 1 month or more	1 Month	Medium	3 - 4
Develop in 1 month or more	1 Month	Low	4 - 5
Develop in 1 month or more	1 Week	High	1
Develop in 1 month or more	1 Week	Medium	1 - 2
Develop in 1 month or more	1 Week	Low	2 - 3
Develop in a week or more	1 Month	Any	5
Develop in a week or more	1 Week	High	2 - 3
Develop in a week or more	1 Week	Medium	3 - 4
Develop in a week or more	1 Week	Low	4 - 5
Develop in a week or more	Daily	High	1
Develop in a week or more	Daily	Medium	1 - 2
Develop in a week or more	Daily	Low	2 - 3
Develop in a day or more	1 Week	Any	5
Develop in a day or more	Daily	High	2 - 3
Develop in a day or more	Daily	Medium	3 - 4
Develop in a day or more	Daily	Low	4 - 5
Develop in 6 hours or more	1 Week	Any	5
Develop in 6 hours or more	Daily	High	3 - 4
Develop in 6 hours or more	Daily	Medium	4 - 5
Develop in 6 hours or more	Daily	Low	5
Develop in under 6 hours	Daily	High	4 - 5
Develop in under 6 hours	Daily	Medium or Low	5

Where seismic or aircraft collision has been assessed as a failure mode the criticality levels are generally 25. Scottish Hydro Electric can not control such events although it can mitigate the results. Seismic risk is considered to be an area requiring specialist study and a separate project has been set up to consider this. Where minor faults can be rectified relatively easily, this will be done. The risk of aircraft collision is not easily reduced. The risk arises mainly from the movements of military aircraft which are not subject to any civilian control. For the present the decision has been made to accept the current risk whilst at the same time streamlining our contingency plans.

The assessments have shown that the greatest risks come from the control mechanisms found on the gates. This is either winding gear, telemetry, ropes, limit switches or supplies. In general where the system fails safe in normal events the criticality is manageable, i.e. the gate fails to open during routine operation. Where the gate fails by spurious opening under a failure event, e.g. rope failure on an overshot gate the criticality value directs us to take remedial actions. There is a higher risk of failure in normal events than of the gates failing in a flood event. High on the priority list is the review of controls on overshot gates, such as drum gates, where failure in a normal event would lead to the gate dropping and an uncontrolled release, causing possible loss of life and property damage. Spurious opening of gates through control signalling without failure will be further investigated as a mode of failure.

In flood events access to sites in the event of a failure has been highlighted as a potential problem. Access for operatives is considered to be difficult at some sites in extreme flood events. Contingency plans need to be reviewed as an ongoing process to take account of this. This is evidenced at sites such as Rannoch Weir where in an extreme event the control tower and parapet would be isolated with control cabling etc. flooded. At a number of our Barrage sites access to the gate to rectify faults may not be possible where the expected depth of water in a PMF is often over 2m above the top walkway with a relatively short build up time.

Brake mechanisms on the winding gear were identified as high risk where reliance has been put on reduction gearboxes or electro/mechanical brakes on the drive gears. The retro-fitting of drum mounted calliper or disc braking will be assessed and acted upon where this is considered to be advantageous ie., vertical lift gates of modern manufacture with direct reduction boxes. A mechanical feasibility study has been done and shown that retro fit brakes are possible and this has been programmed for a start in 1999.

The most important result from Scottish Hydro Electric's point of view is that the risks found are manageable. A programme aimed at managing those risks is currently underway.

Table 4. Gate Refurbishment Ranking List

Location	Type	Description	Priority
Kilmorack	D54	Drum	1
Pitlochry	D54	Drum South	2
Pitlochry	D54	Drum North	3
Clunie	D54	Drum North	4
Clunie	D54	Drum South	5
Awe Barrage	R53	Radial South	6
Awe Barrage	R53	Radial North	7
Awe Barrage	R-OS59	Freshet	8
Beannachran	R53	Radial North	9
Beannachran	R53	Radial South	10
Torr Achilty	FR57	Free Roller	11
Torr Achilty	FR57	Free Roller	12
Torr Achilty	FR57	Free Roller	13
Aigas	D54	Drum	14
Aigas	R52	Radial	15
Kilmorack	R53	Radial	16
Dundreggan	R53	Radial	17
Dundreggan	R53	Radial	18
Dunalastair	FR57	Free Roller	19
Dunalastair	FR57	Free Roller	20
Rannoch	FR57	Free Roller	21
Rannoch	FR57	Free Roller	22
Rannoch	FR57	Free Roller	23
Dundreggan	T55	Tilt	24
Lairg	R-OS59	Overshot	25
Lairg	R-OS59	Overshot	26
Lairg	R-OS59	Overshot	27
Achanalt	R-US60	Undershot	28
Achanalt	R-US60	Undershot	29
Achanalt	R-US60	Undershot	30
Achanalt	R-US60	Undershot	31
Mucomir	R-US60	Undershot	32
Mucomir	R-US60	Undershot	33
Mucomir	R-US60	Undershot	34

FURTHER STUDIES

Hydro-Electric's targeted aim is to both investigate, analyse and formulate any necessary repair and refurbishment programmes. The assessment process is also designed to provide a stepping stone to further more detailed engineering investigations. The assessment to date has achieved the first part of the target in that a refurbishment ranking list is now available as shown on Table 4.

In undertaking this assessment we have tried not to lose sight of the reason for carrying out the work This is to identify and then fix plant problems. We have found that by taking ownership of the assessment the engineer is more likely to pursue the subject to a successful conclusion. This also means that the assessment becomes a live project and stays live as knowledge develops and expands in the light of plant failures and technological advances.

Risk assessment provides an auditable method of targeting expenditure. Scottish Hydro-Electric is not comfortable with the principle of placing a value on human life and then using this with probabilistic risk assessments to decide on whether to undertake safety related works. The Company would prefer to invest resources in physical works to reduce risk from dams rather than use the resources to carry out rigorous probabilistic risk analyses. The costs estimated by some practitioners of around £2 m per dam would allow for very major remedial works at any of Scottish Hydro-Electric's dams. To date, no remedial works project has ever exceeded this cost.

PLANNED REFURBISHMENT

The second stage of the assessment is under way in that various areas noted during the data collection are now being investigated more fully. This is evidenced by the extensive investigative work undertaken at Pitlochry Dam on Loch Faskally. The assessment process has identified the site as an area of concern principally due to the uncertainties. The site inspections and desk studies undertaken in 1997 were aimed at addressing the questions raised by the preliminary studies and risk assessment process. A finite element (FE) analysis was commissioned to identify areas of higher stress within the structure where more extensive non destructive testing (NDT) would be appropriate. Desk top studies also identified areas of concern in the operational equipment.

The FE analysis identified the lower floor plate rib welds as areas of relatively higher stress. This was targeted along with other areas of perceived concern by the on site NDT teams. The on site inspections were able to access the known areas of concern and routinely investigate areas picked out for attention by site inspections. This flexible approach was achieved by the careful selection of both the Contractor and his staff.

The areas targeted were the structural plates and ribs, the hinges, the seals and the control systems. It was deemed prudent to lower the level of the loch to the seal sill level, some 5m, to gain access to flooded areas. This involved a number of operational problems in water management. The control of information distribution to public and interested parties was identified as critical.

A small team of Hydro personnel representing Operations, Control, and Engineering identified the areas of concern six months in advance of the works. As in all such events sufficient flexibility is needed to cope with the unknowns. The bodies notified were ; Scottish Environmental Protection Agency, Scottish Natural Heritage, the local Press, the Tourist Board, the Police, the local Fishery Boards and local water bailiffs. A high degree of interest was shown during the works with large numbers of tourists visiting the site. The investigation was reported in both national radio coverage and the national press. The coverage dwelt mainly on the draw down of the reservoir works rather than the engineering aspects.

The results of the investigation have provided Scottish Hydro-Electric with a great deal of confidence in the structure. Areas of concern were also identified and programmed for action. This investigation process is likely to be repeated at other sites.

PLANNED REFURBISHMENTS (DRUM GATES)

Works on the 8 drum gates studied in the risk assessment process have been programmed over the next three years . The works will include general routines such as coatings, replacement of worn sealing faces, dressing of sprung sealing boxes, and replacement of seals. Works will also be carried out to secure the control systems by updating existing electrical components and introducing back up systems. Appropriate remedial works will be carried out on those drum gates known to have specific problems. This will include overhaul of control systems and the redesign of seal profiles. The drum gate refurbishments will require major investment in temporary works. This is deemed necessary to avoid draw down of the reservoirs and consequential loss of generation and amenity value over extended periods. The temporary works will enable end plates and upstream seals to be repaired and areas subject to running water to have corrosion protection applied.

FUTURE AREAS OF ASSESSMENTS

Areas of future work and investigations on other types of flood gates are identified with the benefit of plant histories, perceived design defects, operational problems, generic defects, and knowledge of failures elsewhere. Currently we are planning to concentrate upon: hinge mechanisms and structures, coupling failures, brake retro fits, control system failures and drive mechanisms.

Although the majority of our gates are believed to be robust this is recognised to be an unproven assumption and further structural assessments will be undertaken. This is reinforced by initial analysis of flood gates where areas of concern have been highlighted through finite element analysis.

Where existing hinge bearings have been stripped out we have so far found minimum wear of the critical parts. A policy of reassessment of the hinge bearings on each gate is considered essential to determine friction factors and induced stresses. The majority of overshot gates now need to be assessed to determine the need for a secondary braking system.

Implications of seismic loading have been recognised but the study of seismic response of gates is being considered under a separate project on seismic risk to dams. The effect of this factor is not yet known. It is recognised that further study in this area may have a significant effect on proposed refurbishments. Some simple measures can be taken at present. Where possible the routing of electrical wiring is considered to reduce the risk of fracture at areas vulnerable to seismic movement. Control systems were identified for particular attention with due consideration given to the effects of vibration in the design.

Future studies will concentrate on those areas identified in both the scheme assessments and the specific assessments on flood gates. Where an individual component has been identified as high risk' this will be subjected to a deeper study. It is possible that a probabilistic approach may be adopted where the cost is considered viable. The assessment process will be continued to include the major machine intake gates to provide a balanced approach to their assessment and refurbishment.

CONCLUSIONS

Once commenced risk assessment is a live process which creates it's own momentum. Initially the process raises more questions than answers and the information generated cannot be ignored It should therefore not be undertaken lightly and may not be appropriate for all owners of dams. The FMECA approach adopted by Scottish Hydro-Electric for the risk assessment of flood gates has, however, provided a number of advantages. It provides a valuable discipline for prioritising refurbishment programmes, provides an auditable framework for decision making and gives a cost effective justification for expenditure on a major work programme. The FMECA method fully meets Scottish Hydro-Electric's needs at the present time but an open approach is maintained to the possible use of more rigorous methods in the future.

ACKNOWLEDGEMENTS
The authors are grateful for the help and assistance given by Prof. J Lewin and J Findlay (Babtie Group) and to the members of the teams involved in the studies for providing advice and local knowledge.

REFERENCES

Sandilands N M and Seaton M (1996) Refurbishment of flood gates at Torr Achilty. 9th Conf. British Dam Society. York, pp 122-127. Thomas Telford, London

Beak C, Findlay J W and Aikman D I, (1997). Experience of Failure Mode, Effect and Criticality Analysis on UK hydro power schemes. Hydropower '97 Trondheim.

Todd R V (1997) Failure of spillway radial gate at Folsom dam, California. 19th ICOLD Congress, Florence, pp 113-126, ICOLD

British Standards Institution, *BS 5760 Part 5 1991*. Guide to failure modes , effects and criticality analysis (FMEA and FMECA).

Hazard and reliability of hydraulic equipment for dams

J LEWIN, Consultant, UK

SYNOPSIS The integrity of a dam installation includes the reliability of devices controlling the release of floods or the facility to empty the reservoir if a serious fault develops, which could cause a dam break. The paper reviews some failure events of spillway gates and bottom outlets, drawing attention to some systematic analyses of operating problems and defects. The objectives and methods of assessing reliability are discussed. Reliability assessments based on fault trees have been carried out mainly for barrages. Results from these investigations and from one spillway gate installation are given, and probabilistic failure criteria are suggested for spillway gate installations and bottom outlets.

INTRODUCTION

In the last decades, dam safety has been re-examined and an extensive technical literature exists on the subject. Statistics of dam failures have been collected and analysed (ICOLD, 1995). Corresponding investigations into the hazard and reliability of reservoir appurtenances are more recent. There is a greater awareness that the integrity of a dam installation includes the reliability of gates controlling flood release and the facility to empty a reservoir if a fault develops.

In an analysis of causes of embankment incidents and failures, according to USCOLD (US NRC, 1983), 2% of 240 dams experienced malfunction of gates. Since the publication of this analysis a few catastrophic events have been recorded involving spillway gates and a number which resulted in a risk.

EVENTS AT SPILLWAY GATE INSTALLATIONS

In 1967 a spillway gate on the Washi Dam in Japan collapsed suddenly (Yano, 1968). The gate was 12 m high and 9 m wide. It was swept downstream. The cause was dynamic instability induced by eccentricity of the trunnion bearings (Ishii et al, 1977 & 1979).

On the 17th July 1995 spillway gate 3 of the Folsom Dam on the American River in Sacramento County, California collapsed and released a flow of approximately 1130 m^3/s to the Lower American River (Bureau of Reclamation, 1996). The gate was 15.24 m high and 12.8 m wide. The failure occurred when the reservoir was nearly full. Corrosion on the loaded side of the steel trunnion pins had increased trunnion friction over time.

Collapse occurred when a strut brace in one of the radial arms sheared at its connection.

A spillway gate of a Swedish dam collapsed due to debris accumulation.

Also in Sweden, a serious breakdown occurred during the remote control of a sector gate (Lagerholm, 1996) due to the gate passing the upper limit switch. The bolts on the gate bearings sheared, causing the gate to break loose and to move down the spillway.

A spillway gate malfunctioned in 1992 at the Tarbela Dam Pakistan (Khan, 1994) when it became stuck during a lowering operation. It fell down, breaking two hoist ropes, damaging the gate and the weir. The gate was 28.6 m high and 15.2 m wide. Over a long period, the clearance between the side sealing plates on the piers (the seal contact plates) and the clamping bar securing the rubber seal on the gate had deteriorated. The cause of the dimensional change was not reliably established.

None of the causes of failure was structural, except for the spillway gate at the Folsom Dam, where trunnion friction was not taken into account during the design of the gate. Of the examples, only the gate at the Washi Dam in Japan failed due to hydrodynamic causes. Although only one example of a gate failure due to hydrodynamic phenomena is quoted, there is extensive literature of malfunction of gates due to vibration, cavitation at high head gates and, to a lesser extent, resonance conditions. (ICOLD, 1996).

INCIDENTS AND FAILURES OF BOTTOM OUTLETS

There are a number of research papers of hydrodynamic problems which have occurred at bottom outlets. Because high velocity flow is experienced at bottom outlets compared with spillway gates, hydrodynamic problems are more frequent.

Bottom outlets have failed to open due to silting. At the Barasona reservoir in Spain (Romeo, 1996), the silt had extended to a depth of 20 m adjacent to the dam and had completely blocked the outlet. The problem became dangerous following a major storm in 1993.

A number of bottom outlets are never, or rarely, exercised. A recent survey of reservoir appurtenances at dams in Indonesia identified a number of bottom outlets which had not been operated since impounding of the reservoirs. These are not isolated cases. Lagerholm (1996) noted similar failures to exercise bottom outlets in Sweden. Seals under high pressure are subject to contact welding after some time. Gates and bottom outlets which have not been regularly moved may be difficult or impossible to raise.

A comprehensive survey of the operation of bottom outlets at 50 large dams was carried out in Romania (Ionescu et al, 1994). While it may not be representative of experience in other countries, significant deterioration, incidents and failures were recorded. Damage had occurred at 38 gate installations. 60% of the incidents and failures were due to vibration problems, including two structural failures, which occurred after 8 and 20 years operation. Four events of intake clogging made the bottom outlets unavailable and nine vibration problems were classified as 'serious'.

CONTROL SYSTEM FAILURES

Incidents of inadvertent operation of gates under automatic control have occurred. Rajar et al (1994) record the self-induced opening of spillway gates on the Mavcice Dam in Slovenia. Two radial gates 20 m high and 13.5 m wide opened, discharging at a rate of 1192 m^3/s, equivalent to a 50-year return period flood.

Other incidents of uncontrolled gate openings have occurred but have not been recorded because they have not resulted in loss of life or damage.

A number of gate designers and reservoir operators require that automatic gate control systems are backed by hard wired electrical circuits which inhibit the time of operation of gates or the distance travelled following a command to move a gate.

COMMON CAUSE FAILURES

Common cause failures, that is failures which affect the operation of a system, are the most serious risk. They range from failure of the mains supply and the back-up system to fire, explosion, earthquake and failure of central control systems.

In electrical installations associated with spillway gates, redundancy is usually provided for transformers, mains switches and supply cables. Standby generating plant is almost invariably provided, either of the permanent or mobile type. Portable plant forms a second standby at some installations. Surprisingly, double busbars are rare in the distribution of electrical power. The failure of a single busbar can be a common cause fault, or at least affect several gates in a multi-gate installation.

The same degree of redundancy in the electrical mains and distribution equipment is rarely provided at bottom outlets.

In a critical industrial installation the electrical supply switchgear and distribution would be divided between two chambers which are not interconnected, with essential services duplicated. In the event of a fire or explosion, part of the plant and essential services would continue to function. This practice does not appear to be followed at spillway gate control stations.

The usual practice where central and automatic control is provided is to site local control systems close to gates. The latter are usually electro-mechanical controls. This satisfies one criterion of reliability of control systems, that there should be duplicate controls and that the two systems should be genetically different.

EARTHQUAKES

The risk posed to dams by earthquakes has been analysed, and some experience of seismic events on dams has been recorded. Little information is available about the effect on spillway gates and bottom outlets due to earthquakes. A quasi-static acceleration coefficient is at best a means of providing a higher factor of safety. It does not represent the dynamic forces which can be experienced by a gate and its hoisting machinery when the gate is suspended by ropes or chains.

Lateral movement of piers must be expected as a consequence of a seismic tremor. It is possible to design gates so that lateral movement of piers or abutments does not jam them by introducing collapse zones; however, this is rarely practised.

Cranes which operate turbine outlet gates are liable to be derailed due to an earthquake shock. Means to prevent crane wheels from jumping rails are established and are fitted where the risk exists. In military and naval defence applications, where electrical switchgear can be subject to severe shocks, shock absorbent mountings are installed. They do not appear to be used for electrical panels controlling gates in areas of high seismicity. An independent view is that the precautions necessary to ensure functioning of spillway gates and bottom outlet installations following a major seismic shock are mostly inadequate and that many appurtenances are at risk.

Damage and disablement of gates following an earthquake are not the only factors to be considered; blocking of access to an installation due to a landslip or damage to roads can inhibit emergency work.

FREQUENT OPERATIONAL PROBLEMS OR DEFICIENCIES

While reservoir appurtenances have a good operational record overall, there have been cases of failure, some potentially catastrophic, as well as areas where persistent operational problems occur. Specific causes of faults (Lagerholm, 1996) were:

- limit switch function
- ice problems (in Northern Europe, Canada and some states of the USA; presumably this must apply also to Eastern Europe and other parts of the world subject to severe winter weather)
- seal leakage, which in winter can cause freezing of the gates
- failure of heating systems
- trunnion bearing problems (the most frequent source of faults)
- loss of communication links

To these must be added:

- gate vibration
- cavitation at the bottom outlet of high head dams

- silting of the intake to bottom outlets
- lack of regular exercise of bottom outlet (also mentioned by Lagerholm, 1996)
- floating debris in extreme floods
- electrical cable fracture
- clogging of the intake and silting of water operated gates

Of lesser frequency are:

- control system malfunction
- uncontrolled descent due to hoist brake failure (two known cases – one reported by Lagerholm, 1996)

There are also records of problems or breakdown of vertical lift spillway gates, bottom outlet rolling gates, pinrack operated gates, and others.

RISK ASSESSMENT OF GATED STRUCTURES OF A RESERVOIR

There is sufficient evidence of failures and malfunction of reservoir appurtenances for spillway and bottom outlet gates to be included in a risk assessment of a dam. Determination of reliability must include potential liability due to design, operation and maintenance, operator training, inspection and supervision and record-keeping of incidents.

Maintenance can be deficient, variable at different stations of the same authority, or completely absent because there is no maintenance budget or authority to order spares. The latter was the case at hydropower plants in a tropical country.

METHODS OF RISK ANALYSIS

A number of detailed hazard and reliability assessments of barrages have been carried out. A barrage is constructed specifically to deal with the hazard of flooding, while the function of a reservoir may be electricity generation, water supply or flood storage and the hazard is perceived as a consequential risk. This may explain why, until recently, more detailed assessments of reliability were carried out on barrages than reservoir flood control structures.

There are a number of definitions of risk. The simplest one is "The likelihood of occurrence of adverse consequences" (McCann et al, 1985). For the purpose of quantifying risk, the definition by BC Hydro (1993) is more useful: "A measure of the probability and severity of an adverse effect to health, property, or the environment. Risk is estimated by the mathematical expectation of the consequences of an adverse event occurring (i.e the product of 'probability x consequence')."

Risk analysis must by definition include probabilistic events, although they may sometimes be implicit. Risk assessment is a combination of art, judgement and science (in that order) constrained in a formalised process (Bivins, 1984).

The most detailed methods used in risk analysis are fault trees and event trees. Fault trees allow the diagrammatic presentation of components that may lead to failure in a system element. A general failure event – the event to be analysed – is at the top of the fault tree, the remainder of which is formed by specific events which can potentially lead to the failure. Analysis of the fault tree results in determination of minimal cut sets, which are the minimal combination of events which cannot be reduced in number and whose occurrence cause the top event. Calculation of the probability of occurrence for each minimal cut set is carried out from the probabilities of the basic events.

An event tree represents all the possible sequences of events which could result from a given initiating event. Unlike a fault tree, it works from the specific to the general. It therefore traces how failure sequences propagate. Branching is limited to 'yes' or 'no' at each system response. There are similarities with operational logic diagrams.

For analysing complex systems, computer programmes have been developed for the calculation of elaborate schematic structures. The one best known in the UK is AEA Technology's programme Fault Tree Manager. A previous version 'Orchard' was used in a reliability assessment of the Thames Tidal Defences and the barrages for the flood prevention of the City of Venice. Hoyland et al (1994) discuss other programmes.

Other techniques which are used to identify different failure modes of appurtenant works consist of structured questions which help to analyse the system. Examples are failure modes and effects analysis (FMEA) and failure modes, effects and criticality analyses (FMECA). BS 5760: Part 5: 1991 describes these as methods of reliability analysis intended to identify failures which have consequences affecting the functioning of a system within the limits of a given application, thus enabling priorities for action to be set. Hazard and operability study (HAZOP) is another analytical tool which concentrates on identifying deviations from design and operating conditions. These techniques use worksheets which are filled out during analysis of a system to document a qualitative assessment.

In dam engineering, a probabilistic risk analysis (PRA) is used as a basis for making decisions when selecting among different remedial actions, and to determine priorities. It is helpful when carrying out PRA if there is a collection of statistics, but this is not essential. When assessing gates and valves, an analysis of service records is a useful guide. This is also used when assigning failure probabilities to fault and event tree branches.

Reliability assessments based on fault trees of the Thames, Barking Creek and other storm surge barriers comprising the Thames Tidal Defences were carried out (UKAEA, 1987 & Duke, 1990), and two reliability assessments of the design of the barrages for the flood defence of the City of Venice (AEA Technology, 1989 & Lewin, 1993).

The Rykswaterstaat in the Netherlands has carried out similar assessments on barriers for flood protection of the Netherlands. Some results of a risk assessment of the New Waterway storm surge gate were given in the papers by Ieperen (1994) and Janssen (1994). At spillway gate installations a fault tree reliability assessment was carried out as part of the deficiency investigations for the Seven Mile Dam in British Columbia (Klohn, 1996). This was a major undertaking – the documentation of the investigation of reliability for normal conditions is extensive, comprising three volumes.

Lagerholm (1996) mentions in his paper on 'Safety and reliability of spillway gates' that fault tree analysis has been performed in Sweden on different types of spillway gate functions. The wording suggests that these were not total system assessments.

The construction of fault and/or event trees and the production of minimal cut sets, together with the computational work required, involves considerable man hours. This type of analysis is considered necessary in special cases such as the Seven Mile Dam, where the operation and reliability of the spillway and drainage systems are crucial to the safety of the dam, or the Folsom Dam where collapse of a spillway gate has resulted in consideration of a fault tree assessment of the spillway system.

In most spillway systems there should be adequate redundancy, so that the malfunction of a gate does not result in a serious risk. If redundancy is provided the overall system reliability depends more on common cause failures, that is, on an event which affects the total installation, such as loss of power supply, a fire or an earthquake. However, redundancy of gates is rarely provided for an extreme event.

Even failure modes, effects and criticality analyses (FMECA) and hazard and operability studies (HAZOP) can involve much technical manpower. They are usually carried out by a team of engineers and technicians familiar with an installation, and can result in lengthy evaluation of specific elements of the control structures.

Where the operator of several dams requires an initial hazard assessment of a number of reservoir appurtenances of different design and age, methods of assessment based on the systematic application of engineering judgement are sometimes used. In Norway, a simplified risk analysis is being applied to dam safety. Scottish Hydro-Electric uses a similar approach to determine priorities for maintenance and improvement of spillway gate and reservoir bottom outlet structures. The analyses could be more appropriately called 'systematic application of engineering judgement'.

Fault trees and minimal cut sets are important tools to assess the reliability of a total installation and for quantifying the contribution of sub-systems and major components to the failure of the top event. They are not, as a rule, extended to include details such as limit switches, an important vulnerable element of gate hoists, local leakages of seals which can cause gate vibration and freezing up of side seals at spillway gates in winter, and others. Unless data of operational problems over an extended period of time are available, it is difficult to assign failure probabilities to these and similar elements. This does not apply to the electrical supply and distribution systems of spillway gate and bottom outlets. General and detailed statistical information is available to assign a failure probability to each element and the result will more accurately reflect the failure probability than the parallel assessment of gates and their mechanical features.

Fault tree reliability assessments are a valuable tool to determine the overall integrity of an installation in relation to the risk of the dam and reservoir. The inclusion in the risk assessment of management, operator training, operational procedures, communication, possible malicious action and failure of advance warning systems results in a comprehensive assessment. In barrages, ship collision is an important risk factor and, more remotely, an aircraft crash.

Operationally, engineering assessments are required when the main objective is to determine the adequacy of maintenance, elimination or improvement of features or elements which are vulnerable, and the reduction in the probability of failures which can put a gate out of operation. A structured assessment system based on engineering judgement is probably the best means of achieving this.

A reasonable record of experience is available of design features of gates and valve systems which are likely to result in operational problems, or are indicators of risk. Instead of structured generalised questions which are the basis of HAZOP, more specific charts should be available for carrying out reliability assessments. They could take the form of diagrams and description of design features or the condition of a component and assign a number. The sum of the numbers would be an index of priorities and individual high numbers would draw attention to areas of urgent action. It would not form a probabilistic index, but if well constructed could be part of a risk assessment.

Reservoir control appurtenances are designed for extreme events and few have been subjected to exceptional loading. However, hydraulic conditions which cause gate vibration, while not necessarily extreme events, may not occur for years after commissioning. Hydraulic conditions, combined with structural mechanical and electrical deterioration, can cause a risk and hazard because they are the coincident event of a number of probabilities. In a formal probabilistic investigation they may not show up, because it assumes that at each demand, the term used in reliability assessment for a gate movement, the structural mechanical and electrical condition is assumed to be the same and that no deterioration has occurred. To factor wear and deterioration is difficult and quantifying it depends on judgement, subject to wide latitude.

RELIABILITY INDICES

The probabilistic reliability derived from a fault tree analysis can be expressed as failure per demand. In the case of a spillway gate, this would be the opening of the gate.

For the Thames Barrier, this was 1.55×10^{-4} per gate per demand (AKAEA, 1987). Expressed differently, there is a chance that a single gate will fail to close on one on 560 closure demands, and that two of the ten gates will fail to close on one full closure in approximately 6000 closure demands.

In the hazard and reliability study of the Flood Prevention Scheme for the City of Venice, failure was defined as flooding of Venice more than 280 mm above Venice datum. The design resulted in 1 event in 800 years (Lewin, 1996).

For the New Waterway storm surge barrier in the Netherlands, the derived reliability targets were:

- Probability of not closing due to human or technical errors less than 10^{-3} on demand
- Probability of collapse less than 10^{-6} in any year
- Probability of not opening due to human or technical errors less than 10^{-4} on demand

For the Seven Mile Dam in British Columbia the reliability analysis (Klohn, 1996) resulted in:

- Probability of failure of spillway gates to open due to environmental hazards 9.68×10^{-6}
- Probability of failure of spillway gates to open due to electrical or mechanical failures 2.07×10^{-7}
- Probability of power supply unavailability to the spillway gates 2.07×10^{-7}

A good industrial system standard is one failure in 10^{-4} per demand. The reliability of a spillway gate installation depends on whether all the gates can pass the probable maximum flood (PMF) or the half PMF. The usual practice in a multi-gate spillway system is that a thousand year return period flood can be passed with one gate out of operation. A failure rate of 10^{-4} per gate per demand would appear to be an adequate assurance under these conditions. If the gates cannot pass the PMF a lower failure rate would be appropriate. This would depend on the hazard resulting from a gate failing to open under flood conditions. Some spillway gates, especially older ones, would not qualify for a failure rate of 10^{-4} per demand or a more severe criterion.

Bottom outlets consist more frequently of a single operating gate with a back-up gate or a discharge valve backed by a butterfly valve or a gate. Two or more parallel fluidways are less frequent. It is suggested that a failure rate of the order of 10^{-5} would be appropriate where only one gate with a back-up gate is provided. Whether a higher reliability is required than that of a spillway gate installation depends on the risk associated with failure of the bottom outlet.

SUMMARY & CONCLUSIONS

Failure probability rating for electrical services, both for details and systems, are available and are statistically valid. This includes standby generating plant.

For spillway gates, their hoisting machinery and control systems, failure probabilities have to be assessed from service records, known incidents or structural and mechanical plant which have some similarity. The available data will probably be of low statistic validity. The selection of a failure probability for each item of a fault tree branch will therefore involve a significant element of engineering judgement. Such judgement, whether exercised by an individual or collectively, depends on experience.

The problems encountered with bottom outlets are more frequently the interaction of structural and mechanical aspects with hydrodynamics. Assignment of failure probabilities to fault tree branches when investigating bottom outlets is therefore even more dependent on judgement and knowledge of theory and practice.

When gate vibration is taken into account, the problem is compounded. ICOLD Bulletin No. 102 gives guidance on gate vibration, although it is difficult to relate it to practical problems. Other publications (Naudascher et al, 1994, Kolkman, 1979 & 1984, Lewin, 1995) and many research papers are helpful but require knowledgeable interpretation.

A number of recent technical papers show that dam engineers are accumulating records of component and system failures of hydraulic equipment, and that these are being systematically analysed. For maximum results these should be carried out on a wide scale, and preferably on an international basis.

Some technical papers recorded failure events which resulted in serious hazards. This would not have been highlighted in a conventionally constructed fault tree and would have resulted in a low failure probability. Integrating experience and knowledge on a wide scale may identify areas where reliability and hazard resulting from a rare combination of factors would result in a different construction of a fault tree, or simply indicate the need for remedial action. A fundamental difficulty is to factor wear, deterioration and corrosion into any reliability assessment.

There is a requirement for a library of operating records and experience of elements of hydraulic equipment for dams, and for a systematic analysis and classification of the data. It would provide the reference to refine fault tree analyses. The records could be used to procure assessment sheets for structured investigations to recognise potential faults or areas of vulnerability to failure, breakdown and hydro-dynamic effects specific to gates and valves. Such assessment sheets would speed investigations as well as making them more comprehensive.

When a hazard assessment of a reservoir is required, a corresponding reliability assessment of the hydraulic equipment would form part of the overall investigation. A fault tree analyses is a detailed and effective method of carrying this out. With a relevant data bank, the value of a probabilistic failure assessment of gates and valves would be increased.

If inspection or operational records suggest that detail or system improvements are required, or that maintenance is deficient, a structured assessment based on engineering judgement is effective in establishing priorities provided it is combined with a risk evaluation. Assessment sheets based on a wider experience than is available to most engineers would shorten the work and result in more focused investigations.

REFERENCES

AEA Technology (1989). Reliability study for the Venice flood defences, Part 1: Reliability assessment of gate performance, Part 2: Hazard assessment. Safety and Reliability Directorate, SRS/ASG/31466, Nov.

B.C. Hydro (1993). Guidelines for consequence-based dam safety evaluations and improvements (Interim). B.C. Hydro, Burnaby, B.C., Canada.

Bivins W S (1984). Risk analysis in dam safety programmes. Water for Resource Development – Proc. of the Conference, Coeur d'Alene, Idaho, Aug, ed David L. Shreiber, ASCE, N.Y., pp115-119.

British Standards Institution (1991). BS 5760: Part 5. Guide to failure modes, effects and criticality analysis (FMEA and FMECA).

Bureau of Reclamation (1996). Forensic report of spillway gate 3 failure, Folsom Dam. Bureau of Reclamation, Mid-Pacific Regional Office, Sacramento, Cal, USA Nov.

Duke A J & Hounslow J (1990). A practical application of reliability analysis to a working installation – the Thames Barrier. I.Mech.E., Seminar Risk Analysis, Nov, pp13-33.

Hoyland A & Rausand M (1994). System reliability theory, models and statistical methods. John Wiley & Sons, N.Y.

ICOLD (1995). Dam failures: a statistical analysis. Bulletin 99, International Commission on Large Dams, Paris.

ICOLD (1996). Vibrations of hydraulic equipment for dams; review and recommendations. ICOLD Bulletin 102.

Ieperen A van (1984). Design of the new waterway storm surge barrier in the Netherlands. Hydropower and Dams, May, pp66-72.

Ionescu S et al (1994). Damage and remedial work during operation of several bottom outlets. ICOLD, 16th Congress, Durban, Q71, R7, pp79-90.

Ishii N et al (1977). Instability of elastically suspended tainter-gate system caused by surface waves on the reservoir of a dam. Am. Soc. Mech, Eng., Fluids Eng. Division, Joint Applied Mechanics, Fluids Engineering and Bioengineering Conference, New Haven, Conn, Jun., Paper No. 77-FE-25.

Ishii N et al (1979). Dynamic instability of tainter gates. 19th Congress Int. Ass. of Hydraulic Research, Karlsruhe, Paper C9.

Janssen J P F M et al (1994). The design and construction of the new waterway storm surge barrier in the Netherlands (technical and constructual implications). ICOLD, 16th Congress, Durban, C15, pp877-900.

Khan K A & Siddique N A (1994). Malfunction of a spillway gate at Tarbela after 17 years of normal operation. ICOLD, 16th Congress, Durban, Q7l, R27, pp411-428.

Klohn-Crippen Integ & Northwest Hydraulic Consultants (1996). Seven Mile Dam deficiency investigations; spillway and drainage systems, reliability for normal conditions for BC Hydro. Task C8 – Part 1A, vol 1; vol 2 – Apendices A-L, vol 3 – Appendices M-0, Sep.

Kolkman P A (1984). Development of vibration free gate design. Delft Hyraulics Laboratory, Publ. 219.

Kolkman P.A (1984). Vibration of hydraulic structures. In Developments in Hydraulic Engineering – 2, Editor Novak P, Elsevier Applied Science Publishers.

Largerholm S (1996). Safety and reliability of spillway gates. ICOLD Symposium, Repair and Upgrading of Dams, Stockholm, June.

Lewin J (1993). System reliability assessment of the definitive design of the Venice flood defences. (Not published).

Lewin J (1995). Hydraulic gates and valves in free surface flow and submerged outlets. Thomas Telford, London, Chapter 10.

Lewin J (1996). Mechanical aspects of water control structures. Dugald Clerk Lecture 1995, Proc. Instn. Civ. Engrs Wat., Marit and Energy, 118, Mar, pp29-38.

McCann M W et al (1985). Preliminary safety evaluation of existing dams, volume 1. Dept of Civil Eng., Stanford University, Vol 1, Report No.69.

Naudascher E & Rockwell D (1994). Flow-induced vibrations – an engineering guide. A.A. Balkema, Rotterdam.

Rajar R & Rryzanowski A (1994). Self-induced opening of spillway gates on the Mavcice Dam – Slovenia. ICOLD, 16th Congress, Durban Q71, R8, pp97-112.

Romeo R (1996). Drawdown of the Barasona Reservoir. Report by the author at the Symposium Repair and Upgrading of Dams, Stockholm, June. Reported in Hydropower and Dams, Issue Five, 1996, p74.

UKAEA (1987). A reliability assessment of the Thames tidal defences. safety and reliability directorate. SRS/ASG/31447. (Not published).

USNCR (1983). Safety of existing dams: evaluation and improvement. U.S. National Research Council, National Academy Press, Washington DC, USA.

Yano K (1968). On the event of the gate destruction of the Wachi-Dam. Disaster Prevention Research Institute, Annals of Kyoto University in Japan, 11-B 1-17.

Should reservoir control systems and structures be designed to withstand the dynamic effects of earthquakes?

G. M. BALLARD, Consultant, UK
J. LEWIN, Consultant, UK

SYNOPSIS. Seismic analysis of all important systems is now an integral part of the safety requirements for major hazard installations in the U.K. This paper considers the value of extending this principle to reservoirs, particularly with regard to control systems and structures. A possible framework for a seismic analysis is outlined and an example given of the type of analysis that may be performed for spillway gates.

INTRODUCTION

The performance and safety of dams during earthquakes world-wide has been remarkably good (Charles et al, 1991). Nevertheless the failure of a dam can have such serious consequences that earthquake safety evaluation of existing dams and of new constructions is a general requirement. Following an earthquake the release of reservoir water can be a critical control function if the dam has been damaged by the seismic motion. Therefore spillway gate installations and bottom outlets need to remain operational after an earthquake and should be included in the seismic analysis. As a New Zealand engineer expressed it "When the big one hits, the likely scenario is that massive power load will be dropped and spilling will quickly be necessary to prevent dam overtopping and serious damage to generating facilities". (Williams, 1996)

This paper discusses the requirement for a seismic analysis which focuses not just on the civil structures but on the control systems that may be required to act to protect the dam after an earthquake or mitigate the consequences of delayed dam failure. An overall methodology is outlined which seeks to identify potential risk issues and then prioritise these issues by detailed analysis of the fault sequences and the potential accident scenarios.

The paper takes an example of spillway gate failure and illustrates the type of analysis that can be used, in a cost-effective manner, to identify concerns and then address these in the installation design.

BACKGROUND

A review of incidents at dams that have been exposed to seismic events (Hinks & Gosschalk, 1993) shows that, while dam performance is on the whole fairly good, there are a number of events in which dams have suffered significant structural damage. In some of these cases the dam has subsequently failed entirely, although such failure has rarely occurred at the time of the earthquake; most failures have occurred either after a few hours or up to 24 hours after the earthquake.

Given this history it is clear that directly after a dam has experienced a significant seismic event there is likely to be an urgent requirement for the water level in the reservoir to be lowered quickly both to reduce the pressure on the potentially damaged and weakened structure and to alleviate the consequences should the dam fail at a later time. Spillway gates and bottom outlets will be used for this purpose, with the spillway gates providing the greater initial capacity for level reduction.

There appears to be little published data on the damage suffered by spillway gates during earthquakes, although there does seem to be some awareness that the effect of earthquakes on gates requires investigation and that strengthening work may be necessary at some installations. It is the impression of the authors that the specification of new spillway gate installations often does require that the plant is designed to withstand an earthquake acceleration, specified at the base of the dam, but that the interpretation of this requirement into the actual implementation is often not carried out in a systematic and comprehensive manner. Since the spillway gates are located at the crest of the dam, the shock can be amplified considerably from that at the dam base and the spillway gates will be subject to this amplified acceleration. Hinks & Gosschalk (1993) quote examples of the amplification of the earthquake acceleration at two rockfill dams in Mexico which had accelerographs on the dam crest and on the rock near to the dam. The acceleration on the crest of the La Villita dam was between 9 and 22 times as great as that measured on rock at the right abutment, depending on whether the transverse, longitudinal or vertical acceleration is considered. At the El Infiernillo dam, which is 2.5 times the height of La Villita, the amplification was significantly less at between 2.7 and 4.8 times. The difference in the amplification factor measured is probably artificial due to the different sub strata of the dams; La Villita has deep alluvial deposits in the valley beneath the dam and so the acceleration at the dam base is likely to have been considerably greater than that measured at the rock abutment. The total amplification measured will thus have probably been the combination of two factors, the transition from rock to alluvial deposits and secondly the height of the dam.

An example of the strengthening that may be required to cope with these enhanced acceleration levels can be seen by reference to three dams in New

Zealand. Here the spillways have vertical lift gates operated by winches on the top of slabs of reinforced concrete structural frames above the spillway deck level. The winch support frames were identified as being at structural risk during an earthquake (Williams, 1996). The problem is similar to that of freestanding outlet towers. The operator of the dams is now initiating checking of the effects induced by earthquakes on mechanical and electro-mechanical structures such as gates, penstocks, headgate servo-motors, transformers, high voltage switchgear and control panels. Scottish Hydro is starting similar investigations.

Seismically induced vibration may not be the only way in which control structures may be damaged during an earthquake. It is possible for seismic activity to set up seiches in a reservoir. Hinks & Gosschalk (1993) give the example of the 35 metre high Hebgen embankment dam in Montana following the earthquake on 17 August 1959 (Sherard et al, 1963). The reservoir was full at the time of the earthquake. The waves created in the reservoir overtopped the dam by 1 metre uniformly over the crest. This overtopping was repeated four times and in each case lasted for about ten minutes. The effects of such an event could cause serious overloading of spillway gates and may result in the collapse of the arms of radial gates. Serious overtopping of spillway gates may also occur when the reservoir is suddenly affected by a landslide. The triggering of landslides by earthquakes is a common occurrence in hilly areas (Skipp, 1980), and may cause destructive waves in addition to the increase in reservoir water level.

Other industries in which an accident could cause a potentially serious threat to human life have been led to a detailed analysis of seismic events. The nuclear industry has had a legislative requirement for a full Probabilistic Safety Analysis (PSA) for a number of years and a structured seismic analysis has been a mandatory part of such analysis. Much methodology and data has evolved, particularly in the U.S.A., as a result of this requirement and this may be of use when assessing reservoir safety. Similarly the chemical industry, which has had to produce Safety Cases as part of the Control of Major Accident Hazards (CIMAH) legislation, is now increasingly being required to include seismic events within those Safety Cases.

OVERALL FRAMEWORK FOR SEISMIC ANALYSIS

Based on the experience from other industry an overall framework for a seismic analysis can be outlined. The major elements are fairly straightforward:-

Seismic Hazard

Determination of the expected frequency at which a particular site might experience an earthquake of a given "size". This involves not just the tracing of the seismic history of particular areas of the country but also the determination of the propagation of ground motions to the specific site given an earthquake with its centre some distance away from the dam.

System Response

The analysis of the response of various dam components to a seismic motion of a given size occurring in the rock adjacent to the dam. This "fragility analysis" involves not just the likelihood of systems being degraded or failing in response to a particular level of input motion but also the extent to which failures of components or systems may be correlated in time.

Systems Analysis

A systems analysis must determine the effect of the failed components or systems on the ability of the dam to withstand the earthquake without major consequence. This involves both the determination of the event scenarios that may occur as a result of damaged or failed systems and the effect on individual systems of specific combinations of component and/or human failures.

Taking each of these areas in turn some additional comments can be made concerning the availability of methods and data to carry out the analyses and the level of uncertainty in the results generated.

There are now a considerable number of databases available which provide the seismic history of various areas of the world, and these can be used to produce an initial frequency curve for earthquakes of varying sizes (see for example Musson & Winter (1994)). However the simple use of historic data is often inadequate. The seismic events, which may be important as risk contributors for a specific dam, have to be significantly larger than the design basis event. Such large earthquakes tend to have return periods that are longer than the typical scientific record so that there is considerable uncertainty in the estimation of the return period.

Another uncertainty arises from the way in which the ground motion is parameterised in the analysis. Typically, a single parameter, such as "peak ground acceleration" is used to characterise earthquake size, even though earthquakes have a large number of parameters such as frequency-dependent acceleration, velocity, displacement, time-dependence of the motion over many seconds of duration, energy dispersion, and so on. Although the historical records can sometimes assist in characterising earthquakes for a given site, these records are limited and the few large earthquakes that have been observed seldom allow the analyst to describe the input motion in detail.

The analysis also requires a description of how the earthquake motion propagates through the soil or rock structure to the dam foundations. There are reasonably good parametric methods for this aspect of the work that can avoid unnecessary expenditure on expensive detailed analysis. For example the Guide to Seismic Risk to Dams in the UK suggests that it may sometimes be appropriate to assume that the vertical acceleration in a seismic motion is 0.67 times the peak horizontal acceleration.

The vulnerability of individual dam components, structures or systems will be determined not just by their sensitivity to the vibration imposed on the dam but also by the potential for amplification of that imposed vibration. The conventional quasi static analysis is unlikely to be an adequate approach in many cases and the reality that it is a dynamic phenomenon must be explicitly recognised. To date it appears that, while dynamic analysis may be applied to the civil structures, it is largely ignored when designing all the ancillary equipment and machinery.

Even if the imposed loads on the dam components and systems are correctly assessed there remains the problem of determining the likelihood of failure under these imposed loads. There is a distinct lack of data on the behaviour of components under earthquake conditions that can be used as a basis for assessing their failure probability. Design calculations can be a useful starting point but they are of limited use in determining the real performance of components. This was also the situation in the nuclear industry during the early days of seismic analysis. Initially the only option was to adopt very conservative assumptions based on an extrapolation of design calculations, but gradually the experience database was strengthened. Even then that database was generally weak for ground acceleration above about 0.5g and a programme of component testing was necessary to make any further progress. Current practice comprises a mixture of analysis for larger components and seismic testing on a tri-axial vibrating table for others. The resulting data may provide a useful starting point for the assessment of dams since many of the components involved carry out similar functions e.g. cable trays, control equipment, electrical relays etc.

Following the methods of the nuclear industry a typical approach to component fragility analysis would be to assess the margin of safety over the design basis for all components of interest. There will be variability in this margin for two main reasons, first because of the distribution in the available data for nominally the same input loading and second because of the distribution in component performance due to the differences in seismic characteristics for the same nominal peak ground acceleration. The overall fragility curve for the component is then represented graphically as in fig. 1 below.

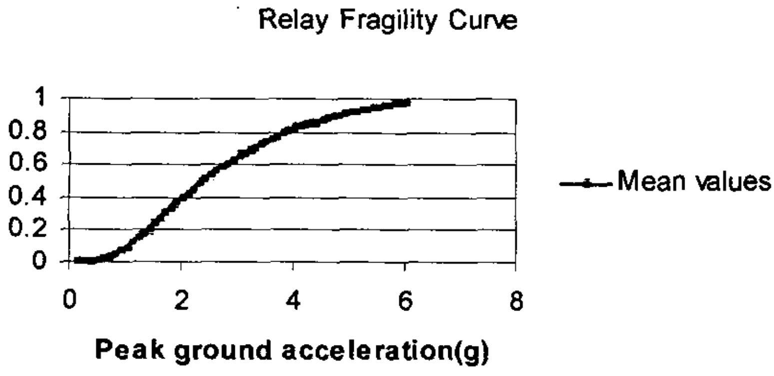


Fig. 1. Cumulative failure probability for relay chatter

The curve shown is really the composite of a family of curves representing the uncertainties noted above. The range of uncertainty is likely to be considerable but this can be reduced where there is better data on a specific component. However the variability due to the simple parametric representation of the earthquake motion will still remain.

The systems analysis is the most well developed but equally the most potentially valuable part of the overall framework. It is fundamentally a structured logical assessment of the events that may lead to serious consequences and of the component, systems and human failures that may contribute to such accidents. The starting point is a set of event trees representing the potential progression of accident scenarios stating with initiating events that were identified either by Hazop or some other structured consideration. These event trees specify the response required from the various control systems and in turn provide the starting point for a set of fault trees which examine how these control functions may fail. The fault trees focus on identifying the combinations of component and/or human failures that will lead to the system failures specified in the event trees.

The great strength of the fault tree method is that it demands that the analyst explores all means by which the system failure, the "top event", may occur and that failure modes are only discarded when there is a cogent argument which is supported by operational experience. Quantification of the fault tree then provides a method of prioritisation of failure modes; frequently the analysis gives a rather different perspective on the relative importance of issues than current thinking might suggest.

For the current analysis the fault tree "cut sets" that contain components susceptible to seismic effects are of greatest interest. Since there are usually multiple components for performing any particular system function i.e. multiple spillway gates, backup power systems, redundant protection systems etc, the events with the potential to cause multiple concurrent failures are particularly important. These so called "common cause failures" are often the least obvious but, from a reliability point of view, the dominant issues. Thus failure of say, limit switches, during a seismic motion are unlikely to be entirely random; if one switch has failed it is likely that a number of other similar switches will also have failed and any redundant arrangement of switches, designed to provide greater reliability, will be short-circuited.

EXAMPLE ANALYSIS

The practical issues of concern are best illustrated by an example of a typical analysis. In the interests of brevity the example will omit the initial steps in the analysis and concentrate on the systems analysis. Since there are obviously many different detailed designs of dam control systems the analysis will necessarily be somewhat general but it can still serve to illustrate the value of the methodology proposed. It is typical of this kind of analysis that it throws up many issues and concerns which can not immediately be answered satisfactorily; further investigation will often be required until the information is sufficiently robust to provide a sustainable argument for the conclusion used. There is a danger of using the immediate "gut reaction" which does not stand up to scrutiny; only objective evidence is good enough if issues are to be discarded from further consideration.

Event Trees

Clearly the range of potential accident scenarios is infinite if the exact details of the event are considered. However for the purposes of analysis scenarios have to be grouped into a number of manageable categories that are driven by the important variables. Any such categorisation inevitably involves some approximation, and this should generally err on the side of safety.

Fig. 2 shows a typical event tree that may be constructed for a seismic analysis for a rockfill or earth embankment dam; other constructions may have rather different characteristics. In practice, of course, there will need to be a family of such trees starting with seismic events of different magnitude. For the present this one tree provides a sufficient example.

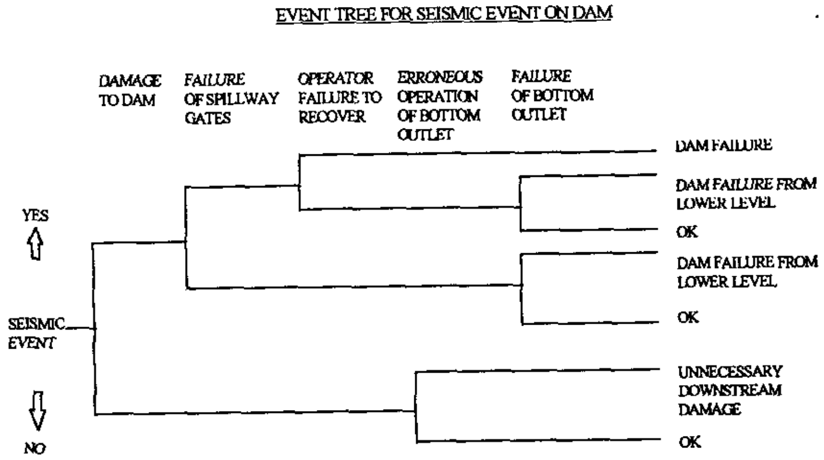


Fig 2. Typical Event Tree

The following notes apply to the specific headings on the event tree.

1. Damage to Dam During a seismic event in a particular size range the dam will have a probability of being damaged to varying degrees. Since the demand on the control systems will depend on the damage to the dam, or perhaps the extent to which this is obvious immediately after the earthquake, a family of event trees for different categories of dam damage may be used for a given earthquake. The analysis will need to reflect the operating procedures determined by the dam owners.
2. Failure of Spillway Gates Depending on the extent of dam damage there will be a requirement for water drawdown over a specific period of time. For the present example it is assumed that this requires the operation of all the spillway capacity, initiated within one hour after the earthquake, followed by continuing operation of the bottom outlet. The full specification for the top event for the relevant fault tree will therefore be "Failure of the system as designed to initiate full spillway flow within one hour of earthquake". In practice this may always be the requirement for the more severe categories of seismic events because of the difficulty of determining the real extent of dam damage. However the potential for downstream damage due to operating the spillways to their full extent will be an important factor in deciding the action recommended in the operating instructions.
3. Operator failure to recover While the spillway gates may fail to respond in the manner intended because of control or other failures the operators may be able to recover the situation in time by various planned or ad-hoc actions. The extent to which this is possible will depend on the time available but also on other factors such as whether the dam is normally manned, whether the operators are practised in fault finding and

recovery, whether advice is available on a communication link which is still operating, etc.

4. Unintended operation of the bottom outlet Either operator action or equipment malfunction may lead to spillway gate or bottom outlet opening when it is not required. The potential for downstream damage as a result of such opening makes these events which need to be considered in the analysis. For the purposes of the present example it will be assumed that the evidence suggests that operators will follow clear operation procedures and will not initiate either spillway gates or bottom outlets unnecessarily. In the case of equipment malfunction it will be similarly assumed that the operators would quickly recognise any unintended operation of the spillway gates because these are immediately visible to them. However it is less clear that they would either see and recognise control indication of bottom outlet initiation or that the outfall of the bottom outlet is visible to them. This event was therefore retained within the event tree.
5. Failure of bottom outlet Dependent on the extent of dam damage, the bottom outlet may be required to open to provide continued lowering of the reservoir water level. Failure to open could lead to dam failure despite successful operation of the spillway gates, although the lowering of water levels resulting from that operation may serve to partially mitigate the consequences of any subsequent dam failure.
6. Event sequence consequences The analysis is driven, and limited, by consideration of the events that concern the dam owners. This example is limited to those events with the potential to kill members of the public. In practice the owner may have an interest in a wider range of consequences such as damage to generating capacity etc. Against this interest has to be balanced the greater complexity that would be required in the event trees and the fault trees.

Fault Trees

Fault trees will need to be developed for each of the decision points identified in the event trees outlined above. It is important that the top event of the fault tree is well specified to avoid confusion in the tree development. For the present purpose a single tree, shown in Figs. 3-6, will be developed for the event "Failure of the spillway gates to initiate as designed within one hour"; for conciseness on the fault trees more abbreviated titles may be used but the fuller specification needs to be borne in mind.

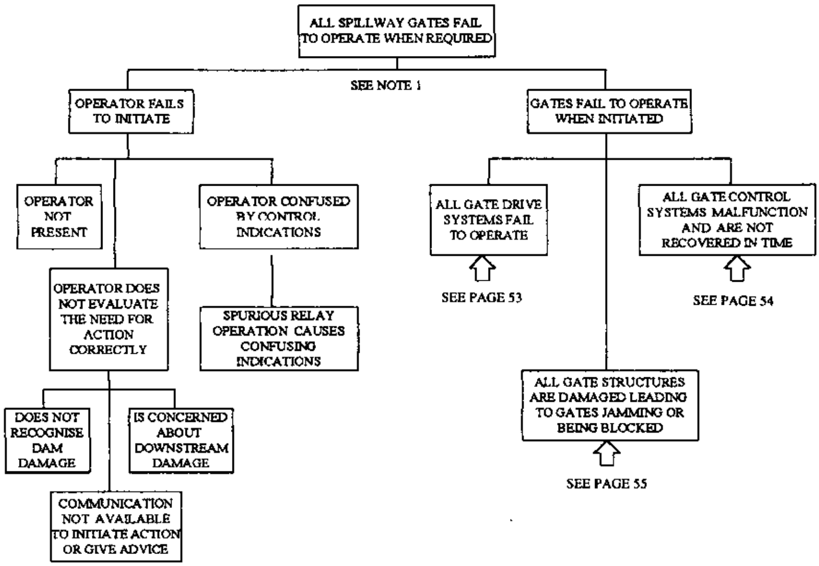


Fig. 3. Page 1 of the Fault Tree

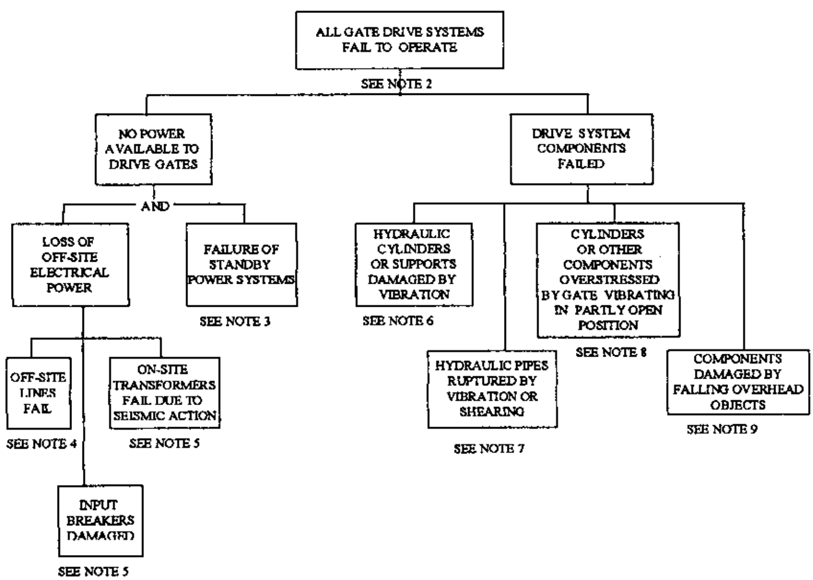


Fig. 4. Page 2 of the Fault Tree

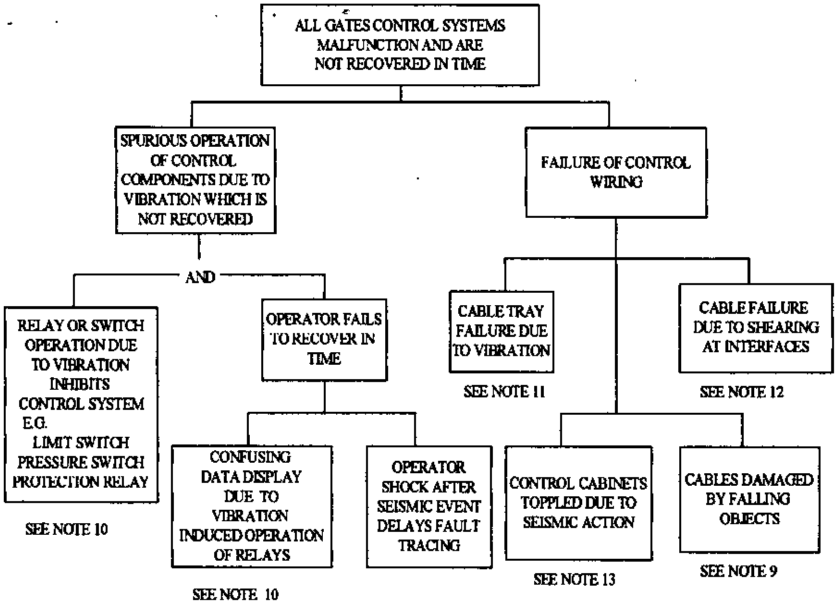


Fig. 5. Page 3 of the Fault Tree

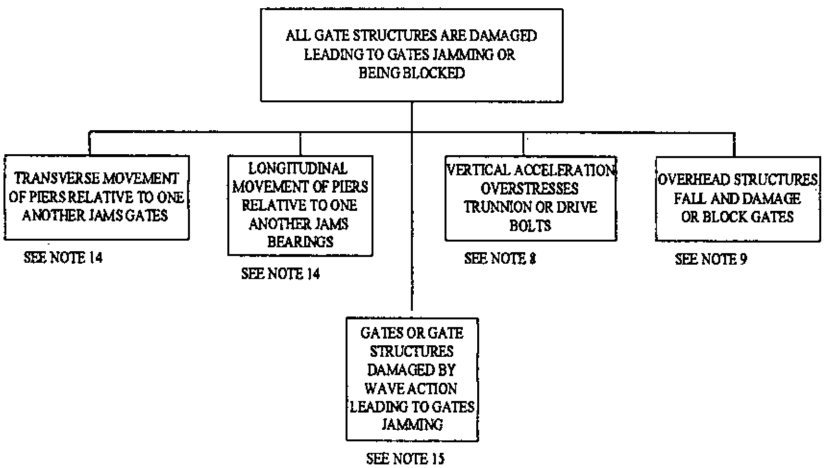


Fig. 6. Page 4 of the Fault Tree

The following notes illustrate some of the trains of thought that occur as the tree is developed and raise some of the issues that require resolution before the analysis of the tree can be completed.

1. Since the interest is in complete failure of the spillway capacity the analysis is likely to be dominated by events with the potential to affect all of a set of multiple gates. Random independent failure of multiple components is clearly possible but its probability of occurrence is likely to be of a much lower order than common cause events. For the present example multiple independent failures have been discounted in the interests of brevity and clarity; in any real analysis they would of course have to be included. Indeed the potential for equipment to be out of action because of maintenance is frequently an important contributor to the failure of intended redundancy but that is not the focus of this paper.
2. There are various designs of spillway gates and the analysis would become too confusing if an attempt were made to include all types. For the purposes of illustration we have therefore chosen radial gates, driven by hydraulic cylinders. Obviously a different drive system or design of gate will throw up different problems for consideration; rope supported gates for example may be particularly vulnerable to seismic motion in a part open state.
3. The gate drive mechanism will require a mains electrical supply for normal operation. There will however be a standby system for actuation of the normal drive, either by providing an auxiliary power supply or by allowing manual operation, or both. Failure of the drive system itself obviously leads to failure even if the standby system functions correctly but this is handled by the logic of the fault tree. Whether the manual system has the capability to open the gates within a reasonable timescale requires to be investigated before any credit could be taken for this arrangement. Similarly the typical arrangement of many diesel auxiliary power supplies appears to make them vulnerable to vibration induced failure; features such as high level fuel tanks and rigid piping to the diesel engine can be a source of failure.
4. In most significant earthquakes it must be expected that the off-site power supply will fail because of the vulnerability of overhead lines. Because it is assumed that the turbines will be tripped by the earthquake the normal on-site electrical power supply will not be available.
5. A variety of components may be damaged if they are insecurely anchored and are overturned by seismic vibration.
6. Long hydraulic cylinders may be vulnerable to low frequency vibration that can damage the hydraulic seals. (This comment may also be relevant in the fault tree for the bottom outlets where servo-motors are very long and mounted on the bonnet of slide gates).
7. Hydraulic pipework to the drive cylinders may also be vulnerable to vibration, particularly if there are poorly supported long pipe runs. (See comment above for the bottom outlet case).
8. Seismic vertical acceleration may be accentuated by the gates being in a partly open position; out of phase motion could result in large relative accelerations between the gate and the trunnions mounted on the piers. In these circumstances the hydraulic cylinders could act as dampers to

alleviate the situation. Alternatively they could be effectively stiff resulting in overpressurisation of the hydraulic system. The pressure transient may be too short for the pressure relief system to react adequately. (If the gate is closed, the gate cill may prevent overloading of the cylinders or gate supports)

9. Overhead structures are a feature of vertical lift gates but there may be a gantry crane, or overhead service gantries. Consideration should be given to whether failure of the overhead structure could result in failure of several gates; is there an effect directly on several gates or is there some vulnerable component, cable or pipework common to all the gates?
10. In a typical control installation there are likely to be tens, or perhaps hundreds, of relays and switches. These are potentially vulnerable to spurious intermittent opening or closing due to seismic vibration. In many cases relay chatter will cause no net effect because the relays will return to their original state when vibration ceases. In other cases, the making or opening of the relay or switch may set in the new condition and this will not be reset when the relay reverts to its original state. This can be true of protection system where a manual reset is often considered a reasonable precaution. Limit switches may be vulnerable to spurious operation, with overtravel switches having the effect of preventing operation of the gates. The full range of spurious effects generated by relay chatter or switch operation will depend on the exact details of the control system design but the potential exists for confusion or even failure of the controls.
11. Cable trays may often be poorly supported and contain cabling for multiple systems. Vibration induced failure is possible and may have wide ranging effects on apparently independent systems.
12. Wiring failures can be serious where there is little segregation of cable runs for different systems and where cable runs cross joints in the concrete structures. Overhead cable runs can be vulnerable to gantry collapse.
13. Cubicles housing control relays and other control equipment can topple if they are not adequately anchored and such a failure can have a wide ranging effect on multiple systems.
14. It is assumed that the concrete structure of the spillway gates is formed by independent piers, flexibly jointed to the sluiceway floor, allowing relative movement between piers, both transversely and longitudinally. An alternative design in which the piers and the sluiceway are monolithic may prevent jamming of a gate.
15. Seismically induced seiches of considerable size have been observed and have the potential to damage gate structures and/or services mounted on the piers. Radial gate arm collapse due to the loading caused by water overtopping the gate is possible.

When the fault trees have been fully developed they may raise a number of concerns which will be reflected in design changes. Using data on the fragility of the components the fault tree top events can be quantified; this provides a relative order of importance of the identified failure modes as well as an estimate of the absolute failure probability. Probably the most useful outcome, however, is the insight gained into the potential vulnerabilities of the system. Even a very approximate quantification can support concerns which are raised by good engineering judgement and experience.

CONCLUSION

The paper has argued that reservoir control systems and structures should be subjected to a systematic seismic analysis because of the potential hazard arising from their failure. An example is given of an outline framework for a seismic analysis and some of the concerns that might arise in the design of spillway gates are raised. The value of this analysis lies in its ability to ensure that potentially important issues are raised at the design stage of a project and that a clear and auditable case is made for resolving each of the concerns. By the use of quantification the issues can be prioritised and arguments based on risk levels can be made.

ACKNOWLEDGEMENTS

The authors are grateful to Mr. Guiseppe Grilli of the Electricity Corporation of New Zealand for information of the effect of an earthquake on a New Zealand dam and on measures to improve reservoir control structures to withstand earthquakes.

REFERENCES

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. Report, Building Research Establishment

Hinks J L & Gosschalk E M (1993). Dams and earthquakes-a review. Dam Engineering, Vol IV, Issue 1, February.

Musson R M W & Winter P W (1994). Seismic Hazard of the UK. Report GNSR(DTI)/P(94)196

Sherard J L, Woodward R J, Gizienski S F & Clevenger W A (1963). Earth and earth-rock dams. John Wiley & Sons

Skipp B O (1980). Earthquake vulnerability of superficial materials-landsliding of natural and manmade slopes. Lecture notes, Course on assessment of earthquakes risk, Geological Society of London.

Williams I S (1996). After the earthquake-ensuring that the spillway can be operated when it really counts. 7th Hydro Power Engineering Exchange, Hamilton, New Zealand, October.

The review of the Reservoirs Act 1975

G P SIMS, Brown & Root
N M PARR, North West Water

SYNOPSIS. This paper describes a review of the Reservoirs Act being carried out by the Reservoirs Committee of the Institution of Civil Engineers. The purpose of the review which has been under way for two years is to identify those aspects of the legislation and current practice which may need to be revised updated or clarified. The review considers the ambit of the Act and the criteria by which a reservoir is to be included within the legislation. The review will also ensure that the Institution is aware of the potential impact of European legislation and other international developments.

INTRODUCTION

In 1996, the Reservoirs Committee of the Institution of Civil Engineers set up a Working Party to review the operation of the Reservoirs Act 1975. Chaired initially by Mr N M Parr, it included members of the Reservoirs Committee and a representative of the British Dam Society. For two years it has been reporting routinely to the Reservoirs Committee. An important element in the approach of the Working Party is to consult with interested professionals and much information was received as a result of a questionnaire circulated in 1997. The Reservoirs Committee has encouraged the preparation of this paper to communicate the Working Party's activities to engineers and managers with a professional interest in reservoir legislation.

TERMS OF REFERENCE

The Reservoirs Committee required the Working Party to undertake a systematic review of the Act, to consult widely to identify and to address the main issues and to consider the applicability of each individual clause. The Working Party was also required to consider the Draft Deregulation Order under discussion by the Department of the Environment, Transport and the Regions, and to prepare a schedule of future actions.

REVIEW OF THE ACT

The questionnaire circulated in 1997 was intended to encourage respondents to identify those sections of the Act where revision or clarification is desirable. Views were sought from those who are affected by the practical application of the legislation including: Panel Engineers, BDS, Environment Agency, Country Landowners Association, NFU, BWB, Water Companies Association, Water Services Association and local authorities. The Working

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

Party has collected all the published observations on the working of the Act since its inception in 1985. The Working Party received nearly fifty replies to the questionnaire. These were summarised by the ICE secretariat in a report to the Reservoirs Committee.

The responses to the questionnaire and the literature search together provide a comprehensive account of professional opinion and identify the issues of concern to practitioners. The consensus was that, despite some shortcomings, the system of reservoir safety under the Act is working well. Many saw a need for a clear statement of the objectives of the reservoir safety legislation. Difficulties with the detailed application of individual clauses, including well documented difficulties with definitions and dimensions, and the need for a clearer definition of the roles of the parties involved with the Act were of concern. An important reminder was that the Panel Engineer must give himself time fully to study the available records. Also proposed are a prescribed format for reports, a code of practice for Panel Engineers, and the requirement for Panel Engineers to be chartered. Some suggest that Review Panels should be mandatory, but others disagree. A systematic hazard assessment of reservoirs was proposed by several respondents as a more rational criterion for inclusion in the Act than the stored volume. Clarity is sought on whether ash and tailings lagoons should be included within the ambit of the Act and the active interest and role of the HSE in these matters. The Committee has confirmed its intention to address all these issues and to progress research aimed at developing the management of reservoirs and risk.

The role of the Enforcement Authorities and Undertakers was the subject of several contributions, including a suggestion that there should be stricter control of some of their administrative actions, consideration of whether a central enforcement authority would be desirable, and whether emergency action plans should be mandatory.

CURRENT PROPOSALS

A joint consultation paper was issued in October 1995 and a draft Deregulation Order was prepared. This has four major considerations:

a) The Institution of Civil Engineers would take over the administration of the Act, including the appointment and reappointment of Panel Engineers.

b) Remove from the ambit of the Act those reservoirs that can no longer hold 25,000m^3, although once they could, owing to sedimentation.

c) Remove the requirement for the continuous supervision of low hazard reservoirs holding less than 100,000m^3 where the Inspecting Engineer indicates that there is no risk to public safety.

d) Transfer of all enforcement duties in England and Wales to the Environment Agency.

It is believed that these proposals are still under consideration by DETR under the better regulation initiative. The Working Party supports items a,b and d but believes that item c and the possible removal from the Act of service reservoirs would better be considered in the light of the current work on risk and hazard assessment.

KEY ISSUES

The new government has a full legislative programme and may have difficulty finding time for amendments to the Reservoirs Act, however worthy they appear to professional engineers. There are three practical ways of dealing with improvements to the Act in these circumstances. The first is to work at the professional level. At this level the Working Party will assemble the comments made on the operation of the Act and seek to define a consensus view that will be supported by all professionals involved. These changes can be implemented through a Guidance Note, perhaps developed from the existing document *Information for Reservoir Panel Engineers*. The second level of activity, and one needing some government time, is to introduce agreed changes through the mechanism of Statutory Instruments. Such changes would include important alterations to the working of the Act and might have far reaching effects. The third level of change would be appropriate for alterations in the scope or thrust of the legislation. This would require changes to the primary legislation itself and would be unlikely to be implemented within the foreseeable future.

It is important to keep abreast of developments concerning reservoir safety in other member countries of the European Union through forums such as the European Club of the National Committees of ICOLD member countries. The current view is that the principle of subsidiarity will, for the foreseeable future, require the United Kingdom to produce its own legislation in this area.

The concept of a Central Enforcement Authority is contained within the draft Deregulation Order and has also prompted a significant response in answer to the questionnaire. Those in favour of a central authority are hoping for improved efficiency and consistency both in terms of administrative organisation and in terms of national record keeping.

Discussion has taken place over the last 10 years as to whether ash lagoons and tailings dams should be within the ambit of the Act, a point raised by several respondents to the questionnaire. Some of these structures are already inspected as if they were within the Reservoirs Act. The Health and

Safety Executive has responsibility for work activity at tailings dams, lagoons and water reservoirs with a capacity of less than 25,000 m^3 under the Health and Safety at Work Act. Waste Management Paper No. 4 deals with the inspection of tailings dams and lagoons but there are those who feel that it should go further.

The final key issue to be considered by the Working Party will be whether Undertakers should be required to prepare emergency plans for their reservoirs, and if so above what threshold of size or hazard should it be required. The Working Party consider this to be a matter of good practice and the topic will be addressed when *Information for Reservoir Panel Engineers* is updated.

THE NEXT STEPS

Under the chairmanship of Dr G P Sims the Working Party will continue its activities reporting to the Reservoirs Committee. The four important next steps for the Working Party are as follows.

1) To update the document *Information for Reservoir Panels Engineers* to encapsulate a consensus view of good practice. This will require appropriate funding. We hope that this document will become a code of practice, published by Thomas Telford and available commercially.

2) To maintain contact with engineers and managers with a professional interest in reservoirs so that the proposed improvements to the legislation represent the considered wish of the profession.

3) To monitor the current CIRIA research project "Reservoirs - Risks and Hazards" and the possible development of a hazard rating system for use in defining those reservoirs that should fall under the ambit of the Act.

4) To continue the work needed to prepare a professional view on emergency plans, hazard ratings, tailings dams and service reservoirs. We intend to formalise this view and discuss it with the DETR to assist them in defining the most appropriate level of action, whether this be at the professional level, the preparation of Statutory Instruments, or amendment of the primary legislation itself.

Investigating internal erosion at Brent Dam

P TEDD, Building Research Establishment, UK
D P M DUTTON, British Waterways Board, UK
I R HOLTON, Building Research Establishment, UK

SYNOPSIS. Brent dam is a 9m high embankment with a puddle clay core. Erosion of material through the brickwork that forms the draw-off culvert raised concerns that voids were being formed within the core. Investigations to determine the extent and cause of the internal erosion have included ground penetrating radar, measurement of the rate of erosion and boreholes to determine the source of the leakage.

INTRODUCTION

Brent dam was constructed in 1835 to supply water to the Regent's Canal. It is situated on the River Brent close to the North Circular Road in North London. The most notable historical event involving the dam was its partial collapse in January 1841 following a seven day period of non stop rain. The extent of the damage to the dam itself is not known but the resultant surge of water from the reservoir wrecked 113 barges and 16 boats, and resulted in loss of life (Makin, 1986, Wembley History Society, 1985). Initially the dam was 7 m high, but following the events of 1841 it was raised in 1852 by 2 m. The 1.8 m wide clay core was raised, supported by fill on the downstream side and a timber-piled wall on the upstream side.

The dam is founded on London Clay and investigations in 1970 indicated the fill to be clayey gravel and sandy clay (Gudgeon & Walbanke, 1985). Brick rubble, and sand and gravel were also reported under the downstream shoulder fill. The main outlet and overflow structures are located near the middle of the dam and control flows to the River Brent. Approximately 21 m south of these structures, two 225 mm diameter cast iron pipes discharge water into a brick lined culvert approximately 3 m high by 2 m wide as shown in Fig. 1. The brick headwall of the culvert is thought to butt against the downstream face of the clay core. The location of the core shown in Fig. 1 is based on the construction drawings for the raising of the dam. The only valves are located close to the headwall inside the culvert. In 1974 a concrete slab was constructed on the crest to allow the dam to be overtopped during the "design flood".

LEAKAGE INTO THE CULVERT

Five issues carrying eroded material are present in the culvert. The most significant in terms of erosion appears to be from the headwall. A shallow trough was constructed in the 1970's to collect the eroded material from the headwall.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

The erosion into the culvert had clearly been going on for a long time and raised a number of concerns:

- Have large voids formed around the culvert and in front of the headwall?
- Has the erosion migrated to the under side of the concrete slab on the crest of the dam?
- Is the water that is leaking into the culvert coming:
  (a) through the core of the dam ?
  (b) along the outside of the draw-off pipes ?
  (c) through a hole or fracture in the pipes ?

To address some of these concerns investigations have been undertaken to detect any voids which have formed under the concrete slab and around the culvert. Also, measurements have been made to quantify the rate of erosion into the tunnel and to determine the nature of the material being eroded.

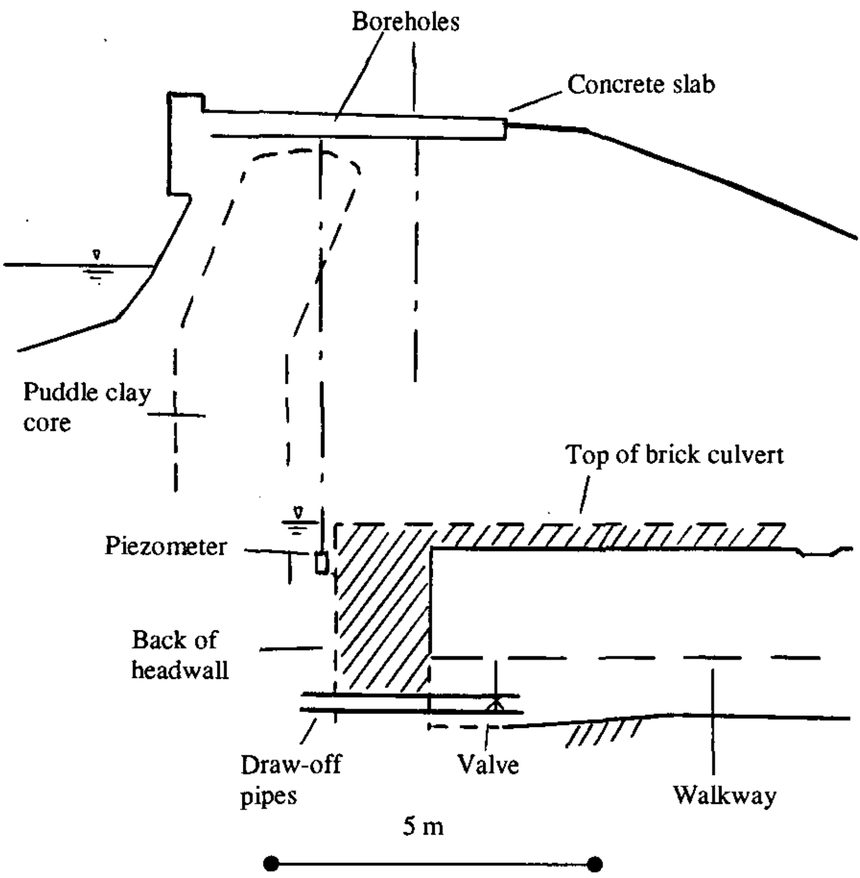


Fig. 1. Cross-section through dam and culvert

INVESTIGATIONS FROM THE CREST

Investigations from the crest involved ground probing radar over a 29 m length of crest, the drilling of two boreholes on the line of the culvert, and the installation of a standpipe piezometer.

Ground probing radar

A survey using ground probing radar was undertaken over a 29 m length of the concrete crest slab spanning the culvert to determine whether it was possible to locate any voids. The concrete crest slab is approximately 4.5 m wide and was constructed in bays with construction joints between them. The survey length consisted of three separate slabs each approximately 10 m long. The first of the three slabs which had been constructed before the other slabs was approximately 200 mm thick and did not contain reinforcing mesh. The other two bays including the one over the culvert were 225 mm thick reinforced concrete cast onto 75 mm of blinding concrete on a 150 mm thick gravel drainage layer.

Ground probing radar has been used to locate voids within structural elements and below floor slabs (Matthews, 1998). It has also been used to locate voids behind the masonry valve shaft at Ogden dam (Charles et al, 1996). The survey at Brent was carried out by BRE using a Pulse Ekko PE1000 digital radar system. Some initial trials were carried out using three antennae operating at nominal centre frequencies of 1.2 GHz, 900 MHz and 450 MHz to determine the most suitable in terms of resolution and penetration. A high frequency antenna gives better resolution of reflecting interfaces whilst a low frequency one gives better penetration of the signal into the concrete and underlying fill. The trials showed that the 900 MHZ antenna gave the best balance between resolution and penetration of the concrete.

A series of longitudinal sweeps at 0.5 m spacing and some transverse sweeps over the culvert were undertaken. Interpretation of the results is a complex task that is primarily concerned with identifying strong reflectors or major changes in signal along a survey line. Strong reflectors are produced at interfaces between different materials such as concrete-gravel and gravel-clay. They are also produced by voids and steel reinforcing bars. A water filled void provides a stronger reflector than an air filled void.

Various features and anomalies were identified within and beneath the concrete. The reinforcing steel mesh in the slabs was clearly identified. No anomalies beneath the concrete were confirmed to be voids by the subsequent borehole investigation.

Borehole investigation

Seven 150 mm diameter cores were taken from the concrete slab to interpret some of the radar anomalies, to determine if any voids were present below the slab and to provide access through the concrete slab for boreholes into the fill below. The cores confirmed the existence of steel reinforcing in two of the slabs and that those slabs were nominally 300 mm thick compared to the slab without reinforcing which was nominally 200 mm thick. At only one location did the coring reveal a void under the slab, however there was a plastic drainage pipe in the ballast immediately below the concrete and this may have caused the void. Record drawings indicate 50 mm diameter drainage pipes running transversely under the slab 3 m at centres. The radar did not make a positive identification of these drainage pipes because of the antenna orientation.

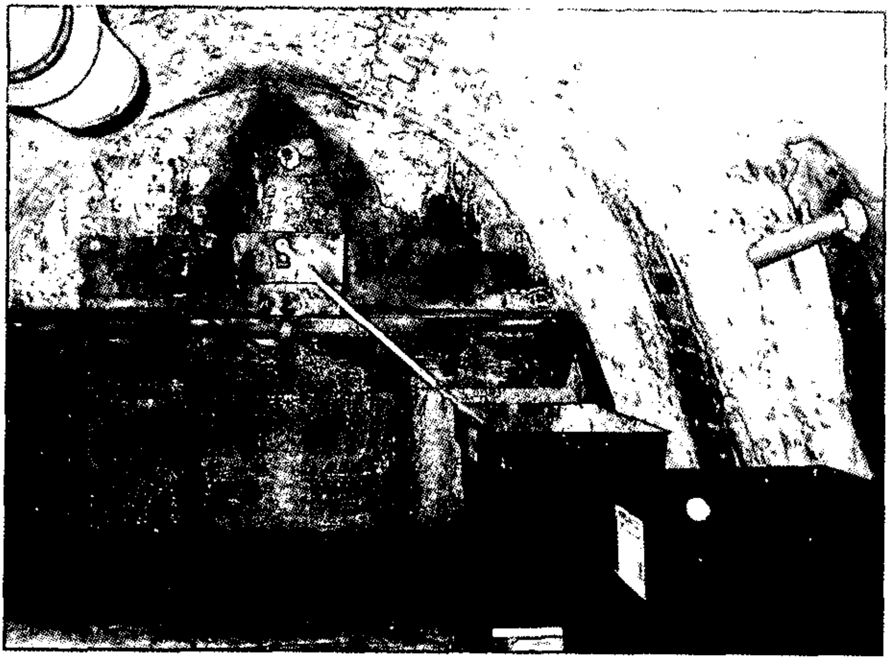
Two boreholes were put down below the concrete slab at the locations shown in Fig. 1 on a line extended from the centre-line of the culvert, to determine whether erosion had migrated up through the fill. Initially a hand auger was used to sink the 90 mm diameter boreholes without the use of water in an attempt not to disturb any evidence of internal erosion. In the event it was only possible to extend the boreholes down to the depths shown in Fig. 1 because of the presence of cobble size stones in the fill and only then by using a combination of the hand auger and a mobile Minuteman drilling rig using a continuous flight auger.

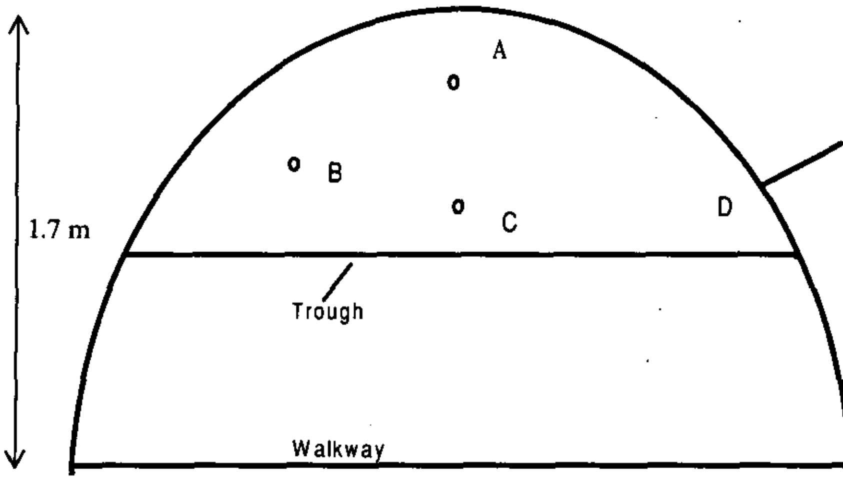
Generally the clay from the boreholes was cohesive, firm to stiff brown (weathered) London Clay with rounded gravel size stones, with the stone content increasing with depth. The additional shoulder fill added in 1852 when the dam was raised appears to be more clayey (Gudgeon & Walbanke, 1985). It is most likely that both boreholes went through the fill immediately downstream of the puddle clay core and not through the upper part of the core as indicated in Fig. 1. Below a depth of 5.4 m the clay was grey and softer. There was no sign of water or evidence of internal erosion at the location of the boreholes. The index properties of a sample from 6 m depth are summarised in Table 1 and show the material to be a clay of high plasticity. A piezometer was installed in the deeper borehole. Figure 1 shows the steady piezometric level at 6.2 m below the crest level which is approximately 2.5 m lower than the levels reported by (Gudgeon & Walbanke, 1985) in piezometers not on the line of the culvert. The lower value is probably caused by the draining effect of the culvert. A falling head permeability test in the piezometer gave a permeability value for the clay of 3.2×10^{-10} m/s which is more than adequate for use as a watertight core.

INVESTIGATIONS IN THE CULVERT

Erosion rate and nature of eroded material

Attention has focused on the leakage from the headwall which was issuing from two locations in the pointing near the crown of culvert. The flow rate from the headwall, although low, has been relatively steady over a three year period between approximately 0.47 to 0.3 l/min, but stopped when the reservoir was emptied in December 1994.





Note: hole D is 2.2 m from the headwall

Fig. 2. Settlement tanks for collecting eroded material and location of exploratory boreholes A, B, C and D in the headwall and side wall of the culvert

In December 1996, approximately 30 litres of very soft orange brown clay was removed from the wooden trough beneath the leaks, see Fig. 2. This amount only represents part of the total amount eroded since the trough was installed approximately 18 years earlier. Although it is not known if the trough has ever been emptied before, it is clear that not all the eroded material was being collected in the trough.

To quantify the current rate of erosion, two settlement tanks were arranged in series (cascade) to collect the leakage and eroded material, see Fig 2. Measurement over a 5 month period gave an erosion rate of approximately 1 litre of in-situ clay per year. This was determined by measuring the dry weight of the collected sediment and then assuming a moisture content of 25% and a bulk density of 2000 kg/m^3 to calculate the in-situ volume eroded. The index properties of the clay from borehole No. 4 at 6m depth, and the eroded material are compared in Table 1 and Fig. 3. The liquid and plastic limits, and plasticity index of the eroded material were significantly higher than those of the in-situ clay such that the material plots beyond the extremely high plasticity range of a normal plasticity chart. This is presumably due to the high clay fraction in the eroded material.

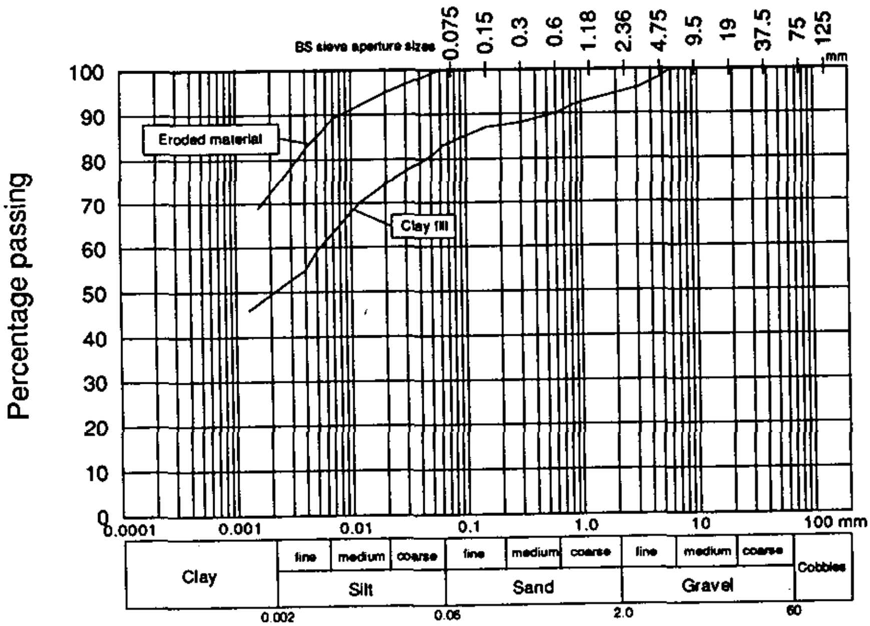


Fig. 3. Particle size distribution of eroded material and in-situ clay

Table 1. Summary of index properties

	w %	w_L %	w_P %	I_p %	Clay %	Silt %	Sand %	Gravel %
Eroded	479	198	121	77	72.2	27.8		
In-situ	30	62	32	30	49.0	33.7	11.9	5.4

23

w moisture content
w_L liquid limit
w_P plastic limit
I_p plasticity index

The relatively high clay content of the eroded material is probably due to the low flow rate along the leakage paths through the clay only having the energy to erode clay size flocs and silt. Erosion is clearly related to the velocity of flow and laboratory experiments at BRE have shown this to be the case (Charles et al, 1995).

<u>Borehole investigations</u>
Exploratory holes, 32 mm and 50 mm diameter, were drilled through the headwall and culvert with the following objectives:

- to determine the thickness of the headwall and the culvert,
- to investigate the source of the leakage causing the erosion,
- to determine the nature of the clay and extent of any voiding behind the headwall and the culvert wall.

Figure 2 shows the location of the holes drilled into the headwall and the side wall of the culvert. At the first attempt to drill through the headwall, it was only possible to drill 0.8 m which did not reach the back of the wall. Having drilled hole A, on the centreline of the culvert, water appeared to be flowing from the back of the hole towards the headwall face at a similar rate to that which had been issuing from the headwall prior to drilling, whereas the hole B to the side of the headwall was relatively dry. The rate of erosion from hole A was monitored for a further two months and was found to be similar to that measured earlier from the face of the headwall. The mortar between the bricks along the hole appeared to be very porous with some voids in places.

Longer holes drilled some months later established that the headwall was 1.6 m thick as shown in Fig. 1. It can be seen that the piezometer was installed only 0.3 m away from the back of the headwall. The clay at the back of the headwall was a firm grey clay similar to that found in the lower part of the borehole drilled from the crest. There was no sign of any of the orange brown soft material similar to the eroded material. On drilling hole C vertically beneath hole A, water flow ceased from the upper hole and started from the lower one. By blocking hole C and observing flows in hole A it appeared that the water was flowing up through the brickwork from a length approximately 0.8m from the headwall face.

Hole D was drilled into the side of the culvert 2.2 m back from the face at the location where a substantial amount of erosion was taking place. The thickness of the culvert wall was approximately 0.35 m (3 courses of brick) and the clay at the back of the culvert was firm and dry, similar to that at the back of the headwall. The water appeared to be coming through the mortar joint behind the first brick. The flow rate from this location is approximately one third of that from the headwall and the erosion rate was slightly under a half of that from the headwall.

The flow through the brickwork which is probably through the porous mortar suggested that the mortar and possibly the brickwork are being eroded by the water. To determine if the material currently being eroded contained any brick or mortar, X-ray diffraction (XRD) analyses were carried out on samples of eroded material, the mortar and the brick. No mineral phases were identified in the eroded clay whereas quartz, albite and haematite were identified in the brick, and calcite and quartz were identified in the mortar. The results imply that the material currently being eroded is probably clay and does not contain mortar of brick. Further analyses are being considered to confirm that the eroded material is clay.

DISCUSSION AND CONCLUSIONS

Many cases of internal erosion of material into draw-off tunnels and culverts have been observed in old British dams. Draw-off tunnels act as drains to the surrounding fill and if not adequately watertight can provide a means of allowing eroded material to exit from the dam. Generally the rates of erosion are very small and there are few reported cases that have led to serious incidents. However, there are many cases where grouting of voids behind culverts has been carried out to prevent continuing leakage and internal erosion.

It has been estimated that the current rate of erosion of clay from the headwall alone into the culvert is at least one litre per year and erosion is taking place at other locations. It is not known for how long this erosion has been going on and what effect it is having on the integrity of the dam. This case history shows that even with very small leakage flow rates, erosion of clay size particles can occur over a sustained period.

There was no sign that the erosion into the culvert was progressing up towards the crest at the location of the boreholes. The drilling investigations, both from the crest and the headwall, have shown that the clay in front of headwall and to the side of the culvert appears to be in good condition and is not being eroded. The leakage is coming up through the porous brickwork, possibly from the draw-off pipe or along the draw-off.

Despite extensive investigations the origin of the leakage and the eroded material is still not known. Further investigations are required to determine whether the water is coming through a fracture or hole in the draw-off pipe as this will determine the most appropriate remedial work.

ACKNOWLEDGEMENTS

Research on the safety of embankment dams at the Building Research Establishment has been supported by the Department of the Environment, Transport and the Regions and this support is gratefully acknowledged. The help provided by Barry Fuller of British Waterways Board in collecting data for this project is also gratefully acknowledged

REFERENCES

Binnie G M (1987). Early dam builders in Britain. Thomas Telford, London, pp 101-111

Charles J A, Tedd P and Holton (1995). Internal erosion in clay cores of British dams. *Proceedings of Conference on Research and Development in the field of Dams*, Crans-Montana, Switzerland, September, pp59-70.

Charles J A, Tedd P, Hughes A K and Lovenbury H T (1996). Investigating embankment dams: a guide to the identification and repair of defects. Building Research Establishment Report BR 303. Construction Research Communications, London

Gudgeon D L and Walbanke H J (1985). Design and performance of 19th Century earth dams. Proceedings of 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, vol 4, pp1981-1984.

Makin G (1986). The great flood. Waterways World, September, pp38 -39.

Matthews S L (1998). Application of subsurface radar as an investigative technique. Building Research Establishment Report BR 340. Construction Research Communications, London

Wembley History Society (1985). The Welsh Harp Reservoir 1835 to 1985.

Observation analysis at Llyn Brenig

ANDREW THOMAS, Independent Supervising Engineer, U.K.

SYNOPSIS. Llyn Brenig dam is a rockfill dam with a sloping rolled clay core and has been well monitored over 20 years. 1,414 readings per year were taken on reservoir levels, rainfall, piezometric pressures (standpipe, hydraulic and pneumatic), total pressure, internal movement (extensometers, inclinometers and settlement gauges) and superficial movement (crest studs and survey beacons). This paper describes a detailed analysis and review of these readings. The objective was to produce new procedures for future monitoring with revised reading frequencies, suspension of non critical readings and detailed proposals for comparing actual readings with predicted readings.

INTRODUCTION

Llyn Brenig dam incorporated many instruments during its construction in 1973-76. Twenty years later there were still over 1,400 readings taken per year and since its completion over 30,000 readings have been taken. Therefore in 1996 it was decided to undertake a detailed review of all readings to rationalise the need for readings, to better understand the readings and to provide recommendations for future readings. Without the aid of modern computer software to manipulate the data quickly and to produce effective colour graphics, this project could have been a daunting task. This paper sets out the findings of the review completed in early 1997.

DESCRIPTION OF DAM

Llyn Brenig dam lies in the uplands of North East Wales. It was promoted by the Dee & Clwyd River Authority, financed by the Welsh Water Authority, designed by Binnie and Partners and constructed by Fairclough Civil Engineering Ltd. The dam has shoulders of Silurian mudstone and a sloping rolled core of boulder clay. It retains 61,525 Ml of water with a maximum height to spillway of 47 m at 377 m a.o.d., with a crest 3m above spillway and 1,200 m long. The upstream face has a slope of 1:2 with berms which made for easier construction and allowed for the effective slope to be adjusted during construction. The downstream slope is 1:4 at the foot increasing to 1:1.5 near the crest. (See Figs. 1 and 2)

The clay core is at a slope of 1:1.25 (vertical:horizontal) being some 14 m thick (horizontally) narrowing to some 10 m at the crest and is protected with sand and gravel filters. The core was laid at this slope to reduce the risk

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

of tensile stresses developing as a result of differential settlements, to allow for a Phase Two raising by 21 m and to provide construction surfaces wider than the core thickness of the core.

The dam is partly formed on rock but mainly formed on the drift of varved clay layers and rises over a natural drumlin some 18 m high at chainage 920. A grout curtain formed the cut-off through the drift and underlying rock The mound constructed on the downstream shoulder, rising 11 m above the crest, sits on the drumlin at chainage 920. It is a landscape feature and would be enveloped in a Phase Two raising.

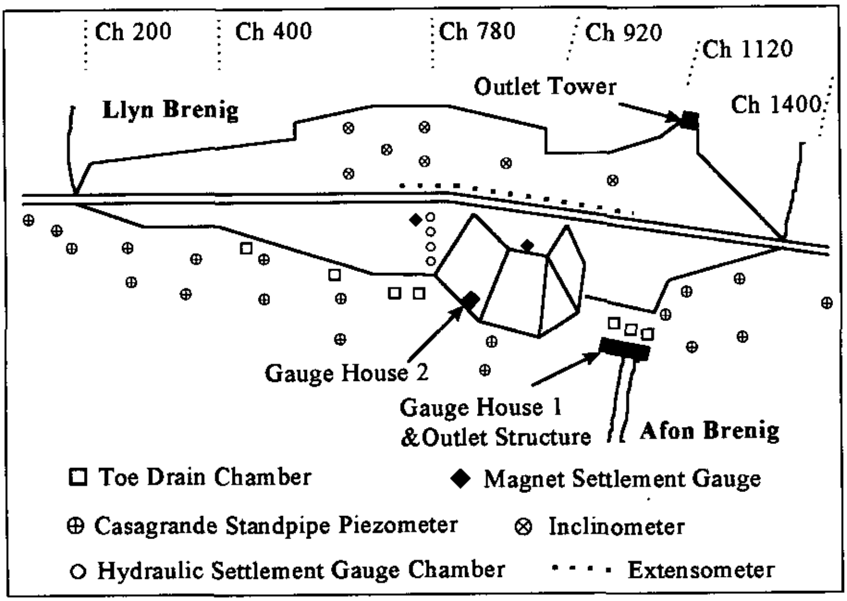


Fig. 1. Plan indicating instrument locations

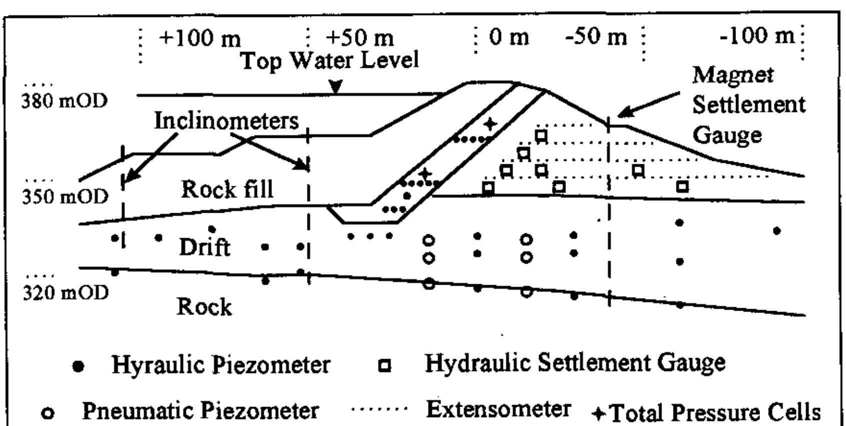


Fig.2. Section at chainage 780 indicating instrument locations

The dam was completed in June 1976 after three years' construction at a cost of £12.2 m, including extensive road diversions, and has been operated in conjunction with Llyn Celyn for regulation of the River Dee. Because of its quicker refill characteristics, Celyn is drawn down first, with Brenig as the back up. For the foreseeable future, Brenig will continue in this role. Raising of the dam is presently considered unlikely.

SUMMARY OF THE REVIEW'S RECOMMENDATIONS

In Table 1, the 1976 column shows the number and frequency of instrument readings recommended for the first period up to 6 months after first overflow with a total of nearly 7,000 readings per year. Thereafter a lesser frequency was recommended for some readings. The 1996 column shows the actual readings prior to the review, totalling over 1,400 readings per year, and the 1997 column shows the recommended readings after the review reducing the readings by over 500 per year. It is fortunate for continuity that one man has taken nearly all the instrument readings for 20 years.

By 1996 some of the piezometer and pressure cells readings had been abandoned because of instrument failures such as blockages in tubes, piezometric pressures being below the instrument level and pressures being above ground level in Casagrande standpipes. Seven of the vertical settlement gauges and inclinometers have been inaccessible since they are underwater on the upstream face and a decision was taken in 1990 to abandon the hydraulic settlement gauges (see Hydraulic Settlement Gauges).
It is interesting to note the high number of readings taken in 1976 which must have involved much time with laborious manual plotting and analyses of the results. Now, with the benefit of 20 years readings to give confidence in the dam's performance and with the aid of computer software spreadsheets and graphics it is easily possible to examine the reduced amount of data in detail.

The review of the readings was therefore undertaken :

- to suspend readings of instruments not at key locations. Since the instrument readings will be needed if the reservoir falls more than 5 m below spillway (see Reservoir Levels) or if the dam were to be raised the readings are suspended rather than abandoned. In practice the only suspended instruments which require regular maintenance are the piezometers which are flushed with de-aired water once per year.
- to reduce the frequency of readings where there is judged to be sufficient confidence in the dam's performance
- to draw trend lines and to provide bands within which the readings are predicted to lie, so that extraordinarily high or low readings are flagged up when they are entered on the spread sheets. The technician is advised to notify the Supervising Engineer if readings lie outside the check limits on

two consecutive occasions or, for six monthly readings, if the reading lies outside the limits on one occasion.

- to set up graphic representations so that any extraordinary readings and trends can be seen immediately.

Table 1. Summary of readings and frequency

YEAR	1976			1996			1997		
Type of Reading	N	F	T	N	F	T	N	F	T
Reservoir Level	1	365	365	1	52	52	1	52	52
Rainfall	1	365	365	1	52	52	1	52	52
Toe Drains	7	365	2,555	7	52	364	7	52	364
Standpipe Piezom's	19	52	988	17	12	204	11	6	66
Hydraulic Piezom's	106	6	636	87	4	348	64	2	128
Pneumatic Piezom's	16	6	96	12	4	48	12	2	24
Total Pressure Cells	31	4	124	14	2	28	0	0	0
Extensometers	33	12	396	25	4	100	0	0	0
Magnet Settle. Gauges	134	6	804	42	2	84	42	2	84
Hydr. Settle. Gauges	8	2	16	0	0	0	0	0	0
Inclinometers	110	2	220	0	0	0	0	0	0
Crest Level studs	25	12	300	25	2	50	25	2	50
Beacon Surveys	42	2	84	42	2	84	42	2	84
TOTAL			6,949			1,414			904

(Column N is the number of readings taken at one time, F is the frequency of readings per year and T is the total number of readings per year.)

OUTLINE OF INSTRUMENT LOCATIONS

The dam is instrumented mainly on three cross sections (See Figs. 1 and 2)

(i) at chainage 780, the greatest height of dam on drift (fill 36 m, drift 23 m)

(ii) at chainage 920, the greatest depth of drift below the dam (fill 22m, drift 45 m)

(iii) at chainage 1120, the greatest height of dam on rock (fill 47 m)

Some inclinometers, extensometers and settlement gauges are placed outside these sections, standpipe piezometers are installed in the ground downstream of the dam and a system of crest studs and survey beacons covers the dam crest, downstream shoulder and adjacent ground.

OBSERVATIONS IN DETAIL

Reservoir Levels and Rainfall

Over the last ten years the reservoir levels have varied between 5.9 m below spillway and 0.2 m above with an average of 1.2 m below. This small variation reflects the use of the reservoir as standby storage and since these levels have occurred during some exceptional droughts it is not expected that this range will vary in the foreseeable future. However, since the dam has only been frequently monitored over this range of levels, it is essential that instrument readings are suspended, rather than abandoned, so that consideration can be given to restarting them if the lake level drops more than 5 m. The weekly rainfall has been at a maximum of 115 mm/week and an average of 22.5 mm/week over the last 4 years. The frequency of both these readings is maintained at once per week to encourage operatives to visit the site and carry out essential visual observations that could surpass the value of any instrumentation.

Toe drains

The are four drains from the toe of the core on the right of the mound, between chainages 200 and 700, one drain on the old river course at chainage 1100 and two drains on either side of the draw off culvert at chainage 1120. V-notch readings on each drain are taken weekly. Health and Safety requirements have recently required the levels at four deeper drainage manholes to be measured by probes from the surface which is proving to be less accurate than direct measurement at the V-notch.

Over the last five years the plots of reservoir level, rainfall and drainage flow against time indicate that the flows may have some reaction to rainfall. However, no direct relationship could be found. This is perhaps not surprising since the drainage flows could be affected by rainfall as it seeps through the shoulder to the drains, by rainfall as it permeates the drift and rock to spring up into the drains or by the reservoir level as water seeps through the core or cut-off. A further complication in finding a relationship was that the instantaneous drainage reading may be affected by the rainfall or reservoir level several hours or even months before. Attempts at assessing the seepage flow through the core using the seepage index proposed by Charles et al (1996) failed. The piezometers show that the hydraulic gradients across the core vary by a factor greater than two. No record of the clay permeabilities were found and values may vary from 10^{-8} to 10^{-10} m/s. Hence estimates of flow could be in error by a factor over 200.

A typical plot of drainage flow against rainfall is given in Fig. 3 for the toe drain near the right abutment. There appears to be a minimum flow of some 0.8 l/s with an indication of flows rising with increased rainfall. Ford et al (1978) state that there is "some natural groundwater flow which has always been evident from the right downstream abutment" but there is no record of the quantity. Because of the variety of sources and the inability to predict the seepage flows, pragmatic check limits were set based on the judgement that previous flows have been acceptable. The limits are shown in the chart and similar limits were set for all the other drains.

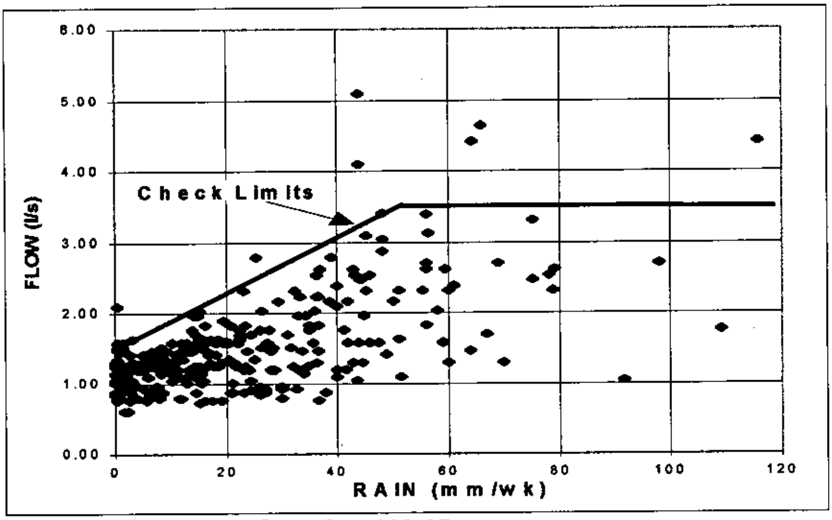


Fig.3. Toe drain No. 1 flows for 1992-97

Casagrande Standpipes

The standpipes are installed on the abutments and downstream of the dam in the drift and rock. Plots of piezometric levels against reservoir levels showed a good correlation in most cases whereas no good relationship was observed with rainfall. Trend lines based on a least squares fit are drawn by computer and check limits are usually set to 1 m either side of the trend lines.

Figure 4 shows some typical results with the piezometric levels rising with reservoir level as shown by the trend lines. Other standpipes gave plots with excessive scatter (+/- 3 m) and some level or negative gradients. It was decided to suspend readings where the results were not meaningful and where other standpipes could reflect pressures at the location.

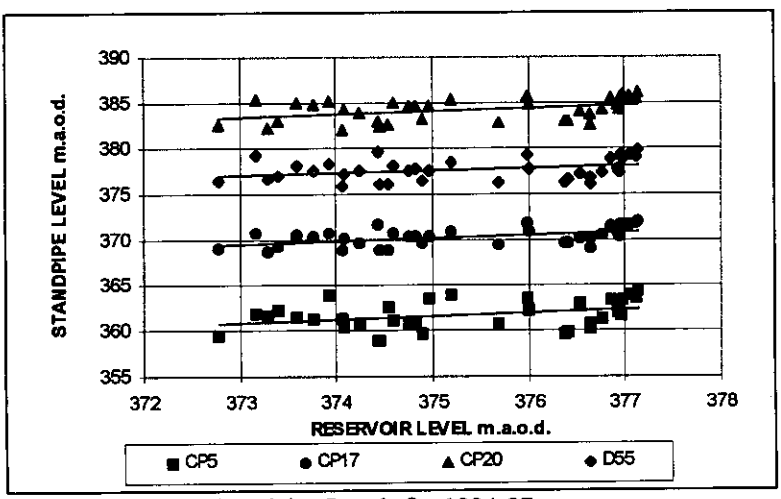


Fig. 4. Casagrande Standpipe Levels for 1994-97

Hydraulic Piezometers

A comprehensive set of hydraulic piezometers is installed at the three cross sections mentioned previously and located in the shoulders, the core, the drift and the rock. Readings were analysed as for the standpipes with trend lines set for check limits and the plots enable a check that the various components of the dam are behaving as expected.

Figure 5 shows typical readings in the upstream rockfill shoulder for two piezometers with some readings nearly identical. Both are about 10 m upstream of the clay core and No.130 is at approximately one third height whilst No. 137 is at two thirds height. The slope of the trend lines nears 1:1 as the piezometric pressure directly reflects the reservoir level .

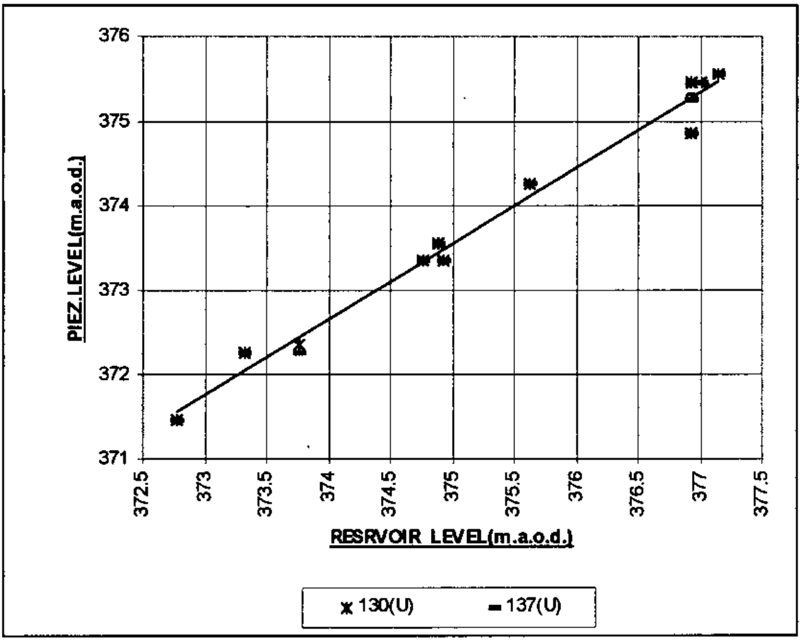


Fig. 5. Hydraulic piezometer levels in the upstream fill at chainage 1120 for 1994-97

Figure 6 shows plots in the downstream shoulder; No.114 being located near bedrock; No. 123 at about third height and No. 131 at two thirds height. The readings appear with the level gradients, as expected, indicating that the reservoir level is having no effect. However, all these readings are 1 m or thereabouts, below the level of the piezometer tip so all that can be deduced is that the pressures have not risen above the piezometer tips. This also demonstrates that, although modern computers are excellent for analysing and presenting readings, slavish adherence to the results without due consideration can be misleading .

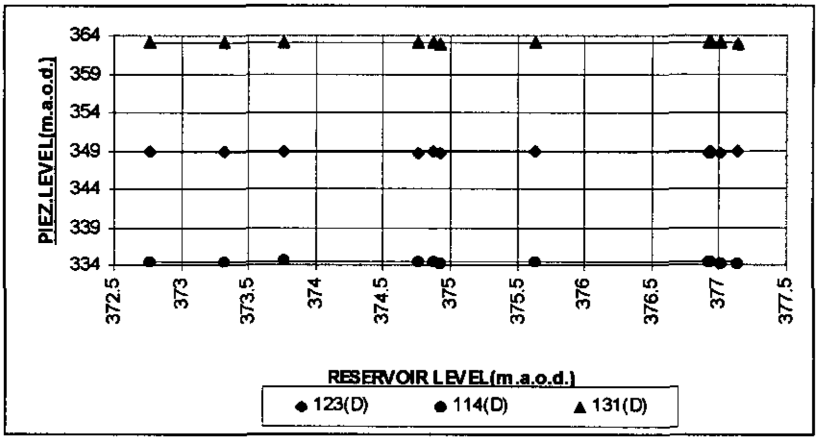


Fig. 6. Hydraulic piezometer levels in the downstream fill at chainage 1120 for 1994-97

Figure 7 shows typical readings for a line of piezometers in the core at a third height; No. 125 being near the downstream face and No.129 near the upstream. The gradually steepening piezometric gradients towards the upstream face give confidence that the core is functioning correctly. In another line of piezometers the gradient downstream was steeper than the upstream gradients which gives rise to the suspicion that the piezometer tubes had been wrongly numbered. Ford et al (1978) mentioned that "with the hazards of installing banks of leads through the busy construction, readings from some piezometers are suspect"

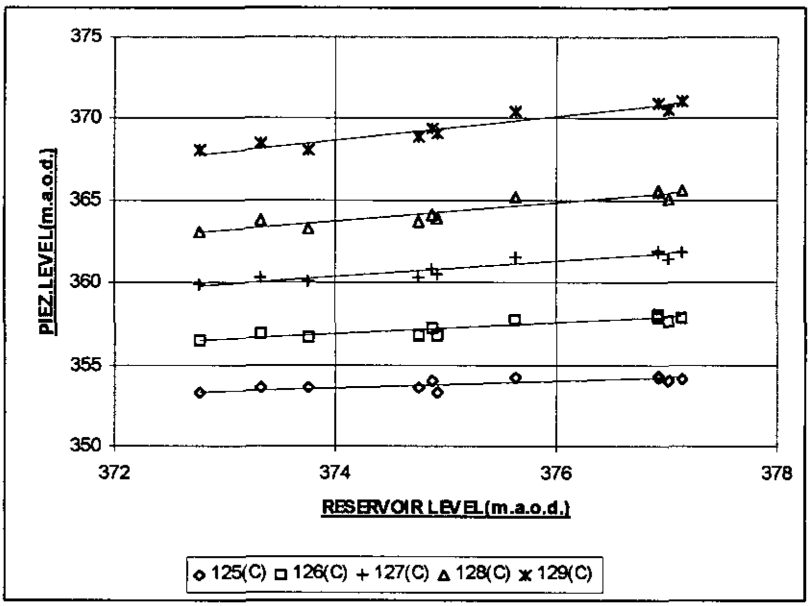


Fig. 7. Hydraulic piezometer levels in the core at chainage 1120 for 1994-97

The readings of piezometers in the upstream drift generally adhere closely to a trend line with typical positive slopes whereas those in the downstream drift show a wide scatter and a trend line that is usually flat or negative. This indicates that in the upstream drift piezometric levels are influenced by the reservoir level but there is little evidence for this in the downstream drift which is to be expected.

Most of the piezometers have given good and acceptable readings although some of their tubes had become blocked for unknown reasons soon after construction and subsequently others have ceased to work. Out of the 106 hydraulic piezometers, 19 are defunct, 64 continue to be used and 23 have been suspended. Those suspended are in non critical locations such as the downstream and upstream shoulders or in the drift where other pneumatic piezometers are available. To ensure that the piezometers can be reinstated, de-aired water is circulated through the tubes once per year. Furthermore the reading frequency has been reduced from 3 monthly to 6 monthly.

The check limits vary from 0.3 m to 0.6 m either side the trend lines depending on the scatter experienced at each location. No alarming changes have been recorded other than in one month when most of the readings were outside the check limits. This was later found to be due to an inexperienced operative taking erroneous readings.

Figure 8 shows a plot with crude straight lines drawn between the piezometric levels. At 365 m level the hydraulic gradient is 1.4 rising to 2.1 at the lower 350 m level and to 3.2 at 335 m level. This is as expected but at chainage 780 (see Fig. 9) the hydraulic gradient is 0.8 at 364 m (accepting a wider scatter of results) rising to 2.3 at 350 m and falling to 1.6 at 341 m. The reason for the low gradient at 341 m may be that the section of the clay core widens to some 25 m at 342.5 m level to fill the key trench in the drift. Hence there is a greater length for pressure reduction between upstream and downstream faces.

Pneumatic Piezometers

The pneumatic piezometers are placed in the downstream drift at chainages 780 and 920 and were installed as checks on the hydraulic piezometers and to measure pore pressures at levels too low for hydraulic piezometers.

The plots of pore pressures against reservoir level are similar to those for hydraulic piezometers in the downstream drift. Four have been defunct for some time and the remaining 12 are maintained with check limits up to 0.6 m either side the trend line.

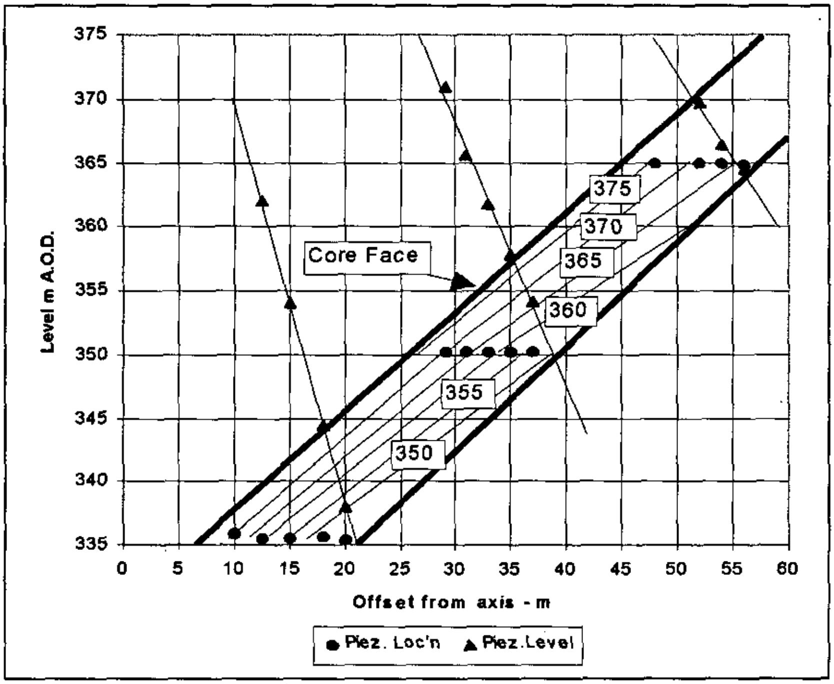


Fig. 8, Equipotential lines in core at chainage 1120 for reservoir full at 377 m

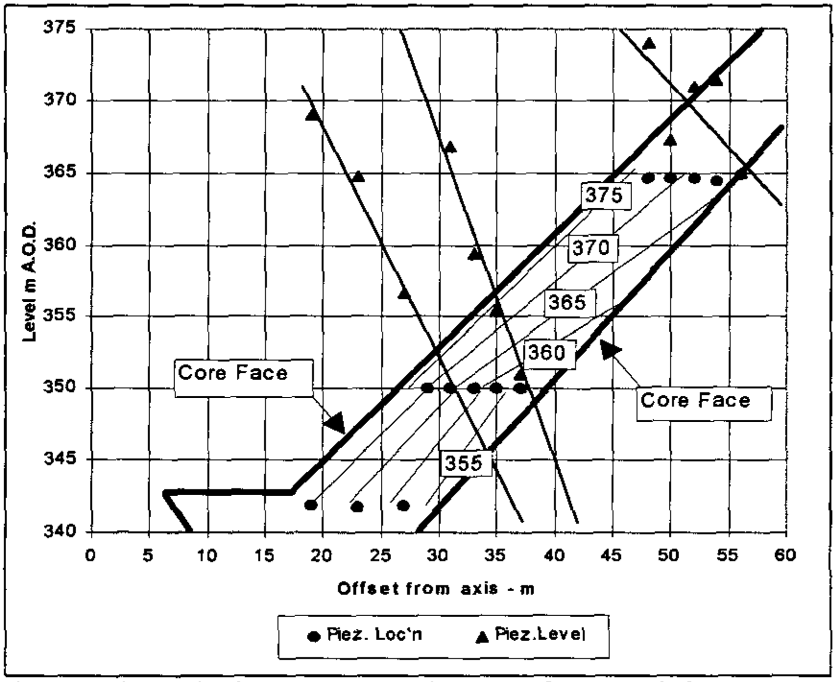


Fig. 9. Equipotential lines in core at chainage 780 for reservoir full at 377 m

Total Earth Pressure Cells

The oil filled earth pressure cells were placed in eight clusters in the core at the three main sections and the cells were set at differing orientations. Up to 1996, readings were only being taken at 12 cells out of the original 31 since suspected blockages prevented readings at many cells. Generally the readings have shown very little variation over 10 years and attempts at predicting the total pressure, taking into account the clay particle pressure, the pore water pressure and the cell orientation, have failed. Therefore all these readings have now been suspended since the pore pressure and settlements are being monitored by other means.

Extensometers

One line of the potentiometric extensometers was placed in a line parallel to the axis of the dam (See Fig.1) a few metres above the drumlin in the drift and just downstream of the core to help detect any vertical cracking in the core over the drumlin. Other lines of extensometers were placed at different levels in the section at chainage 780 (See Fig. 2). Over the last 10 years there have been no movement trends detected. This gives confidence that there is now little movement within the dam and that no cracks have formed in the core. Hence these readings have been suspended and reliance is placed on the surveys.

Magnet Settlement Gauges and Inclinometers

Seven magnet settlement gauges are integral with inclinometers and are installed in the upstream fill and drift down to bedrock (See Fig.1 and 2). It is noted by Ford et al (1978) that early anxiety over apparently large movements were dispelled once the foibles of the instruments were overcome by taking account of temperature, damp atmosphere and the stretching of the combined power and suspension cables. Because of the high reservoir levels it has not been possible to access the inclinometers over the last ten years except on a few occasions but the expertise to take measurements was not readily available. Therefore the decision has been taken to suspend these readings as none are accessible whilst the reservoir level is within 5 m of top water level.

A further two magnet settlement gauges were installed in the downstream shoulder. One of the gauges with 27 magnets surfaces at the top of the mound at chainage 920 and the other with 18 magnets is in the downstream shoulder at chainage 780 (See Figs 1 and 2). Until 1996 movements of the magnets were recorded relative to magnet No. 1 at the base of the hole. At chainage 920 it was noted that the records over 10 years showed a wide scatter ranging up to 40 mm at some magnets. On examining the tape readings, of some 75 m length, at magnet No. 1 the scatter was over a range of 30 mm affecting the datum for all the upper magnets. Consequently the readings at both gauges are now measured relative to the top of the tube which is regularly surveyed. It is assumed the datum was originally taken as the bottom magnet because the tube was being extended during the construction but the longest tape reading to the bottom is also potentially the most inaccurate. The readings at chainage 920 indicate that the settlement increases with the height of the magnet up to a maximum of 370 mm at 376 m a.o.d. (top water level is 377 m a.o.d.) and then reduces to 80 mm at the top of the mound. This may be explained if the top of the mound was constructed after the main body of the dam had initially settled. Check limits are now set against the ten year average and are +/- 10 mm for depths down to 60 m and +/- 15 mm for greater depths; these limits being based on the previous observations. There will be a review the new procedure in two years time.

Hydraulic Settlement Gauges

These readings were abandoned in 1990 upon the recommendation of an Inspecting Engineer. Although contemporary readings at each location were very similar over the years, they were not consistent within themselves. For instance, since 1975 gauges near the crest recorded settlement of 150 mm, whereas some near the foot recorded 450 mm settlement and others showed a heave of 150 mm. Because of these inconsistencies and the fact that survey beacons were on the line of these gauges, they were abandoned.

Surveys

Crest studs are set along the crest at 50 m spacing and levelled at 6 monthly intervals The survey beacons are set on the downstream slope mainly on chainages 780 and 1120 and are surveyed by triangulation to detect vertical and horizontal movement every 6 months. These surveys are taken by an outside firm and are the only readings not taken by the directly employed instrument reader.

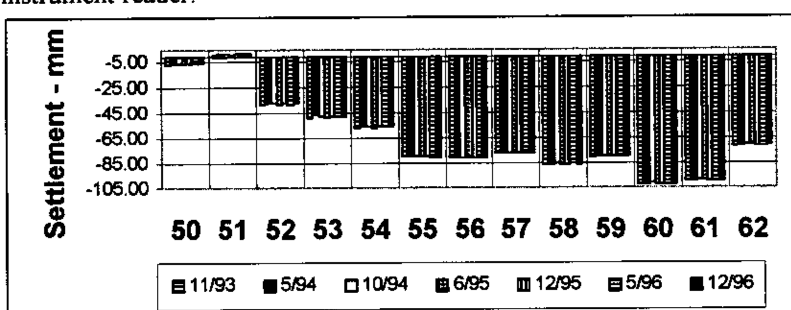


Fig.10, Crest studs' vertical movement since January 1977

Figure 10 shows the crest settlements from chainages 800 (stud No.62) to 1,400 (stud No.50) with a scatter of a few mm over the last four years. The crest had built-in settlement allowances. The maximum allowance was 500 mm at chainage 860 m, between stud Nos 60 and 61, whereas the maximum settlement recorded since January 1977 is 102 mm at stud No.60. The simplistic calculation of the settlement index proposed by Charles et al (1996) for the Brenig readings during the period 1977 to 1997 is in the range of 0.001 to 0.002 which is less than for other rolled clay core dams. This, coupled with the knowledge of original settlement allowance, gives comfort that the actual settlements are well within acceptable limits. The crest surveys are continued at 6 monthly intervals with a check limit +/- 1.5 mm on the average over 5 years. The survey beacons show settlement up to 65 mm since 1977, lateral movement up to 6 mm and longitudinal movement parallel with the axis up to 16 mm. These surveys are also continued at 6 monthly intervals with a check limit of +/- 4 mm on the average over 5 years.

CONCLUSIONS

Although the review took many hours to complete it achieved :

- a greater understanding of the readings and of the dam's performance
- a rationalisation of measurements taken at particular locations
- a check on the validity of the readings and of the methods of measurement
- a set of revised procedures which handle the data more efficiently and which provides a greater confidence in effective measurement.

The lessons learnt from the review were:

- the receipt of too many continuous repetitive readings leads to complacency and an inability to see real trends
- the ability of computers to follow set mathematical procedures should not blind one to challenging the results and applying common sense
- visual observations on site can be more valuable than any instrument reading. (Instrument readings only monitor the dam's performance at one particular location.)

This review would not have been undertaken without the aid of modern computer spreadsheets which enabled:

- easy manipulation of data from different files
- the production of predicted readings to which check limits can be applied
- the production of graphics to set trends and spot anomalies easily

ACKNOWLEDGEMENTS

The author is grateful to the Water Resources staff of Dwr Cymru Welsh Water for their ready assistance with collating the data and executing the recommendation and to Mr. J.G. Cowie and Mr. D.N.W. Earp for their invaluable comments and guidance throughout the review and in the preparation of this paper.

REFERENCES

J.A.Charles, P.Tedd, A.K.Hughes & H.T.Lovenbury (1996). Investigating embankment dams. Building Research Establishment.

S.E.H.Ford, R.M.Arah, J.W.Reeves, J.H.Flemming & A.Cockshaw (1978) Llyn Brenig. Part II: Design and Construction with Discussion. Journal of the Institution of Water Engineers & Scientists, Vol 32, No 4. pp 288 et seq.

The Lower Lias Clay at Barrow No 3 Reservoir

N G SWANNELL, Sir William Halcrow & Partners Ltd, UK

SYNOPSIS. Barrow No 3 Reservoir is a fully bunded reservoir constructed of the Lower Lias Clay. Construction commenced in 1887. However, final filling was delayed some 49 years due to a succession of slips. In 1992 a seismic safety evaluation was undertaken (Swannell, 1996) with an investigation of the Lower Lias Clay which included (i) review of published results, (ii) back analysis of the old slips, and (iii) sampling and testing. These together indicated, *inter alia,* the important residual strength envelope to be non-linear. A plot of the angle of effective residual strength against normal effective stress is suggested as being generally applicable to the Lower Lias Clay.

INTRODUCTION

Barrow No 3 reservoir is a non-impounding fully bunded structure which overlies the Lower Lias Clay. The embankment has a puddle clay core with clay fill shoulders. The shoulder fill was all sourced from the Lower Lias Clay, but the generally higher plasticity core material is thought to be a mixture of alluvial and weathered Lower Lias clays. The height of the embankment varies from a minimum of approximately five metres above original ground level to the south of the site, to a maximum of approximately 12 metres on the opposite, or northern, bank (Swannell, 1996).

Construction commenced in 1887, but was only completed to full height with some difficulty, with the embankment suffering a succession of well recorded slips in the period from late 1887 until approximately 1910. Final filling was not achieved until 1936, water level not being allowed to exceed approximately 0.5 metres below top water level (TWL) since 1914.

The history of the reservoir was comprehensively researched in 1981/82 by Messrs Watson Hawksley, consulting engineers, on behalf of the then Bristol Waterworks Company (Watson Hawksley, 1982).

In 1992 the reservoir was subject to routine statutory inspection in accordance with the Reservoirs Act 1975. An appropriate seismic safety evaluation was recommended to be carried out in accordance with the BRE Guide (Charles *et al*, 1991). Details of the evaluation and the methodology adopted have been published (Swannell, 1996).

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 required to carry out the necessary pseudo-static stability analyses. An appropriate investigation was therefore devised to target the particular information required.

The investigation afforded the opportunity to study and carry out specific tests on the Lower Lias Clay which, in general, appears to have been subject to relatively little investigation compared to other clays in the United Kingdom.

It was recognised that in view of the history of slope failure it would be particularly important to determine with confidence the residual strength of the clay. The detailed and well maintained records of some of the pre-existing failures gave scope for back analysis to augment the results of a literature review and from sampling and testing.

Therefore, in summary, the scope of the geotechnical investigation consisted of

i) a desk study of the Lower Lias Clay;
ii) back analyses of historical slope failures at the site; and
iii) ground investigation and laboratory testing,

with particular attention being given to establishing site specific effective stress soil strength.

The paper describes the three parts of the investigation and summarises the key results relating to the Lower Lias Clay in this area, including comparison with results from other sites.

DESK STUDY

The desk study consisted of two principal components thus:

i) a geological and geomorphological review of the site;

ii) a study of published results relating to the properties of the Lower Lias Clay.

Geology and geomorphology

The general geology of the area is described by Hawkins (1973) and, with particular reference to Barrow No 3 Reservoir, by Watson Hawksley (1982).

The Barrow Reservoirs overlie the Jurassic Lower Lias deposits which comprise stiff to very stiff dark bluish grey, laminated silty clays, thinly interbedded with very weak to weak calcareous mudstone and moderately strong to strong grey limestone. The deposits weather to brown and bluish-grey clays.

Part of No 3 Reservoir, principally towards the north east, overlies colluvium and periglacial soliflucted material having its origins on the slopes of Dundry Hill to the south of the site (Hawkins, 1973). Both Hawkins (1973) and Chandler (1970a and 1970b) conclude that when the Lias Clay (Lower and Upper) outcrops in areas where the slopes are greater than 9° the slopes are often disturbed and also veneered by superficial deposits. Chandler (1972) also records that the Upper Lias Clay, and probably other heavily overconsolidated clays, can be severely disturbed as a result of permafrost conditions and this disturbance can result in reductions in shear strength in near-surface layers. These general conclusions appear to be equally applicable to the Lower Lias Clay at the Barrow site.

Below the northern section of the embankment there are known to be three valleys infilled with soft sandy-silty-clays of high moisture content, apparently not removed prior to embankment construction (Watson Hawksley, 1982).

Published results

The Lower Lias Clay has been less widely studied than the Upper Lias Clay. However, there appear to be no clearly recorded significant differences in geotechnical properties (Cripps and Taylor, 1981).

Some typical published results for the Upper Lias Clay are given in Table 1.

A documented slope failure in the Lower Lias Clay was the Evesham Railway embankment slip. The index properties of the fill measured at this site were as follows (Chandler *et al*, 1973):

w_L	61%
w_P	25%
I_P	36% (37% in landslip zone)
w	28% (29% in landslip zone)
Unit Weight	19.3 kN/m^3

where the notation is as for Table 1 and w = natural moisture content.

The peak effective strength measured in the laboratory was c' = 5 kPa, $\phi'_p = 27°$. This compares with an average value of c' = 10 kPa, ϕ'_p = 28° measured in the 1981 investigation at Barrow No 3 (Watson Hawksley, 1982).

Residual strength tests at Evesham showed some scatter, but a value of residual angle of shearing resistance (ϕ'_r) of approximately 17° was estimated at the effective normal stress level of 30 kPa acting on the failure plane.

Most importantly the residual strength failure curve was noticeably non-linear, rising fairly steeply at low effective normal stress levels, that is below 50 kPa approximately. This effect is consistent with other clays of medium to high plasticity (Skempton, 1985). The implication for the Barrow investigation, where the effective normal stresses were expected to be relatively low, was that $\emptyset'_r$ on pre-existing failure planes was likely to be relatively high, probably in excess of 16° to 17° and possibly as high as 20°.

Table 1. Typical Index Properties and Strength Values for the Upper Lias Clay

Reference	w_p (%)	w_L (%)	I_p (%)	Clay Fraction (%)	Activity Index	Peak Strength		Residual Strength[1]	
						c' (kPa)	$\emptyset'_p$ (°)	c'_r (kPa)	$\emptyset'_r$ (°)
Skempton (1985)	28	64	36	52	0.7	-	-	0	9.9-12.7
Chandler (1970a)	30	60	30	63	0.5	14	24	0	17-18
Chandler (1970b)	-	-	-	-	-	-	-	0	18-19
Chandler (1972)	-	-	-	53-76[3]	-	10[2]	20[2]	-	-
Chandler (1977)	-	-	-	-	-	-	-	0	10-17.5
Cripps & Taylor (1981)	-	-	-	50-60	-	-	-	0	10-20
Bridle *et al* (1985)	23	58	35	-	-	0	18-24[4]	-	-
Lupini *et al* (1981)	-	-	-	-	-	-	-	<5.4	<11.1

w_L Liquid Limit
w_P Plastic Limit
I_P Plasticity Index

Notes

(1) effective normal stress dependent (see text)
(2) completely remoulded, weathered material
(3) soliflucted material
(4) intact clay at Empingham Dam

RESERVOIR HISTORY AND BACK ANALYSIS

Reservoir history

The history of Barrow No 3 Reservoir has been comprehensively investigated and documented (Watson Hawksley, 1982) and the present investigation owes much to the meticulous detail of this work.

Construction of the embankment commenced in 1887 generally using Lower Lias Clay dug from within the reservoir area. Construction proceeded only with some difficulty and a succession of substantial slips occurred over a number of years on both the inner and outer slopes. Failures appear to have occurred at random locations around the reservoir rim, although with a general bias towards the northern and eastern slopes, and took place both during construction and also during various attempts at impounding. Progressive flattening of the slopes and staged reservoir filling finally allowed top water level to be achieved some 49 years after the start of construction.

The following aspects were considered to be of particular relevance to the 1992 investigation:

- construction started in May 1887 approximately
- the form of construction was a relatively narrow puddle clay core (typical base width : height 1:4.5) with clay fill shoulders
- major remedial works at a number of locations were necessary between 1888 and 1905
- the outer slopes, particularly to the north, were progressively flattened from 1 on 3 to their present composite slope of between 1 on 5 and 1 on 8 with a rockfill toe
- the inner slopes were progressively flattened from 1 to 2½ to 1 on 4 and subsequently to 1 on 6, and, in places, to 1 on 8 with a rockfill toe
- little further movement seems to have occurred since approximately 1910
- seepage rather than stability was the main concern between 1933 and 1967; however, grouting of sections of the core in 1971-72 appeared to solve the problem, although further leakages occurred in other areas as recently as 1995 and further grouting was put in hand in 1996

the slips are noted in the records as having moved slowly; this suggests non-brittle soil behaviour which would generally be considered as being characteristic of clay fill or relict slips rather than of shearing in undisturbed foundation strata.

Opportunities for residual strength determination

The seismic safety evaluation (Swannell, 1996) required, in particular, that the highest section of the outer slope of the embankment be evaluated. This section was identified by Watson Hawksley as an area where repeated slippages had occurred and the outer slope progressively flattened. Therefore, any seismic evaluation of the reservoir had to take account of the presence of pre-existing slip (shear) surfaces at this section. Thus it was apparent that the determination of reliable estimates of the residual strengths of the clay fill and / or the foundation clays was fundamental to the study.

In addition to laboratory determinations of residual strength it was recognised that the history of slipping of the embankments presented an opportunity to determine an estimate of average residual strength by back-analysis of one or more well defined existing slips.

A review of the slips identified by Watson Hawksley indicated that there were three principal occurrences which had the potential to yield values for residual strength. These were:

i) first time slips which failed as a result of movement on a foundation solifluction shear which could be assumed to be at or close to residual strength

ii) reactivation, meaning renewed movement at a factor of safety of 1.0, of the slips defined in (i)

iii) reactivation of other slips not necessarily associated with solifluction shears, but where the shear strength could be assumed to be at or close to residual strength.

It was considered reasonable to assume that the residual strength of the Lower Lias Clay would be approximately the same whether arising from either:

- solifluction shearing
- extensive shearing of upper layers of weathered foundation clays, or
- extensive shearing through remoulded clay fill.

The literature review had indicated the effective stress dependency of the residual strength of the Lower Lias Clay, but each of the above situations was considered to be at low normal effective stress and therefore the residual strengths were expected to be approximately equivalent, although there was obviously some scope for variation and possible inaccuracy.

It was considered that there was sufficient information available from the Watson Hawksley report to attempt the back-analysis of two sections as follows:

i) Ch 6400 inner slope - a slip which occurred in 1895, but which was almost certainly a reactivation of a previous slip in late 1887

ii) Ch 1250 inner slope - an 1893 'first time' slip of the completed embankment on a solifluction shear surface with a reactivation in 1895 after remedial works (probably following drawdown after impounding).

Ch 6400 is located at the eastern end of the north embankment and is a section studied by Watson Hawksley for different purposes in 1982. Ch 1250 is on the south east section of the embankment in an area where it appears most likely that a solifluction lobe extends below the embankment. This section was also investigated by Watson Hawksley, but not analysed at that time.

<u>Back analysis at Ch 6400</u>
At Ch 6400 a slip occurred in 1887 in a 12m high section of the embankment. This resulted in the inner slope being flattened. New fill and core material were placed in the upper levels of the embankment where this had been disrupted. The slope failed again in 1895 by what could reasonably be assumed to have been renewed movement on the 1887 slip surface. The section is shown on Fig. 1.

The 1895 slip may have resulted from drawdown after partial impounding to an unknown level (Watson Hawksley, 1982). Therefore, as a matter of engineering judgement, the pore pressure ratio (r_u) was probably high at the time of failure, that is 0.4 to 0.5 approximately.

A parametric study carried out using Sarma's Method (Sarma, 1975) for non-circular slip surfaces indicated that the following combinations of average $\emptyset'_r$ and r_u resulted in a factor of safety of approximately 1.0 (zero cohesion being assumed throughout):

r_u	=	0.35,	$\emptyset'_r$	=	15°
r_u	=	0.45,	$\emptyset'_r$	=	18.5°
r_u	=	0.5,	$\emptyset'_r$	=	20°

The most likely conclusion from these results was therefore considered to be that an average value of $\emptyset'_r$ would be within the range 18° to 20°.

This was considered to be generally consistent with the results suggested by tests on the Lower Lias Clay at Evesham, for example, (Chandler *et al,* 1973) because the mean normal effective stress on the failure surface was calculated to be approximately 35 kPa, which is a relatively low value for earthworks of this scale.

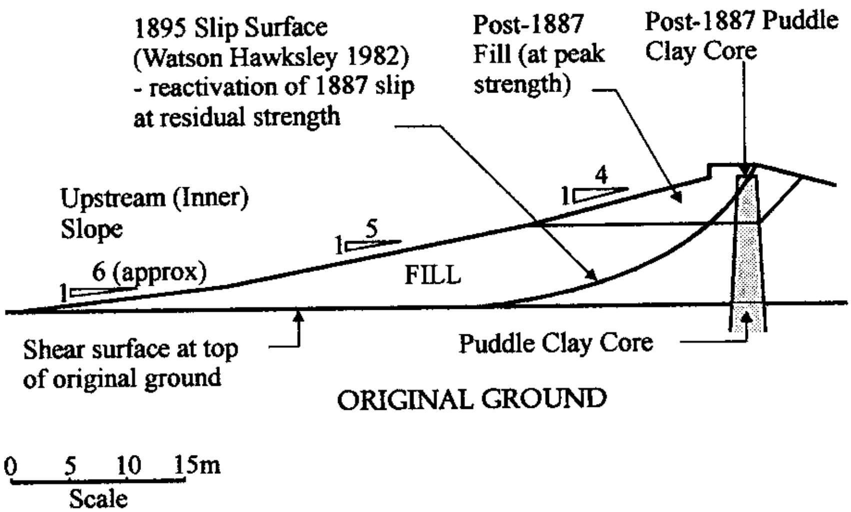


Fig. 1. Back analysis section at Ch 6400.

<u>Back analysis at Ch 1250</u>

At Ch 1250 a 'first time' slip occurred in 1893 on the inner slope of the reservoir 'after the reservoir water level had been drawn down to 19 feet below top water level' (Watson Hawksley, 1982). The embankment had been completed to full height at the time of the slip (see Fig. 2).

A parametric study was carried out on this slip using Sarma's Method assuming

i) peak strength in the fill was c' = 10 kPa, $\emptyset'_p$ = 28°, being the average from the 1981 site investigation;

ii) fully softened strength existed in the foundation colluvium with a strength of c' = 0, $\emptyset'$ = 28°;

iii) residual strength existed on the solifluction shear surface below the colluvium.

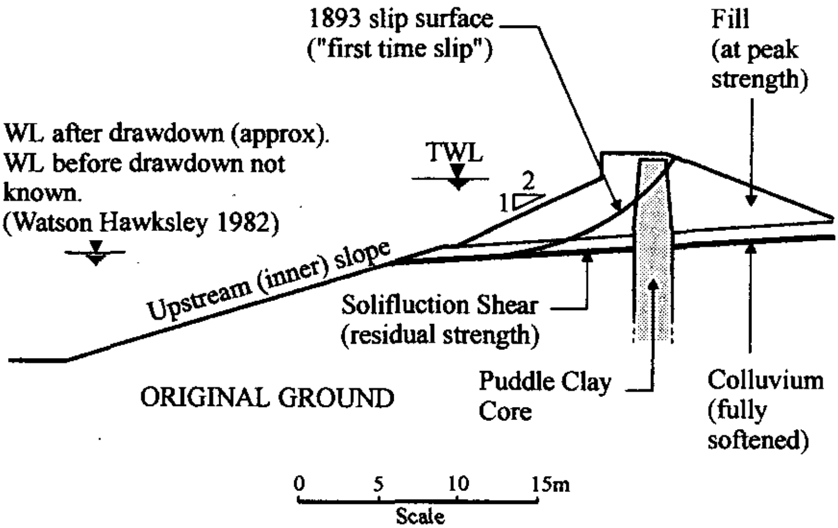


Fig. 2. Back analysis section at Ch 1250.

The results indicated that for r_u = 0.45, $\emptyset'_r$ was approximately 22°. This was recognised as being a relatively high value, but possibly not unreasonably so because the embankment at this location is low (5m approximately) and the effective normal stresses small (12 kPa approximately). The residual strength could therefore be expected to be high on the basis of the stress dependency observed elsewhere.

After the 1893 slip the slope was flattened, the crest zone rebuilt and a rockfill toe added. Nevertheless the slope failed again in 1895, probably following drawdown after impounding (there was a high concentration of slips at this time). Drawdown was at least to the level of the top of the new rockfill berm. Other details of the drawdown are not known, but it is considered reasonable to assume that pore pressures were high at the time of failure, that is r_u = 0.4 - 0.5.

Parametric studies carried out on this further slip also indicated that $\emptyset'_r$ could be approximately 22° for an r_u value of 0.45. However, it was noted that for r_u of 0.5 and the inclusion of a small amount of cohesion, $\emptyset'_r$ = 20° also gave a factor of safety of approximately 1.0. It was thought possible that some cohesion could have been regained on the slip plane as a result of the remedial works carried out after the 1893 slip. It was also recognised that the assumed slip geometry may have been more complex than that assumed, particularly in the toe area. Therefore, in general, the results of this analysis were considered to be slightly less conclusive than the back analysis at Ch 6400.

Summary of back analysis results

Although the results of the back analyses cannot be considered to be conclusive, or necessarily accurate, because of the uncertainties existing (for example over the exact slip geometries and the pore pressures existing at the time of failure) the results appear to be generally consistent with other published data on the Lower Lias Clay and suggest that:

i) at Ch 6400 the average strength on the existing slip surface is probably in the range $c'_r = 0$, $\phi'_r = 18\text{-}20°$ at a mean effective normal stress of approximately 35 kPa

ii) at Ch 1250 the average strength on the existing slip surface may exceed $\phi'_r = 20°$, possibly 22°, (assuming $c'_r = 0$) at a very low mean effective normal stress of approximately 12 kPa

iii) the residual strength envelope for the Lower Lias Clay at the Barrow site is non-linear as reported elsewhere

iv) the minimum average value of ϕ'_r for the Barrow No 3 embankments appears to be approximately 18°; lower values of the order of 15° would have required r_u values in the existing slips of 0.35 or less which are considered to be unreasonably low in the known circumstances.

SITE INVESTIGATION

Fieldwork

The site investigation was targeted on the outer embankment slope at the highest section (Ch 5600). Four cable percussion boreholes were sunk to penetrate original ground in the Lower Lias Clay. The maximum depth of hole was approximately 19m.

In addition to sampling and standard penetration testing, 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. The piezometers gave information required for the subsequent stability analyses.

Laboratory testing

Laboratory testing comprised index testing, undrained triaxial tests with pore pressure measurement, direct shear strength tests, multi-reversal shear strength tests and ring shear tests on samples of the Lower Lias Clay taken from both the fill zones and from original ground. All testing was performed in commercial laboratories and in accordance with the appropriate British Standards.

Test results

The average laboratory test results are summarised in Table 2. The index test results are plotted on Fig. 3. The relatively high plasticity of the puddle clay core material can be noted. Otherwise the Lower Lias Clay at the site is confirmed as being of medium to high plasticity.

Table 2. Summary of Average Laboratory Test Results for the Lower Lias Clay

Sample Origin[(1)]	w (%)	w_p (%)	w_L (%)	I_p (%)	Bulk Density (Mg/m^3)	Peak Strength		Residual Strength	
						c' (kPa)	θ'_p (°)	c'_r (kPa)	θ'_r (°)
PCC	36	25	66	41	-	-	-	-	-
C	37	24	62	38	1.84	0	33	-	-
F	32	25	55	30	1.84	8.5	25	0	19[(2)]
OG	26	25	56	31	1.96	8.5	25	0	16[(2)]

PCC - puddle clay core
C - colluvium
F - fill
OG - original ground

Notes

(1) all samples are derived from the Lower Lias Clay

(2) results are given for a normal effective stress of 30 kPa; values are stress dependent

The undrained strength was also measured and was found to have a value of 188 kPa for material from original ground (one test only) and an average of 51 kPa for samples from the fill (six tests).

The natural moisture content of the clay in the original ground was noticeably less than in the fill and colluvium derived from the same material, as was to be expected.

Of most direct relevance to the investigation were the results of the effective stress strength tests. Fig. 4 shows the peak strength failure envelope derived from triaxial shear tests (from samples taken from fill and original ground) with the results of direct shear tests also shown. There was found to be reasonable agreement between the two forms of testing. A peak shear strength of c' = 8.5 kPa and $\emptyset'_p$ = 25° was considered to be a representative average result.

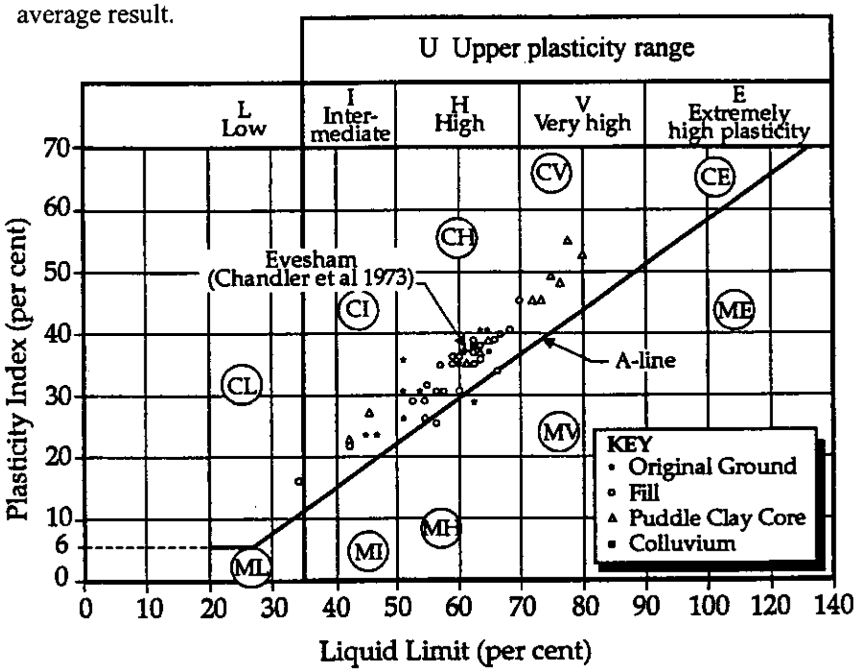


Fig. 3. Plasticity chart for the Lower Lias Clay at Barrow No 3 Reservoir

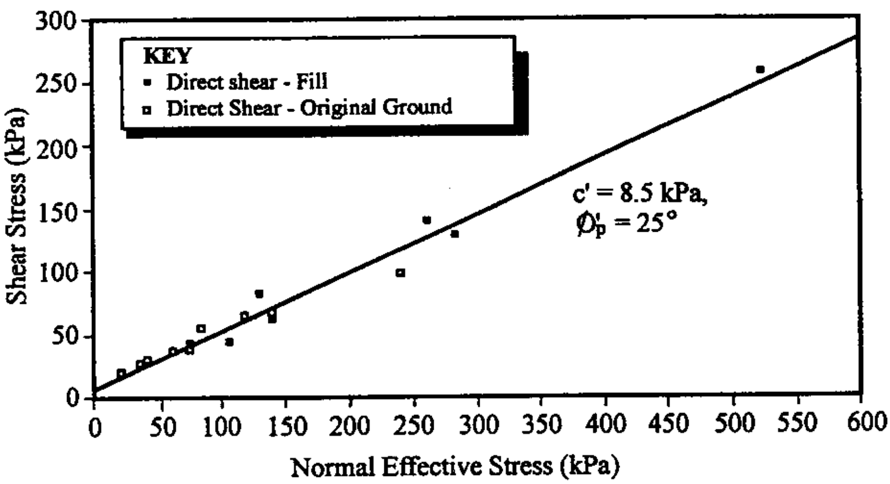


Fig.4. Peak shear strength envelope for the Lower Lias Clay at Barrow No 3 Reservoir

Fig. 5 shows the results of residual strength testing by multi-reversal shear box and by ring shear apparatus. Although there was reasonable agreement between the two methods of testing, a discernibly higher residual strength was exhibited by samples taken from the fill compared with that for samples taken from original ground. For example, from Table 2 it can be seen that at an effective normal stress of 30 kPa, ϕ'_r = 19° in the fill and 16° in original ground. The fill at Barrow was observed to contain hard inclusions of mudstone, limestone and other material which may not occur at other sites and may be an explanation for the difference. The non-linearity of the residual strength envelopes was discernible, although in retrospect it is regretted that the opportunity was not taken to carry out more tests at very low effective stresses. However, the results of the back analyses could be used to supplement the data at low stress levels for practical purposes.

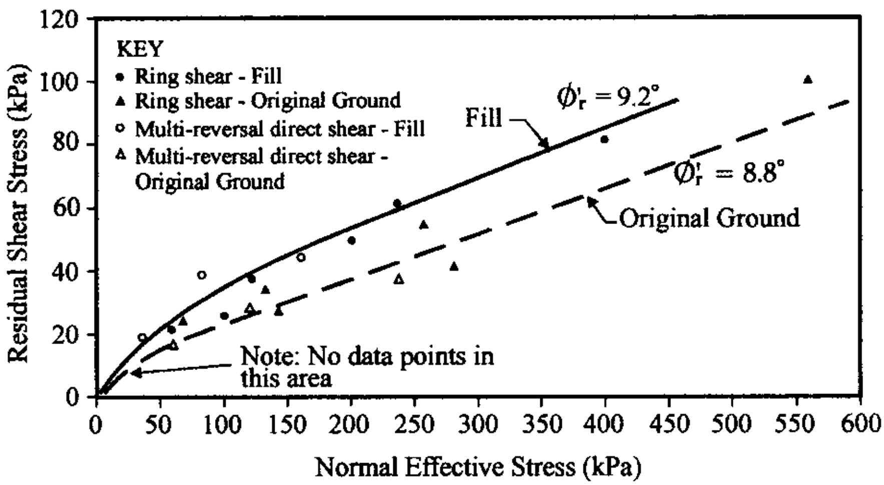


Fig. 5. Residual shear strength envelopes for the Lower Lias Clay at Barrow No 3 Reservoir

SUMMARY OF SOIL STRENGTH STUDIES

Peak shear strength

The peak shear strength of the Lower Lias Clay at the Barrow site was found to be c' = 8.5 kPa, ϕ'_p = 25° by laboratory testing. This compared reasonably well with c' = 10 kPa, ϕ'_p = 28° determined in the 1981 investigation (Watson Hawksley, 1982), and with c' = 5 kPa, ϕ'_p = 27° measured at Evesham (Chandler *et al,* 1973).

Although available data are limited (Table 1), the peak strengths of the Lower and Upper Lias Clays appear to be similar for engineering purposes.

Residual strength

The residual strength envelope for the Lower Lias Clay at Barrow was found to be non-linear which is consistent with previous findings by Chandler *et al* (1973) and a general characteristic of medium to high plasticity clays observed by Skempton (1985). The residual strength of samples taken from original ground were found to be slightly lower than those from the fill (Fig. 5). However, it should be noted that the fill at Barrow was observed to contain material other than clay.

Plots of ϕ'_r against normal effective stress (σ'_n) based on the combined results from laboratory tests and back analysis are shown on Fig. 6. The results from Evesham (Chandler *et al*, 1973) can be seen to show good agreement with the curve representing original ground. Above a normal effective stress of approximately 200 kPa a constant value of $\phi'_r = 9°$ appears appropriate for all samples, but at lower stresses the strength rises significantly and appears to vary with the source of material, that is whether from fill or original ground. It is suggested that the 'original ground' curve is probably generally representative of the Lower Lias Clay. The 'fill' curve may be site specific.

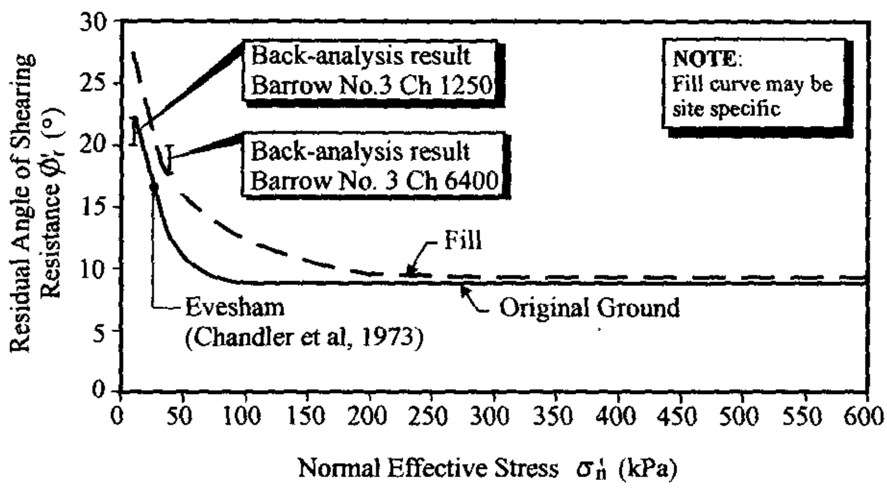


Fig. 6. Suggested variation of residual angle of shearing resistance with normal effective stress in the Lower Lias Clay for analytical purposes.

CONCLUSION

The investigation of the Lower Lias Clay at Barrow No 3 Reservoir, in connection with a seismic safety evaluation of the reservoir, allowed some key engineering characteristics of the material to be studied. Of necessity the investigation was targeted at particular parameters. However, there was an unusual opportunity to combine laboratory testing with back analysis of well documented historical slips and results, although fairly limited, from other sites, to arrive at reasonably reliable values for analytical purposes. Of particular relevance was the confirmation, for practical purposes, of the non-linearity of the residual strength failure envelope. Because of the generally good agreement observed between the various approaches it is suggested that the plot of θ'_r against normal effective stress for samples from original ground (Fig. 6) is probably sufficiently accurate for general application elsewhere in the Lower Lias Clay. However, it should be noted that local hard inclusions of mudstones and limestones may affect the average strength at specific sites.

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 investigations and of Mr R J Matthews and the staff of Geotesting Services Ltd for undertaking the ground 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.

REFERENCES

Bridle, R. C., Vaughan, P. R. & Jones, H. N. (1985). Empingham Dam - design, construction and performance. Proc. Instn. Civ. Engrs, Part 1, vol 78, pp247-289.

Chandler, R. J. (1970a). The degradation of Lias Clay slopes in an area of the East Midlands. Quarterly Journal of Engineering Geology, vol 2, No. 3, pp161-181.

Chandler, R. J. (1970b). A shallow slab slide in the Lias Clay near Uppingham, Rutland. Geotechnique, vol 20, No. 3, pp253-260.

Chandler, R. J. (1972). Lias Clay : weathering processes and their effect on shear strength. Geotechnique, vol 22, No.3, pp403-431.

Chandler, R. J., Pachakis, M., Mercer, J. & Wrighton, J. (1973). Four long term failures of embankments founded on areas of landslip. Quarterly Journal of Engineering Geology, vol 6, Nos. 3 and 4, pp405-422

Chandler, R. J. (1977). Back analysis techniques for slope stabilisation works - a case history. Geotechnique, vol 27, No. 4, pp479-495.

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. Watford, UK. Building Research Establishment.

Cripps, J. C. & Taylor, R. K. (1981). The engineering properties of mudrocks. Quarterly Journal of Engineering Geology, vol 14, No. 4, pp325-346.

Hawkins, A. B. (1973). The geology and slopes of the Bristol region. Quarterly Journal of Engineering Geology, vol 6, Nos. 3 and 4, pp185-206.

Lupini, J. F., Skinner, A. E. & Vaughan, P. R. (1981). The drained residual strength of cohesive soils. Geotechnique, vol 31, No 2, pp181-213.

Sarma, S. K. (1975). Seismic stability of earth dams and embankments. Geotechnique, vol 25, No 4, pp743-761.

Skempton, A. W. (1985). Residual strength of clays in landslides, folded strata and the laboratory. Geotechnique, vol 35, No. 1, pp1-18

Swannell, N. G. (1996). Barrow No 3 Reservoir - a case history of simplified seismic safety evaluation. Proc. 9th British Dam Society Conference, York, pp. 248-257.

Watson Hawksley (1982). Barrow No 3 Reservoir. Review of the history of the embankment dam with the soils investigation of 1981. Report to the Bristol Waterworks Company. Unpublished.

Investigation of a possible sinkhole at Walshaw Dean Upper Dam

A C ROBERTSHAW, Yorkshire Water Services Ltd
M S ATKINSON, Soil Mechanics
P TEDD, Building Research Establishment

SYNOPSIS. Walshaw Dean Upper dam has a puddle clay core and a deep narrow clay filled cut-off trench. It is nearly one hundred years old. In January 1997 a hole, approximately 1 m cube, was discovered on the upstream slope near the crest of the dam. As internal erosion of the clay core and clay filled cut-off trench had occurred at the two dams immediately downstream which are of similar construction, there was concern that the hole in the upper dam was also due to internal erosion. This paper reviews the problems and remedial works at the middle and lower dams and describes the current investigations undertaken at the upper dam to determine the cause of the hole.

INTRODUCTION

Walshaw Dean Upper dam is one of three dams designed by G H Hill and constructed between 1901 and 1907. The dams are located 5 miles north-west of Hebden Bridge in West Yorkshire. All three dams are of similar construction with a central puddle clay core, 2.6 m wide at the top with batters of 12:1 vertical to horizontal down to rockhead below which there is a 3 m wide deep clay filled cut-off trench with a concrete shoe. The upstream embankment slope is 1:3 and the downstream slope is 1:2. Selected fill described as "clayey or more adhesive material" was placed close to the core and "driest or most stony material" was placed in the outer zones. In view of the similarities in construction, it is useful to review the internal erosion and settlement that occurred at the middle and lower dams.

On first filling of the reservoirs there were no significant problems with the upper dam but large leakage occurred at both Walshaw Dean Middle and Lower. At Walshaw Dean Middle substantial settlement of the embankment occurred mainly in the vicinity of the valve shaft, (see Fig. 1). The leakage was stemmed by grouting between 1911 and 1915 (Barnes, 1927, Wood, 1946) and the reservoir was brought into commission in 1915. The grouting was confined to two areas: where the outlet tunnel goes through the core and where a vertical step of 9.1 m occurred in the base of the trench. These undesirable features could have led to differential settlement of the puddle clay resulting in very low stresses and the possibility of hydraulic fracture causing a leakage path and subsequent internal erosion.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

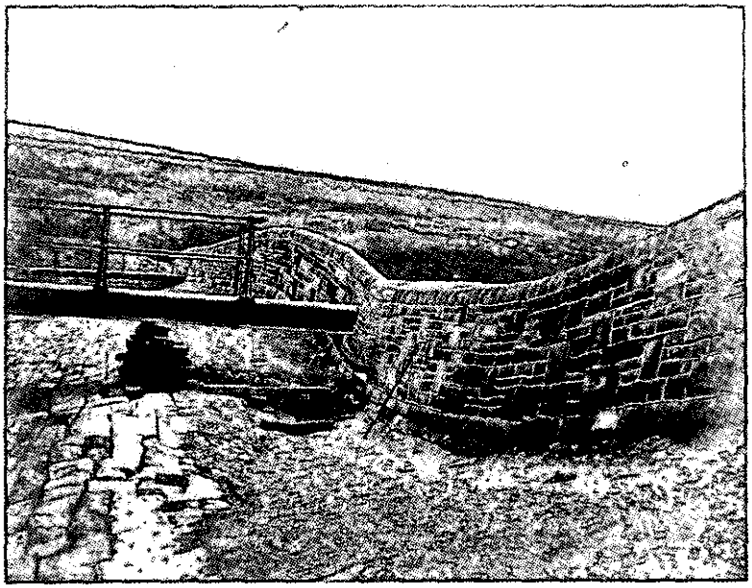


Fig. 1. Settlement of Walshaw Dean Middle dam on first filling

Fig. 2. Subsidence of the upstream fill of Walshaw Dean Middle in the 1930's

About 20 years later small subsidences (see Fig. 2) occurred on three separate occasions on the upstream side of the middle embankment just above top water level (Wood, 1946). Boreholes revealed that the puddle clay was of inferior quality, containing large quantities of peat, stone and timber fragments and in the lower part of the cut-off there was no trace of puddle clay. The water levels in the boreholes downstream of the core were only 0.6 m below that of the reservoir and fluctuated in sympathy with it. The remedial measures were carried out in two phases. Cement grout was injected into the upstream foundation strata and the fill upstream of the core, the works being limited to the central section of the dam. In drilling for the second phase of injection, the central section of the core was found to be interlaced with grout. Elsewhere the drilling water was lost entirely at the junction of the puddle clay and the concrete shoe and in a number of holes it was also lost at the level of the original ground surface. The grouting proved hydraulic connection between some adjacent holes put down through the puddle clay. All the above observations are characteristic of internal erosion.

Extensive gravity grouting of the rock immediately upstream of the cut-off trench was also carried out during 1911 to 1915 at Walshaw Dean Lower to stem the leakage (Barns, 1927). Investigations in 1980 revealed softening of the puddle clay below original ground level, with evidence of water paths around which the clay was very soft. Remedial grouting carried out in 1982 involved cement grouting of the rock foundation immediately downstream of the cut-off trench as well as cement-bentonite grouting of the core and cut-off.

Until the appearance of the hole in 1997, no significant problems had occurred at the upper dam and no major remedial works have been carried out to control leakage or internal erosion, although the core has been raised and improvements have been made to the spillway (Harrison & Drabble, 1996). The lack of long term problems reflected the absence of construction problems; the upper embankment made consistently the best progress (Bowtell, 1979).

DISCOVERY OF THE POSSIBLE SINKHOLE

In January 1997, a hole was discovered beneath the upstream face pitching and wavewall near the crest of the upper dam. Pitching stones had fallen into a hole of approximately 1 m cube, see Fig. 3. An AR Panel Engineer was informed and Yorkshire Water Services emergency procedures were followed. The reservoir was drawn down to 2 m below top water level.

The hole was located 28 m from the eastern end of the dam. Although the embankment at this location is only 11 m high, the 3 m wide cut-off trench extends to 25 m below original ground level, see Figs 4 and 5. The history of leakage, internal erosion and associated settlement at the two lower dams naturally raised concerns that the hole was a sinkhole caused by internal erosion. Unlike the middle dam where the initial problems of settlement and leakage were associated with the outlet tunnel and a 9 m vertical step at the base of the cut-off trench, no such features are present at the location of the hole on the upper dam. Also, the repeated subsidences that occurred at Walshaw Dean Middle in the 1930's covered a larger area (1.8 to 2.4 m) than the very localised hole at the upper dam. However, cracking by hydraulic fracture of puddle clay in deep narrow cut-off trenches is likely because of stress reduction in the clay caused by arching. Both measurements (Charles & Watts, 1987) and analysis (Dounais et al, 1996) indicate that low stresses exist in deep narrow cut-offs. From case records, including evidence from Walshaw Dean Lower, Skempton (1989) suggested that if a hydraulic gradient across the core exceeded 6, hydraulic fracture is likely to occur. This value is exceeded in the 3 m wide cut-off trench from 18m below top water level (20 m below crest level) at the location of the hole, assuming the full reservoir head acts against the upstream side of the cut-off.

Fig. 3. Hole on the upstream slope of Walshaw Dean Upper dam, 1997

INVESTIGATIONS AT THE UPPER DAM

An investigation was commissioned to determine whether the hole had been caused by internal erosion and to assess the safety of the dam. The investigation began with cone penetration tests in the core and cut-off, followed by continuous sampling by cable percussion and the installation of piezometers and earth pressure cells. The investigations were completed with a shallow trial pit 2.5m deep at the location of the hole into which two magnet settlement gauges and a piezometer were installed.

Cone penetration tests

Static cone penetration tests (CPTs) were carried out by Fugro Ltd from the crest of the dam into the core and the cut-off trench at 13 locations immediately downstream and either side of the hole. An additional test was carried out in the centre of the dam, remote from the hole. The test locations are shown in Fig 4. Cone resistance values were generally between about 0.4 and 0.8 MPa from which the undrained shear strengths were derived to be between about 25 and 55 kPa, (assuming the undrained shear strength is approximately 1/15 of the cone resistance). However, there were occasional larger fluctuations.

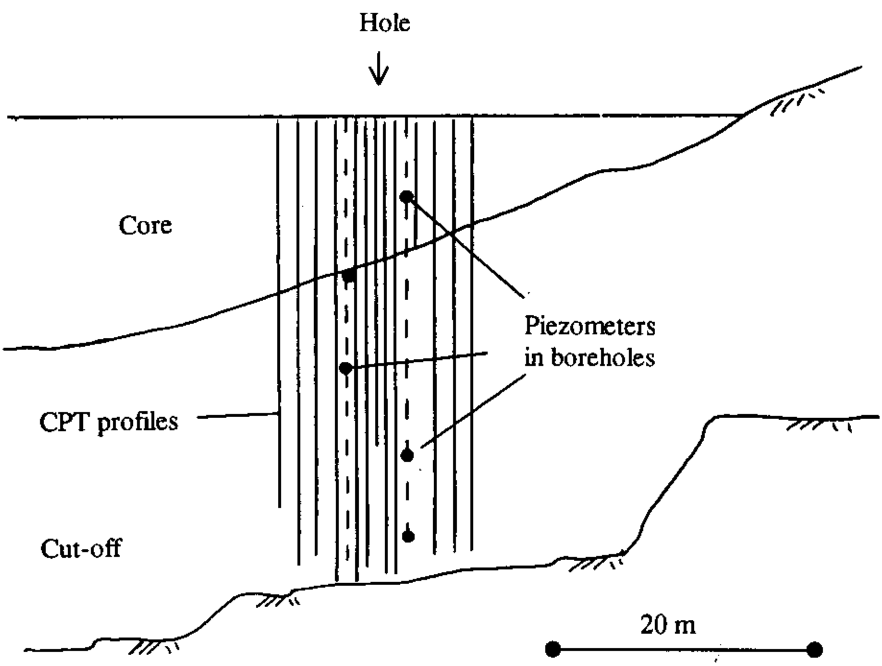


Fig. 4. Longitudinal section showing location of CPT and piezometers in the core and the cut-off trench

Derivation of soil type from the friction ratios indicated the soils to be generally soft to firm CLAYS with some sandy layers. In three of the profiles immediately downstream of the sinkhole, larger cone resistance up to approximately 7 MPa were measured between depths of 18 and 23 m where the soil types have been interpreted as:

- stiff sandy or very sandy CLAY
- medium dense silty SAND
- medium dense silty clayey SAND

Large cone resistances were also measured in another profile to one side of the hole, between depths of about 17 to 20 m, and the material was again interpreted as loose to medium dense silty SAND with some clay layers. Between 20 and 22 m depth the cone resistance fell to below 0.4 MPa indicating the puddle clay here to be very soft. These results appear to show that a local but relatively thick sandy layer exists in the cut-off trench and imply the possibility that internal erosion has occurred. This contrasts to the CPT investigation undertaken at Walshaw Dean Lower in 1980 where cone resistance went down to near zero in the cut-off trench in a number of profiles.

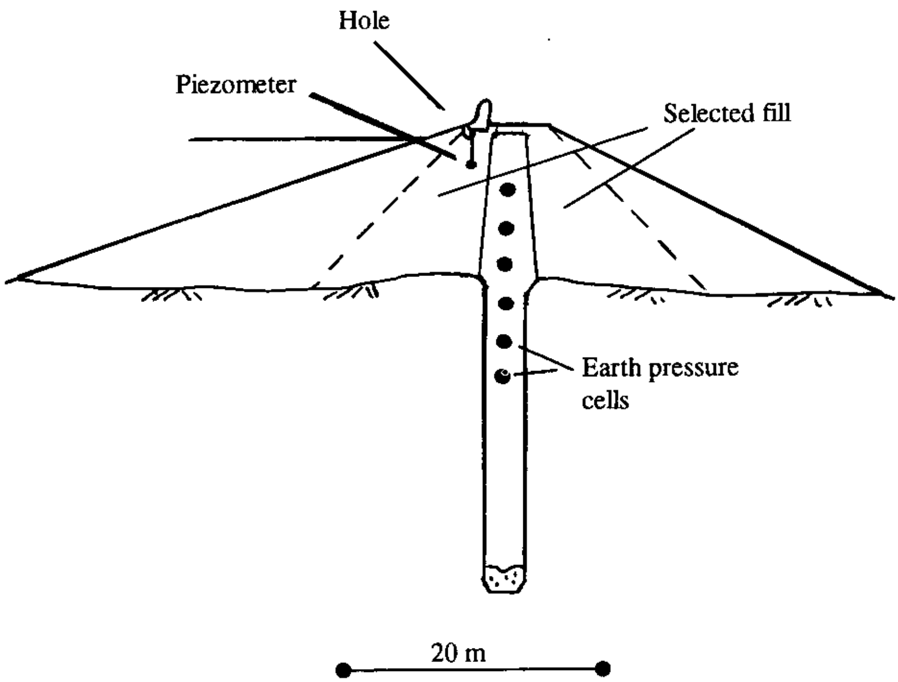


Fig. 5. Cross-section of dam at location of the hole showing positions of the pressure cells, and piezometer and magnet settlement gauge in the trial pit

Borehole investigation

A borehole investigation of the clay core and cut-off trench was carried out by Soil Mechanics using light percussion drilling. Nominal continuous sampling was undertaken using a 100 mm open drive sampler. Three boreholes were put down close to the hole and three in the middle of the dam. Except for the first borehole near the middle of the dam all boreholes were cased throughout their depth. The boreholes were drilled dry down to a depth of approximately 15 m. Below this depth, the holes were filled with water to a nominal depth of 2 m below the reservoir level to prevent squeezing in of the clay at the base of the borehole. In filling the boreholes with water consideration was given to the possibility of causing hydraulic fracture of the clay in the cut-off trench where stresses could be expected to be low. Throughout the drilling there was no evidence of loss or gain of water, and negligible changes were measured in the water levels in the boreholes when left overnight. These observations indicate that the clay surrounding the boreholes has a low permeability.

Puddle clay descriptions and laboratory test results

The first borehole was drilled near the middle of the dam to provide reference samples of clay well away from the possible sinkhole. Unfortunately samples from this borehole, particularly near to its base at a depth of 10 m, consisted of lumps of very soft clay in a slurry matrix and understandably caused concern. The poor quality of the samples from this preliminary borehole was possibly related to the method of drilling without casing. Samples from the remaining boreholes, including a further two close to the middle of the dam, were generally of good quality puddle clay from both the core and the cut-off trench and showed no sign of erosion. Down to a depth of about 15 m the puddle clay was described as soft, varying to firm, generally slightly sandy clay with a little gravel. Below this level there was a general absence of gravel and only infrequently was the clay described as slightly sandy. There were occasional sand partings throughout the puddle clay but they are noticeably less frequent in the less sandy clay of the lower layer.

The Atterberg Limits indicate similar plasticities for the puddle clay from the central and hole locations. The data lie around the A line with the majority from above about 15 m depth being CH and MH (clays and silts of high plasticity respectively) whilst those from deeper than about 15 m are predominantly CI with some MI values (clays and silts of intermediate plasticity). The change confirms the visual description of the cores and is possibly associated with a different source of puddle (Bowtell, 1979). Clay from the first borehole with soft clay was predominantly of intermediate pasticity. The scatter and values of the index properties of the clay at Walshaw Dean Upper are typical of those measured at other Yorkshire dams and are not considered to be a cause for concern.

Trial pit investigation

Lack of evidence of internal erosion of the clay in the core or the cut-off from the borehole investigation led to the hole being investigated more thoroughly. A trial pit was excavated at the location of the hole to a depth of approximately 2.5 m below the pitching (3.7 m below the crest) to determine if there was any evidence of erosion extending below its base. It was extended over a 3 m width to include an area which would not have been affected by the hole.

The excavated fill generally consisted of soft to firm sandy clay with some coarse gravel. A timber, 75 mm by 220 mm, running the full length of the trial pit was found approximately 1m below the pitching. The very limited evidence of possible erosion comprised soft or soft to firm lumps of stony clay in a slurry matrix, a slurry infilled fissure and a short "pipe", 110 mm long by 10 to 12 mm diameter, infilled with clay slurry. Both the fissure and the "pipe" appeared to be isolated features and may well have been formed during the placing of the fill. There was no evidence of erosion below the base of the hole.

To monitor any further settlement at the base of the hole a magnet settlement gauge, with three magnets and piezometer was installed in the trial pit as it was back filled. A control settlement gauge was installed in the fill approximately 2 m away from the hole.

PIEZOMETER AND PRESSURE CELL OBSERVATIONS

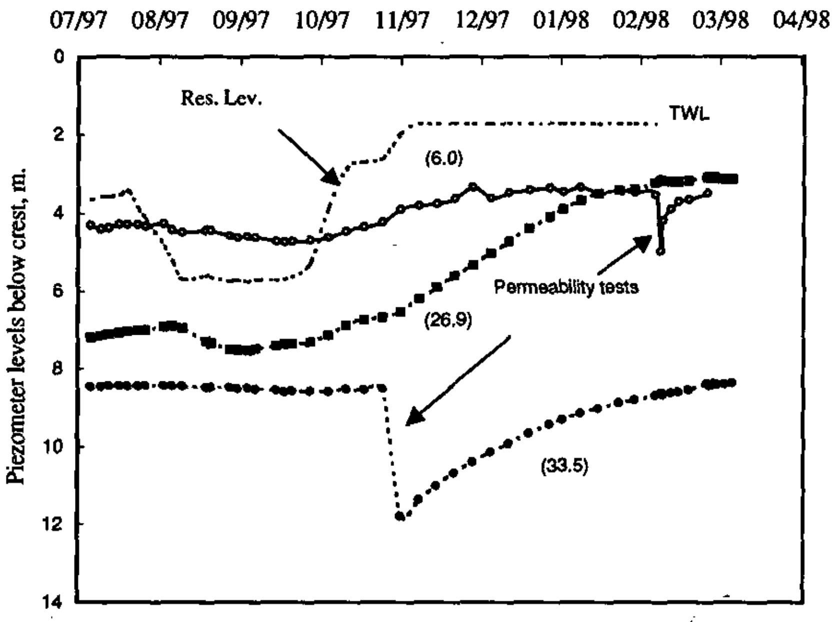
A total of eight standpipe piezometers were installed in boreholes in the core and cut-off trench. In addition one piezometer was installed in the trial pit in the upstream fill. Figure 4 shows the location of the piezometers close to the hole. Any conclusions made from the piezometer observations must take into account that the piezometers were installed in boreholes 2 m either side of the hole. Boreholes for piezometers were not drilled immediately downstream of the hole because of the close proximity of the CPT holes.

Susceptibility to hydraulic fracture can be investigated by measuring in-situ total stresses in puddle clay cores (Charles & Watts, 1987). For hydraulic fracture or cracking of the clay to occur, the hydraulic head due to reservoir pressure must be larger than the total stresses. To measure the earth pressures in the clay core and cut-off trench, six BRE miniature earth pressure cells (Watts & Charles, 1988) were installed in a borehole downstream of the sinkhole at the depths shown in Fig. 5. All the cells were oriented to measure vertical stress. The action of installing the pressure cells generates excess pressure which dissipates to an equilibrium value in the following 4 to 6 weeks depending on the stiffness of the clay. In puddle clay, the over-read due to installation can generally be ignored.

Piezometer observations

Figure 6 shows the changes in levels in some of the piezometers downstream of the hole together with the changes in reservoir level. Readings have been generally taken at weekly intervals. The 2 m drawdown in August 1997 was to allow the trial pit to be carried out more safely. There was no response of the piezometer at 33.5 m depth in the cut-off to the decrease or increase in reservoir level, however there was a response in the piezometer at 26.9m which is still very deep in the cut-off trench. Lowering the water level in the piezometer at 33.5 m depth confirmed its operation, giving a permeability of 3×10^{-11} m/s.

Clearly the piezometer at 26.9 m depth had not reached equilibrium prior to the reservoir drawdown in August. It only came to equilibrium 3 months after the reservoir reached top water level, at a level only 1.4 m below top water level and slightly above the level in the piezometer at 6 m depth. The piezometer at 6 m depth has shown a slow but definite response to the changes in reservoir level. The level in this piezometer was lowered in February 1998 for a rising head permeability test.



( ) depth of piezometer tip below the crest in metres

Fig. 6. Variation in piezometer levels in the core and the cut-off trench near the hole

The equilibrium levels with full reservoir in all the standpipe piezometers downstream of the hole are shown in Fig. 7. It can be seen that except for the deepest piezometer, all the levels in the cut-off are very close to the reservoir head. These pressures are significantly higher relative to reservoir head than those measured in the cut-off trench at Walshaw Dean Lower (Charles & Watts, 1987). However, the Walshaw Dean Lower data came from pneumatic piezometers.

The level in the piezometer in the upstream fill installed at the bottom of the sinkhole has followed the changes in reservoir level. A permeability test in this piezometer indicated a permeability of 4 x 10^{-6} m/s.

Rising head permeability tests in the piezometers have indicated that the permeability at all locations is less than 1 x 10^{-9} m/s and generally much lower. However, tests have not yet been carried out in the piezometer at 26.9 m with the particularly high piezometric head. Permeability data are still being assessed.

Earth pressures observations

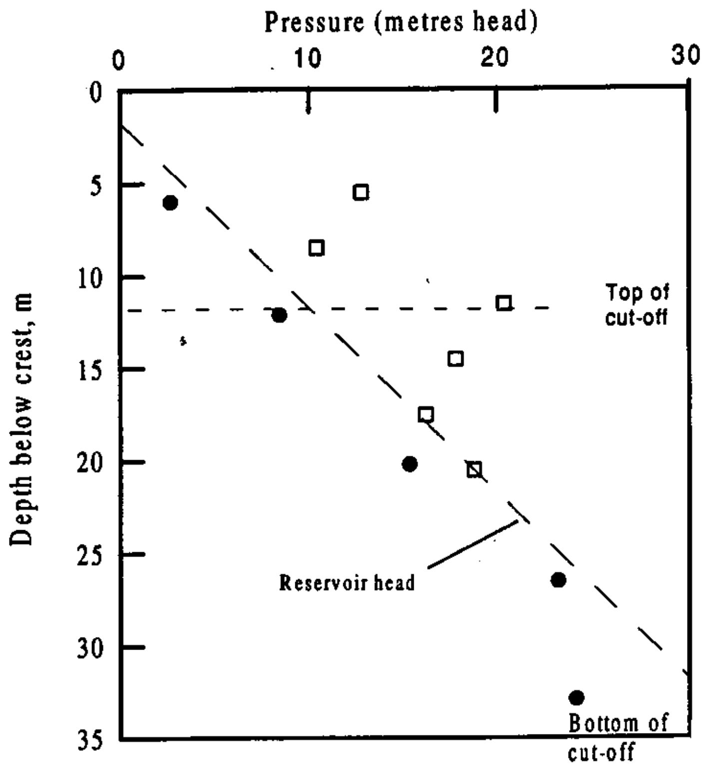
Figure 7 shows the pressures measured when the reservoir had been refilled to top water level. All the measured earth pressures except for the two lowest in the cut-off trench were larger than reservoir head. The large scatter appears to be related to the variation in stiffness and nature of the clay into which the cells were installed.

Although there have been relatively small changes in reservoir level since the investigation began it is interesting to examine the ratio of the change in total vertical stress ($\Delta\sigma_v$) in the core and cut-off to changes in reservoir head, ($\gamma_w\Delta h_w$).

Table 1. Response of pressure cells to changes in reservoir head

Cell No	Depth of cell, m	$\Delta\sigma_v/\gamma_w\Delta h_w$	
		Drawdown from 3.6 to 5.7 m	Refilling from 5.7 to 1.7 m
1	20.5	0.97	0.79
2	17.5	0.60	0.74
3	14.5	0.82	0.79
4	11.5	0.28	0.52
5	8.5	0.14	0.69
6	5.5	0.09	0.79

Note: changes in reservoir heads are relative to crest level



- Porewater pressure
- □ Earth pressure

Fig. 7 . Earth pressures and porewater pressures in the clay core and cut-off trench close to the hole

On reservoir drawdown, the response of cells in the core (cells 4, 5 & 6) was very small, however it was much larger in the cut-off and compares with observations at Walshaw Dean Lower by Charles & Watts (1987) where horizontal earth pressures were measured. The results imply that changes in reservoir head are affecting water pressures in the upstream foundation. Refilling the reservoir has had a similar response on the cells in the cut-off, however there has been a much larger response of the cells in the core than during reservoir drawdown. The response of cell No 4 was relatively small possibly because it is at the widest point of the core and therefore furthest away from the upstream side of the core.

<u>Magnet settlement gauge</u>

The settlements measured between the pitching and the deepest magnet has been less the 2 mm, which is within the accuracy of the measuring system.

DISCUSSION

Many of the internal erosion problems observed at British dams have occurred during first filling of the reservoir. Where problems have occurred at a later stage, the development of internal erosion has generally been sufficiently slow that it has been detected by routine surveillance or monitoring in time for remedial action to be taken. The hole at Walshaw Dean Upper was seen following works to the spillway during 1996 which involved traffic across the crest and considerable vibration from sheet piling works. It is likely that the hole had existed for a long time and was being bridged by the pitching. Prior to the investigation, it seemed likely that internal erosion was the cause of hole because of its location adjacent to the core and its limited extent. However, wave action, the increase in reservoir operation in recent years and the recent remediation works were all considered as possible causes.

The CPT investigation indicated the presence of a sandy horizon with clay layers of significant thickness (2 m or more thick) between 18 and 23 m below crest level in three profiles in the area immediately downstream of the hole. The close proximity of these three holes suggested continuity of the sandy horizon between the CPT positions. However, the three boreholes drilled in the location in which continuous sampling was carried out revealed no sign of any distinct sand horizons, either at this or any other depth. The cone penetration tests are thought to be misleading in terms of implying a significant band or pocket of sand, although the reasons for this are unclear. The use of a piezocone could have given more conclusive data as a measure of permeability could have been obtained.

Based on the borehole evidence and the permeability data from the piezometers, the puddle clay appears to be of good quality with a low permeability. The trial pit at the location of the hole did not provide any conclusive evidence that the hole had been caused by internal erosion. There is some evidence in the form of localised softening, traces of a possible "pipe" and a slurry infilled fissure that could be interpreted as relics of a former leakage path or paths which have largely self-healed.

The relatively large piezometric heads in the cut-off could possibly be explained by the boreholes being located close to the upstream edge of the cut-off trench. If this is the case, they also imply that the cut-off is effective in terms of maintaining high pore pressure in the upstream foundation. The relatively large response of the earth pressure cells in the cut-off to changes in reservoir level also implies that pressures are being measured close to the upstream side of the cut-off. The measured earth pressures are above the hydrostatic reservoir pressure except at the two lowest cells where they are equal to the reservoir pressure. The clay here is likely to be more susceptible to hydraulic fracture. However, during the drilling of the boreholes to install the pressure cells there was no evidence of erosion in the recovered samples.

CONCLUSIONS

The cause of the hole in the upstream shoulder is not known. There was no clear evidence from the investigation that the hole in the upstream fill was a sinkhole caused by internal erosion. The clay recovered from the core and the cut-off trench immediately downstream of the hole appeared to be of good quality. If it has been caused by internal erosion then the process appears to be very slow. The CPT results did indicate possible problems in the cut-off but this was not substantiated by the borehole investigation. The piezocone should be considered as an investigative tool in clay cores as crucial permeability data can be obtained.

Although further investigation has been considered, the AR Panel Engineer has recommended that the various instruments should be monitored together with vigilant surveillance of the dam, and that the results should be reviewed by an AR Panel Engineer in 1999.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the permission of Yorkshire Water Services Ltd to publish this paper. The opinions expressed here, however, are those of the authors and are not necessarily those of Yorkshire Water Services Ltd or the Panel AR Engineer responsible for the reservoirs concerned.

REFERENCES

Bowtell H D (1979). Reservoir railways of the Yorkshire Pennines. Oakwood Press, Blandford.

Barnes A A (1927). Cementation of strata below reservoir embankments. Transactions of the Institution of Water Engineers., vol 32, pp42-68

Charles J A & Watts K S (1987). The measurement and significance of horizontal earth pressures in the puddle clay core of old earth dams. Proceedings Institution of Civil Engineers Part 1, vol 82, February, pp123-152.

Dounais G T, Potts D M & Vaughan P R (1966). Analysis of progressive failure and cracking in old British dams. Geotechnique, vol 46, no 4, December, pp 621-640.

Harrison P C & Drabble J (1996). Walshaw Dean reservoirs – spillway improvements. The Reservoir as an Asset. Proceedings of the 9th Confernece of the British Dam Society, York, September, pp 11-19. Thomas Telford, London.

Skempton A W (1989). Historical development of British embankment dams to 1960. Clay Barriers for Embankment Dams. Proceedings of Conference organised by the Institution of Civil Engineers. pp15-52. Thomas Telford, London, 1990

Tedd P, Claydon J R & Charles J A (1988). Detection and investigation of problems at Gorpley and Ramsden dams. Proceedings of the Conferences Reservoirs Renovation 88 Conference, Manchester, paper 5.1, pp 1-15. BNCOLD, London

Watts K S & Charles J A (1988). In situ measurement of vertical and horizontal stress from a vertical borehole. Geotechnique, vol 38 No 4 pp 619 - 626.

Wood J N (1946). Discussion; Leakage of a clay core-wall. Transactions of the Institution of Water Engineers, vol 51, pp 118-121

Kentmere – past, present and future mining subsidence

A K HUGHES, RKL ARUP, Director of Ove Arup & Partners and AR Panel Engineer, London, UK

SYNOPSIS. The paper describes how, in the authors opinion the Reservoirs Act 1975 should work in that the concerns of a Supervising Engineer were discussed informally with an All Reservoirs Panel Engineer before taking some 'formal' action with the owner. The owner demonstrated a responsible attitude and brought the statutory inspection forward. The paper goes on to describe the problems found and the remedial works carried out to bring the reservoir back to a good state of repair.

INTRODUCTION.

Kentmere Reservoir was formed by the construction of Kentmere dam in about 1845 to afford supplies of water, mainly for power purposes to certain works on the banks of the river below the dam. It is located approximately 17 kilometres North West of Kendal (Fig 1). It was constructed by Messrs Shuttleworth and Dobson to the designs of J F Bateman, although Bateman did not appear to be involved with the construction. It is owned by James Cropper Plc.

Originally it had a capacity of 280 million gallons (1.27Mm^3) and a water area of 40 acres (16 ha). The maximum height was 59 feet (18m). The dam is sited at an elevation of over 950 feet (289.76m AOD) the original top water level being 974.5 feet (297.02m AOD).

The reservoir now has a capacity of 220 million gallons, (1Mm^3) up to a top water level of 295.3m AOD when the surface area is approximately 0.16 sq.km. The reservoir was the only one of five which were allowed to be constructed on the Rivers Sprint, Mint and Kent in the 'Act for Making & Maintaining Reservoirs in the Parish of Kendal' (1845).

The dam has an upstream side with a slope of 3:1 and a downstream slope of 2:1. The waterproof element is provided by a puddle clay core. The embankment has a rock foundation and a 'tongue-trench' with puddle filling excavated to rock below the puddle core. Where the discharge pipe passes through the puddle wall it is supported by a masonry-arch. The puddle material is said to consist of a 'light clay permeated with vegetable fibres, and probably obtained from local drift material'. The embankment material is of local 'sammel', a drift product consisting of a large proportion of small stones and sandy material. The downstream slope is grass covered. The upstream slope is entirely protected by closely packed stone pitching set on edge.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

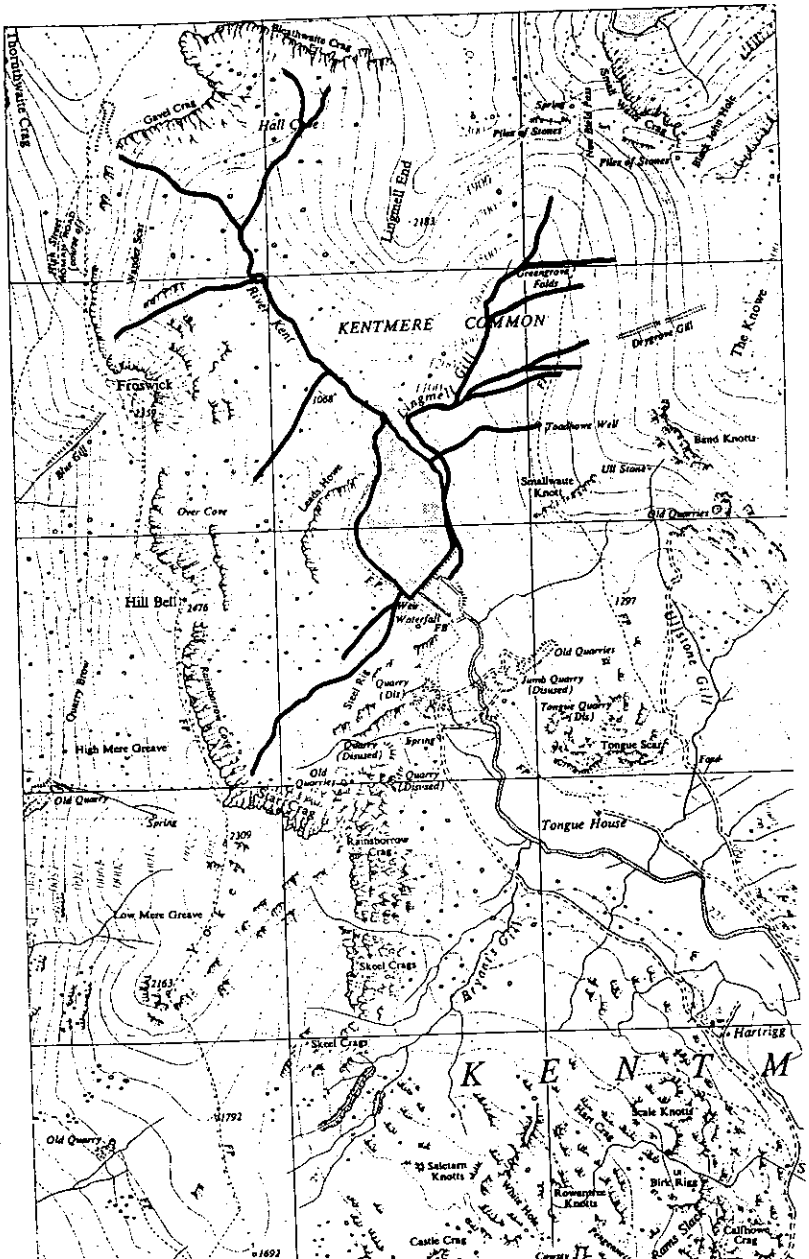


Fig. 1. Location of Kentmere Reservoir

A description of the dam construction is given in a book entitled 'Early Victorian Water Engineers' by Mr G M Binnie (Thomas Telford, 1981):

> "The clay corewall was specified to be worked in courses 8 inches thick, each layer to be completed to its full extent before the next layer was laid; 'the puddling to be done by soaking each course 12 hours in water and then cutting it lengthways and crossways and treading it, each course to be cut and worked into the one below'. The bank was to be constructed in concave layers 2ft thick on the water side and 4ft thick on the outer side of the corewall; 'the inner part of the bank to be composed of the most adhesive material and so laid, carted over and watered in dry weather that it shall become either quite or nearly watertight, the outer part of the bank to be composed of dry or stony material'.
>
> Instead of supporting the pipeline where it crossed the puddle trench on pillars, Bateman in this case specified an arch as follows: 'the pipes to be supported in passing through the puddle upon a stone platform formed by throwing an Arch across the puddle trench so that the pipes may have equal bearing throughout and can be at the same time surrounded by Puddle at the Centre of the Bank'. The detailed drawings of the arch no longer exist but it was probably narrow, not more than one stone wide. The method of construction may have been that, after bringing up the clay in the puddle trench to the level of the underside of the pipeline, timbered trenches were excavated in the puddle clay to the shape of the arch and the stones were then placed in position with the clay acting as centring. Finally, with or without removing the timbering, the slot above the arch was backfilled with puddle clay. The laying of the pipeline would then have been able to proceed (Fig 2).
>
> For the culverts, the specification reads:
>
>> The culverts to be excavated out of the solid ground a trench being cut for them no wider than sufficient to contain the masonry of the sides without backing. To be built of good flat bedded Stone or Slate set in hydraulic mortar and covered 3 feet with Puddle or pounded clay united with the centre puddle of the bank.

> The inner culvert having open communication with the reservoir was specified to have an opening of rectangular shape of 4ft high by 3ft 6in wide.
>
> It was also specified that 'the inner or mouth of the inner culvert be covered with an iron grating part of which must be moveable by means of a rod or chain worked by a Crab or other Apparatus on the top of the bank.' This would appear to have been some mechanical arrangement for raking the iron grating under water to free it when necessary of debris. With the bars running from the top to the bottom of the grating, perhaps the device consisted of hinged rods moving in vertical planes between the bars which could be lifted or lowered by a lever on the crest of the embankment."

CAUSE FOR CONCERN.

In 1995, the Supervising Engineer for the reservoir became concerned about the magnitude of the flow that he was measuring from the outlet of the reservoir. This flow had been measured for a number of years and was thought to be due to the poor sealing valve fixed to the upstream end of the drawoff facilities. The Supervising Engineer had manipulated the valve with the owner but the flows could not be stemmed. A statutory inspection was due in a couple of years time but the owner had agreed, following discussions with the Supervising Engineer, who had talked informally with an All Reservoirs Panel Engineer, to bring the statutory inspection forward.

The owner was asked to assemble data prior to the inspection. This included previous statutory inspection reports, drawings and instrumentation data. The data indicated an interesting history of problems at the site:-

It was clear from research that the dam has been subject to a large degree of modification and remedial works have been carried out on a number of occasions, indeed the dam was not considered to be in a satisfactory state almost from the date of first filling. Successive modifications moved the control of the pipework from the centre of the dam to the upstream end of the culvert through the upstream shoulder.

The minutes of the meetings of the Commissioners on various dates subsequent to the completion of the works indicate that the Reservoir was not considered to be in a satisfactory state almost from the date of construction.

On 24 April 1852 it was resolved that the embankment be raised (this may have been necessitated by the unusual settlement of the embankment during the years immediately following its construction).

On 16 September 1854 it was reported that a leak, or leaks existed through the masonry at the bottom of the well and it was resolved to have the leaks thoroughly caulked with mill waste.

A Report by Thomas Hickstead dated 13 April 1864 states that the reservoir was raised three or four years previously to make good several feet of natural subsidence and that at the time of inspection the appearance of the dam was good. The principal findings of the Report were:-

(i) leakage from the culvert carrying the discharge pipe - this leakage was stated to be increasing from time to time
(ii) the sluice gate in the valve shaft was out of order and had not been used for 13 years
(iii) a leakage into the watercourse below the discharge pipe, stated to come from outside the culvert under the centre of the embankment
(iv) a leakage under the bywash, only observed when water was passing over the waste weir
(v) the embankment was originally constructed in layers from 3ft (0.9m) to 6ft (1.83m) in depth this being criticised for a structure intended to be watertight.

Hickstead recommended that the dam should be thoroughly inspected by Mr Bateman to advise what remedial works were necessary.

Records also suggest that the original bywash constructed in 1851 failed and had to be rebuilt in 1864. In June 1864 emergency repairs were made following an 'outflow from the gauge house' which required drawdown of the reservoir, repair of the bywash and puddle clay repairs. In April 1865 Bateman seems to have been asked to draw up plans for repairs and that these plans, involving sinking a shaft, were discussed in 1865. In May/July 1868, it was resolved 'to have the leakage again stuffed with wool waste without delay'. In December 1868 the minutes state that 'repairs of reservoir uncertain until Mr Bateman's account is settled!'.

In August 1898 the penstock valve in the shaft was installed to control the supply to the 24 inch discharge pipe under the embankment.

During 1926 the embankment was again raised to its original height for the whole of its length and the overflow weir lowered 1ft and a moveable timber sill fixed upon it, controlled by a hand operated trip mechanism.

Very extensive settlement adjacent to the valve shaft was noted in 1933 and at the time of the 1933 inspection the water level in the valve shaft was the same as that in the reservoir.

Oakes in his 1933 Report states 'the subsidence of the inner slope of the dam around the valve shaft already referred to, is doubtless due to cavitation in the heart of the embankment caused by the long–continuing leakage of water carrying away the finer materials to the culvert at the stream below.

In 1935 it was found that the pipe into the bulkhead (in the downstream culvert) had cracked up to the next joint both at the top and bottom of the pipe and was found to be crushed out of circular. A special connecting piece was made up following removal of the pipe and a VJ coupling introduced.

In 1938 leaks in the valve tower were noted and 20 tons of ashes were placed around the valve tower and along the line of the culvert to try to check for leakage and leakage to the upstream culvert was recorded as being up to 350 cubic feet per minute.

Mr Oakes report of June 1943 states 'The valve shaft is now reasonably accessible – the discharge pipe may now be shut off from the valve shaft, since the re–fixing within the shaft of the old controlling valve which formally was located at the downstream end of the discharge pipe.' (see Fig 3).

Later in 1943 the valve shaft was pointed internally and the valve at the downstream end of the culvert transferred to the upstream end of the inner culvert. Subsidences were also filled in at that time (see Fig 3).

In December 1959 – two breaches in tunnel repaired and pitching replaced.

Between July 1964 and October 1964 the following works were undertaken:

1. Removal of the sluice valve installed in the bottom of the valve shaft at the end of the existing 24 inch diameter cast iron discharge pipe.
2. Fitting and grouting in 24 inch diameter light weight pipes (oil drums!!) in the outer culvert from the end of the 24 inch diameter cast iron discharge pipes to the inlet chamber at the toe of the inside slope.
3. Fitting a new draw–off slide valve with protecting grating to the existing iron frame on the inlet chamber.
4. Filling in the subsidences in the immediate vicinity of the draw–off valve and above the culvert on the upstream slope of the embankment and replacing and grouting the pitching stones.

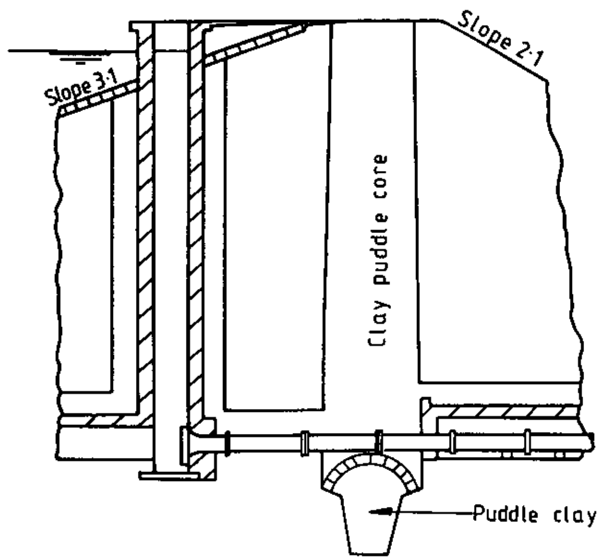


Fig. 2. Kentmere Reservoir – Arrangement of valve well core and culvert

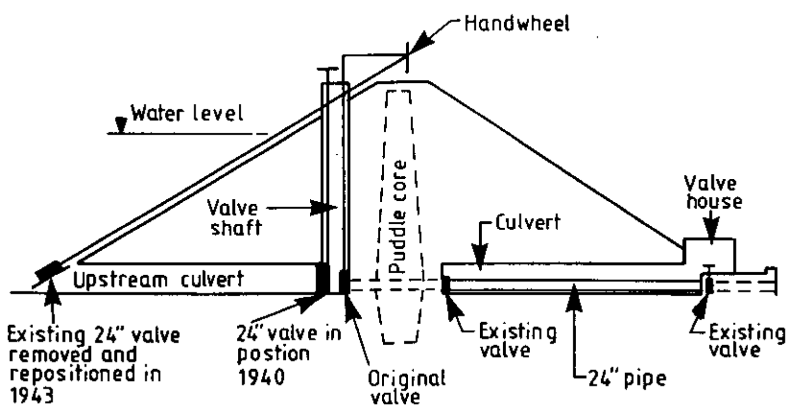


Fig. 3. Kentmere Reservoir – Diagram showing position of valves

5. Pressure grouting of the loose filling and sections showing subsidence on the inner slope of the embankment was carried out for a distance of 30 feet on each side of the outlet culvert 90 holes being drilled of a total length of 3540 feet into which 1473 tons of grout was injected. All drills were taken 5 feet below the invert of the culvert. In grouting in the pipe some 104 tons of grout was used. It was further stated in the Specification that men of a 'suitable stature' will be used to install the new 'lightweight pipes'. The Contractor was Cementation.
6. A plug of concrete was constructed at the bottom of the valve shaft and a reinforced concrete slab placed on top.

Part of this work was carried out because of 'an appreciable depression adjoining the tower' (some 18 inches (450mm)), and a leakage of 40 gallons per minute as reported in the Engineers Report of June 1964.

The original culvert through the upstream shoulder was 3ft 6 inches (1.1m) wide and 3ft 9 inches–4ft (1.24m) high. A report of an inspection in 1964 suggested that it was constructed in heading because a layer of peat existed above the crown of the tunnel.

An almost complete collapse of the shoulders of the tunnel occurred on two occasions and springs were noted in the base. In 1964 it was noted that the leakage to the tunnel was serious. Binnie reports on the tunnel and valving work as follows:

> "a new valve was fitted which closed off the end of the inner culvert. The operating rods for this valve were carried up the stone–pitched slope to a large handwheel on the crest of the dam. The consequence of this was the inner culvert of rectangular shape was now subject to water pressure, a condition for which it had not been designed by Bateman, and the entrance to this culvert collapsed in 1964. With a steel tube, made of old oil drums welded together through the middle to allow water from the reservoir to gain access to the gate well, the culvert was grouted up. It was expected that when in due course the thin shell of the pipe corroded, the concrete surround would provide an adequate watertight pipeline but the culvert had to be grouted again in 1977 before all the voids in the stonework surrounding the pipe were filled."

In July 1977 Mr Delwyn Davies found 'leakage holes of some size' in the walls of the concrete intake well. He pointed out in his interim report of the time that it was likely that the grouting to the culvert in 1964 had not completely filled the voids in the stonework surrounding the pipe made of oil drums.

An immediate grouting programme was taken because the leaks were of the order of 2.3 mgd which following grouting reduced to 150,000 gallons per day.

At this time it was found that settlement and general level of the crest warranted remedial action due to reassessment of flood capacity. The resulting action involved the lowering of the weir by some 3 feet 6 inches (1.07m), the work being carried out in 1978.

Current Inspection (1995)

As a result of concerns on the part of the Supervising Engineer for the reservoir about increases in flows being measured at the toe of the dam a Statutory Inspection was carried out in May 1995. The reservoir was emptied to carry out an inspection of the upstream side and to allow a CCTV survey of the culvert..

It was immediately apparent that there were areas of settlement either side of the culvert on the upstream face of the dam. During emptying of the reservoir precautions were taken to prevent large amounts of silt being discharged to the river.

A closed circuit television (CCTV) survey of the culvert was carried out starting at the upstream end of the culvert in the upstream shoulder of the dam.

> The CCTV camera mounted on a motorized trolley was able to travel some 58m, before getting stuck in silt which resided in the pipe as a result of the emptying process. The camera therefore was able to travel the full length of the upstream tunnel/culvert, through the section where the valve shaft had been blocked off and into the cast iron pipe in the downstream culvert.
>
> The CCTV showed that the first metre of the culvert from the upstream chamber to be masonry walling. After that the culvert was formed in oil drums which have been joined together. The drums seem to have an internal diameter of 20 inches (500mm).

A number of points were noted both in the masonry section and the oil drum section where there were major ingresses of water, and in the case of the oil drum section where the drums had corroded and there were large and small holes in the fabric of the drums. An ingress of water at these points demonstrated the ineffectiveness of the surrounding grouting annulus and many of the holes were in a position which roughly correlated with depressions and holes in the pitching. Clearly there was direct connection between the culvert and the reservoir and the formation of "pipes" through the upstream fill leading to erosion and loss of material/subsidence which could eventually to an uncontrolled emptying of the reservoir.

At the end of the oil drum section it was not possible to detect any feature which indicated the position of the old valve shaft but it was possible to detect the entry point into the cast iron pipes. This occurred at around 46m from the upstream toe but the oil drums stopped at 40m. It is thought that this junction was formed with a special pipe and a flexible joint formed with lead wool or something similar. The general condition of the cast iron pipe appeared to be good.

The line and level of all sections of the pipes appeared to be good although there did appear to be some steepening in the vicinity of the core at 21m and again at 42m.

In view of the condition of the upstream culvert it was not possible to recommend that the reservoir be refilled. There was evidence that the structural integrity of the oil drum liner had been compromised and that an erosion process had started which could have led to total loss of reservoir storage, subsidence, collapse of the culvert, etc. Remedial works or works associated with discontinuance were recommended to be carried out as soon as possible.

Remedial Works

A number of ways to overcome the problems being experienced at Kentmere reservoir were investigated including:

(i) breaching the embankment
(ii) reducing the volume stored in the reservoir to less than 25000 cubic metres
(iii) carrying out remedial works at the site

The first option, of discontinuance of the reservoir, was disregarded. The reservoir is situated in the Lake District National Park and the 'scar' caused by a breached embankment would have detracted from the natural beauty of this part of the Lake District and be opposed by the Planning Board and the public in general. Even emptying of the reservoir caused a public outcry with the press describing the site as a 'wilderness of bare rocks and mud' and the prospect of an abandoned reservoir as a 'hideous blot on the landscape'!

The second option, of reducing the capacity of the reservoir to less than 25000 cubic metres would have involved such a large reduction in embankment level with the corresponding reduction in water level that again a 'scar' would have been caused have detracted from the natural beauty of the area. This option was not considered further.

The owner decided, even though he had no use for the reservoir to carry out remedial works.

Remedial Works

The remedial works proposed at the site comprised three elements:–

(i) grouting of the foundation and shoulder of the dam
(ii) the installation of a liner to the scour pipe and a proving CCTV survey
(iii) the construction of a wave wall on the crest

The programme of working involved:

(i) excavation of the pipe lead–in area and demolition of the upstream wall and lid of inlet chamber, removing and storing existing valves
(ii) CCTV survey of existing pipe
(iii) scraping of existing pipe
(iv) CCTV survey of existing pipe
(v) proving of pipe prior to lining
(vi) slip lining of existing pipe
(vii) CCTV survey of new HPPE liner
(viii) grouting of pipe annulus
(ix) CCTV survey of new HPPE liner
(x) reinstatement of wall and lid of invert valve chamber and valves
(xi) grouting of shoulder

Other works included painting of the pipework, desilting of the downstream tunnel, and grouting of the spillway channel.

The liner was a HDPE pipe with a diameter of 400mm OD supplied by Stewarts and Lloyds Plastic. The pipes were introduced into the culvert after partial demolition of the upstream chamber. Joints were formed using dual pressure butt fusion techniques.

The annulus was sealed at the upstream end with bentonite/cement grout and then the annulus was injected with PFA/OPC grout in stages from a series of injection pipes around the perimeter of the pipe commencing with the lower injection tubes, causing the water to be displaced upwards. The grout had additives to minimise plastic shrinkage and had a minimum 28 day strength of $16N/mm^2$. Seventeen cubic metres of grout was used to fill the annulus.

Work commenced in August 1996 with any water entering the reservoir basin being overpumped via 2 No. 6 inch pumps and 1 No. 4 inch pump. Unfortunately the bywash which existed around the left hand side of the reservoir is in an advanced stage of decay and hence could not be used. The Contractor therefore had to be prepared to deal with any runoff entering the reservoir basin. The 1 in 2yr, 10yr, 100yr and PMF storm inflows were calculated as 11, 21, 34 and 138 cumecs respectively.

Consolidation grouting was carried out from two lines of vertical holes, one on either side of the inlet culvert and offset by 2.5m from the culvert centreline. Holes were spaced, initially, at 3 metre centres (staggered along the line of the culvert) and taken to 1 metre depth below the culvert formation level. Holes were drilled with air flush and supported by temporary casing through the embankment fill and were 50mm diameter.

Grout was introduced by tremie pipe under gravity head and the casing withdrawn in 1.5 metre stages keeping the grout level topped up to ground level.

The grout was bentonite–cement grout in the proportion of 1:1 by water at a water/cement ratio of 5:1 by weight, although the latter ratio was varied as necessary. The bentonite was mixed with water first using a high shear colloidal mixer before adding cement. A total of 5 tonnes of grout was injected through 27 holes.

The contract was led to Askan Construction Ltd of Lancaster. The tender price was £100,125 and the final certificate value was £72,760.

PRESENT AND THE FUTURE.

Refilling of the reservoir was done in a controlled manner using the outlet valve to try to limit the speed of rise of water level. The flow over the V–notches at the 5 points of known leakage were monitored as the water level rose, as well as taking crest levels.

As a result of the work two of the flows continue at a much reduced rate and the others have ceased entirely. Crest levels taken over the last 8 years have shown very little movement; the largest being a maximum of 8mm of vertical movement when the reservoir was emptied.

CONCLUSION.

Good records enabled the performance of the reservoir over an extended period to be investigated which gave some clues to the reasons for the conditions being experienced.

As a result of the efforts of a very vigilant Supervising Engineer who developed a good working relationship with a Panel Engineer, and the responsible attitude of the owner, Kentmere Reservoir has been repaired and modified to meet modern standards and hopefully will now operate in a satisfactory way for many decades to come.

Distributed temperature sensing in dams

J Dornstädter, GTC Kappelmeyer GmbH, GERMANY
M Aufleger, Versuchsanstalt für Wasserbau - TU München, GERMANY

SYNOPSIS. The development of fibre optic temperature laser radar systems allows distributed temperature sensing (DTS) along fibre optical cables, which opens new possibilities for dam surveillance. The cable can easily and economically be integrated into the dam construction and thereby enabling sensitive and reliable long-term monitoring of water retaining structures. Two applications of DTS technology in dams will be presented. The first DTS application in an embankment dam has been performed during the rehabilitation of the sealing system of the *Mittlerer Isarkanal*, a hydro power supply channel in Munich, where a fibre optical cable was deployed underneath a 1200 m long section of asphaltic lining and a 200 m long section of concrete surface sealing in order to monitor seepage on a long-term basis. The second example shows the results of monitoring concrete curing temperatures during the construction of the Birecik gravity dam in Turkey using DTS.

INTRODUCTION

Since 1955 soil temperature measurements have been used for the surveillance of dams and dikes. It is well known that the temperature of the water in reservoirs, channels, rivers, etc. acts as a natural tracer when it percolates through the embankment dam. Percolated dam areas stand out during summer time as positive soil temperature anomalies and during winter time as negative anomalies (Kappelmeyer, 1957). Thus, temperature measurements inside dams can be used to locate temperature anomalies and thereby detect seepage zones. With a conventional temperature measurement method more than 350 km of dam have been investigated successfully for leakage (Dornstädter, 1996, 1997). This measuring equipment is usually deployed temporarily. A low cost alternative for long term monitoring is the application of distributed temperature sensing (DTS) systems. This holds specially for new dams, where the temperature sensor, i.e. fibre optical cable, can be integrated into the construction. Another application for DTS in dams is the monitoring of the curing temperature in concrete. The development of the curing temperature in mass concrete determines the thermal stresses and the resulting cracking.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

DISTRIBUTED TEMPERATURE SENSING.

The method of distributed temperature sensing (DTS) was developed at the beginning of the eighties for the surveillance of power cables (Darkin et al., 1985). DTS works by sending a short laser pulse (< 10 ns) down an optical fibre. The determination of the temperature is performed with the help of Raman spectroscopy on the backscattered light. The temperature is calculated from the Stokes to anti-Stokes ratio. The localization of the measurement point results from a precise time measurement under consideration of the propagation velocity of light in the optical fibre. Thereby a measurement accuracy of up to ±0.2 °C and a spatial resolution of up to ±0.5 m can be obtained.

Modern fibre optic temperature sensing methods enable temperature measurements along an optical fibre of up to 40 km of length. Therefore, they are extremely suitable for the surveillance of dams and dikes. By integrating an optical fibre built into the structure of a new dam or within the scope of rehabilitation, the temperature can be measured readily along the inexpensive optical fibre and emerging leaks can be detected exactly.

INVESTIGATIONS

Mittlerer Isarkanal - hydro power supply channel

A fibre optical temperature sensing system for surveillance purposes was first built into a dam at the *Mittlerer Isarkanal*, a hydro power supply channel. During the rehabilitation of the sealing system two fibre optical cables were deployed in a ditch filled with drainage gravel parallel to the length axis of the channel; one underneath a 200 m long section with concrete facing (section I) and one underneath a 1100 m long section with asphaltic facing (section II). The cable was embedded in fine gravel about 1.2 m underneath the facing. The scope of the deployment was to detect percolating channel water on a long-term basis. During the flooding of the channel in October 1997 measurements were taken daily, then weekly for two months and then every two months. In the long term two measurements per year are scheduled.

The results of the measurements at the *Mittlerer Isarkanal* show a well functioning sealing system in both sections. Nevertheless, during the first two weeks after the flooding a higher permeability of the concrete sealing, probably due to open construction joints, could be observed. Further, one small leak in section II was detected, but it was sealed after a few weeks by fine sediments in the channel water, (see Fig. 1).

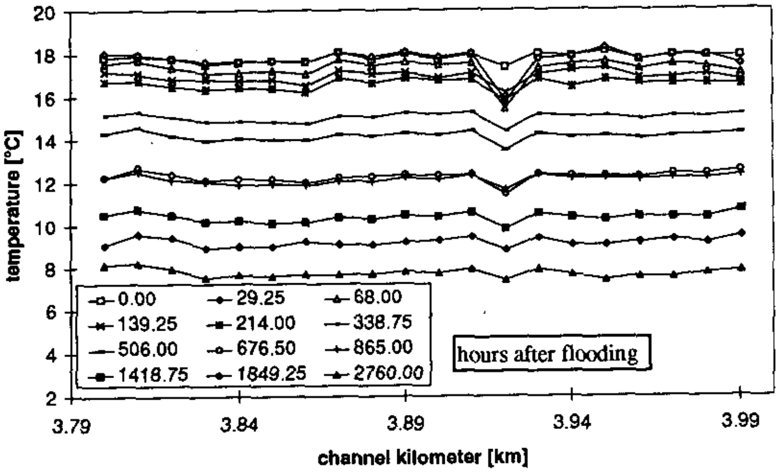


Fig. 1. Temperature measurements in section II (asphaltic facing) from just before flooding to 2760 hours after flooding. During the first hours a leak occurs at km 3.920. After a few weeks it was sealed by fine sediments.

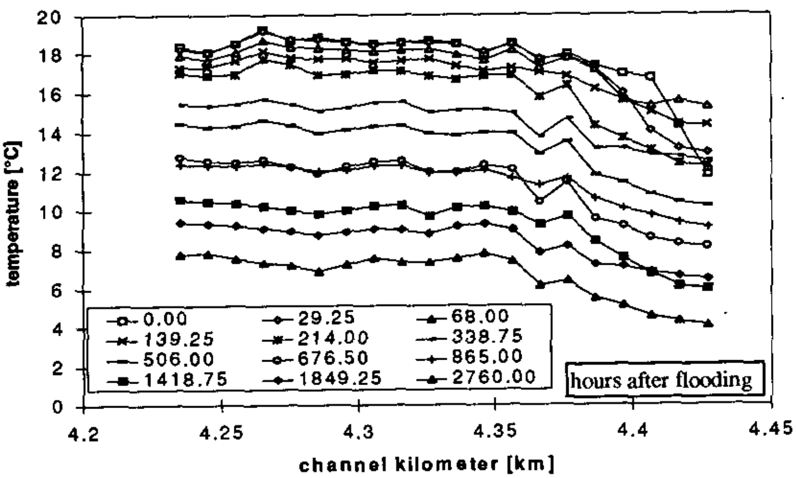


Fig. 2. Temperature measurements in section II (asphaltic facing) from just before flooding to 2760 hours after flooding. Seepage water from the adjacent non-rehabilitated downstream section was flowing upstream underneath the asphaltic sealing (km 4.380 to 4.435).

In section I at channel kilometer 1,970 to 1,990 a groundwater flow underneath the bottom sealing could be observed. From channel kilometer 4,380 to the end of the rehabilitation section II, seepage water from the adjacent non-rehabilitated downstream section, is flowing upstream underneath the asphaltic facing (see Fig. 2).

Birecik gravity dam

During the construction of concrete dams it is essential for the successful outcome of the entire project to confirm the fracture stability of concrete. Therefore, threshold values for the temperature of just poured concrete and the development of curing temperature are specified. The employment of the innovative fibre optic measurement method enables a nearly continuous temperature measurement along an optical fibre built into the dam structure during concreting. This technology has been applied and examined for its reliability for practical operation at the Birecik gravity dam. The dam is being built on the Euphrat river in Southeast-Anatolia.

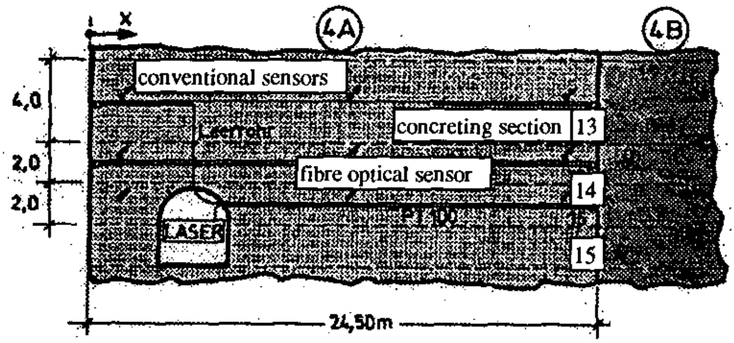


Fig. 3. Part of the cross-section of the Birecik dam with the fibre optical cable layout.

An optical cable with a length of about 155 m was deployed in three concreting sections in block 4A of the gravity dam. Additionally, as a reference conventional sensors (PT100) were installed near the fibre optical cable. The DTS (laser) was installed in the inspection gallery (see Fig 3).

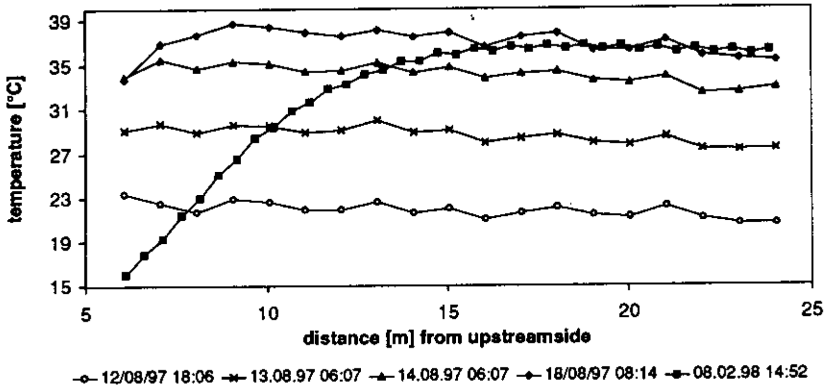


Fig. 4. Curing temperature development in concreting section 15. The left hand side is adjacent to the inspection gallery.

The recorded DTS data show an excellent correlation with the data obtained from the conventional sensors. The development of the curing temperature over time is given in Fig. 4. By 6 hours after the pouring of concrete the temperature rose to 22 °C and reached its maximum temperature of about 38 °C after 140 hours. The adjacent inspection gallery, with an average temperature of 27 °C during August '97, caused a cooling of the concrete and therefore a high temperature gradient. Five month later, in February '98, the temperature measurements show the cooling of the block due to the low outside temperatures in wintertime on the upstream surface.

CONCLUSIONS.

Distributed temperature sensing is well suited for the use under construction conditions. The monitoring of the curing temperatures in concrete has proved successful. The method offers a reliable way to cover extensive areas with extremely high information density at low cost and leaves conventional surveillance and monitoring systems redundant.

ACKNOWLEDGEMENTS.

The investigations at the Mittlerer Isarkanal have been carried out in close cooperation with the dams owner „Bayernwerk Wasserkraft AG". Funds were also given by the „Bayerische Forschungsstiftung" and the Bavarian Government. The support of the examination at Birecik dam by GAMA, Philipp Holzmann AG and STRABAG is gratefully acknowledged.

REFERENCES.

Darkin J P, Pratt D J, Bibby G W and Ross J N (1985). Distributed optical fiber Raman temperature sensor using a semiconductor light source and detector. Electron. Lett., 21, 569.

Dornstädter J (1996). Sensitive monitoring of embankment dams. Proceedings: Repair and Upgrade of dams, Symposium. Stockholm June 5-7, 1996. Royal Institute of Technology, Stockholm.

Dornstädter J (1997). Detection of internal erosion in embankment dams. Proceedings: 19. ICOLD Congres 1997. Florence. Q73R7.2:87-101. International Commission on Large Dams, Paris.

Kappelmeyer O (1957). The use of near surface temperature measurements for discovering anomalies due to causes of depth. Geophysical Prospecting, 3: 239-258. The Hague.

Bewl Water spillway investigation

IAN DAVISON, Montgomery Watson, UK

SYNOPSIS Bewl Water is a major impounding reservoir in Kent, UK. Over the last few years it has become evident that the crest of the spillway shaft is suffering from severe cracking. This paper describes the investigation carried out to determine the cause of the cracking and identify possible remedial measures to the shaft. Following *in situ* investigations and laboratory testing it was concluded that the cracking was caused by a combination of the Alkali Silica Reaction (ASR) in the pre-cast concrete crest units and the flared geometry of the bellmouth at the top of the shaft.

BACKGROUND

General

Bewl Water is a large, raw water reservoir situated approximately 10 km south east of Royal Tunbridge Wells. The reservoir is primarily filled by the Yalding Pumping Station with additional water from the natural catchment.

A 30.5 m high zoned rockfill embankment with a central rolled clay core retains the reservoir. A bellmouth spillway shaft located within the reservoir was designed to discharge floods up to a Catastrophic Flood of 115.5 m^3/s. Water is abstracted from the reservoir through a 36 m high reinforced concrete tower adjacent to the spillway tower. Impounding of the reservoir started in 1976 and was almost full by mid 1978.

Spillway Shaft

The overflow structure, as shown in Figure 1, consists of a vertical shaft from the reservoir bed up to the full supply level of the reservoir. The lower section of the shaft is 3.5 m in diameter internally and has 500 mm thick reinforced concrete walls. Over its upper 7.75 m, the shaft flares out to a diameter of 10.8 m at the top. At the top of the shaft is a series of 32 precast concrete blocks that form the lip of the weir. These are divided into quadrants by anti-vortex piers that prevent a vortex forming in the shaft when it becomes submerged. A 1500 mm deep beam boat fender surrounds the weir reducing the size of waves impinging the weir and preventing boats from getting too close to the overflow crest. The boat fender is located 2.0 m away from the weir crest and is supported by four radial beams spanning out from the rear of the anti-vortex piers.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

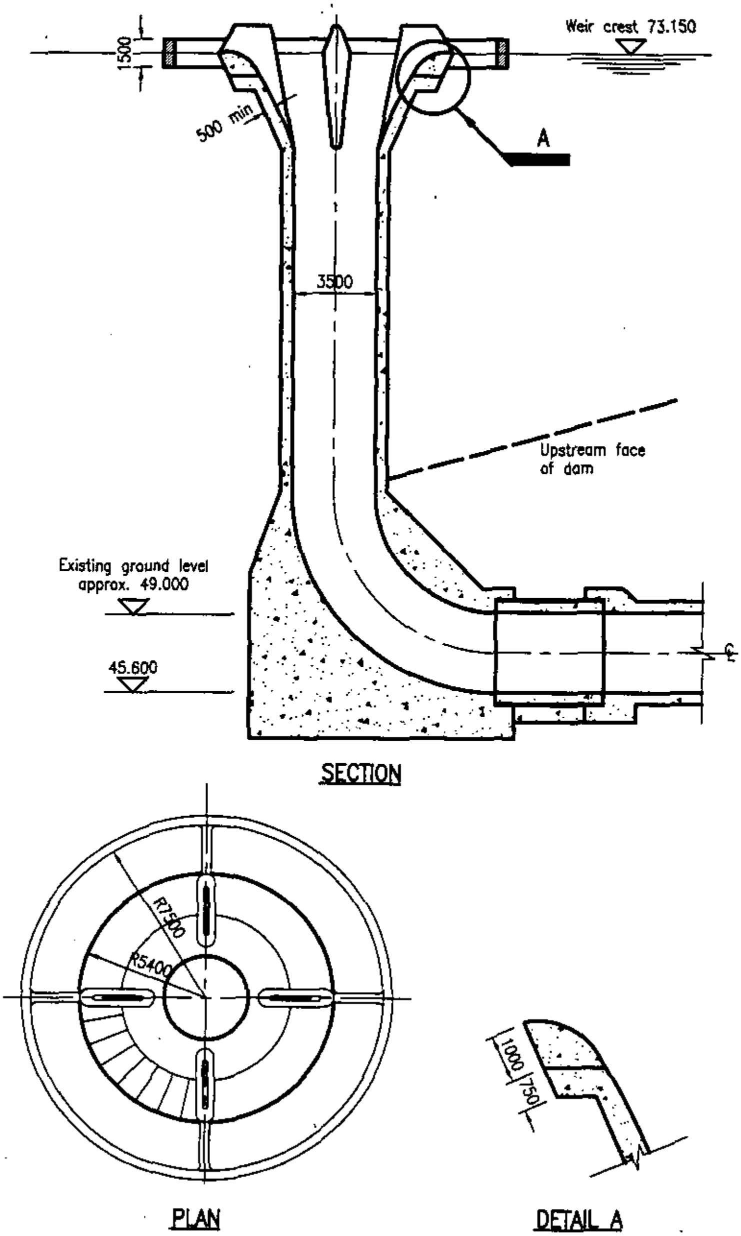


Figure 1 – Detail of the Overflow Structure

At the base of the tower the shaft turns through a 90° bend into the discharge tunnel that then passes under the dam before discharging into the river downstream. The tunnel is a horseshoe in section and is approximately 3.3 m in diameter.

THE PROBLEM

The Supervising and Inspecting Engineers noted cracking and spalling of the precast concrete blocks around the spillway crest during routine inspections. In February 1996, McDowells Consulting Engineers carried out a preliminary investigation of the cracking. They recommended further investigation of the boat fender and upper section of the shaft, including removal of core samples from the crest blocks and *in situ* concrete. The cause and extent of the cracking were unknown and it was unclear whether the condition was deteriorating.

THE INVESTIGATION

Original Construction

Copies of the record drawings and some of the construction drawings, including the reinforcement details for the spillway shaft, were obtained from the Southern Water records. Details of the original concrete mix design for the *in situ* concrete in the shaft were available. However there were no details of the mix design for the pre-cast concrete used in the weir crest units except that it was known that they were cast off site using a different concrete mix.

The Survey

Specialist sub-contractors were employed to carry out an internal survey and testing of the concrete in the overflow shaft. These works were carried out by Abtec, using roped access techniques, during the summer of 1997 when the reservoir level was approximately 3.0 m below the spillway crest. A scaffold platform was erected around the top of the boat fender to assist in accessing the inside of the shaft. As part of the survey the following tasks were carried out:

- photographic record of the inside of the face of the shaft from the weir crest down to the base of the flared section.
- random carbonation testing was carried out to determine the depth of carbonation within the concrete.
- cores were removed from the pre-cast units and the *in situ* concrete for detailed examination and testing in a laboratory. Difficulties were encountered in removing the 100 mm and 75 mm cores from the *in situ* concrete because of the closeness of the reinforcement.

- covermeter survey was carried out to check the size and depth of cover to the reinforcement.
- ultrasonic pulse velocity testing was used to examine the general integrity of the concrete and to estimate the depth of the cracks in the anti-vortex piers.

A visual crack survey of the external surface of the tower was also carried out by Montgomery Watson engineers from a small boat.

Testing

The following laboratory testing was carried out by STATS on the cores removed from the spillway:

- a visual examination of the cores
- a microscopic examination of the cores
- ultrasonic pulse velocity testing on the cores
- estimation of the concrete density
- estimation of the concrete mix design using micrometric analysis
- Chemical analysis - cement content
  - sulphate content
  - alkali content
- Alkali Immersion Test in accordance with the BCA working party report on diagnosis of ASR.

The laboratory testing showed that the cores from the pre-cast and *in situ* concrete were formed from two different types of concrete, both of which were showing signs of cracking consistent with shrinkage. However the pre-cast concrete cores also had microcracks caused by ASR but there was no evidence of this reaction in the *in situ* concrete. The testing also indicated that neither the pre-cast nor the *in situ* concrete appeared to exhibit evidence of sulphate attack or delayed ettringite formation and that the risk of chloride induced corrosion was low. There was a risk of carbonation-induced corrosion of the reinforcement where intersected by the deeper cracks. Table 1 summarises the findings of the laboratory testing on the cores.

Findings

The visual inspection and the photographic record indicate several different crack patterns in the *in situ* and pre-cast concrete of the shaft. Most of the cracks are generally in the horizontal direction on both the internal and external faces of the shaft. There are also some radial vertical cracks in the outer lip of the pre-cast units. These cracks are caused by stresses in the

Table 1. Summary of Laboratory Test Results

Concrete Type	Precast	*In Situ*	Precast	*In Situ*
Coarse Aggregate Petrography	Quartzite & sandstone	Chert	Quartzite & sandstone	Chert
Fine Aggregate Petrography	Quartz / quartzite & chert	Quartz / quartzite & chert	Quartz / quartzite, sandstone & chert	Quartz / quartzite & chert
Estimated excess voidage	1.0%	0.5%	1.0%	0.5%
Carbonation	2-3s, 9c	12s	5s, 75c	7-10s
Surface deposits	none	calcite layer	none	none
Macrocracking	none	normal to surface	one normal to surface	none
Microcracking	normal to surface, emanating from chert, passing through matrix and aggregate	normal to surface	emanating from chert and passing through matrix and aggregate	normal to surface, and minor confined to matrix
Crack types	ASR / shrinkage	Shrinkage	ASR / shrinkage	Shrinkage
Secondary deposits	gel in cracks, ettringite in voids	ettringite in voids	gel & ettringite in cracks & voids	none
Comments	ASR	none	ASR	none
Pulse velocity	4.0 m/s	3.9 m/s	3.6 m/s	4.6 m/s
Water/cement ratio	0.5	0.5	0.5	0.5
Cement content %	22	17	25	16
Density, kg/m^3	2335	2365	2285	2365
Cement content, %	23.1	14.4	-	-
Alkali content kg/m^3	3.8	2.0	-	-
Sulphate content, % SO_3	2.99	3.06	-	-
Chloride content % Cl^-	<0.01-0.03	<0.01-0.05	-	-
Alkali immersion test	gel after 4 days	no change	-	-

UPV testing on the flow splitter piers

Typical cracking at the top of the spillway shaft concrete and have led to the cracking scenario described below. Some of the pre-cast crest units also show map cracking which is typical of ASR.

ASR is a chemical process in which alkalis, mainly from the cement, combine with certain types of silica in the aggregate, when moisture is present. This reaction produces an alkali-silica gel that can absorb water and expand to cause cracking and disruption of the concrete. Only certain forms of silica react with silica to a significant extent and the proportion of reactive silica must lie within a small range. Chert which is present in the concrete at Bewl Water is a now a widely recognised form of reactive aggregate.

The on site carbonation testing of the *in situ* concrete on the inside face of the shaft indicated depths of carbonation of less than 1 mm whilst in the anti vortex piers the depth was only of the order of 3 mm. The laboratory testing was consistent with these depths although carbonation had taken place to depths of 75 mm along some of the cracks. These results confirm that there is not a carbonation problem with the spillway concrete.

The laboratory testing and the inspection of the cores confirmed that the pre-cast units were cast from different mix to that used in the *in situ* concrete. They also concluded that the pre-cast units were suffering from ASR whilst this did not appear to be a problem with the *in situ* concrete. Four of the six cores taken from the spillway had macrocracks running normal to the surface of the concrete and in one case this extended the full 225 mm length of the core. This crack was probably formed very slowly and was possibly initiated by an existing shrinkage crack.

CAUSE OF CRACKING

The ASR that is taking place in the pre-cast concrete has caused the units to swell. These units form a tight ring around the top of the shaft, each securely held in place by two anchor bolts grouted into the *in situ* concrete and wedged between the anti vortex piers. Although the units are suffering from ASR, there is unlikely to be a significant reduction in the compressive strength of the concrete and the units have not crushed as they expanded. Instead they have moved radially outwards and upwards. It is thought that this has put the outer lip of the units into tension and thereby caused:

- the vertical radial cracks seen in the top surface of most of the blocks and
- the horizontal cracking that can be seen in most of the external faces of the blocks.

The outward movement, in combination with the flared geometry of the upper section of the shaft, has also generated tensile stresses along the inside face of the shaft. The reinforcement is unlikely to have been designed to take the tensile forces generated there and a pattern of generally horizontal cracks has occurred over the flared section of the inside face of the shaft. The cracking seen in the core taken from the *in situ* concrete suggests that this pattern was initiated by early shrinkage cracks. Due to the good connection between the pre-cast units and the *in situ* concrete, the anti-vortex piers have also been forced to move radially outwards. The inside edge has also been put into tension and several horizontal tension cracks have occurred on the inside of each pier.

REMEDIAL WORKS

Bewl Water is a crucial part of Southern Water's water supply system and so it is necessary to retain the full working volume of the reservoir. Three remedial works options were considered:

1. "Do Nothing"
2. Replace all the crest units thereby removing the problem and preventing further expansion.
3. Selective Replacement of Units and provision of expansion joints which would allow further expansion of the blocks without causing further distress to the *in situ* concrete.

1. The "Do Nothing" option, would leave the spillway in its present condition. The laboratory testing has suggested that the swelling due to the ASR is likely to continue. From the alkali immersion test carried out it is not possible to estimate the degree of swelling which is still to occur. Mortar bar testing was considered as an alternative method. This test could provide additional information on the extent of swelling but the test takes six months to carry out and it would not provide reliable information on the time for the swelling to occur. However it is safe to assume that, if no remedial work is carried out to relieve the stresses in the upper section of the spillway, then the condition of the tower will deteriorate.

There are three likely scenarios that might result from the "Do Nothing" option:

a) the fixings between the pre-cast units and the *in situ* concrete will fail and the pre-cast units will be free to move outwards and upwards without causing further serious distress to the tower.

b) the pre-cast units will remain firmly fixed to the *in situ* concrete and the expansion due to the ASR will cause the cracks to propagate further.

c) the amount of further expansion will be minimal and there will be an insignificant increase in the distress to the tower.

In scenario (a) the failure of the fixing system is unlikely to have a serious effect on the operation of the spillway unless the units fall apart. In this case there could be the loss of up to one metre of reservoir storage (approximately 3.1 Mm^3). During large flood discharges there could also be a possibility of the blocks being dislodged into the shaft, which may in turn lead to damage to the culvert.

In scenario (b) a propagation of the cracks would increase the risk of corrosion to the reinforcement in the *in situ* concrete. The purpose of this reinforcement, particularly on the inside face of the shaft, is to resist the bending moments induced by the flared rim of the shaft and the cantilevered boat fender. Further increases in the crack width could lead to yielding of the reinforcement if this has not already occurred. Depending upon the conservatism of the original design, a decrease in the available strength of the steel due to yielding or corrosion could eventually lead to major damage to the top of the shaft.

In scenario (c), the cracks are not likely to propagate further but there is still a risk of deterioration of the reinforcement, as described in scenario (b), due to water ingress through existing cracks in the *in situ* concrete.

The laboratory tests indicated that there is likely to be further expansion of the pre-cast units but they could not estimate the amount of further expansion that will occur. It is therefore not possible to estimate the time scale for the development of the above scenarios or the likelihood of one scenario occurring rather than another.

Due to the importance of the reservoir and the uncertainty in the timing with a "Do Nothing" option, it has not been recommended. The width of the cracks already present in the *in situ* concrete indicate that the shaft has already suffered severe distress which will in time probably lead to the break up of the affected section.

2. Replace all the crest units. In order to prevent further deterioration of the existing *in situ* concrete, this option would remove the ASR problem by removing the pre-cast crest units. New pre-cast units would be formed, transported to the crest of the shaft by barge and lifted into place using a floating crane. The main difficulty with this option is in the restricted working space around the top of the tower and the need to position the new pre-cast units from a floating barge over the top of the boat fender.

3. Selective Replacement of Units. Not all of the crest units have serious cracking and complete replacement may not be necessary. An alternative to replacement would be to relieve the stresses induced by the ASR expansion and repair or replace the more seriously damaged units. Sawing through the joints between the pre-cast units would relieve the stresses. It is difficult to predict the effect of the stress relief on the badly cracked units and it is possible that some of them may fall apart. These units would be removed completely and replacement units installed. The joints would be resealed using a flexible polyurethane grout that will allow further expansion to take place. To prevent seepage into the existing cracks the remaining blocks would be sealed with polyurethane or acrylic elastomeric coating.

In both of the repair options, the *in situ* concrete will be sealed on the reservoir side to reduce the risk of water entering the cracks and encouraging corrosion of the reinforcement.

PRESENT SITUATION

Although the scale of the repairs is not large and the techniques are mainly relatively simple, the cost of the repairs is high. This is due principally to the difficult access. In its present condition, the spillway shaft is operational and the deterioration does not affect the integrity of the dam from a reservoir safety point of view. However continual expansion of the pre-cast concrete crest units could lead to significantly higher repair costs in the future.

The decision on whether or not to proceed with the repairs depends upon the degree of further expansion of the crest units and the extent of further damage to the *in situ* shaft section. A simple monitoring program using tell tales is therefore planned to measure further expansion over the next year. The results of this monitoring will be used to determine the form, if any, of the repairs.

ACKNOWLEDGEMENTS

The author is grateful to Southern Water for their permission to present the findings of the investigation.

Enguri Dam: horizontal and vertical displacement, retrospective analysis

D. MIRTSKHULAVA, Centre of Diagnostics and monitoring of constructions, Georgia
I. NONIEV, centre of Diagnostics and monitoring of constructions, Georgia
J. EBANOIDZE, Centre of Diagnostics and monitoring of constructions, Georgia
T. KOBASNIDZE, Centre of Diagnostics and monitoring of constructions, Georgia

---

SYNOPSIS The present paper, which is devoted to the Enguri arch dam, presents the results of monitoring of the displacements of the dam-foundation system obtained by geodetic methods. The results of field measurements for the ten-year operation period (1987-1997) provide material to estimate the technical condition of the dam.

INTRODUCTION

The Enguri High Arch Dam is a double curvature arch dam with gravity blocks at both abutments. The height of the dam is 271.5m and the length of the crest is 728m that includes 605m of arch length. Supporting part of the dam is the saddle that is separated from the arch by the perimetral joint. The height of the saddle is 15-20m at the edges and 60m in the lower part of the gorge. The thickness of the dam in cantilever cross-section is 10m along the crest, 56m along the perimetral joint. It is 490m along at the joint with the foundation. Construction of the dam began in 1971. The reservoir was filled on a stage-by-stage basis from 1978 and in 1987 reached its designed level of 510m (Chogovadze, 1987). The volume of concrete in the dam is 3.96 million. The body of the arch dam is cut into 42 sections by joints located normal to the arch axis (Fig. 1).

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

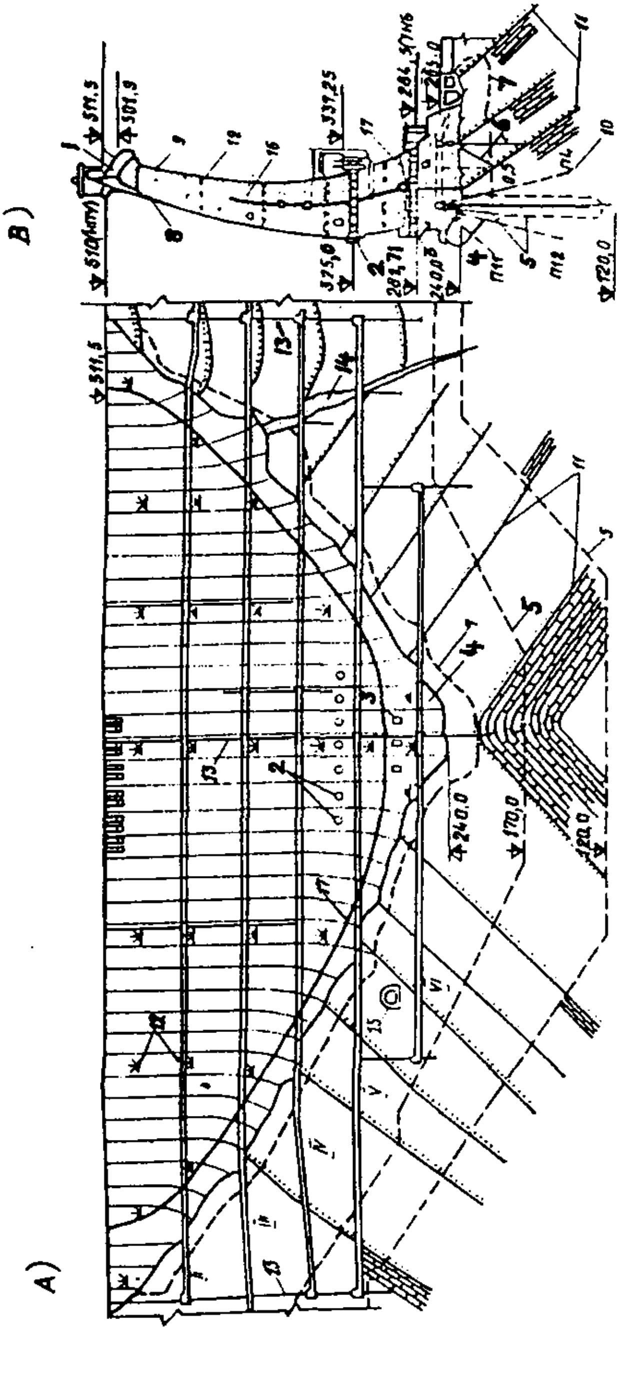


Fig. 1 Arch dam; A) development along the axis of grout curtain; B) crown block; 1) crest spillway; 2) low level outlet; 3) construction outlet; 4) saddle; 5) grout curtain; 6) curtain drain; 7) boundary of blanket grouting; 8) tracks of leaning gate; 9) service catwalks; 10) piezometers; 11) tectonic fractures; 12) points of monitoring stress-strain state of dam; 13) plumbline; 14) sealing of fault; 15) construction tunnel; 16) longitudinal joint; 17) perimetral joint; I-IV horizons of service galleries and tunnels.

GEODETIC MONITORING SYSTEM

The programme of geodetic monitoring at Enguri Dam (Kuznetsov, 1983) included:

1 Horizontal displacements
   a) measurements by direct and inverted plumblines;
   b) polygonometry on the crest and in control galleries;
   c) angular intersections at downstream face;
   d) triangulation of the supporting and control network points.

2 Settlements – geometrical levelling on the crest in 5 sections and along access roads.

Table 1 Number of measuring systems in the arch dam

Nos.	Name of the measuring systems	Number of Instruments		
		At Design Stage	In position on 01.01.1998	On completion of monitoring system rehabilitation
1	Fundamental and working bench marks	34	25	35
2	Settlement points on the surface	220	193	205
3	Settlement points in the galleries and adits	402	343	355
4	Triangular points	40	11	17
5	Polygonometry points	82	76	118
6	Bench marks on downstream face	89	64	64
7	Plumblines in the body of the dam	13	13	17
8	Plumblines in the foundation	25	20	8
9	Plumblines downstream	14	12	12

Due to a grant allotted by the government of Switzerland in 1997, the necessary equipment has been delivered and work has commenced on the design and installation of the new and refurbished equipment. This includes new plumblines in the body of the dam, a new system of polygonometric record keeping in the new control network as well as allocation of the benchmarks and settlement monitoring points.

HORIZONTAL DISPLACEMENT

Horizontal displacements are monitored by the system of direct and inverted plumblines in the body of the dam located in its three sections (12, 18 and 26). Fig. 2 summarises displacements of sections 12, 18 and 26 as recorded by the plumblines in these sections. The displacements from the plumbline readings are provided in relation to the deepest inverted plumbline (CGE-3) fixed in the rock foundation at the benchmark of 211m.

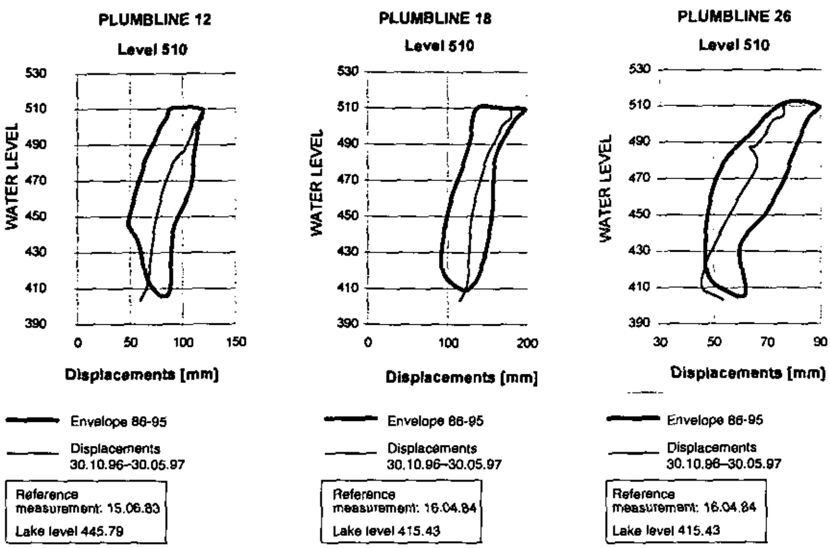


Fig. 2: Horizontal displacements measured at the dam. Plumblines P12, P18 and P26

The results of displacement measures in the radial direction in 1987-1996 show that under the influence of the rising water level in the reservoir the left abutment is moderately displaced downstream, whereas the displacement of the right abutment above the elevation 425m is high at 3-7mm annually (ref. Table 2).

Table 2 Annual Increase in the Displacement

	Total Increase	Mean Annual Increase
- left abutment	9 - 13mm	2 - 3m
- left quaterpoint	10 – 14mm	2.5 – 2.5mm
- crown section	10 – 21mm	2.5 – 3.5mm
- right quaterpoint	1 – 29mm	3 – 7mm
- right abutment	13 – 28mm	3 – 7mm

Considering the relatively young age of the dam, the displacement increments of less than 5mm/year are quite acceptable since there is a clearly noticeable tendency for displacement increments to decrease with time. As for the displacement of the right abutment, it is now being studied and analysed. The situation will be clarified after installation of an additional range of plumblines and deepening the existing range of specified within the new project.

Horizontal displacements are also monitored by survey of points fixed on the downstream faces of the four horizons of the dam at elevations 510m, 475m, 425m and 380m. Fig. 3, 4, 5 and 6 present the results of the measurements conducted in 1990, 1991, 1995, 1996 and 1996 (versus 1986 at the lowest reservoir level of 396m) at the horizons.

At Enguri Dam horizontal displacements were also measured by the control – crest polygonometry (traverse) method at IV and VI operational dam horizons. Fig. 7 demonstrates the results of the polygonometry on the dam crest conducted in 1990, 1991, 1995, 1996 and 1997 (in relation to the cycle performed in April, 1990 at WL of 406.3m) at various water levels in the reservoir.

Measurements of the displacements determined using the method of angular intersections and polygonometry were conducted simultaneously with triangulation of the arch supporting network. High precision measurements of the distances between the individual points of this network which were chosen as base marks.

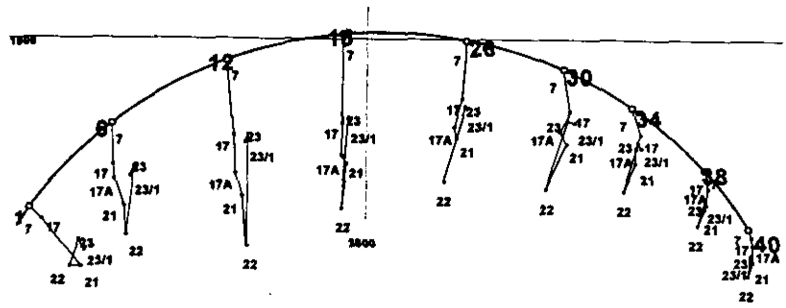


Fig. 3 Horizontal displacements at level 510m

LEGEND

° 7 – 05.1986	W.L. 396.0	° 21 – 11.1995	W.L. 488.0 – 486.0
° 17 – 05.1991	W.L. 441.0	° 22 – 10.1996	W.L. 510.0
° 17A – 09.1990	W.L. 494.5	° 23 – 05.1997	W.L. 405.0

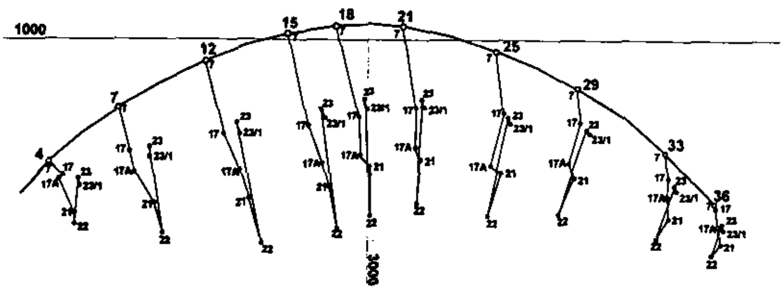


Fig. 4 Horizontal displacements at level 475m

LEGEND

° 7 – 05.1986	W.L. 396.0	° 21– 11.1995	W.L. 488.0 – 486.0
° 17 – 05.1991	W.L. 441.0	° 22 – 10.1996	W.L. 510.0
° 17A – 09.1990	W.L. 494.5	° 23 – 05.1997	W.L. 405.0
° 23/1 – 07.1997	W.L. 440.0		

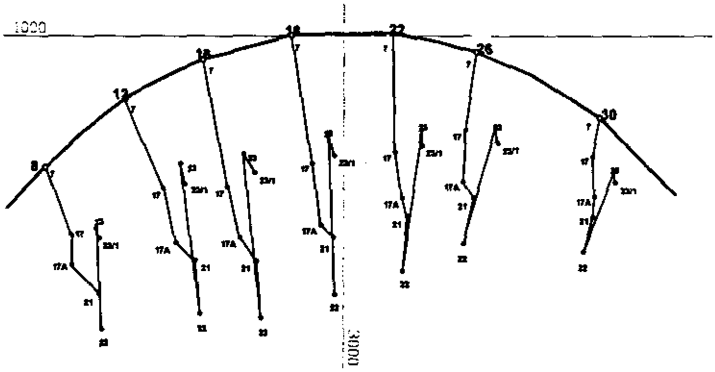


Figure.5 Horizontal displacements at level 425m

LEGEND

° 7 – 05.1986	W.L. 396.0	° 21 – 11.1995	W.L. 488.0 – 486.0
° 17 – 05.1991	W.L. 441.0	° 22 – 10.1996	W.L. 510.0
° 17A – 09.1990	W.L. 494.5	° 23 – 05.1997	W.L. 405.0
° 23/1 – 07.1997	W.L. 440.0		

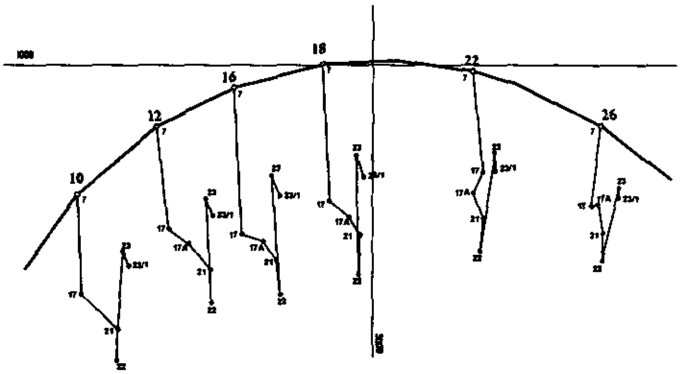


Figure 6 Horizontal displacements at Level 380m

LEGEND

° 7 – 05.1986	W.L. 396.0	° 21 – 11.1995	W.L. 488.0 – 486.0
° 17 – 05.1991	W.L. 441.0	° 22 – 10.1996	W.L. 510.0
° 17A – 09.1990	W.L. 494.5	° 23 – 05.1997	W.L. 405.0
° 23/1 – 07.1997	W.L. 440.0		

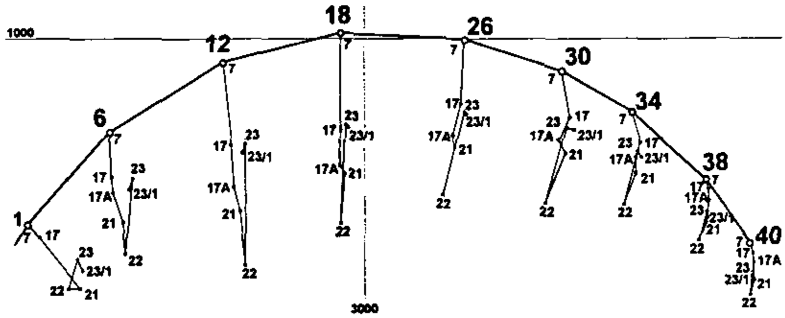


Fig. 7 Horizontal displacements of the dam crest by the traverse measurements

LEGEND

° 7 – 05.1986	W.L. 396.0	° 21 – 11.1995	W.L. 488.0 – 486.0
° 17 – 05.1991	W.L. 441.0	° 22 – 10.1996	W.L. 510.0
° 17A – 09.1990	W.L. 494.5	° 23 – 05.1997	W.L. 405.0
° 23/1 – 07.1997	W.L. 440.0		

VERTICAL DISPLACEMENT (SETTLEMENT)

One of the basic parameters to monitor the tectonic state of Enguri Dam is the summary value of dam body and foundation settlement and its distribution along the support outline of the construction. These observations were conducted using high precision levelling of the settlement benchmarks on the crest, in the 5 operational dam horizons and across roads. In the course of construction the recorded settlements did not exceed 5mm. During the initial filling of the reservoir the settlements progressively increased (23.1 mm at the VI horizons) but ceased once the design head was reached; in fact, in some horizons the settlements of the benchmarks (up to 22mm at II horizon) were observed to be reversible. Figs. 8, 9, 10 and 11 present the results of levelling at horizons 265, 315, 360 and 511.5m obtained in 1991, 1992, 1996 and 1997.

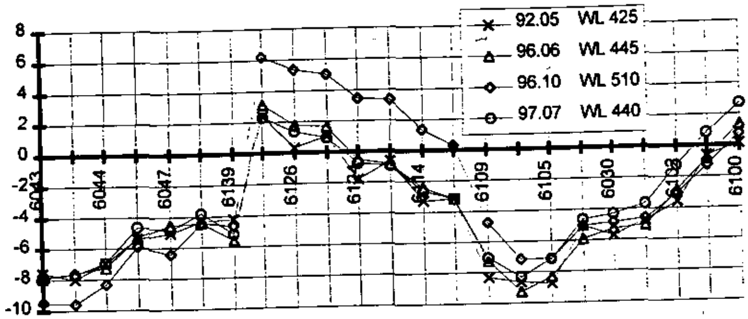


Figure 8 Vertical displacement at elevation 265m

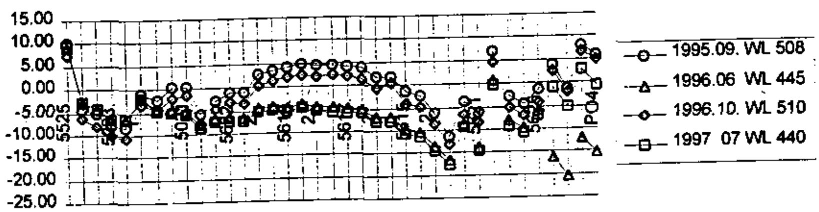


Figure 9 Vertical displacements at elevation 315m

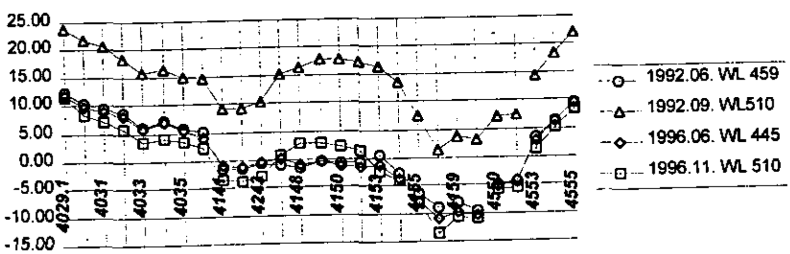


Figure 10 Vertical displacements at elevation 360m

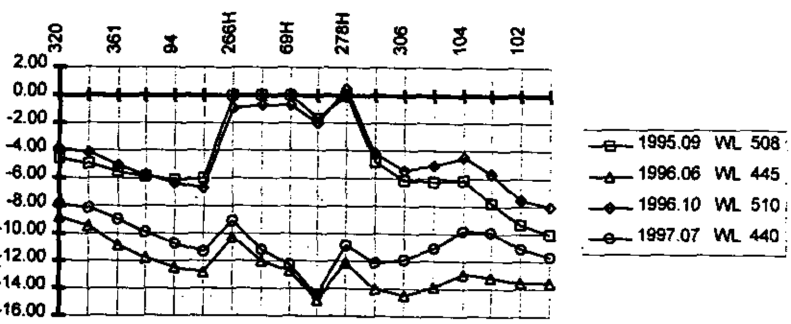


Fig. 11 Vertical displacements at elevation 510m

CONCLUSIONS

The data from geodetic and plumbline measurements of the dam-foundation system provide very valuable material for the analysis of its state and play an important role in the monitoring system of the Enguri Dam. This makes the restoration and elaboration of its measuring network a priority task.

The results of field monitoring of the general displacements of the dam obtained during its ten-year operation allow the identification of the following important points:

- the displacement range measured by the plumbline is increasing and is basically in conformity with the results of the geodetic measurements. The exception is left-side plumbline where the gap between the results of the two measurement types remains unexplained so far;

- In the horizontal displacements in the upstream-downstream direction recorded plumblines there are significant discrepancies in the readings below the right-bank wing of the dam. A level higher than 400m (above the right-bank fault) is undergoing significant displacement towards the valley; this evidently is the cause of the increase of deviation in the upstream-downstream direction along the plumbline 12;

- levelling conducted in recent years indicates that left-bank abutment of the dam foundation is higher than the right-bank (by 10-15mm); at the same time the character of displacements in these two zones (left-abutment at elevation 265m and right-bank of the dam at 315m) is different and cannot be explained unless addition measurements are made;

- the data are indicative of the fact that the foot of the dam (including the saddle) tends to displace towards the valley as the reservoir is being filled. This has led to the appearance of a tension zone in the rock foundation under the upstream face. This results in water penetration into more or less closed fissures in the rocky foundation under exceedingly high pressures (22-25 atmospheres) with subsequent damage to the grout curtain. Thus the part of the "pulvino" under the upstream face is affected by the increased backpressure and experiences uplift of 12-15mm in the course of reservoir filling. This phenomenon is also encountered in some other arch dams in wide valleys and requires special attention and thorough observation to ensure permanent control over the situation.

REFERENCES

Ghogovadze, G.I., Mgebrishvili I.M., Lomov I.E. (1987). Enguri Hydro Power Plant (in Russian). 200p. Energoatomizdat, Moscow

Kuznetsov G.I., (1983), Result of geodetic measurements of horizontal displacements of Enguri dam. Hydrotechnical construction N3, p 34-38.

Water storage for the Middle Level Cambridgeshire Fens

A ROWLAND, Binnie Black & Veatch, UK

SYNOPSIS. The Middle Level Commissioners are investigating options for the construction of a reservoir as an additional resource for water management and to supplement irrigation supplies. During winter, water is pumped from the arterial drains to the tidal Ouse: in summer, water is released into the Middle Level from the River Nene. Restrictions on abstraction have to be imposed frequently and water levels have been reduced unacceptably. Storage requirements could be 6000 Ml by 2020. Construction of a reservoir would safeguard supplies for irrigation and preserve the ecology of the river banks.

INTRODUCTION

The Middle Level is that part of the Great Level Cambridgeshire Fens which lies between the River Nene to the north west and the Great Ouse (Old Bedford River) to the south east and which is bounded by low clay hills to the south west (the highland) and by the marine silts of Marshland to the north.

Of the total catchment area of 70 000 hectares approximately 48 500 hectares are rateable fenland. The rateable area is mostly below mean sea level and is divided into 39 Internal Drainage Districts from which run-off is pumped to the main Middle Level drainage system by 78 pumping stations.

Attempts to drain the Fens were first made by the Romans and then throughout the Middle Ages. The present flat man-made landscape of black peat soils did not evolve until Dutch engineers substantially improved the drainage in the seventeenth century. The fenland areas of the Middle Level have never been heavily populated, most of the centres of population being located on the higher ground. Agriculture has historically been the main industry in the fens and remains so today.

WATER USAGE

Irrigation has always been practised in the fens to increase agricultural output. In the past this has been achieved by using subsurface pipes drawing water from the drainage ditches in which water is retained. Many farms still rely on these unlicensed abstractions as their only form of irrigation. The abstractions are made from the IDB drains sourced as necessary from the Middle Level watercourses through sluices, penstocks or other structures known locally as slackers. Recent years have shown a trend to the use of spray irrigation with water again being drawn from the drainage ditches.

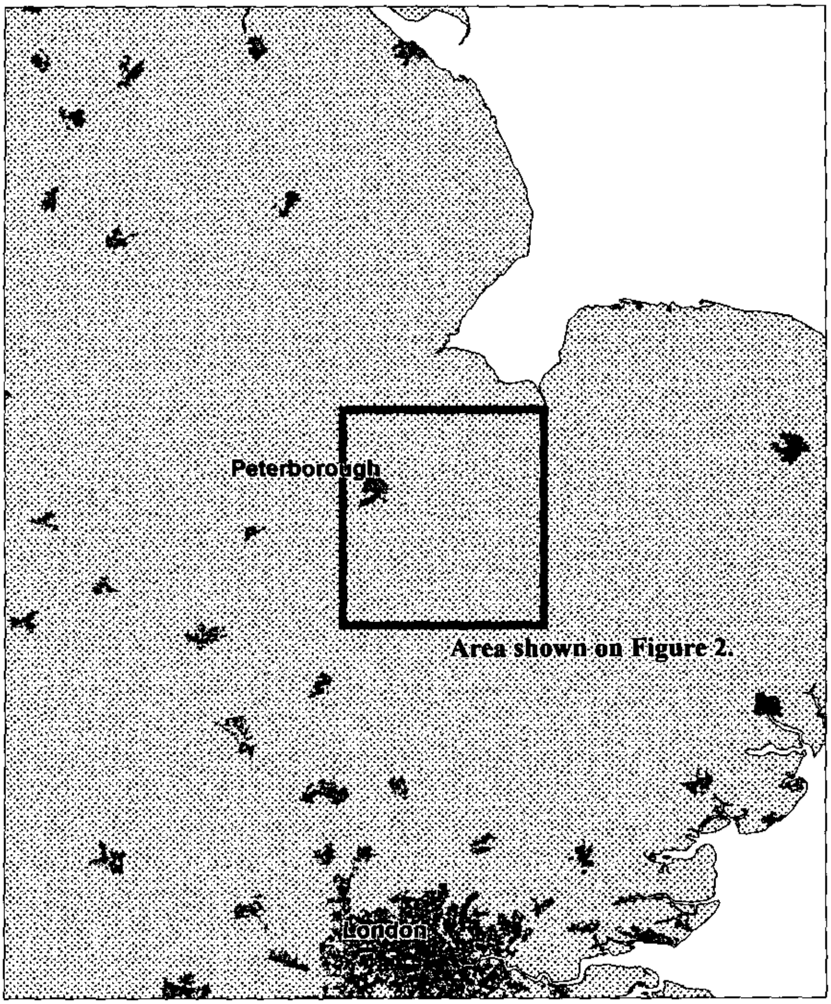


Figure 1. Location Plan

Abstractions for spray irrigation have to be licensed by the Environment Agency. After the 1976 drought an embargo was placed on issuing new summer abstraction licences because of the water resource problems. After the promotion of the Middle Level Transfer Scheme circa 1985 some additional new spray irrigation licences were granted but no new summer abstraction licences have been issued since then. New licences are being issued for summer abstraction only if backed by releases from storage reservoirs.

In dry years abstraction has to be limited in order to maintain water levels in the Middle Level main arterial drains. This limitation is generally effected through conditions in the licences that come into effect when water levels in the main drains drop below a given elevation or when flow in the River Nene is below a set minimum. In drought years the Environment Agency has the power to impose further restrictions.

As demand requires, water is released from the Middle Level arterial drains into the IDB drainage ditches from where the majority of the irrigation abstractions are made. Water levels in the Middle Level arterial system are supplemented by releases from the River Nene through Stanground Lock. Although there is no statutory limit to these releases, the capacity of the sluices limits the quantity to about 140 Ml/d. In dry years the Environment Agency requests that releases are reduced to maintain flows in the river for dilution, fish ladders and small releases to the North Level IDB and to ensure that the Nene Washes SSSI does not suffer from lack of water.

WATER SUPPLY PROBLEMS

In five of the last seven years summer abstraction for irrigation has had to be limited. Changing agricultural patterns and the demand for higher crop yields and improved consistency of quality are driving up the demand for water for spray irrigation. Reducing employment in thedominant agricultural sector can only be reversed by generating new industry in growth areas such as tourism and leisure. This will further stretch the demand for water during the critical summer months. The Middle Level drains have a high aquatic environmental value. Reductions in water level that occur during dry periods threaten the ecology of the banks of the watercourses and thus of the fens as a whole.

Although water level management has improved over the past few years in response to the increasing demands, the ability of the Middle Level Commissioners to fulfill their functions is limited by the lack of available water. A large storage reservoir is the only means of providing additional summer resources without endangering the environmental value of adjoining watercourses, such as the Nene and the Ouse.

ESTIMATE OF WATER DEMAND

There is no monitoring of the amounts of water released into the IDB drains through slackers and used for sub-surface irrigation. Annual returns by abstractors to the Environment Agency record the total annual usage for spray irrigation but do not give any direct measure of peak daily demand. Analysis of water levels and releases from the Nene through Stanground Lock for the summers of 1995 and 1996 suggests that the peak demand in dry periods is of the order of 170 Ml/d of which about 60 to 70 Ml/d is for spray irrigation and the remainder for sub-surface irrigation and maintenance of water levels.

If the current cropping patterns of one of the major farms, which cultivates an area of 7000 acres and has well managed water resources, were adopted by the whole of the Middle Level, the annual demand for spray irrigation would increase by nearly 4000 Ml a year.

It is estimated that the volume of storage that would have been needed to maintain water levels in the Middle Level and IDB drains in 1995 and 1996, which are considered critical years, is 1600 Ml. A further 400 Ml should be set aside to allow reduction in summer abstractions from Morton's Leam (River Nene). The minimum required additional storage is thus 2000 Ml and the maximum 6000 Ml to allow for increased abstractions for both agriculture and other industries.

SITE SELECTION
The client has requested that at this stage the locations of most of the sites are not revealed. The description of the sites is therefore in general terms.

There are three sources of water that can be considered independently:

1 St Germans Pond (run-off from the Middle Level otherwise pumped to the Ouse)
2 River Nene via Morton's Leam and Stanground Lock
3 Run-off from high ground in south west of the catchment (impounded before it enters the Middle Level drain system)

The 50 year return period minimum November to March run-off to St Germans Pond, as calculated from the pumping records for St Germans pumping station for the period 1933/34 to 1996/97, is 8000 Ml.

The average annual run-off to the River Nene is 295 000 Ml of which 74% occurs during the months November to March. Not all this is released through Morton's Leam, but the likely lowest volume in the period of record would have been of the order 12 500 Ml, well in excess of the proposed maximum storage volume.

Run-off from Bury Brook is unreliable. The lowest annual inflow in the period of analysis (1960/61 to 1996/97) was just 120 Ml in 1975/76. However, a storage volume of 1000 Ml would allow a two year storage cycle, which gives a yield of 750 Ml with a 5% chance of failure.

Twelve options were considered initially but the number was reduced to five sites. Options 1A and 1B abstract from St Germans pond water that would otherwise be pumped to the Great Ouse: Options 2A and 2B abstract directly from the River Nene; Option 3A impounds natural run-off.

1A Bunded non-impounding reservoir near Ramsey
1B Bunded non-impounding reservoir near Chatteris
2A Bunded non-impounding reservoir near Stanground Lock
2B Clay pit near Whittlesey
3A Impounding reservoir in south west highland area

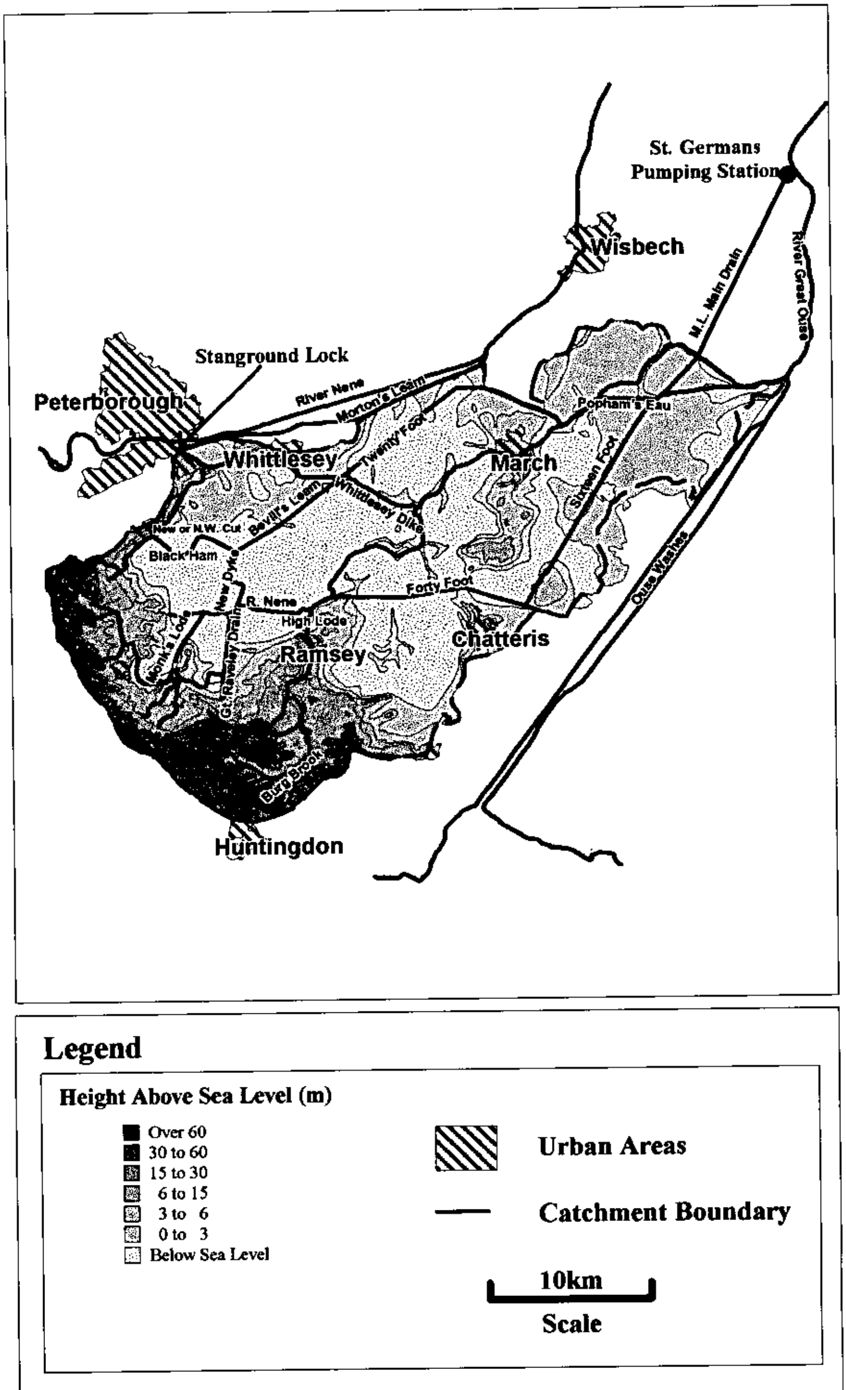


Figure 2. The Middle Level

GEOLOGY

Most of the area is covered by superficial deposits (drift). The solid formations are represented by parts of the Jurassic system. The formations present in the area are set out in Table 1.

Table 1. Geological succession

AGE	FORMATION	THICKNESS (metres)
Recent	Alluvium Nordelph Peat Barroway Drove Beds Lower Peat	Variable
Pleistocene	River Terrace Deposits Head Boulder Clay Glacial sand and gravel Glacial lake deposits	Variable
Jurassic	Corallian (Ampthill Clay, West Walton) Oxford Clay Kellaway Beds	30 65 6

The area is in the southern part of the fen basin and is bounded in the west by the dip slope of the Jurassic limestones and in the east by the Ampthill Clay. The fen islands (outcrops of the solid formations) are erosion relics often capped by gravel.

The geological structure of the area is relatively simple. The regional dip of the beds is uniformly east-south-east and does not exceed 5 degrees, although there are also localised flexures or folds. There has been some faulting of the Jurassic strata at Stilton and in the Peterborough area with associated flexuring. Elsewhere minor faulting or flexuring will occur. It is probable that these folds and faults were developed during tertiary times. Superficial disturbances have occurred throughout the district. Gentle cambering is probably present on all valley sides where localised limestone beds outcrop on the upper slopes.

TECHNICAL CONSIDERATIONS

The site selection has taken account of the regional geology and concentrates on areas where the solid deposits are likely to be close to the surface. Reservoirs located in areas where there are expected to be large depths of superficial deposits, particularly alluvium, are likely to present technical difficulties and be relatively expensive. The Lower Peat layers would compound these problems. Whilst gravel is suitable structurally for dam construction, the cost of providing a cut-off can be high.

No account has been taken of the groundwater level in the site selection process as it has been assumed that the rest water level would be within 1 or 2 metres of the ground surface at all sites. Where granular deposits are present, dewatering during construction and the need for hydraulic cut-offs have similarly not been considered for site selection but have been taken into account in assessing the suitability of the sites and in costing the selected options.

It was assumed for initial site selection that, if the site were generally suitable, there would be deposits of clay or clayey silts available for formation of a cut-off within the embankment and the foundation.

ENVIRONMENTAL APPRAISAL

The environmental appraisal was undertaken to identify the key issues that would need to be considered prior to implementation of any of the schemes. Responses to written consultation were received from:

Cambridgeshire County Council
Countryside Commission
English Heritage
English Nature
Environment Agency
Royal Society for the Protection of Birds

The main concerns expressed related to the possible deleterious effects of winter abstraction from the River Nene on the Nene Washes SSSI, the cumulative effects of winter abstraction on the Wash SSSI, the landscape implications of a bunded reservoir and potential damage to sites of archaeological importance. The desirability of including recreation as part of the proposals was also raised, there being a shortage of water based recreation in Cambridgeshire.

In the long term all of the five options are likely to have a similar level of environmental impact. At this stage the two most favourable options would appear to be one of the bunded options (Option 1B) and the clay pit (Option 2B). Option 1B seems likely to have limited impact on archaeology, habitats or species and does not require abstraction from the River Nene. Option 2B appears likely to have minimal impacts on archaeology, landscape, habitats and species and it offers the greatest opportunities for wetland habitat creation and recreational use. Its source is the River Nene but the quantities abstracted are unlikely to have any significant effect on the winter flooding regime of the Nene Washes.

The banks of the rivers and arterial drains maintain a delicately balanced ecology that is very dependent on the water level regime. The water shortages of the past few years have put that balance at risk and the Middle Level Commissioners in association with the Environment Agency have had to balance the needs of the farmers with their duties to the environment. It is acknowledged that drying out of the soils in the fens is causing damage to the ecology. The provision of additional storage allowing releases during dry summer months has a benefit not only to the water supply situation but also to the environment.

ECONOMIC ANALYSIS

The estimated capital and operating costs of the five options are set out in Table 2. Present values are calculated using a discount rate of 6% over a 50 year period.

Table 2. Summary of costs of options

OPTION	YIELD Ml	CAPITAL COST £	ANNUAL COST £	PRESENT VALUE £
1A	1000	3 474 000	7 500	3 334 577
1B	2000	6 093 000	12 500	5 817 210
2A	1000	3 976 000	6 500	3 772 806
2B	1000	2 254 000	7 500	2 211 123
3A	750	1 512 000	5 500	1 504 303

Options 1A and 1B are the pumped storage reservoirs drawing water from the Middle Level drains. Option 2A is the bunded reservoir drawing water by gravity directly from the River Nene. Option 2B is the use of an existing clay pit and Option 3A is the impounding reservoir on a river draining the higher ground in the south west of the area.

The value of water has been taken as 40 p/m^3, based on the improved cropping potential of crops irrigated in the Middle Level. Not only are yields increased but higher quality can be maintained as demanded by the supermarkets.

Environmental benefits are more difficult to quantify but work by other consultants in a similar context used a value of 10 p/m^3 of additional water available to maintain water levels in rivers by reducing abstractions. Using these values the benefit cost ratios and rates of return are set out in Table 3.

Table 3. Benefit cost ratios

OPTION	YIELD Ml	BENEFIT COST RATIO (40 p/m^3)	BENEFIT COST RATIO (40 + 10 p/m^3)	RATE OF RETURN (40 p/m^3)
1A	1000	0.84	1.04	1.75%
1B	2000	0.96	1.20	3.25
2A	1000	0.74	0.92	< 1%
2B	1000	1.26	1.58	7.2%
3A	750	1.39	1.74	7.9%

CONCLUSIONS

Although two of the options appear economic based solely on the value of water, consideration of environmental benefits improves the economic viability considerably. Assessment of environmental benefits for this study has been based on a simple approach and may underestimate their true value. A more rigorous estimate could be obtained by carrying out a contingent valuation. This does involve considerable work and surveys of public opinion.

It is most unlikely that the relevant consents or public and commercial support could be obtained easily for a winter storage reservoir in the Middle Level on the basis that its only benefit would be to secure increased irrigation abstraction. The environmental benefits, both in terms of protecting the ecology of the existing watercourses and of providing a potential leisure amenity, make the development much more attractive.

As engineers, we have often undersold and undervalued the environmental benefits of reservoirs and this has made their promotion much more difficult. A database of the value of such benefits would allow engineers and environmentalists to make a more objective evaluation of development of a reservoir compared with other options

ACKNOWLEDGEMENTS

We would like to thank Graham Clemmow of the Middle Level Commissioners and Pat Sones of the Environment Agency, who provided assistance and substantial amounts of data throughout the study.

We also acknowledge the time and information given by Richard Hall of English Nature, the staff at Cambridgeshire County Council, the Government Office Eastern Region, Cambridge County Council Estate Farms, Hanson Brick Ltd, Abbotts Ripton Farms Ltd and numerous farmers and landowners.

Challenges and opportunities for flood storage reservoirs

A T PEPPER, ATPEC Ltd, UK
D PETTIFER, Jeremy Benn Associates, UK
J FITZSIMONS, Environment Agency, UK

SYNOPSIS. Reservoirs used for flood storage have to comply with very different criteria to those used to store water for supply purposes, whether that supply is for potable water, irrigation or hydro-electric power. These differences in design and operation can affect the way the reservoirs are inspected and supervised under the Reservoirs Act. Historically little regard was paid to the aesthetics of these reservoirs, which might appear to be little more than dry embankments for the vast majority of their lives. Recent thinking is to incorporate existing features, e.g. old railway embankments, into new structures, and to ensure that any new works appear more natural than was often the case in the past.

BASIC CONCEPTS OF STORAGE

Flood storage occurs in all rivers where flow is varying with time, and is a function of river width, reach length, the stage-discharge relationship, and the steepness of the flood hydrograph. The effect of flood storage is to reduce peak flows, but to prolong the duration of high discharges, according to the continuity equation:

$$Qi = Qo + \frac{V}{t}$$

In this equation the flow into a reach (Qi) must equal the flow exiting a reach (Qo) less the change in volume (V) per unit time (t) within that reach.

Lakes and reservoirs are a special (and perhaps simpler) case of the river situation, where a large volume of storage can be utilised with a small increase in level. This is typically the case with most reservoirs, where fixed crest controls, such as weir spillways, are used. The water level over the crest rises to pass the increased flow, but this increased water level utilises a significant additional volume of storage, so attenuating the flood flow.

If gated outlets are used it is possible to pass increasing flows through the reservoir with no increase in level - hence no increase in storage and no attenuation of flood flows.

FLOOD STORAGE RESERVOIRS

The flood storage effects occurring naturally in rivers and lakes can significantly reduce peak flood flows, but in many cases some of the natural river storage has been lost through historic river improvement schemes.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

Often the 'natural' storage is insufficient to prevent high peak flows from causing flooding problems, and so the river engineer must look for solutions. Typically these might include channel enlargements (to carry peak flows at a lower water level) or raised flood banks (to contain high water levels), but another option is to reduce the peak flow by engineered flood storage at a point upstream.

There are a large number of reservoirs throughout the world that were constructed with the prime purpose of flood storage, to reduce the risk of flooding downstream. In the UK many are normally dry, or they may contain a low level of water for ecological or recreational purposes. A large number of these were constructed in the 1970's in connection with New Towns, such as Milton Keynes, and other towns earmarked for major expansion, often with their own Development Corporations, such as Northampton.

The Register of British Dams (1994) indicates that 42 reservoirs that fell within the scope of the Reservoirs Act 1975 were operated by the National Rivers Authority. Practically all of these would have flood storage as their prime or only function, and would be retained by earth embankment type dams. These are now the responsibility of the Environment Agency, and that figure has now increased with additional schemes having been constructed subsequently.

Flood storage reservoirs can be impounding or non-impounding reservoirs, (on-line or off-line), as shown in figure 1.

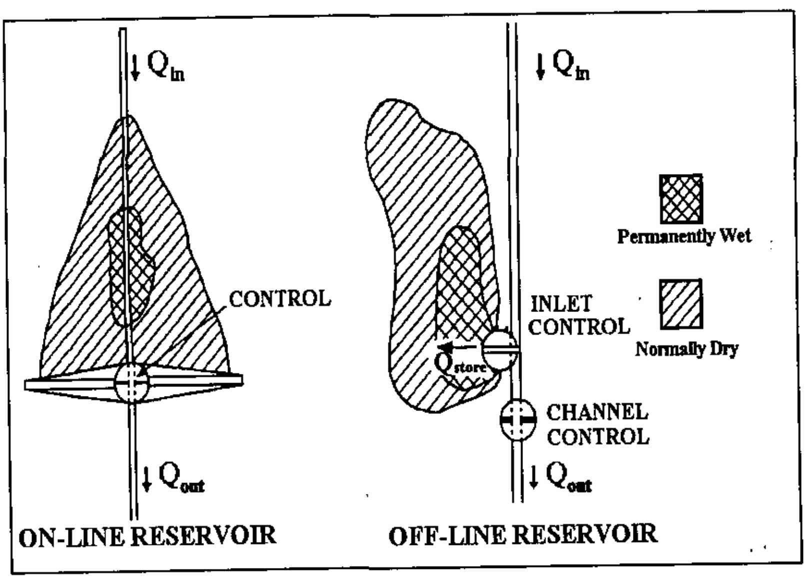


Fig 1 - Types of Flood Storage Reservoirs

Flood storage reservoirs differ from water supply reservoirs in a number of ways:

1. The reservoir may remain totally empty for several years in succession.

2. The land upstream of the dam, but within the reservoir, may have an alternate use, such as farming, when it is not storing flood water.

3. The land liable to be inundated at reservoir-full level may not be owned by the dam owner.

4. The dam can experience rapid fluctuations in the level of the water it is retaining, ranging from empty to full and back to empty again in a matter of hours.

5. The reservoir may never be filled at all while under the supervision of the Construction Engineer. Although in some cases it may be possible to carry out a trial filling, this could well be undesirable because of the time it would take, the effects on flows downstream, and of land use within the reservoir.

6. As the reservoir may be designed to be full only once in (say) 100 years - and then only for a few hours - it may never be seen by an Inspecting Engineer, or even a Supervising Engineer, when it is full, or nearly full.

7. Seepage, in terms of loss of water, is rarely a problem with flood storage reservoirs, although obviously seepage should not endanger the structural stability of the dam.

DESIGN OF FLOOD STORAGE RESERVOIRS

In the design of any reservoir the two primary criteria are a design outflow and a design inflow. In the case of water supply reservoirs the design outflow relates to consumer demand, and the inflow may be natural catchment runoff, possibly augmented by pumping or diversions. Both inflow and outflow are usually considered over a time period of several years, the reservoir providing long-term water storage.

This contrasts with a reservoir used for flood storage, where the maximum design outflow is the greatest flow that can be passed downstream without causing flooding, and the design inflow is invariably the runoff from a discrete flood event. The hydrological timescale is thus very much shorter when considering flood storage aspects of reservoirs.

The most effective provision of storage will be when the volume required is minimised to achieve a specific standard of flood defence downstream of the reservoir. Two criteria for this are:

a) Maximise the downstream channel capacity. It may be possible to increase the capacity of the downstream watercourse significantly for relatively little effort. However, as more and more works become necessary downstream it may be become more cost-effective (and environmentally desirable) to increase reservoir storage volume.

b) Store no floodwater in the reservoir until the maximum capacity of the downstream channel is reached. This will almost certainly require more complex controls, and the cost of these must be set against the need for a larger reservoir with simpler controls.

Typical inflow and outflow hydrographs are illustrated below. Figure 2 shows the ideal case, of a flat-topped outflow hydrograph, where no storage is utilised before the maximum permissible outflow is reached. This cannot, however, be achieved without mechanical flow control devices, although off-line storage can come close to the ideal.

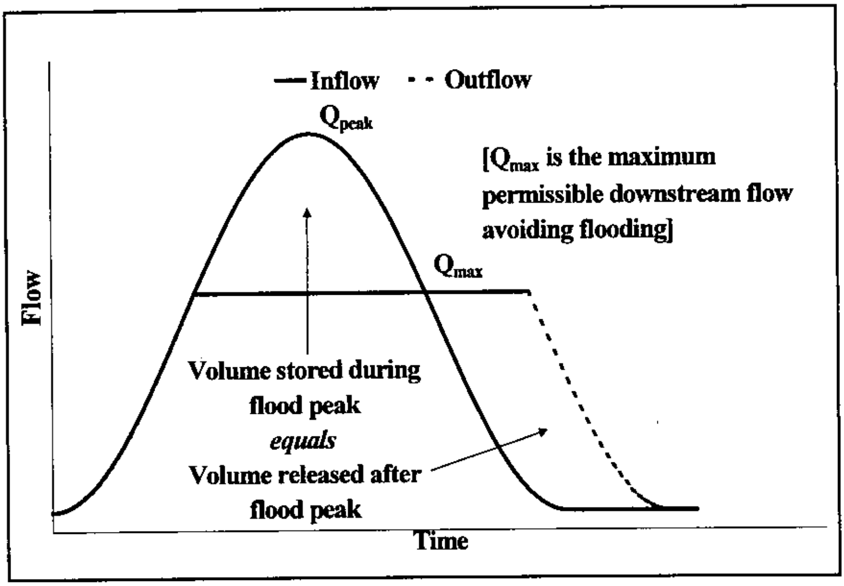


Fig 2 - Ideal Outflow Hydrograph for Maximum Useful Storage

This case may be contrasted to that of a very simple flood storage reservoir, comprising a dam with a culvert through its base, where the outflow at full reservoir level must not exceed the downstream channel capacity. Consequently at low reservoir levels the outflow is correspondingly reduced, and unnecessary storage takes place.

Figure 3 illustrates the resulting outflow hydrograph, as well as the attenuation in peak flow that can be achieved by storage - which also requires a delay in the time at which the peak flow occurs (translation). Storage volumes can typically be double in the case illustrated by figure 3 when compared to the ideal situation to minimise storage volume shown in figure 2.

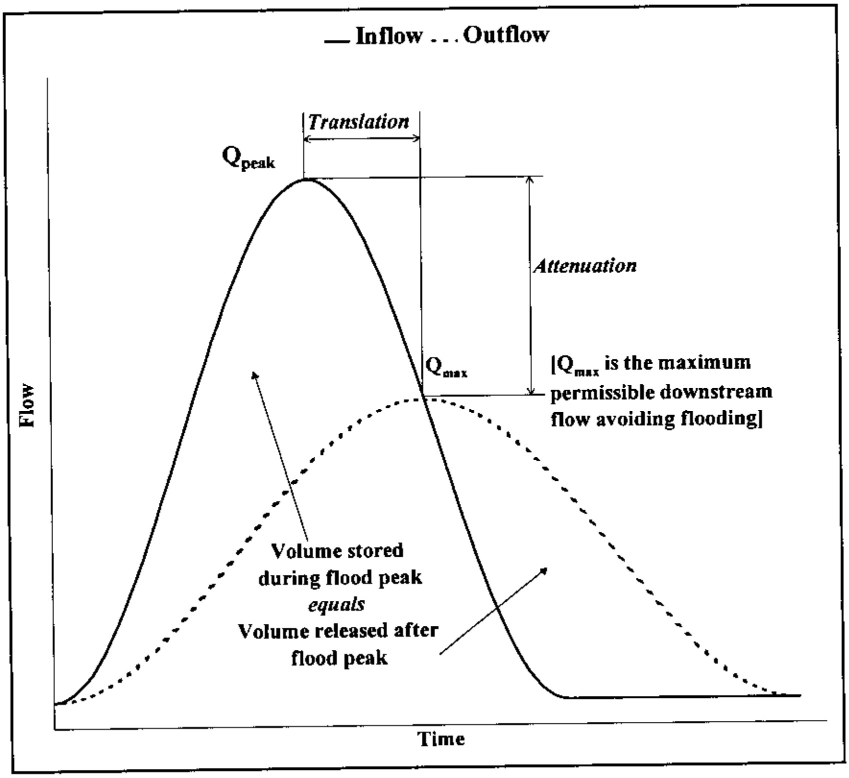


Fig 3 - Outflow Hydrograph from Fixed Orifice type of Control

For this reason complex control systems can often be justified in cost terms alone, although now there will also be a requirement for any new works to minimise any adverse environmental impact. By reducing the volume of storage needed the land area and the reservoir depth can both be reduced.

However, minimum volume and land take may not be the overriding design criteria in all cases. A fixed orifice type of control will typically release stored water more gradually than a constant outflow control, and this may be beneficial when the addition of hydrographs from any downstream tributaries is taken into consideration.

Once the maximum downstream flow is defined, the type of control can be considered in detail. Fixed controls usually lead to variable outflows during the storage cycle. For on-line reservoirs these can include culverts (with or without orifice plates) or flumes (for low-head reservoirs).

In all cases the correct operation of the control is crucial, and it is essential to allow for means of screening the orifice from the risk of debris build-up, as this would reduce outflow, and potentially fill the reservoir before the peak inflow.

Adjustable controls for on-line reservoirs can achieve constant outflow during the storage cycle, or at least achieve a more efficient use of storage than fixed controls. A variety of such controls are in use, and include the following, as examples:

Hydrobrakes - limit outflow to a near-constant value, regardless of head difference (e.g. Hothfield and Aldington)

Tilting gate within a concrete channel through the embankment - normally lying on the river bed, but rise to limit flow downstream, and hence raise levels upstream within the reservoir. (e.g. Garstang)

Penstock at the end of outlet culvert - normally fully open, but once the downstream water levels reach bank-full the penstock closes to limit outflow, from the reservoir. Automatic electrical operation. Penstock sized to take maximum downstream flow with no reservoir storage. (e.g. Weldon)

Penstock at the end of outlet culvert - As above, but radial gate closes as tailwater rises - maybe no power requirement (e.g. Barnwell), or closed by electric actuator (e.g. New Mill).

Off-line reservoirs usually achieve a near-constant downstream outflow, as they are typically located beside the upstream river channel, and the dividing bank may be in the form of a weir to allow floodwater to overtop and flow into the reservoir once the channel is full. A long side weir, at the highest possible level, maximises the available storage, but with shallow depths of flow the performance of a long side weir can become uncertain. A means of rapid drain-down must be devised for such reservoirs, as once the peak flow in the river drops below the maximum permissible downstream flow the reservoir needs to be quickly evacuated to be ready for a further flood event.

To avoid a long length of overspill bank, however, tilting gates can be installed on the dividing bank. These are normally fully raised, and lower to spill water into the reservoir when river levels become critical. Examples can be found at Catterall, and at the two washland areas providing flood storage for Lincoln.

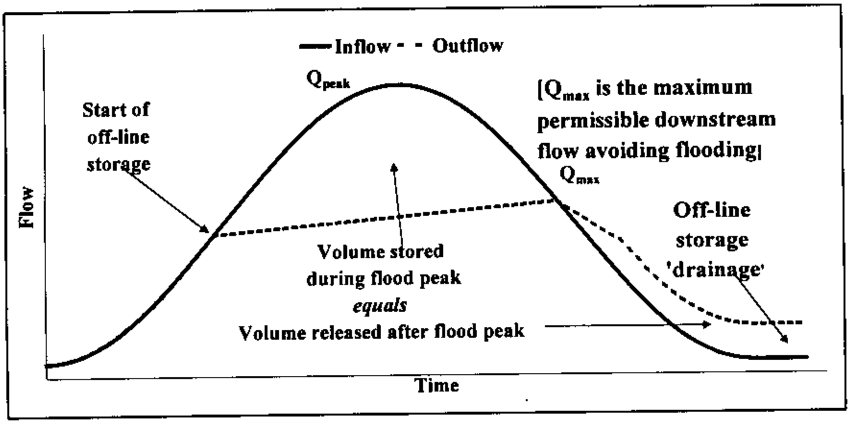


Fig 4 - Outflow Hydrograph from Off-line Flood Storage Reservoir

Figure 4 shows a typical outflow hydrograph from an off-line reservoir that is filled by a side weir, hence it must start filling just before the maximum permissible downstream flow is achieved. Often a separate outlet structure drains the floodwater back to the river, normally at a modest flow rate, when downstream levels permit.

Where mechanical gates are adopted as the means of filling an off-line reservoir they may be able to be used to return stored water to the river. In such cases it is possible to vary the rate of 'drainage' of the reservoir.

A more extreme means of spilling water into an off-line storage area can be found in Hungary, where a 'fuse-plug' method is used at a site in the southwest of the country. Here a 33km² area of forested land can be flooded by blasting out a long section of the river bank. This part is delineated by sheet piled walls at each end, and a substantial concrete weir (doubling as blast protection) at 2.4 m below the bank crest level. When the event has passed the earth bank is rebuilt, complete with plastic pipes to enable explosives to be inserted when needed. The storage area is drained back to the river via triple 2.4m x 1.8m box culverts with penstocks, capable of discharging over 20m³/s at the design differential water levels.

It may be considered ironic - and perhaps an anathema to water supply engineers - that a reservoir can be empty for several years, but when it is filled there is an urgency to empty it. However, the climatic and catchment conditions that create one flood are very likely to still pertain after that event, and a second flood peak is not uncommon.

Flood storage reservoirs are typically designed to contain the volume of the worst 1 in 100 year flood at reservoir-full level. Any subsequent inflows that are greater than the outlet capacity will then flow over the emergency spillway. Most such spillways are fixed crests - and many are built into the dam itself, rather than in original ground adjacent. When conditions are such that these spillways operate there may be some attenuation of flood peak due to the large reservoir area at and above top water level, but it has to be accepted that flows downstream will be greater than the channel capacity, and some flooding will occur.

The design of flood storage reservoirs is an iterative process, where the volume to be provided for a given standard of flood defence depends on the permissible outflow, type of control, and (if fixed controls) the difference between empty and full reservoir levels. At the extreme events for which these reservoirs are designed (typically to fill once in 100 years) it is often assumed that the rainfall return period equals the flood return period. There are, however, an infinite number of 1 in 100 year storms, of varying length and rainfall depth. To ascertain which of these storms will require the greatest storage volume it is necessary to carry out an extensive range of hydraulic modelling. This should initially be hydrological catchment modelling, which can produce hydrographs to feed into a hydrodynamic model of the proposed flood storage reservoir, complete with the proposed form of control.

It may be impossible to provide the necessary volume of storage in a single reservoir - perhaps because there are more than one branches of the river upstream of the area to be protected, as indicated in figure 5.

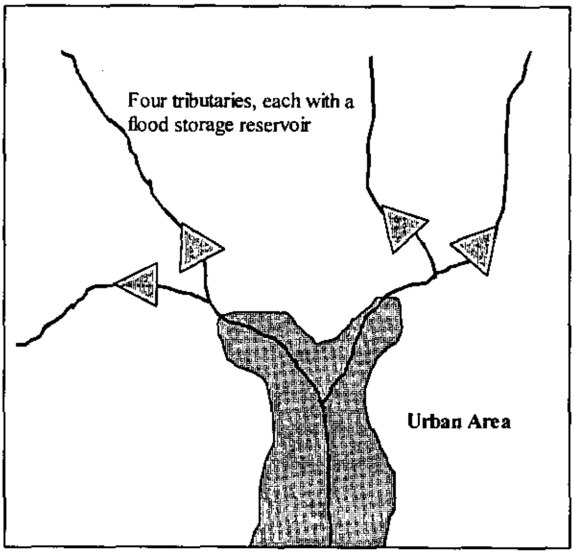


Fig 5 - Multiple Flood Storage Reservoirs

In this case more variables are introduced, as the total reservoir outflow into the urban area may be defined, but the proportional contribution from each of the four reservoirs can varied at the design stage to optimise the size of each, or take into account site-specific constraints. Examination of the numerous options in such a case makes mathematical modelling indispensable.

Multiple flood storage reservoirs may also be required along a watercourse, as is the case for the Tame Lakes Scheme in the densely populated and industrialised West Midlands. Here three controlled washlands and three flood storage reservoirs are used in conjunction to progressively store floodwaters, and thus limit flows to the capacity of the downstream reach. On this scheme areas normally used for playing fields have been designed to be flooded less frequently than those areas used for casual recreation or pasture.

ENVIRONMENTAL CONSIDERATIONS

Flood storage reservoirs constructed in past decades were formed by purely functional earth embankments, with little input from landscape architects or conservation interests. Consequently the embankment alignments were invariably straight, and the slopes uniform. These would be grassed, and mown at regular intervals. In many examples spillways are built into the embankment, to avoid additional land-take to one side of the dam. These spillways may then have a concrete crest and downstream glacis, although in some instances reinforced grass was used.

The control structures were often very obvious and substantial structures - more so for constant outflow reservoirs, which demand a more complex form of control than the fixed orifice that often suffices for variable outflow reservoirs. In addition, control systems often require power and telecoms, both of which are often supplied by overhead cables.

The creation of the Environment Agency has brought about a cultural change in the thinking of Flood Defence Engineers with an agenda to create even more environmentally acceptable schemes. All options for flood defence works are now rigorously examined for their environmental impact, and this includes alternative types of flood defence works.

Flood storage reservoirs constructed to reduce downstream peak flood flows in urban areas are often constructed upstream in rural areas, where the impact of "less than natural" construction methods can be detrimental to the ecology, habitat and landscape of a significant length of river. Here again the choice between several smaller reservoirs in different locations, compared to a single large reservoir, may be affected by environmental, as well as engineering, considerations.

Structures associated with flood storage areas constructed over the last three decades have in many cases caused an intrusion into the natural environment and new techniques should now be considered for the future to reduce the impact of these structures.

Controls hidden underground, embankments planted with acceptable vegetation wildflower seed mixes whilst still allowing a stable structure, or even the shape of the embankment altered to give a less uniform appearance.

The latter can be achieved more readily if there is surplus spoil that cannot be used in the embankment structure itself. Downstream velocity protection and spillway protection can now be achieved using natural materials, or geotextiles hidden beneath the surface. Bare-faced concrete has surely had its day in rural areas.

Consideration should be given to using existing natural and man-made features to create impounding dams. Many natural features exist that can be adapted, as well as old and not so old railway embankments, road embankments etc. The country contains many remnants of old structures that have become obsolete, being in many cases too costly to remove. It may be possible to use some of these for new flood storage schemes. In any case embankments built in the future must be sustainable, so that if they become obsolete they should not become eyesores and blots on the landscape, but blend into the local environment. Decommissioning of each structure should be considered at the design stage and "hard" structures avoided where possible.

A good example of using an existing feature is where the Agency is proposing to use a 200 year old embankment at Kenilworth Castle to re-create a defence - in this case from "war defence" to "flood defence". Doubtless some strengthening would be required to meet modern standards, but this would be carried out sensitively and with due consideration.

Old railway embankments have already been used, one example being at the Garstang flood storage reservoir, where the former embankment forms a substantial part of the main embankment across the valley.

Sometimes it is possible to create a new feature from a flood storage reservoir. This is the case with some of the lakes in Milton Keynes, and also at the National Exhibition Centre in Birmingham, where a major storage facility on the Holywell Brook now provides a focal point for many exhibitions. This would not, however, have been constructed without the flood defence requirement of storage provision.

CHALLENGES OF FLOOD STORAGE RESERVOIRS

There are a number of challenges that the Flood Defence engineer must consider when designing a reservoir for flood storage purposes. Whilst many of these are common to other types of reservoir design, there are some which are unique to reservoirs used solely for flood storage purposes. The list of considerations includes :

1. Determining the most cost-effective and environmentally acceptable combination of downstream channel works and storage reservoir works. This is complicated by the options for reservoir control, which will affect the reservoir size required.

2. Analysing a comprehensive range of hydrographs - and combinations of hydrographs - that will provide the worst case for storage once the downstream capacity and the method of control have been chosen. The hydrograph requiring the greatest volume of storage will not be the same one that gives the peak flow downstream on an *unreservoired catchment*, and it will vary depending on the type of reservoir and its control system.

3. Land take and land use - in many cases the operating authority may not wish to own and maintain the land that will be flooded on occasions, but in this case an agreement must be reached with the landowner to flood the land to different depths at unspecified intervals. It may also be necessary to restrict the use of the land - for instance, in one case no hay or straw was allowed to be left within the flooded area in case it was carried by floodwater and blocked the outlet works.

4. The time when a reservoir will be filled will be the time when surrounding land should be allowed to drain unhindered. The effects of a full reservoir on local drainage, perhaps some distance upstream, must be fully investigated. This is a particularly acute problem with the proposed Kenilworth Castle scheme.

5. Rapid changes in water level may occur in the reservoir - perhaps after months or years of standing empty. The geotechnical design of the dam must ensure that there will be no failure or weakening of the dam under such conditions.

6. Although not required under the Act, many Construction Engineers wish to see their reservoirs filled before issuing the Final Certificate. The testing of a flood storage reservoir under controlled conditions of filling and emptying may be impossible, and thus the Final Certificate may have to issued with the reservoir never having contained water.

CONCLUSIONS

Flood storage reservoirs can be a cost-effective means of flood alleviation, and can avoid or significantly reduce the environmentally damaging effects of channel works for what might be many kilometres downstream of a reservoir.

The impact of a flood storage reservoir on the environment can be very significant in terms of landscape, habitat and land use. However, not all these impacts need be adverse, as sympathetic design can create worthwhile environmental enhancements in certain situations. These may be wildlife areas - especially wetlands, or recreational facilities.

Flood storage reservoir design requires a great deal of iterative mathematical modelling if the optimum solution is to be achieved, which will take into account both the site-specific constraints and the need for a cost-effective solution.

There are several basic types of flood storage reservoirs, each having different hydraulic and hydrological characteristics. The choice will largely be influenced by available land and its topography. Within each type there are a wide variety of control structures to ensure that the reservoirs store floodwater that would otherwise lead to excessive flows, and flood damage, downstream of the site.

The design and operation of flood storage reservoirs have many fundamental differences to those of water supply reservoirs, not least being the rarity of filling, and both Supervising and Inspecting Engineers need to be aware of these differences.

Reservoirs and flood control: A Northern perspective

A MACDONALD, Director and
G A McINALLY, Technical Director, Babtie Group

SYNOPSIS. Recent flood events within the United Kingdom have focused attention on the use that can be made of reservoirs to reduce the impact of flood flows on downstream areas. This paper reviews three different approaches that have been adopted for the control of flood flows within the North of England and Scotland. These involve operational changes to existing reservoirs, as well as the construction of new areas for flood storage.

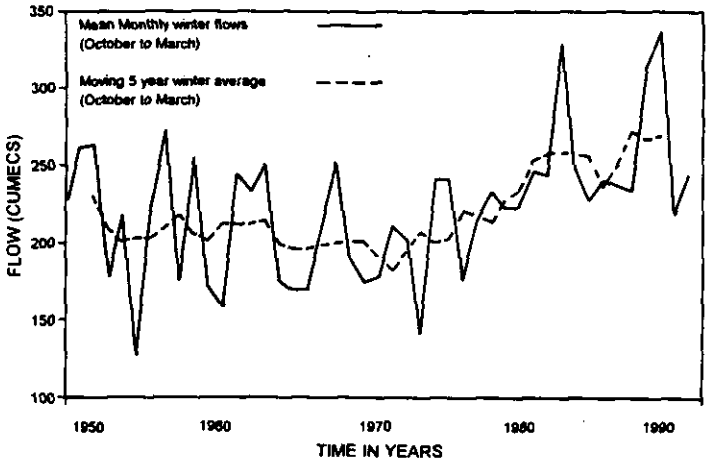
INTRODUCTION

There is an increasing amount of evidence being accumulated which indicates that there is a trend towards climate change giving a substantial increase in winter run-off from upland catchments. This is illustrated in Figure 1 which indicates a significant increase in the moving 5 year winter average flow in the River Tay at Ballathie Gauging Station in the period from 1970 to 1990. This is indicative of a trend affecting many Scottish river systems (Smith & Bennett, 1994).

The concensus of scientific opinion indicates that there will be a rise in annual global-mean surface air temperature over the next decade and beyond. Research (Hulme et al, 1993) suggests that this increase may be of the order of 1.7° by the year 2050 with a potential increase in precipitation of between 6 and 15%.

It is possible that increases in global mean temperatures may mask considerable regional variation. Over the period 1957 to 1992 the last decade has been identified as the wettest with a generally linear increase in precipitation particularly evident over the last two decades (Smith, 1994). Higher precipitation and river flows over this period have also been associated with increased storminess and westerly derived weather generally. It is possible that such weather patterns may lead to a greater incidence of snowfall followed by rapid snowmelt events accompanying the frequent passage of warm and cold fronts associated with Atlantic depressions.

Major flooding events have occurred within the central belt of Scotland in recent years, at Perth, 1993, Paisley, 1994, and Kilmarnock, 1996. The public are becoming less tolerant of flooding affecting their property and this, together with the apparent increase in the frequency and severity of flooding, has prompted feasibility and design work on a number of flood protection schemes. Some of these schemes have looked at the potential for the use of existing reservoirs as part of the comprehensive flood control system. This paper describes studies that were undertaken following the Perth and Paisley flood events in respect of the role that reservoirs could play in flood mitigation.



1 - Increased Mean Monthly Winter Flows at Ballathie

In the North of England extensive use has been made for many years by the Environment Agency of washland areas to take the peak off a flood prior to it passing through urban areas. The Environment Agency have now accepted that many of these washland flood storage areas should be classed as large raised reservoirs under the Reservoirs Act 1975 and have instigated a programme of supervision and inspection to comply with the requirements of the Act. However, new washlands are still being constructed and this paper outlines the approach being taken to floodplain storage on a scheme on which construction commenced in 1997 on the River Irwell upstream of Salford.

STANELY RESERVOIR, PAISLEY

The use of existing reservoirs for flood attenuation purposes is an option that may increasingly be considered by flood authorities when the provision of direct defences downstream may not be the most economic or environmentally acceptable course of action. It is often the case that existing reservoirs if properly managed can provide significant flood attenuation of flood flows where the catchment upstream is relatively small.

One such example is currently being considered by Renfrewshire Council to provide attenuation of storm flows from the Gleniffer Burn and to provide protection to the properties located immediately downstream. The location of the reservoir is shown in Figure 2.

The reservoir in question is Stanely Reservoir, located in the south west of Paisley which historically supplied the mills and local industries with water. The reservoir is currently owned by West of Scotland Water who have been reviewing its usage and consider it to be surplus to requirements at this time. Its capacity is approximately 0.9 million m^3 and it is formed of two discrete units with a shared embankment connected by a lowered weir section.

Inflow to the reservoir was originally by a diversion channel from the Gleniffer Burn controlled by a series of sluice gates. These gates have since been removed and all flow now bypasses the reservoir flowing along a section of the crest in an open channel. Outflow from the reservoir to the Gleniffer Burn is via three 3.5 m spillweirs.

The area downstream of Stanely Reservoir has a history of flooding. The areas at most risk are those of Park Avenue, Stockholm Crescent and Neilston Road. The largest flood event in recent times was in December 1994 when significant inundation occurred to these areas. The cost of direct damage for this event has been estimated at approximately £1 m.

A range of mitigation measures were identified to provide protection to the areas directly affected in 1994. An integral part of this scheme would be the use of Stanely Reservoir as an on-line flood attenuation scheme. It has been demonstrated that if Stanely Reservoir was to be used in isolation the risk of flooding downstream could be reduced from the current 1:2 year recurrence interval to a 1:10 year recurrence interval. If however additional off-line storage was provided at playing fields at Moredun immediately downstream of the reservoir, the risk of flooding downstream could be reduced to a 1:100 year return period.

In order to determine the viability of the use of Stanely Reservoir a full hydrological study was carried out for the Gleniffer Burn catchment. In addition a full flood study was undertaken to determine the existing freeboard of Stanely Reservoir at probable maximum flood. The PMF figure was determined to be 14 m^3/sec as a result of the limitations of the channel and culverts upstream. Using this figure the works required to adapt the reservoir to act as an on-line storage reservoir were assessed. The works required were limited in scale and included new inlet works, modifications to the existing draw-off works and minor works required in the interests of safety.

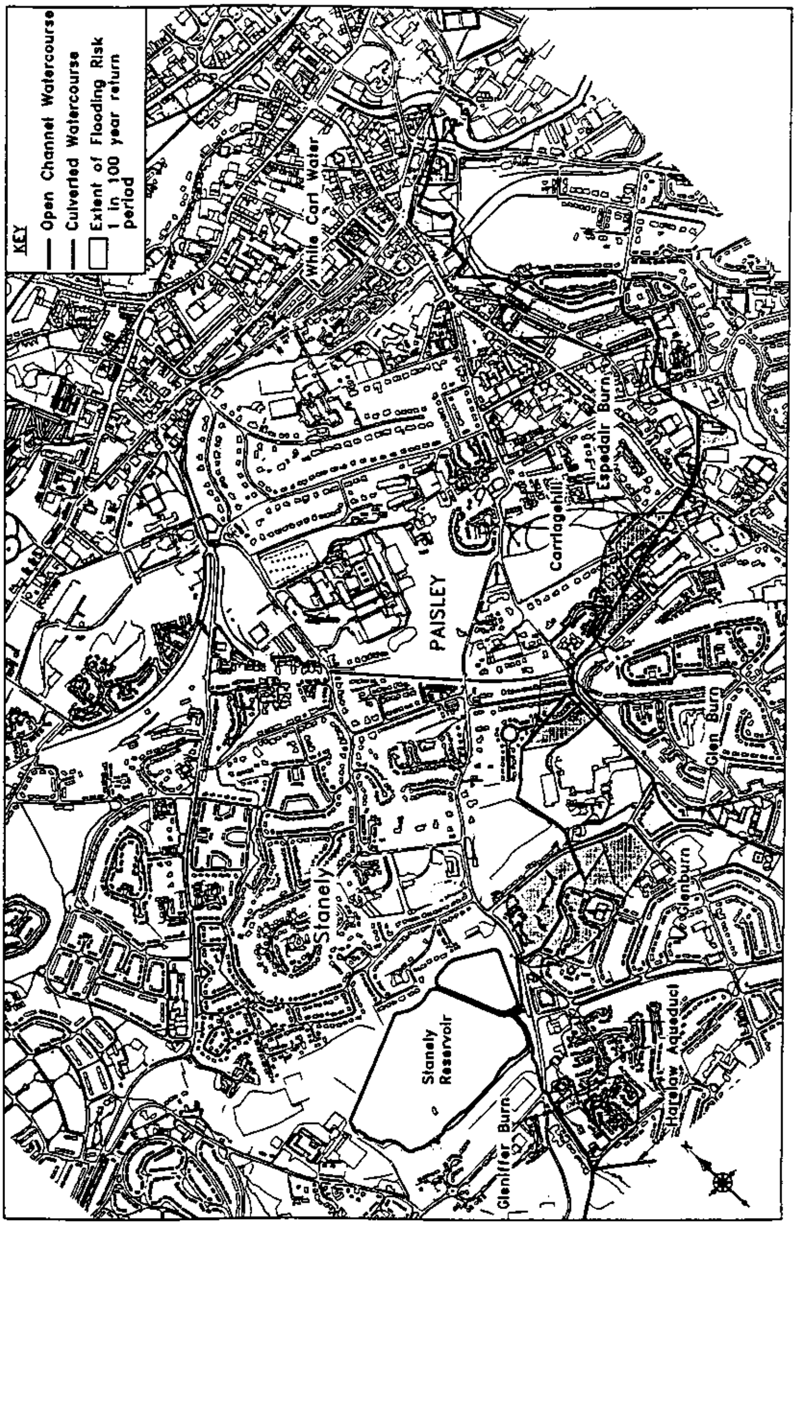


Figure 2 – Stanely Reservoir

The estimated capital cost of the works to the reservoir is of the order of £60,000. The cost of the provision of additional storage at Moredun has been estimated at £500,000. The cost benefit analysis has indicated a ratio of approx. 1.1 for the 1:100 year level of protection based on current cost estimates.

Negotiations are currently underway with West of Scotland Water Authority to hand over ownership of the reservoir to Renfrewshire Council for flood attenuation purposes.

The incorporation of Stanely Reservoir into the flood alleviation scheme for south west Paisley has demonstrated that it is possible to adapt existing reservoirs to act as attenuation storage ponds at a relatively minor cost. However, in this case the potential conflict of continuing use of the reservoir for water supply purposes has not occurred and has made its incorporation into the scheme very much easier.

PERTH FLOOD PREVENTION SCHEME

The River Tay is the largest river in the United Kingdom in terms of discharge with a catchment area upstream of Perth of approximately 4,970km^2. It drains a large area of the central Highlands and has a mean daily discharge of 167 cumecs and a maximum daily mean flow recorded in January 1993 of 1965 cumecs. The River Tay is renowned for its salmon fishing and, more recently, for its propensity to flood extensive low lying land along its length.

Scottish Hydro Electric plc (SHE) operate two schemes in the upper reaches of the Tay Catchment. These are known as the Tummel Valley and Breadalbane Schemes. These systems consist of a series of pipes, tunnels and aqueducts which re-route a proportion of flows to various lochs and reservoirs to augment supply to the power stations. In total flows from 1980km^2 of the Tay Catchment are influenced by SHE Operations. The Tummel Valley scheme alone covers 1339km^2, representing 46% of the total catchment area upstream of the confluence of the Rivers Tay and Tummel. Fig. 3 identifies the River Tay Catchment and areas influenced by SHE operation.

Tummel Valley Scheme is the most complex of the systems. The general rules for operation use the intermediate catchments of Lochs Eigheach, Rannoch and Tummel for generation whilst the main storage reservoirs of Lochs Ericht and Errochty are used to top up the intermediate lochs as necessary. The schemes are operated to minimise spill as any lost water is effectively lost revenue.

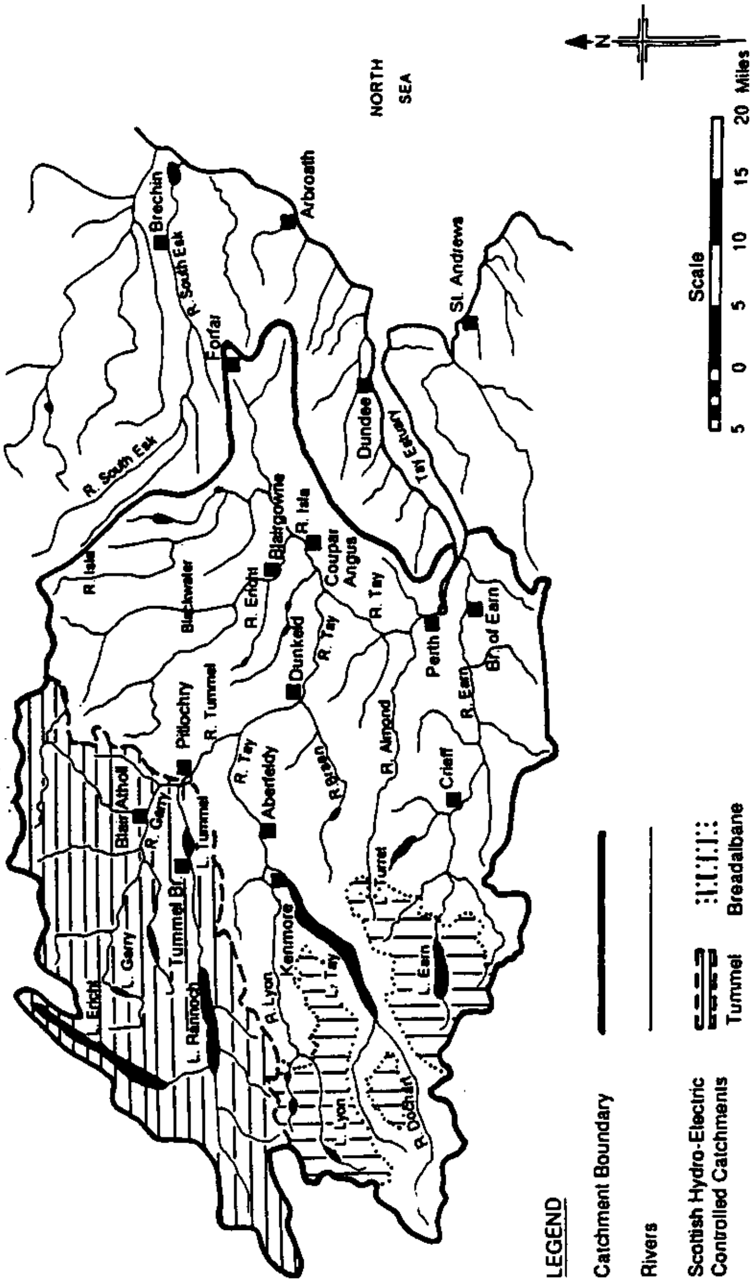


Figure 3 – River Tay Catchment

During a flood event the system is run under a modified set of rules whereby the efficiency of generation and revenue considerations are important. Prior to a flood event, or if a forecast of extreme rainfall is given, the schemes may run at full generation in order to keep the levels of the reservoirs low. In contrast during a flood, generation may cease to enable impounding to commence while the peak flow passes through the system downstream. During a potential spill situation the water normally diverted from other catchments will be turned back.

The operators must not only consider the immediate requirements of managing the flood flows but must have due regard for the condition of the main storage reservoirs for the following season. The Tummel Valley Scheme is a cascade system and has been designed to run as a balanced system during an average winter month ie. the storage in the lochs remains unchanged. This means that during a wet period, when heavier than average run off is sustained, the storage in all the reservoirs will be steadily depleted even during heavy to full generation. Thus the only substantial impounding potential lies in the main storage reservoirs. The main storage reservoir is Loch Ericht with a storage volume of 230 Mm^3. This not only acts as an impounding reservoir, but must also supply compensation water during dry spells. It is therefore important that this reservoir is used with great care as a low winter level could result in a shortfall the following summer if a long cold winter is experienced.

The Tummel Valley and Breadalbane hydro-electric schemes have an important effect in increasing the available storage throughout these catchments and in providing the capability of holding and diverting flows during flood events. The release of water from the hydro schemes during flood events can also introduce large volumes of water downstream in a relatively short period. The operating procedures adopted by SHE during floods can therefore have significant implications for flood warning and close liaison is essential between SHE and the Scottish Environment Protection Agency who operate a flood warning scheme for the Tay and Earn Valleys at these times.

<u>The February 1990 and January 1993 flood events</u>

Following the construction of the hydro schemes in the 1950's there was a relatively flood free period when extreme flood flows were not experienced. However, on 4 and 5 February 1990 there was a significant flood event on the River Tay which led to serious inundation of farmland and disruption to road and rail services. Three years later, between 14 and 18 January 1993, a more severe event occurred. The return periods of the February 1990 and January 1993 events have been estimated to be of the order of 1 in 20 and 1 in 100 years respectively using data from the period 1948 to 1994 at Ballathie.

The flood flows registered in February 1990 and January 1993 were the highest recorded at Ballathie since records began. Both floods were similar in that extremes of rainfall were experienced before and during the flood event, together with a build up of snow immediately preceding the flood followed by a rapid thaw which contributed significantly to the peak flow. During both events significant areas of agricultural land were inundated in the Tay and Earn Valleys. Of particular significance was the widespread flooding in the City of Perth which occurred in 1993 and the breaching of embankments at North Muirton and the subsequent inundation of the North Muirton housing estate.

The February 1990 flood was generated principally in the north and western areas of the catchment. Antecedent conditions in January left the catchment saturated and rivers running at a dangerously high level. By the end of January a considerable amount of snow had accumulated on the higher ground in the catchment. During the first five days of February an occluded front crossed Scotland bringing extreme rainfall in the western catchment and unusually high temperatures. The result of this was massive runoff and peak flows experienced throughout the Tay system.

During this flood the main storage reservoir of Loch Ericht did not spill although the volume of stored water increased markedly from 52% of capacity on 1 January 1990 to 94% on 1 March 1994. Following the excessive wet period in January, Rannoch power station was generating heavily 24 hours per day from the 27th January. However, during the flood event itself both Rannoch and Errochty stations were shut down for a period while the River Garry Peak passed through Pitlochry Dam. This led to a reduction in flow of the order of $90m^3/s$. The Rannoch station recommenced generating after the peak flow in the River Garry had reduced by $250m^3/s$. The peak spill from the Clunie dam was $189m^3/s$ and the peak flow over Pitlochry Dam estimated as $970m^3/s$.

The January 1993 event was the result of runoff from catchment wide rainfall and snowmelt. Throughout the first 18 days of January rain or snow was recorded every day at most stations in the Tay and Earn catchments. Recorded rainfall figures were substantially greater than average for this period and, in contrast to February 1990 were experienced across the entire catchment. In addition by the 14th January widespread snow cover was present over the entire catchment, even at low elevations. Between the 14th and 16th January a general increase in temperature occurred rising to a recorded maximum of 13.2°C in the south of the catchment. This temperature, coupled with the heavy rainfall was enough to cause widespread snowmelt at all levels and the subsequent peak flows recorded. The peak flow recorded at Ballathie was approximately 30% higher than the

February 1990 peak. This increase is almost wholly attributable to the contribution from the River Isla.

Flows generated upstream of the River Isla were broadly similar for both events. Flows were released early from the Rannoch-Tummel Scheme in an attempt to allow the peak flow in the Tummel to pass prior to the peak from the Garry arriving. Nevertheless the peak flow recorded downstream was $1048m^3/s$ demonstrating the limited ability of the Hydro Schemes in controlling flows.

During this event main storage lochs of Ericht, Errochty, Lyon and Giorra did not spill and continued to impound flow during the flood event. Prior to the event controlled spill was initiated from the intermediate reservoirs following a warning from the Meteorological Office of a sudden rise in temperature and the known existence of widespread snow cover. Rannoch power station was shut down on January 17 at 02.40 hrs and did not recommence generating until 18.40 hrs on January 19.

Following each of these floods there were calls for a review of factors that could have contributed to the magnitude of the events and measures that could be adopted to reduce the impact of the flooding. Amongst the factors that were considered was the operation of the hydro schemes and the possibility of SHE playing a greater role in the control of flood flows from those catchments affected by their operations.

Hydrological Studies

A hydrological study of the Tay catchment was commissioned in 1993 by Tayside Regional Council. One of the aims of this study was to consider the role of the hydro-electric schemes during flood events and to determine if there was scope for a reduction in flood risk in the lower Tay valley through positive operation of the reservoirs or through the provision of increased storage capacity.

The results of the study suggested that the existing hydro scheme operation provided a reduction of peak flows at Ballathie of between 3 and 4% for the January 1993 event. If further storage was provided, for example by raising crest levels, the flood peak could theoretically be reduced although it was noted that this was unlikely to be a realistic proposition due to commercial and environmental considerations.

The option of construction of a new dam in the River Garry was examined in outline. A provisional estimate of storage volume required to prevent flooding at North Muirton was 50 million m^3. Provision of 35 million m^3 of storage on the lower River Isla would also theoretically have reduced peak flows at Perth such that North Muirton would not have flooded in 1993. Both these options were considered to be impractical.

A feasibility study was carried out to examine the options available for the provision of direct flood defence measures for the city of Perth. Indirect flood defences such as upstream storage or greater control of the hydro schemes were considered but were found to be insufficient as measures on their own in terms of significantly reducing the risk of flooding at Perth. Thus a comprehensive flood prevention scheme was prepared for the city of Perth. This scheme is due to commence construction in 1998 at an estimated cost of £21 m.

Scope for the Control of Floods

In general the primary constraints on the effective control of flood flows in hydro schemes are the location and size of the storage reservoirs and the uncertainties that currently exist in forecasting. If reservoirs are to play an enhanced role in reducing flood flows during an event then it would be necessary to draw down the levels of these reservoirs prior to a flood event occurring. Depending upon the size of the reservoir this could take a period of weeks or months and given their location relative to the whole catchment would not necessarily assist in alleviating the flooding situation downstream. The experience of the January 1993 flood event was that the main storage reservoirs of Ericht, Errochty, Lyon and Giorra did not spill during the flood event and the majority of the flood runoff was generated from the uncontrolled catchments.

If the main storage reservoirs were drawn down in anticipation of a perceived flood event and the event did not materialise then the consequences for future operation of the scheme could be severe. As indicated earlier the main storage reservoirs are not only used to impound water during wet periods but are also used to provide compensation water during dry spells. If these storage reservoirs did not fill over the winter period following a forced draw down then there would be serious implications for the future generating potential in the following summer period. Long term forecasting and flood prediction techniques are not yet sophisticated enough to provide the operators of the hydro schemes with sufficient confidence to draw down storage reservoirs to low levels in anticipation of flood events.

The control of flood flows in the intermediate lochs such as Rannoch and Tummel is limited due to the relatively low storage capacities in each. A prolonged period of greater than average runoff will reduce available storage in these reservoirs and there is limited scope for a dramatic drawing down of levels in advance of a serious flood event.

There is however some potential for a limited drawing down of levels in these intermediate lochs and reservoirs in the period immediately preceding a flood event, based on short term weather forecasts and the use of weather radar in tracking and assessing the implications of storm conditions and precipitation/snowmelt over the catchment. Judicious operation of the reservoirs during medium scale flood events can be beneficial in influencing the relative timing of peaks, particularly those of the Rivers Tummel and Garry. By continuing to store water in Lochs Ericht and Errochty and ceasing generation from these sources for a period the rate of rise in the levels of Lochs Rannoch and Tummel can be reduced, delaying the onset of spill. Spill can then be introduced downstream in a controlled fashion after the peak on the River Garry has receded. In medium scale flood events (ie. to 10 year Return Period) such action can be beneficial.

Problems occur in major floods when high flows in uncontrolled spate rivers such as the Garry are prolonged. In such events there is a tendency for available storage in Lochs Rannoch and Tummel to become exhausted and spill from the system then has to be introduced when the River Garry is still running at high levels.

Improved short term weather forecasting, the use of weather radar and the links developed between SHE and SEPA for flow monitoring and flood warning are now being utilised to improve the options for short term flood management with the aim of reducing flood flows still further.

RIVER IRWELL FLOOD CONTROL SCHEME

The River Irwell rises at Irwell Springs north of Bacup, Lancashire and drains a catchment area of 560 square kilometres to the north of Manchester. The catchment combines predominately bare moorlands with steep sided, heavily urbanised valleys, producing a river which displays a rapid response to rainfall due to the generation of high levels of run-off.

The river passes through Salford, Manchester, where there is a well recorded history of flooding with reliable estimates of flow dating back to 1880. Flooding in Salford has caused significant damage over the years with the 1866 and 1946 events being the worst on record. The estimated peak flow in the latter was in the order of 562 cumecs and affected 243 hectares of land, flooding 5000 residential and 300 industrial premises.

The most recent major flood occurred in October 1980, being estimated at 412 cumecs, following which there were calls for improvement works to guard against the possibility of future flooding. Short term improvements were undertaken at that time by means of local bank raising to eliminate low spots in the defences. A detailed hydrological and hydraulic study of the River Irwell through Salford was undertaken in the period 1994/95. This confirmed that the level of flood protection was unsatisfactory and identified that a flood storage scheme utilising two major floodplain areas, coupled with improvements to the defences in downstream reaches would be the preferred solution to providing a satisfactory level of protection.

The proposed solution

The proposed solution to provide a 1:100 year standard of flood protection to the Salford area of Manchester consists of:-

a)Construction of two flood storage basins in the area of the existing Littleton Road and Castle Irwell Playing Fields.

b)Construction of flood walls and embankments downstream of the flood storage basins to form a channel capable of conveying 450 cumecs with 600 mm freeboard.

c)Installation of non-return flap valves at various outfall locations.

d)Installation of a telemetry system for operational and flood warning purposes.

Design parameters

The Adelphi weir is around 2.5 kms downstream of the Castle Irwell basin. A 100 year peak flow at this weir was estimated to be around 550 cumecs. The aim of the flood storage scheme was to reduce the peak flow in the river at this location to around 450 cumecs.

In order to achieve this the hydrological model of the proposal indicated that the storage capacity of the Littleton Road basin should be around 657,000 m^3, with the Castle Irwell basin having a storage capacity in the order of 680,500 m^3. The flood embankments around the storage basins were to be constructed using material from within the basin area itself, by means of a cut and fill operation. The total volume of material to form the structural part of the embankments was around 85,000 m^3. In addition, significant volumes of fill were to be used in landscaping the flood embankments due to the sensitive nature of the sites and their proximity to urban areas.

A minimum freeboard allowance of 700 mm was adopted to the top of the flood embankments from the 1:100 year return period level in the adjacent river.

Geotechnical design

Both sites are underlain by the Sherwood Sandstone, which outcrops locally in the river. Above this lie alluvial deposits of clay and of sands and silts, as well as sands and gravels and laminated clays. However the majority of the material which could have potential to be won easily as fill consisted of sandy silts and silty sands to a depth of around 2 m. The site investigation indicated significant inter-layering of the sands and silts with variation in moisture content from close to optimum to well wet of optimum.

A trial embankment indicated that the material could be successfully excavated and placed using normal construction plant and the embankments around the Littleton Road Playing Fields were constructed in 1997.

The fill specification called for the material to be placed in 200mm layers using four passes of a vibratory roller with a mass per metre width of 3,400 kilograms. Fill suitability was assessed using the moisture condition apparatus. The contract commenced in June 1997 and was completed in October the same year. The total volume of material excavated and placed during this period was 69,000 m cumecs^3.

<u>Inlet/Outlet structures</u>

Each of the flood storage basins has an inlet/outlet control structure near to its upstream end. This comprises a reinforced concrete channel incorporating a single, hydraulically operated, automatically controlled over-shot gate of the fish belly type. The gate is designed to normally rest in the raised position so as to maintain the flood defences and will be lowered during extreme events to divert river flow into the flood basin.

The hydraulic design of these inlet/outlet arrangements was checked using Babtie Group FLOODTIDE software. A fully dynamic river model was constructed representing the adjacent reaches of the River Irwell with the flood embankments in place. This model was also used to simulate a complete flood hydrograph and hence check that the proposed scheme provided the required level of flood storage under various flow conditions, and to assess velocities for scour protection along the embankment.

A small land drainage pumping station has also been incorporated into the design. This is to ensure that the playing fields continue to have adequate drainage even at relatively high water levels in the river, and to assist the final evacuation of flood waters after the operational use of the storage basins.

A telemetry out-station is provided on each of the flood storage basins to provide data on rainfall, river level, stored water level, and sluice gate status. This is used to control the operation of the flood storage basins and provide real time modelling of the river/storage system as part of the Environment Agency North West Region's flood warning service.

CONCLUSION

In considering the potential for reservoirs to play a greater role in flood control it must be recognised that this is entirely dependent on operational, geographical and economic constraints. In the case of gated structures such as exist on the Scottish Hydro schemes or the River Irwell flood storage basins, a complete knowledge and understanding of conditions throughout the catchment and the consequence of operating decisions is required. Access to an extensive hydrometric network together with reliable short and medium term weather forecasts, and a knowledge of the flows and weather patterns in uncontrolled catchments are therefore essential for the scheme to operate as designed.

In the case of the more passive systems such as that proposed at Stanely Reservoir, the basis for the design of the flood prevention scheme should be made clear to the owner such that he can ensure the reservoir water level is held down to the required degree. This in itself has implications for reservoir safety in that the core of the embankment may be above water level at its higher levels for a considerable part of its life. The Supervising Engineer should therefore be instructed to closely monitor the embankment following major flood events, taking particular note of any increase in seepage flows.

As the effects of climate change become more apparent it is likely that further consideration will require to be given to the multi-purpose use of reservoirs. This requires a balance to be drawn between the needs of the owner, who may wish maximum storage to be available for water supply or hydro generation purposes and the conflicting need of lower water levels for the provision of flood storage. More advanced and accurate rainfall forecasting systems and knowledge of the location, amount, duration and type of precipitation and variations in temperature would allow a better compromise to be drawn between these requirements and may result in more use being made of existing reservoirs for flood control purposes.

ACKNOWLEDGEMENT

The authors would like to thank Renfrewshire Council, Perth & Kinross Council, Scottish Hydro Electric and the Environment Agency North West Region for their kind permission to publish this paper.

REFERENCES

SMITH K and BENNETT A M (1994). Recently increased river discharge in Scotland : effects on flow hydrology and some implications for water management. Applied Geography, 14, 123 - 133.

HULME, M, HOSSELL J E and PARRY M L (1993). Future climate change and land use in the United Kingdom. The Geographical Journal, 159, No. 2, 131 - 147.

SMITH K (1995) Precipitation over Scotland 1957 - 1992: some aspects of temporal variability. International Journal of Climatology, 15, No. 5.

Essential engineering criteria for the abandonment of tailings lagoons as environmental wetland features

D. R. LAMONT, Health and Safety Executive, UK
J. R. LEEMING, Health and Safety Executive, UK
M. BRUMBY, RJB Mining, UK

SYNOPSIS. The paper describes agreement between the Health and Safety Executive (HSE) and the principal operator in the now-privatised coal mining industry (RJB Mining (UK) Ltd.), over requirements for converting colliery tailings lagoons to environmentally diverse water features as an alternative to overcapping them when abandoning coal mine spoil heaps. The paper reviews engineering safety concerns and considers factors influencing the residual level of risk. Essential engineering design criteria that should be adopted during the construction of such features to ensure their long term stability are described.

INTRODUCTION

Unsurprisingly, the Civil Engineers and Legislators after Aberfan were not greatly exercised by considerations of beneficial land afteruse, of environmental enhancement and of aesthetics. However, over the past thirty years the expectations of the public, environmental organisations and Minerals Planning Officers have changed. At a number of sites, due to the closure of mines as a result of the scaling down of the mining industry, there is significant over-capacity of tipping space. At a time when good agricultural land is being put to 'set-aside,' there is less enthusiasm for restoring land to second rate agricultural use. At all but the most modern sites there is only a fraction of the soils needed for a satisfactory agricultural restoration. Conversely, there is increasing demand for sensitively designed restoration schemes, for areas of public access and amenity and for ecological diversity. (There is a national and, indeed, international shortage of wetland habitats.) Without addressing these aspirations, planning approvals will be increasingly difficult to obtain.

Coal washing and preparation for the market at large coal mines almost invariably produces a tailings fraction, suspended in water. This fraction is predominantly dirt, usually mudstones, and has a size range typically of 100% < 2mm; 99% < 1mm and 95% around 0.1 mm. Large quantities of this fraction are produced, and are disposed of by settling out of suspension in lagoons, usually constructed on top of active tips, with cleaned water being recycled.

When a lagoon has filled with tailings, standard practice has been to remove supernatant water, allow a period for the deposit to dry sufficiently, then to overcap with coarse discard material. Eventually as the tip is created, a lagoon will be progressively overtipped to finish within the body of the tip. An old tip may contain a number of old lagoons.

Overcapping is only possible if sufficient coarse material is available. When a tipping site is nearing capacity, provision is made to stock a sufficient amount of material to complete the cap on the final lagoon, and to shape the tip to an agreed profile. With the sudden or premature closing of a mine, a shortfall in the availability of coarse discard could mean the expensive importation of material to complete the cap. As a strategy to deal with this problem, proposals to leave parts of lagoons open, as water features, have in some suitable cases been put forward, with a supporting statement arguing that the resulting feature would have an important amenity value once the site has been restored and disposed of. Such features would be constructed by only partial overcapping of lagoons and by appropriate contouring of adjacent areas.

In the early 1990s, restoration proposals including wetland features on spoil heaps were made at Ledston Luck and Allerton Bywater Collieries. British Coal implemented the former and planning permission was obtained for the latter. After privatisation, RJB Mining (UK) Ltd. have submitted revised restoration proposals incorporating wetland features for several further sites including Askern, Bilsthorpe, Clipstone (Rufford), Gedling (Stoke Bardolph) and Point of Ayr. These fully designed proposals have been accepted and (together with Allerton Bywater) are being implemented.

In addition, there are numerous spoil heaps which have been abandoned by British Coal with lagoons which are partially or completely uncapped. Wetland features occur at these sites through neglect. A significant number of these have passed to RJB Mining plc as prospective opencast or tip wash sites, while many others have passed to local authorities.

HM Inspectorate of Mines on behalf of HSE has considered the proposals from the privatised mining industry over this matter and has agreed to them in principle. It was felt that guidance was required on the minimum engineering standards acceptable for safe abandonment and these were subsequently agreed between both parties.

TIP AND LAGOON CONSTRUCTION

Three principal contributory factors to the Aberfan disaster were the slope of the ground that the structure was built on, the fact that water was allowed into the tip and, crucially, that experienced and suitable qualified civil engineers had little or no input into the design and construction of the spoil heap.

Tips and lagoons at coal mines are now constructed in accordance with the NCB Technical Handbook "Spoil Heaps and Lagoons,"[1] and "Codes and Rules"[2] ; which were drawn up following lessons learned from the Aberfan disaster, and as a result of intensive technical investigations into the properties of mine waste and the behaviour of spoil heaps. Tips are now built on essentially level land, and are constructed in such a way as to enhance the strength of deposited material and to prevent the ingress of water. They are formed in discrete, compacted layers of maximum thickness 1.5m which are relatively impermeable. The structure is anisotropic and drainage paths are approximately horizontal and parallel to the layer surfaces. The external flanks are sloped to shed water in a controlled manner and to facilitate restoration.

Lagoon banks are built up from much thinner layers, 0.3m maximum, also compacted, to ensure even greater strength and impermeability. Tailings in a lagoon are deposited from varying discharge points around the lagoon and tend to form layers of different permeability. Overall however the vertical permeability of the tailings in the lagoon body is low and is significantly less than its horizontal permeability. The vertical permeability decreases with time due to consolidation of the deposits.

LEGISLATION AFFECTING TIPS

The framework

On 12 October 1966, the major tip slide at Aberfan in Glamorgan claimed 144 lives, the majority of them school children, mostly between 7 and 10 years old. The Tribunal of Enquiry held to investigate this disaster determined that to prevent a recurrence and to ensure stability, the construction and maintenance of tips needed strict regulation[3]. This resulted in the enactment of the Mines and Quarries (Tips) Act 1969[4], "the Act", and subsequently the Mines and Quarries (Tips) Regulations 1971[5], "the Regulations".

There are three classes of tip recognised, which are:

- Active tips: those in current use.
- Closed tips: associated with mines or quarries that are still being worked but the tip no longer in use.
- Disused tips: those associated with mines or quarries that are themselves abandoned.

Active and closed tips can be further categorised as classified (or unclassified) active or classified (or unclassified) closed. Classification depends primarily on the size of the tip. Classified tips cover an area greater than 10 000 square metres or are higher than 15m or are situated on land sloping at more than 1 in 12. In the case of lagoons the criteria are that any lagoon more than 4m high or having a volume more than 10 000 cubic metres is a classified tip. The requirements for unclassified tips are somewhat less exacting than those for the larger, classified tips.

The Act is split into two Parts, denoted I and II. Active and closed tips are covered by Part I,
with Regulations imposing requirements to ensure that the structures are built and managed in such a way as to maintain their stability. Commencement or cessation of tipping and any dangerous occurrences are events notifiable to HSE.

The Reservoirs Act 1975[6] Section 1(1), specifically excludes a "mine or quarry lagoon which is a tip within the meaning of the Mines and Quarries (Tips) Act 1969", from the definition of "reservoir".

The requirements of the regulations

The Regulations impose general requirements to ensure that tips are made and kept secure. In particular, operations must not cause an accumulation of water in, under or near the tip which may make the tip insecure, and the tip must be efficiently drained.

In addition, managers must appoint a competent person to supervise tipping operations at active tips, and to supervise maintenance, drainage and security at both active and closed tips. Defects must be recorded in a special book (M&Q Form 320(T)).

Before tipping operations commence, the owner must provide a geological map of the area, construct sections of the underlying strata showing significant faulting, and produce a plan of the site showing mine workings, water sources and courses and any topographical feature which may affect the security of the intended tip.

Following this, a report (Regulation 9 Report) must be made covering the design of the tip. This includes specifying the intended method of tipping and detailing all matters which may affect the security of the tip including the topography, geology, hydrology and hydrogeology. Amounts to be tipped are estimated and site preparation, drainage and fencing specified. A tip plan must be constructed showing this detail.

Tipping must be controlled by tipping rules, which specify not only the technical specification for construction but also the supervision, and nature and frequency of inspections. Regulations require inspections to be carried out weekly by a competent person appointed to carry them out. This inspection is primarily directed at the drainage of the tip, and '*such other inspections as are required* by Tipping Rules.'

Although both tipping rules and the Regulation 9 Report are primarily designed to satisfy the requirements of the Mines and Quarries (Tips) Act and Regulations, it is normal for them also to address the requirements of other legislation and regulations which may apply, including other health and safety, environmental and planning requirements.

The Regulations also require a civil engineering security report every two years, known as the 'Regulation 12 Report.' A supplementary report should be prepared as soon as practical after any "dangerous occurrence", or after a change has been made to the design, location or nature of deposits that might affect the stability of the tip. These record the works that have been carried out since the last report, address any changes in situation, specify remedial or maintenance work which must be carried out and focus on external factors, e.g. underworking, which could affect the stability of the site. The report must also include an opinion on the present and future stability of the tip. National Coal Board / British Coal Codes and Rules stipulate that both Regulation 9 and Regulation 12 Reports had to be countersigned by a chartered civil engineer.

An accurate plan of the tip, up to a date no more than 15 months past, must be kept. A record must also be kept of the amount of refuse deposited each week (M&Q Form 321(T)).

The Codes and Rules also require a full inspection at monthly intervals by the mine mechanical engineer, a full inspection at three monthly intervals by the mine manager and a six-monthly stability inspection by a competent civil engineer. The reports were recorded on special forms, denoted P224 (weekly), P225 (monthly) and P226 (six-monthly). This regime of inspections has served the industry well and operators are to be commended for continuing the system post privatisation.

Closed classified tips generally pose a lesser threat as no tipping operations are being carried out and, if properly maintained, consolidation of the material and the dissipation of elevated pore pressures will increase stability. Nevertheless their size or location demands a heightened awareness of their state and precludes complacency. The tip must continue to be subjected to inspections, now at 6-monthly intervals for liquid tips (lagoons) and 12 monthly intervals for solid tips. In addition, a civil engineering stability report (Regulation 18 Report), similar to that prepared under Regulation 12 for an active classified tip, must be made, now at intervals of 5 years for a liquid tip and 10 years for a solid tip. As part of the chartered civil engineer's Regulation 12 or Regulation 18 Report, the frequency of inspections and reports must be addressed and, in particular circumstances, a lesser period between inspections may be specified.

Disused tips pass into Part II of the Act, which gives local authorities (LAs), at county and metropolitan level, the responsibility of ensuring that the owners of any tip prevent any public danger. The LA does not become the owner of the tip but is given powers to seek information from the owners, to enter sites to inspect or carry out tests; and to require owners to carry out remedial operations. If the LA believes that any apparent instability constitutes a danger to the public, the LA can carry out the remedial operations itself and recover its expenses from the owner. Again, under the NCB, British Coal and (now) RJB, it is recognised that the most responsible way of managing disused spoil heaps is to treat them as closed tips and the inspecting and reporting regime is thus significantly extended.

The reasoning behind all these inspections was to ensure that tips were constructed and maintained in a stable condition. It is worthy of note that since the enactment of the Act and Regulations and following an intensive programme of investigation, analysis and major remedial works on tips in the early 1970s, no significant tip instability problems have been reported.

HSE can also direct that all or part of a tip can be taken out of Part I of the Act. The part must have been put some other use not consistent with the resumption of tipping. The tip would then no longer be within the mine curtilage. This action removes the need for detailed inspections and allows the land to be put to other, possibly commercial use, while the mine is still working. It is self-evident that before considering such a request, HSE would need to be convinced that the tip was stable.

TIP ABANDONMENT

With most tips, the final structure is a domed shape, and the land restored to forestry, amenity or agricultural use. Once a tip site has been vacated by the mine or quarry operator, the Local Authority has a responsibility for ensuring the long term stability of the site, and taking corrective action as required. It is essential that the mine or quarry owner leaves the site in a stable condition when the site is vacated, regardless of whether or not a water feature is planned.

Two important principles have to be satisfied. These are:

- That as the LA will have limited resources (and may not have in-house expertise) to examine and maintain the site, the water feature should require minimal maintenance and its safety should not easily be compromised by vandalism, and

- That the water feature when in both its intended state and in any condition of overflow, must not compromise the stability of the structure.

As noted above, one factor in ensuring stable tip construction is the elimination of water from the body of the structure. The intentional leaving of a body of water, perched on a tip above the natural ground level, therefore raises some questions as to long term stability which have to be addressed.

ENGINEERING FACTORS INFLUENCING STABILITY ON ABANDONMENT

Various factors which could adversely affect the stability of the tip structure must be identified and engineering measures taken to counter them. They are related to the existence on the tip of a body of water subject to changes in level due to climatic variations, and the effects that the margins of this water have by wave action and gully cutting during overflow. In countering them, robustness and minimising the need for maintenance must be considered. The obvious concerns are as follows:

Water could percolate into the tip and through the lagoon bank causing eventual failure.

It is important to be sure that during the active service of the lagoon, there had been no history of significant movement, cracking, slumping or excessive seepage. Any of these would infer poor standards of construction or some inherent weakness in the lagoon banks. This must be confirmed by review of inspection records and Reports, etc. In the absence of any recorded problem, it can be argued that the reduced loading from the water feature would not present any future problems. It must also be the case that the construction of the water feature or any regrading of the outer slopes has not reduced the width of the lagoon bank from that available during its active life. Any such reduction would invalidate the above argument and would necessitate a thorough geotechnical investigation before any proposals could be accepted. With a controlled, shallow depth of water in the lagoon after abandonment, the load on the tip through hydrostatic head can be no greater than with the lagoon active.

Not completing the cap could maintain slurry in a wet state with consequently high pore pressure.

The pore pressure will remain high for a number of years, but would have been higher had the cap been completed as a result of the dead weight of the capping material sitting on the lagoon deposit. The tailings will have been deposited in layers, evened out by the repositioning of the slurry infeed discharge point. Consolidation will occur which is likely to lead to settlement of any crust on the lagoon deposits. Recovery of tailings from old lagoons has shown that the lower layers of the deposit have consolidated considerably, even when the lagoon has remained uncapped. Consolidation coupled with low vertical permeability will minimise any tendency for the low head of water to keep the deposits liquid.

Water will migrate into the edge of the partially completed cap, liquefying it and causing the feature to grow.

This could be the case if the edge of the partial overcap coincided with the water level, and would be exacerbated by wave action. The overcap material will seasonally be at least partially saturated with water and this together with any climatic or wave effects will accelerate the natural tendency of the shale materials to weather into silty and sand size fractions. This is very far, however, from suggesting that any kind of 'liquefaction' of the cap could occur. Only excessive wave action or inadequate overflow or spillway facilities are likely to lead to an increase in the size of the feature.

There is a possibility of seismic activity or blasting causing liquefaction.
This possibility has been intensively investigated and "all investigations have indicated that the liquefaction of lagoon sediments as a consequence of overtipping operations, or ground shaking by (British) earthquakes, is extremely unlikely." [7]

Wave action will erode the banks.
This could be the case if the inner banks were steep, or if the water was of sufficient area and depth to promote waves larger than a ripple. Provided that inward banks were kept to a shallow slope, and the water depth was strictly limited, adequate control should be assured. It is expected that some beaching effect will develop, with wave action reaching a stable situation, where the farther the beach got cut back, the less the wave energy at the top. The effects could be mitigated by establishing plants at the waters edge to stabilise the margin, and/or by placing a pebble beach.

If overtopped, the flank would be washed away.
If overtopping was uncontrolled, this would certainly be the case. To maintain the water level at the planned depth, a maintenance free overflow system must be provided, possibly by the insertion of a pipe through the bank, with inflow and outflow through stone chimneys to prevent vandalism. To prevent uncontrolled events, a high level overflow must be provided. An ideal method, which would also satisfy the minimal maintenance principle, would be constructed in the form of a lowering of the bank at a selected point, with a spillway formed of sufficient width to cope with a 1 in 150 years event, while keeping water velocity through the spillway to a level low enough to prevent erosion. It would be advantageous if the spillway is lined with a geotextile membrane, and planted with suitable vegetation with a well developed root system.

Statutory requirement for drainage of tips
The practice of leaving a water feature contradicts Regulation 4(2) of the Mines and Quarries (Tips) Regulations 1971 which states that, 'Every active and closed tip shall be efficiently drained.' A dictionary definition of "drain" includes both '...carry off superfluous water ...' and '...get rid of moisture by its flowing away.' It can therefore be argued that providing water levels are controlled and there are no unplanned events, the tip could be regarded as being efficiently drained.

ESSENTIAL ENGINEERING CRITERIA

Following examination of the above concerns, HM Inspectorate of Mines on behalf of HSE and RJB Mining (UK) Ltd agreed on the establishment of a set of design criteria for this work, the adoption of which should ensure that the stability of the tip structure including water feature, is assured for the long term.

These criteria covered :-

- stability and other geotechnical engineering requirements
- water level

- scour protection
- spillway capacity
- guidance on engineering standards

The text of the agreement containing the detailed criteria is set out in Appendix A to this paper

PUBLIC SAFETY

Any wetland feature in the landscape is a potential hazard to the public. There have been accidents where trespassers have fallen into active lagoons and there is clear evidence at a number of disused spoil heaps that trespassers are regularly walking or riding motorcycles over uncapped tailings deposits, in some cases for many years. Such admittedly irresponsible behaviour is not persuasive of the notion that lagoon deposits are like quicksand and will remain in this state more or less in perpetuity.

If a lagoon is to be retained and adapted as a wetland feature with potential for public access, there are certain basic precautions which will minimise risk. Surrounding internal slopes should be reduced by the deposit of suitable material, from a typical 1 in 2 to a flatter slope, preferably 1 in 5 or less. This will facilitate any inadvertent trespasser getting out of the lagoon and will also have ecological and aesthetic benefits. Suitable aquatic plants should be established where possible around the margins to increase surface shear strengths by their root systems and to act as a demarcation and barrier to discourage access. Reeds and bulrushes are particularly suitable.

Before any consented access by the public could be contemplated, an adequate crust should have formed on the deposits. Shear strengths of the top 200mm around the perimeter should be at least $6kN/m^2$. This is around the minimum desirable strength to permit normal overcapping operations using a low ground pressure bulldozer. It is also the kind of strength which can be found between high and low tide levels in a tidal estuary and will support the weight of a pedestrian. Even where the deposits are flooded, this strength should develop in a reasonable period but this should be verified using a vane tester.

Consideration must be given to the provision of fencing, warning notices and life-saving equipment where appropriate. Normally, this would be a temporary requirement until shear strengths are adequate. Subsequently, there is no reason to suppose that a wetland feature would be any more hazardous than any river, pond, lake or seashore.

CONCLUSIONS

Following the announced intention of the mining operator to leave bodies of water in a partially uncapped state on the tops of existing tips, work was undertaken to assess the likely hazard and mechanisms of potential failure of the tip structure.

Following this work, it was concluded that providing certain basic design criteria are adhered to, the resulting feature should have no detrimental effect on the stability of the tip structure as a whole. The feature itself should be no more hazardous than any other body of open water to which the public has access. It is concluded that no additional burden will be placed on the LAs by the establishment of these features.

REFERENCES

1. NCB Technical Handbook 'Spoil Heaps and Lagoons'. (Second Draft 1970). National Coal Board.

2. NCB (Production) Codes and Rules: Tips. First Draft 1971. National Coal Board.

3. Report of the Tribunal appointed to enquire into the Disaster at Aberfan on October 21st 1966. HMSO 1967.

4. Mines and Quarries (Tips) Act, 1969. (1969 Chapter 10.)

5. Mines and Quarries (Tips) Regulations 1971. SI 1971 No. 1377.

6. Reservoirs Act 1975. (1975 Chapter 23)

7. Composition and Engineering Properties of British Colliery Discards: R.K. Taylor, N.C.B. 1984.

Appendix A   Text of Agreement

"ESSENTIAL ENGINEERING CRITERIA FOR THE ABANDONMENT OF TAILINGS LAGOONS AS ENVIRONMENTAL WETLAND FEATURES"

STABILITY AND OTHER GEOTECHNICAL ENGINEERING REQUIREMENTS

- The embankment shall have been constructed to an appropriate engineering standard e.g. in compliance with section 15.4 "Design of Lagoon Banks" in the NCB Technical Handbook "Spoil Heaps and Lagoons", except that "Design 5" shall not be used. This requirement necessitates that all spoil in the embankment has been placed and compacted in layers not exceeding 300mm depth unless a more permeable outer zone has been specified in the design.

- No reduction in embankment width or increase in height of water or lagoon deposits from operational conditions shall be permitted.

- There shall be no record of instability with the lagoon and the adjacent part of the tip indicated in the statutory inspection reports.

- There shall be no significant settlement of embankments reported in statutory inspection reports in preceding 10 years.

- There shall be no record of significant seepage potentially affecting the stability of the embankment, reported in statutory inspection reports in the preceding 5 years.

- On abandonment, the slope of the inner face of the lagoon shall be graded (by partial overcapping if necessary) to be no steeper than 1 in 5 and preferably less near water's edge. Any overcapping shall not adversely affect stability.

- Prior to abandonment, crust on lagoon deposits should ideally be 300mm thick and at least 200mm thick and of 6 kN/m2 minimum shear strength. Nothing shall be done needlessly to disturb the crust on lagoon deposits as part of forming a wetland feature.

- Drawings including cross sections through lagoon shall be prepared as part of a comprehensive Reg. 12 or Reg. 18 Report, indicating the shape of the lagoon on cessation of operational use and the proposed shape for abandonment

WATER LEVEL

- Maximum depth of water shall initially be no greater than 1 metre and consideration shall be given to the increase in depth which will result from consolidation of the deposits.
- Prior to abandonment, lagoon deposits forming the base of the wetland feature shall be essentially horizontal .
- Top water level when the main spillway is operating at design capacity, shall be not higher than top water level when lagoon was in operational service.

SCOUR PROTECTION

- Scour protection shall be provided around water's edge to protect embankment against wave action. This can be :-
  - by the use of vegetation e.g. rushes or grass.
  - by the provision of extra top soil to give more luxuriant vegetation.
  - by the use of stone protection.
  - by the use of geotextiles which should be buried 50 to 150 millimetres below the surface of the ground and protect the zone susceptible to wave action (see Guidance on engineering standards below).
- Protection against scour due to surface water run-off shall be provided if necessary.

SPILLWAY CAPACITY

a) Main spillway (overflow pipe).

- The main spillway shall be a pipe through the embankment or a channel in the embankment crest as required.

- The main spillway shall have properly constructed intake and outfall structures its ends and shall be designed to minimise the risk of interference from vandals.

- All discharges from the main spillway shall be piped or channelled to the foot of the embankment.

- The spillway shall be constructed to prevent scour of the embankment side from leakage or overtopping whilst in use and be engineered to be maintenance free. Care shall be taken in backfilling around any pipe or channel to avoid any possibility of piping failure.

b) Auxiliary spillway

- the auxiliary spillway shall be a broadcrested weir set 100 mm above top water level when the main spillway is in operation at design capacity.

- the capacity of the auxiliary spillway shall be not less than that of the main spillway and shall prevent the top water level rising to within 500 mm of the embankment crest.

- adequate scour protection shall be provided to the base and sides of the weir and be extended to the foot of the embankment in the path of the discharge.

c) Minimum dimensions

- The normal operational lagoon design freeboard of 1 metre shall be maintained for the entire embankment (other than the auxiliary spillway) at all times. A design check shall be carried out in accordance with the guidance listed below to ensure that this freeboard is adequate considering both the physical characteristics of the wetland feature, its location and the consequences of any overtopping by flood and/or wave action.

- The embankment including any landscaping fill on the inner face, shall be at least 10 metres wide at the level of the high water mark with the main spillway operating at design capacity.

GUIDANCE ON ENGINEERING STANDARDS

- Guidance on appropriate engineering standards for spillway capacity, scour protection and wave height /freeboard calculations can be found in :-

  - "Small embankment reservoirs", published as Report 161 by the Construction Industry Research and Information Association (CIRIA) , 6 Storey's Gate, London, SW1P 3AU.

  - "Design of reinforced grass waterways", published as Report 116 by the Construction Industry Research and Information Association (CIRIA) , 6 Storey's Gate, London, SW1P 3AU.

  - "Floods and reservoir safety" 3rd Ed., Inst. of Civil Engineers Gt. George St., London SW1P 3AA .

- Guidance on appropriate engineering standards for embankment construction can be found in NCB Technical Handbook: 'Spoil Heaps and Lagoons' 1970.

- Guidance on the drainage of tips can be found in NCB publication "Technical Management of Water in the Coal Mining Industry." (National Coal Board, 1982).

GENERAL REQUIREMENTS (FOR GUIDANCE PURPOSES ONLY)

- Consideration should be given to the management of authorised traffic on and the exclusion of unauthorised traffic and pedestrians from the lagoon site after abandonment.

- Consideration should be given to the siting of appropriate warning signing and the provision of lifesaving equipment adjacent to the water.

Sediment management studies of Tarbela Dam, Pakistan

L J S ATTEWILL, TAMS Consultants Ltd, UK
W R WHITE, HR Wallingford Ltd, UK
S M TARIQ, Tarbela Dam PMO, WAPDA, Pakistan
A BILGI, TAMS Consultants Ltd, UK

SYNOPSIS. Tarbela dam, a key component of the Indus basin scheme in Pakistan, was completed in 1974 for irrigation and hydropower. With an annual sediment inflow into the reservoir of some 200 million tonnes, the live storage is being rapidly depleted and unless action is taken hydropower generation will cease within a decade, with irrigation releases declining over the next 30 years. The paper presents the results of computational sediment modelling studies undertaken to simulate the sedimentation process and the effect of various mitigation options. These are described and the arguments leading to the recommended policy is discussed.

PROJECT DESCRIPTION

Tarbela dam was constructed in the 1970's to help regulate the seasonal flows of the upper Indus both for irrigation of the Indus plains downstream and for the generation of hydropower. It is still, 25 years on, the only major storage reservoir on the Indus and as such plays a key role in the provision of dry season releases of water for irrigation. Tarbela irrigation releases amount to 11 600 Mcm, or 50% of the WAPDA (Water and Power Development Authority of Pakistan) total with a corresponding agricultural revenue of Rs 2.8 billion. In addition, with an installed capacity of 3 478 MW and a firm electrical energy of 14.8 GWh/year Tarbela provides 32% of both Pakistan's total power and energy needs with a corresponding annual revenue of Rs 6 billion. It is, therefore, a strategic national resource whose continuing future efficient operation is of paramount national interest.

The Project comprises a reservoir formed by three dams, the main embankment dam with a length of 2 750 m and a height of 143 m and two auxiliary dams. The combined fill volume of the three dams is 145 Mcm. The main dam is founded on deep deposits of river alluvium, the seepage through which is controlled by an upstream clay blanket and downstream drainage wells.

Releases from the reservoir are generally made through four tunnels situated in the right abutment. Of these tunnels, Tunnels 1 and 2 serve power station no. 1 and Tunnel 3 serves power station no. 2. When irrigation demands exceed the capacity of these three tunnels releases of the balance are made through Tunnel 4. A fifth tunnel, also for irrigation releases, is located in the left abutment.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

Because the live storage volume of the reservoir, originally 11 900 Mcm is only 14.6% of the mean annual inflow, the reservoir spills every year during the months of July and August. Two gated spillways, the service spillway (7 gates) and the auxiliary spillway (9 gates), with a combined capacity of 39 900 m^3/s are both situated at the left abutment.

The project was constructed between 1968 and 1974, although construction work continued on Tunnel 5 and remedial works to Tunnels 1 and 2 and the spillway stilling basins until 1982. The total cost of the project was US$ 2.56 billion (Rs 14.58 billion), the foreign component of which was funded by the World Bank with bilateral loans from the USA, UK, France, Canada, Italy, Australia and Germany.

OUTLINE OF THE SEDIMENTATION PROBLEM

Tarbela dam impounds the waters of the Indus, which carry a heavy sediment load. This is the case particularly in the spring and summer when the melting snows cause heavy erosion of the upland catchment. Most of the sediments brought down by the Indus are trapped in Tarbela reservoir. Thus with an average annual sediment inflow into the reservoir of approximately 200 Mt per year the live and dead storages of the reservoir have diminished by 16% and 21 % to 10 000 and 1 360 Mcm respectively in 1997.

The accumulation of sediment within the reservoir causes two major problems:

- A loss of live storage which results in a gradual reduction in the regulated yield of the reservoir. This in turn results in a reduction in the water available for agriculture and a reduction in the firm energy available from the project.
- The physical effect of sediment, which includes the risk of blocking the outlets particularly in the event of an earthquake, and erosive action of sediment laden water on the dams outlet works and turbines, which will result in increasing maintenance costs to the point when the scheme will eventually become inoperative.

Unless remedial action is taken, the reservoir will be largely filled up with sediment by the year 2030, giving the project a useful life equal to that estimated at the time of the original design. However, in view of the size of the investment already made in the Tarbela project, and of its critical national importance outlined above, a policy of inactivity is unthinkable and clearly a programme of future actions have to be undertaken to maximise the economic returns from this resource.

STUDY OBJECTIVES

This Paper presents the findings of the Tarbela Dam Sediment Management Study, which was carried out by TAMS Consultants in association with HR Wallingford in a six month period from June to December 1997.

The purpose of the study was to determine a strategy for the economical preservation of the assets of the Project on a sustainable basis. To this end all possible options were considered, including measures to reduce sediment inflows, measures to increase sediment outflows, the creation of additional live storage in addition to the improved management of sediment deposition within the reservoir. The study comprised the collection and critical review of hydrological and sediment data, the construction and operation of sediment and system operation mathematical models, preliminary engineering designs and cost estimates and economic and financial analyses.

HYDROLOGY

The catchment

The source of the River Indus is situated in the Tibetan Plateau, at an elevation of 5 500 metres above sea level. From there it flows across some of the highest mountain ranges in the world before emerging onto rain-fed lower-lying country. Downstream of Tarbela, the Indus flows along a broad valley until it reaches Attock Gorge, some 51 km downstream. Here, the valley becomes a narrow gorge, which transports the flow for about 160 km to Kalabagh. The Indus then flows onwards for a further 1 600 km, being joined by various tributaries, to its mouth on the Arabian Sea.

The Indus basin upstream of Tarbela Dam, an area of 169 650 km^2, consists of two distinct hydrological regions. Over 90% of this narrow basin lies between the Karakoram and Himalayan mountain ranges; the meltwaters from the snow and ice that cover approximately one quarter of this mountainous portion of the basin contributes a major part of the annual flow reaching Tarbela. The remainder of the basin, about 10 400 km^2, lying immediately upstream of the dam, is subject to monsoon rainfall, primarily during the months of July, August and September, the runoff from which causes sharp floods of short duration which are superimposed on the slower responding snowmelt runoff.

Climate

Climate in the Tarbela basin is subtropical and semi-arid in the headwaters. It is divided to form two distinct seasons: kharif (summer) extending from April to September; and rabi (winter) covering the remaining months. Average daily temperatures at Tarbela range from 7°C in January to 41° C in June. The relative humidity is generally quite low, only exceeding 50% during the pre-monsoon season. The annual rainfall averages around 890 mm of which two third falls between June and October.

Indus river flows

The Tarbela inflow hydrograph (Fig. 1) shows the contribution to the run off made by rainfall and snowmelt. It is estimated that the monsoon (rainfall) contribution to the total runoff is approximately 10% of the whole. The mean annual inflow into the Tarbela reservoir is 81 billion cubic metres. Variability of flows from year to year is small, with a coefficient of variation of only 15%.

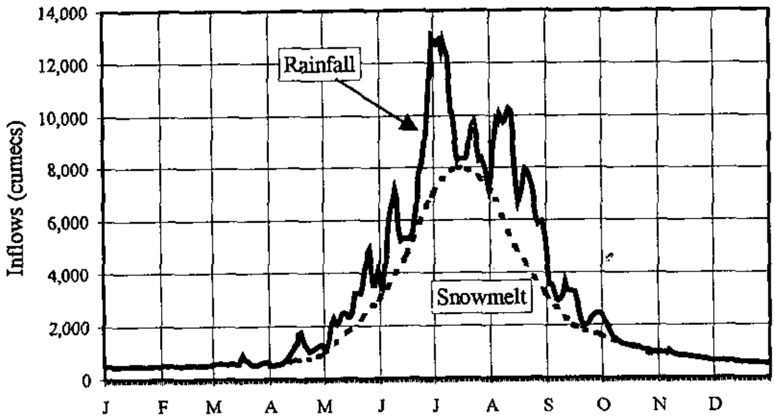


Fig. 1 Tarbela inflow hydrograph

SEDIMENT DATA

Sediment inflows

Monthly sediment inflows into the reservoir, derived from sediment sampling carried out at 10 gauging stations, is shown in Fig. 2. The mean annual sediment inflow is 200 Mt.

Sediment deposition

Since the reservoir was first impounded in 1974 the annual sediment inflows into the reservoir have been deposited to form a delta which has been advancing towards the dam. This has been monitored by surveys of the reservoir bed level which have been carried out annually since 1979 on 73 range lines covering the whole of the reservoir.

The delta profile is sensitive to the way the reservoir is operated, in particular to the minimum pool level and the length of time the minimum pool level is maintained. During the initial years of operation, until the year 1988, the reservoir was drawn down close to the minimum operating level of 1300 ftAD every year and consequently the delta advanced towards the dam. As the rate of advance of the delta downstream is related to the extent to which the reservoir is drawn down, the policy since this time has been to operate the reservoir with a higher minimum water level. This, however, has encouraged the deposition of sediment further upstream, in the middle reaches and within the live storage.

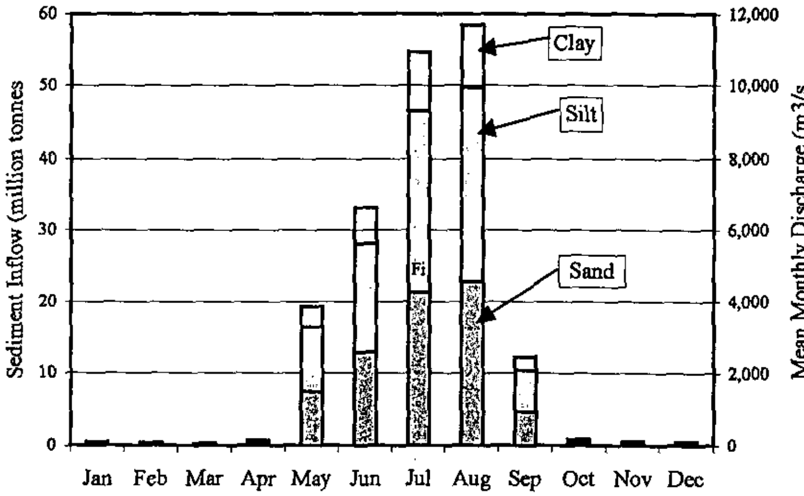


Fig. 2 Distribution of Sediment Inflows

However, in the spring of 1997 the reservoir was drawn down to a level of 1318 ftAD in order to meet the high irrigation demands that year. This lowering of the minimum reservoir level caused the delta to advance substantially, as shown in Fig. 3.

Stability of sediments

The sediments in the delta are deposited in a loose state with void ratios of over 0.8 and consequently have a high potential for liquefaction. It is estimated that any earthquake which results in a peak ground acceleration of over 0.13g (an event to be expected about once every 30 years) will be sufficient to induce liquefaction of the delta sediments that will thus result in a flow slide. In this event the flow slide will come to rest with an angle of repose of about 0.5% or 28ft/mile. If before failure the delta had crossed the limit line (Fig. 3), the resulting flow slide will inundate the tunnel intakes.

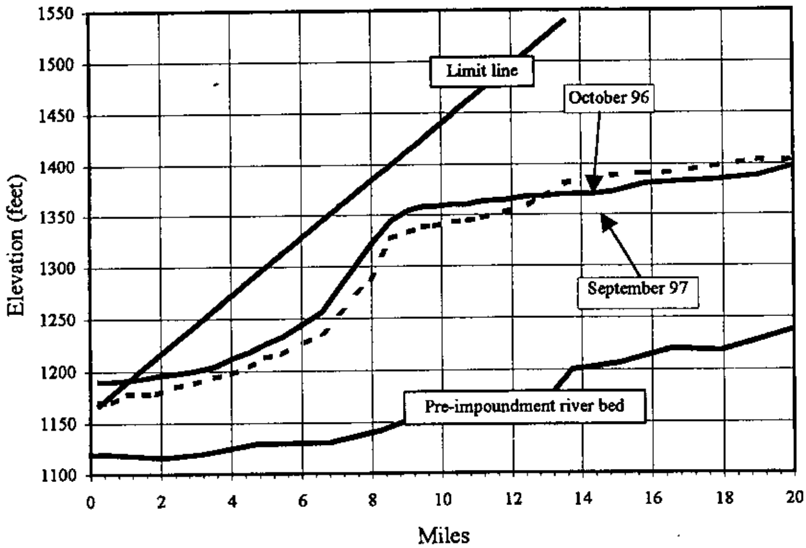


Fig. 3 Delta Profiles

SEDIMENT MODELLING

Description of the model

The numerical model RESSASS was used to simulate reservoir sedimentation in Tarbela reservoir. The model takes the original reservoir cross sections and a 60 year sequence of water and sediment inflows into the reservoir and computes the sediment profile at each cross section using equations that relate sediment movement and flow for a range of sediment sizes.

Verification

The model was verified by simulating the observed sediment deposition from the time the reservoir was impounded to 1996 and comparing the profiles predicted by the model in 1996 with those observed. The model gave excellent predictions in the 15 miles immediately upstream of the dam.

Reservoir operation policies

Eight runs of the model were carried out to explore the influence of the minimum operating level and the annual duration of the minimum operating level. The initial runs of the model confirmed that the higher the minimum pool level, the later the date when the tunnel intakes are likely to be inundated, as shown in Fig. 4.

The subsequent runs of the model indicate that for the safe short-term operation of the reservoir the minimum reservoir level should be raised by 4 ft each year and that the annual drawdown period should be limited to a maximum of 15 days.

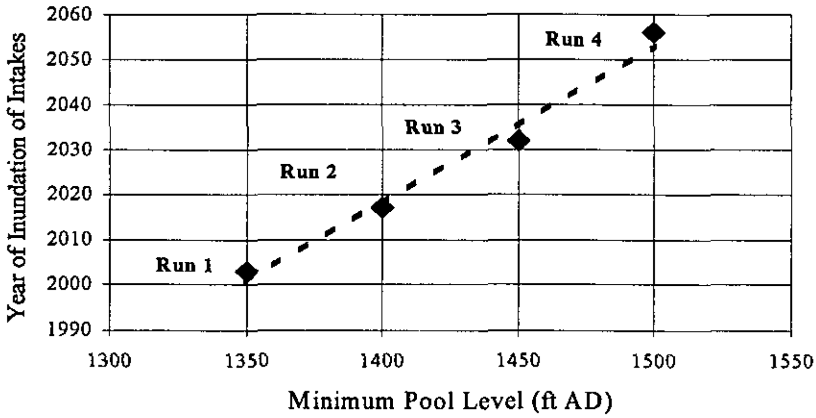


Fig. 4 Minimum pool level and year of inundation of intakes

Reservoir flushing

Five model runs were carried out to simulate reservoir flushing, in which the flushing level, the flushing period and the date flushing commences were varied. The results show that:

- flushing provides a substantial long term live storage with only a small annual reduction;
- low level flushing is more effective than high level flushing;
- flushing over a 30 day period is more effective than over a 20 day period;
- the sooner flushing commences the greater the live storage.

Fig. 5 shows the live storage volumes attainable with the chosen flushing regime compared with some of the other operational policies that are available. In the analysis 'high level' flushing refers to the reservoir being drawn down to 1450 ftAD while 'low level' refers to a reservoir level of 1350 ftAD. The reservoir normal operating level is 1550 ftAD.

Impact of Basha dam

There are plans to construct another large storage dam on the Indus upstream of Tarbela. If constructed, such a dam would act as a sediment check dam to Tarbela and as such its effects have been studied in some detail. Unfortunately, the model results show that the effect is small, partly because 2016 is too late for Basha to have a substantial positive impact, and partly because Basha itself will probably be operated in a sustainable way involving the flushing of sediments.

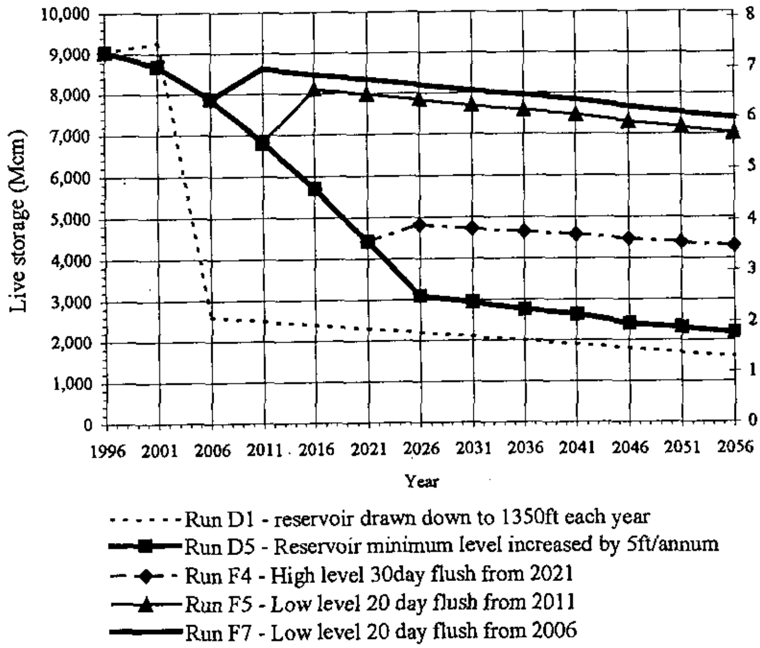


Fig. 5 Comparison of predicted live storages

THE ACTION PLAN

Objectives

Two policy objectives were set by the study:

- total security should be provided to the tunnel intakes against clogging by sediment; and
- the long term target live storage should be approximately 7 000 Mcm.

In the Action Plan selected the security of the intakes is provided in the short term by the adoption of a suitable reservoir operating policy and in the long term by the protection of an underwater dike. A sustainable live storage can only be assured by means of flushing, which thus forms phase III of the Action Plan.

Phase I - Reservoir Operation

Phase I of the Action Plan comprises a revised reservoir operating rule which will ensure the security of the tunnel intakes from blockage for at least ten years until the year 2008. This period is considered enough for the construction of the underwater rockfill dike, which comprises phase II of the Action Plan. The selection of the optimum reservoir operating rule depends on the balance of the limitation of short term losses on the one hand and longer term security on the other. An annual increase in minimum pool level of 4 ft together with a limit on the duration of the annual draw down to 15 days has been selected as the Phase I operating policy.

Phase II - Underwater dike

Phase II of the Action Plan comprises the underwater dike, which is assumed to be constructed within the ten year period of security of the intakes provided by phase I. The underwater dike will be nearly 100 m high and will be constructed of dumped rockfill. It will be located far enough upstream of the tunnel intakes so that dike construction does not interfere with power generation. The dike will be founded on rock in the area formerly occupied by the diversion channel. The dike location is shown in Fig. 6.

Phase III – Flushing

Capacity. There are several design criteria for the selection of the discharge capacity, including:

a) capacity equal to the 1 in 5 year flood peak, which will result in very large and expensive outlet structures;
b) the Chinese experience that the capacity should be equal to half the bed forming discharge, which was taken to be the 1 in 2 year flood;
c) the Russian experience that the discharge capacity should be between 2 and 4 times the MAF.

The 1 in 2 year flood at Tarbela is approximately 11 000 m^3/s so that criteria b) would give a capacity of 5 500 m^3/s. Criteria c) would give a capacity of between 5 000 and 10 000 m^3/s. The capacity selected for Tarbela is 7 500 m^3/s.

Availability of water In an average year it is estimated that 18 000 Mcm of water will be discharged through the bypass to flush the reservoir, generally in June. The distribution of water releases from the reservoir during a typical year is shown in Fig. 7. By holding the reservoir level down to a low level, the release of this water has a considerable impact on energy generation. However, the security of the irrigation releases is not jeopardised as there is sufficient water to facilitate flushing and subsequently fill the reservoir each year.

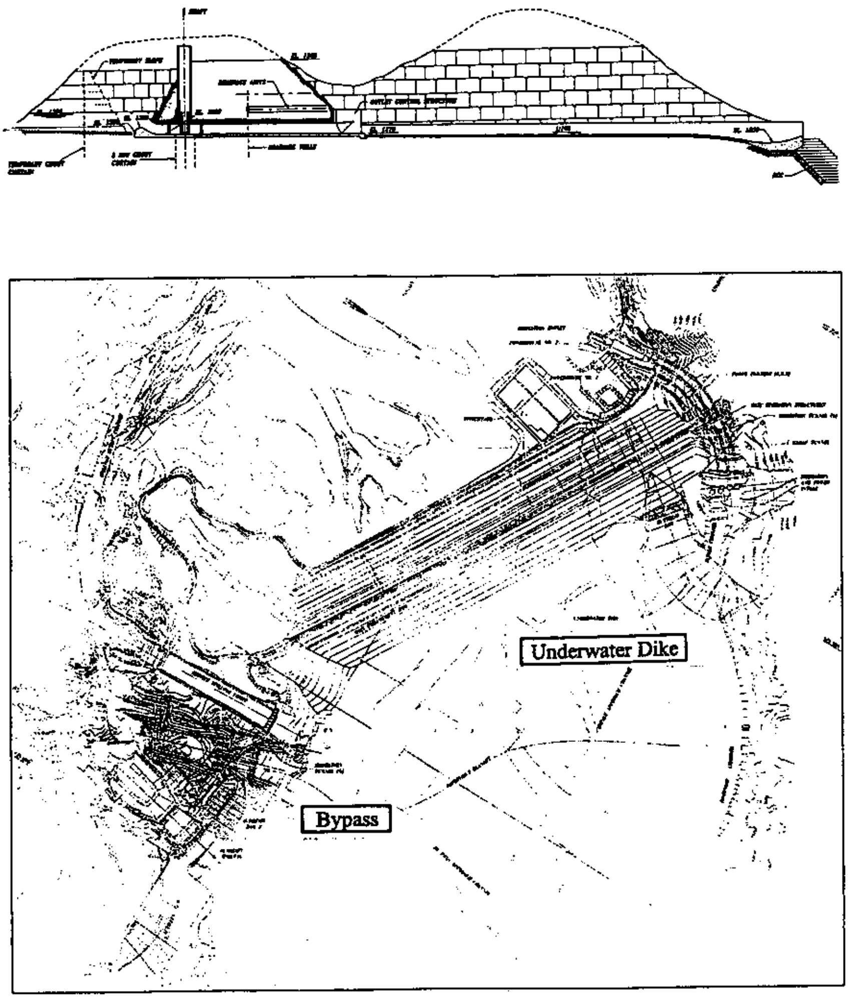


Fig. 6 Site plan and bypass section

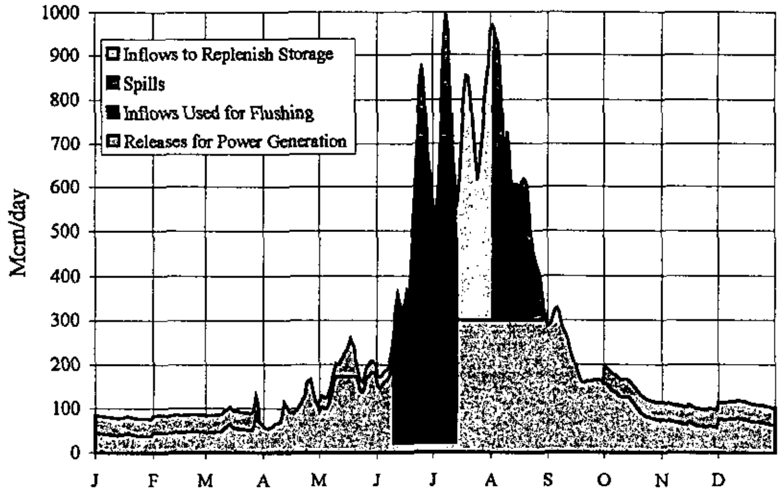


Fig. 7 Availability of water

Description It is proposed that the reservoir will be flushed through a low level bypass to be situated on the left abutment, between the service spillway and the auxiliary spillway, as shown in Fig. 6. The bypass will comprise four new 11m diameter tunnels which, together with tunnel no. 5, will have a capacity of 7 500 m³/s at a reservoir level of 1350 ftAD. Typically, during flushing 18 000 Mcm of water will be discharged through the bypass over a 30 day period in June of each year. During this period the reservoir will be flushed for 16 hours a day, to allow 4 hours of power generation during the evening peak. The bypass will discharge sediments into the Dal Darra channel and thence into the Ghazi pond. Preliminary model tests indicate that these sediments will be flushed through the pond with little deposition.

Additional tests are required to confirm this. A provision has been made in the economic estimates for the need for training bunds, dredging and a small increase in the Tarbela tailwater level.

Alternatives

There are two alternatives to the proposed Action Plan. These are:

1. modified reservoir operations alone with no capital works; and
2. phased dike construction.

The first of these alternatives will result in a gradual decline in live storage ending with the cessation of power production in about 2025. The second, requiring a second phase heightening of the dike of 30m would guarantee run of river power production but the live storage would be reduced to its residual value of approximately 2 200 Mcm in the year 2020. Fig. 8 illustrates the effect of the Action Plan and these alternatives on live storage.

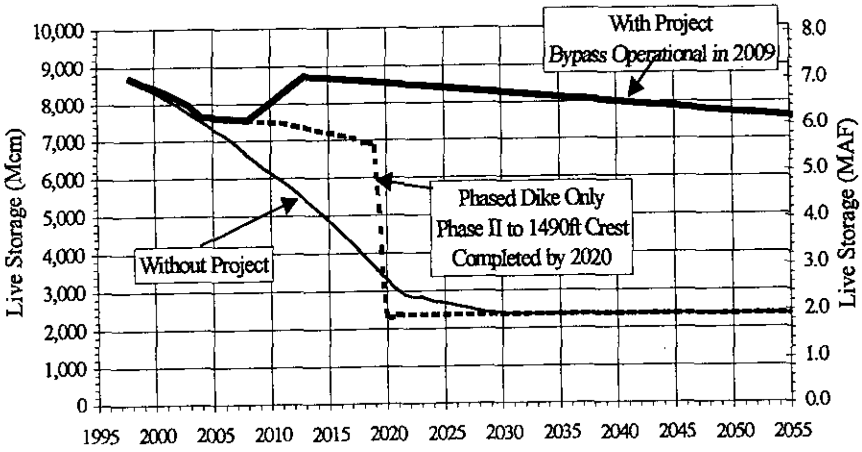


Fig. 8 Comparison of live storage

ECONOMIC ANALYSIS

Cost estimates

The capital cost of the underwater dike and the low level bypass are estimated a US$ 170 and US$ 500 million respectively:

Recurrent costs are estimated as follows:

routine operational and maintenance costs:	1% of the capital costs p.a.
dredging costs: (phase III only):	US$ 900 000 p.a.
bypass chute concrete replacement:	US$ 3.6 m every 5 years

Benefits

The main benefits attributable to the Action Plan are agricultural and energy.

Concerning irrigated agriculture, Tarbela fulfils two main functions:

1. provision of irrigation water during the dry season (rabi) for the wheat crop: it is estimated that if the Action Plan is not implemented wheat production will decrease by 500 000 tonnes/year; and
2. provision of irrigation water during the spring (pre-kharif) to foster sugar cane and cotton: it is estimated that if the Action Plan is not implemented cotton and sugar cane production would decrease by 70 000 tonnes/year and 7 million tonnes per year, respectively.

The total annual incremental agricultural benefit of the Action Plan will increase steadily to an estimated US$ 210 million by the year 2030. Thereafter the benefits will gradually reduce to US$ 180 million by the year 2055.

The power benefit comprises two components, capacity benefit and energy benefit. Capacity benefit is the cost of replacing Tarbela power stations nos. 1 and 2 that would be necessary without the project. These benefits are estimated at replacement costs of US$ 1,740m and US$ 1,720m for power stations 1 and 2 respectively.

The energy benefit is estimated as the incremental cost of the thermal generation that would be required without the project, which mostly comprise the cost of fuel, and is estimated to reach an ultimate value of US$ 595 million per annum by the year 2040.

Economic Internal Rate of Return

The Economic Internal Rate of Return (EIRR) for the base case is calculated to be 21.6%. This analysis has been subjected to a range of sensitivity tests to assess the robustness of the base case to variations in project parameters, as follows;

Sensitivity Tests	**EIRR (%)**
Base case (bypass by 2009 & power capacity replaced by 2020)	21.6
Power capacity replacement programme brought forward by 5 years	27.4
Power capacity replacement programme delayed by 5 years	18.7
Bypass completion delayed to 2012	25.8
Irrigation benefits reduced by 25%	19.7
Construction costs increased by 25%	18.7
Construction costs increased by 25% & all benefits reduced by 25%	15.7

OTHER PROJECT IMPACTS

Environmental

The environmental impacts of the project, which will prolong the economic life of an existing asset, are generally positive. In particular the project will save over 50 000 tonnes of fossil fuel that will otherwise be used each year in thermal power stations. The project will also help restore the sedimentation regime of the Indus river to approximately its original state.

Flooding

By preserving significant live storage in Tarbela reservoir, the project will help preserve the limited but important contribution that Tarbela makes to the control of floods downstream.

<u>Employment</u>

The project will preserve many thousands of jobs in the agricultural sector and will make a valuable contribution to the rural economy.

CONCLUSIONS

The conclusions of the study are:-

1. Flushing at the correct elevation, discharge and for the correct duration will ensure that a substantial live volume will be sustained at the site well beyond the original economic life of the project.
2. Although there are substantial engineering difficulties that need to be surmounted to construct the dike and the bypass, all of the proposed works are well within current engineering experience.
3. Although the cost of implementing the project is approximately US$ 670 million, the sustainable irrigation, energy and power benefits ensure that the project has a good economic internal rate of return.

On a more general level, it is concluded that the preservation of live storage by flushing through high-capacity, low-level outlets is likely to be effective in all reservoirs which have a suitable shape and where there is adequate and predictable river flows.

ACKNOWLEDGEMENTS

The Authors wish to thank Mr. Mottadullah, Member (Water) of the WAPDA Board and Mr. Gohar Chief Engineer (O&M) Tarbela Dam Project and all the staff of Tarbela Dam Project Monitoring Organisation for their continued support and encouragement.

Reservoir operation to control sedimentation: techniques for assessment

E ATKINSON, HR Wallingford, UK

SYNOPSIS. There are few cost effective techniques available to engineers which can prevent loss of storage or recover storage volume in a reservoir. One such technique is flushing and another is the lowering of water levels in a reservoir to enable large sediment quantities to be passed through during floods. Changes in reservoir operation to control sedimentation are often not feasible for technical reasons. The paper presents methods, ranging from simple assessment criteria to 3D numerical models, for the assessment of changes in reservoir operation. Use of the techniques to assess options for controlling sedimentation at the Tarbela Dam, Pakistan, is presented.

INTRODUCTION

It has been estimated that approximately 1% of the total storage capacity in the world's reservoirs is lost each year due to sedimentation. Sediments can also block intakes in reservoirs and damage tunnels or turbines. There are few cost effective techniques available to engineers which can prevent loss of storage or recover storage volume in a reservoir. One such technique is flushing, whereby water levels are lowered sufficiently to re-erode deposits and flush them through the dam. Flushing has proved to be highly effective at some reservoirs. For example, at the Mangahao reservoir in New Zealand 59% of the original operating storage had been lost by 1958, 34 years after the reservoir was first impounded. The reservoir was flushed in 1969 and 75% of the accumulated sediment was removed in a month (Jowett, 1984). Atkinson (1996) lists eight other reservoirs where flushing has been successful.

Flushing is the most extreme change in reservoir operation. It generally involves the greatest disruption to the benefits of the reservoir and uses most water. Another technique to prevent storage loss is the lowering of water levels in a reservoir to enable large sediment quantities to be passed through during floods. This practice is termed 'sluicing' in the literature. It has been successfully applied at the Roseires Reservoir on the Blue Nile in Sudan (Ahmed & Hamad, 1986).

Measures such as these can be expensive, for example the cost of addition low level outlets at the dam or loss of benefits due to reduced reservoir levels. Alternative measures can be even more expensive, however.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

Removal of accumulated sediments by dredging or excavation, and the subsequent disposal, is only rarely economically viable. Replacement of storage by constructing a new dam involves high expenditure, is environmentally and socially damaging and is often politically unacceptable.

Changes in reservoir operation to control sedimentation are often not feasible for technical reasons. An incised channel scoured into sediment deposits may be narrow relative to the reservoir, or river discharges may be insufficient to transport large sediment loads through the reservoir.

The purpose of this paper is to present a range of techniques that can be used to assess changes in reservoir operation. Techniques include simple assessment criteria and numerical models. Use of the techniques to assess options for controlling sedimentation at the Tarbela Dam, Pakistan, is also presented.

SIMPLE ASSESSMENT CRITERIA

Techniques are required which can be used to provide an initial assessment of whether temporal lowering of water levels will prove successful at a reservoir. The assessment techniques should require relatively little data and should give a reliable first estimate of the feasibility of the change in operation. Once the engineering feasibility of the change is established then economic and environmental assessments can be undertaken.

Empirical criteria for successful flushing, such as a maximum value for the ratio of capacity to annual runoff, have been suggested but they have not proved reliable when compared against observed results of flushing (Atkinson, 1996).

Rational criteria for assessing flushing or sluicing (the term 'flushing' is largely used to describe both processes henceforth) can be derived from two basic requirements:

(i) sediment quantities passing through low level outlets in the dam during flushing should equal or exceed sediment quantities depositing between flushing operations ('sediment balance'), and

(ii) the storage volume in the reservoir that can be created or maintained by flushing flows should be a relatively large proportion of the original reservoir volume before sedimentation ('sustainable reservoir capacity').

Sediment Balance

When flushing is attempted without drawing down water levels, the high flow velocities at the outlets are very localised and the impact is

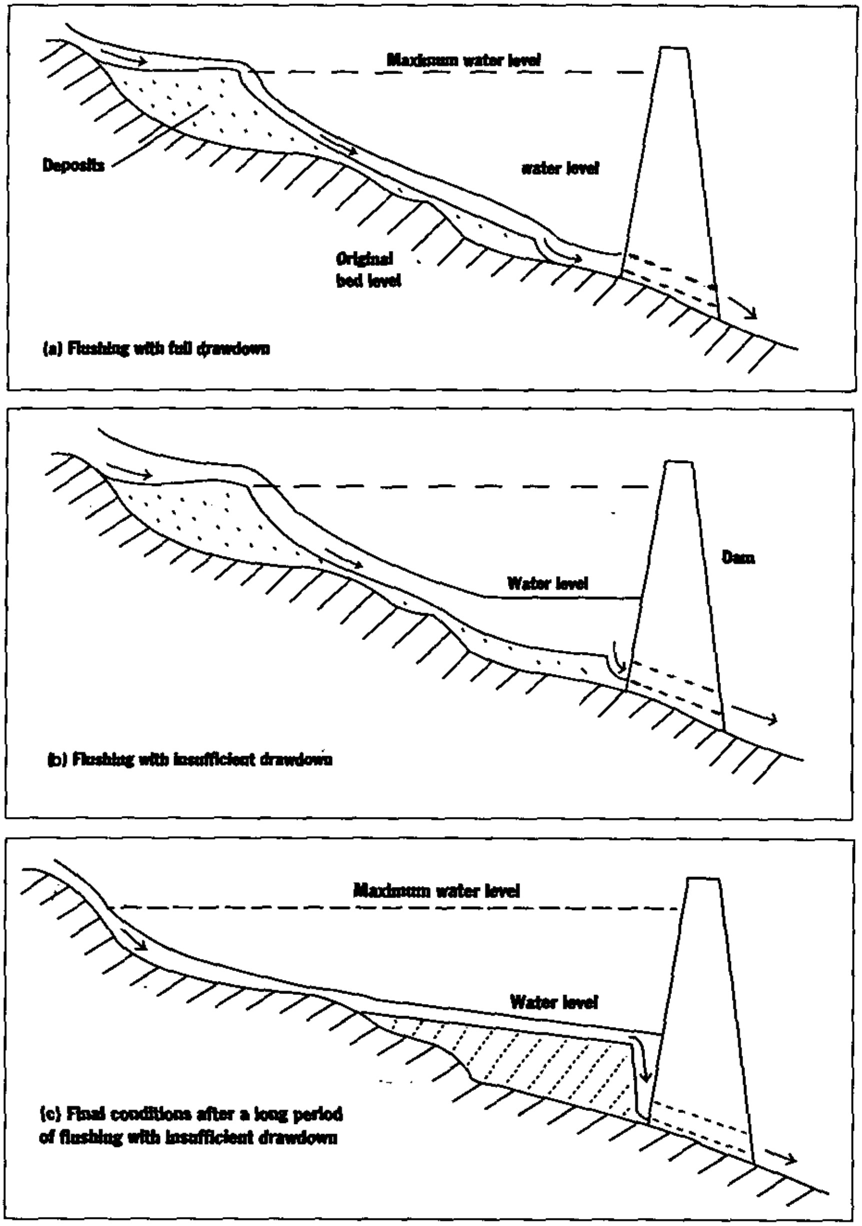


Fig 1 Longitudinal profiles during flushing

insignificant. The water level in a reservoir must be drawn down to close to the bed elevation at the dam before flushing can be effective (Fig la). Many authors have confirmed this with observation, theory or modelling (for example, White & Bettess, 1984 and Atkinson, 1996). However, moderate lowering of water levels during flushing will still significantly increase flow velocities at the upstream end of the reservoir, where bed levels will be above the water level at the dam (Fig lb). Large sediment volumes will be scoured from these upstream reaches and will redeposit nearer the dam. Eventually, bed levels upstream from the dam will rise to the water level during flushing and then significant sediment quantities will be transported through the low level outlets (Fig 1 c).

If flushing water levels are close to bed elevations at the dam (either as in Fig la or as in Fig 1 c) then the sediment mass flushed will, in the long term, balance the sediment mass depositing between flushing operations. This balance can be expressed:

$$Q_s \, n \, T_f = N \, M_{in} \, TE \qquad (1)$$

where Q_s is the sediment transporting capacity (t/s) of the flow in the channel upstream from the dam, n is 86400 (seconds per day), T_f is the duration of flushing (days), N is the interval between flushing operations (years), M_{in} is the sediment inflow rate (t/year) and TE is the trapping efficiency of the reservoir (TE can be taken as 1.0 if the bulk of the sediment inflow will occur during the flushing period). A sediment balance ratio SBR, the ratio of sediment flushed to sediment depositing, can be expressed: $SBR = Q_s \, n \, T_f / N \, M_{in} \, TE$

The transporting capacity, Q_s, will be a function of discharge, channel roughness, width and slope, and the properties of the deposited material. The only method for predicting Qs during reservoir flushing, which has been widely tested, is an empirical equation derived by Tsinghua University and reported in IRTCES (1985):

$$Q_s = \Psi \frac{Q_f^{1.6} \, S^{1.2}}{W^{0.6}} \qquad (2)$$

where Q_f is flushing discharge (m /s), S is longitudinal bed slope, W is channel width (m) and Ψ is a constant related to the sediment type:

1600 for loess sediments
650 for other sediments with median size finer than 0.1 mm
300 for sediments with median size larger than 0.1 mm, and
180 for conditions of flushing with a low discharge.

Discrepancies between predictions of sediment load derived from Eq.2 and observations were relatively small for the flushing data from China, on which the method is based. Discrepancies were within a range of half to twice in 87% of cases, which is very good for sediment transport predictions. Atkinson (1996) compared the method with observations from reservoirs in India, the United States and the former USSR. Eq.2 was found to overestimate sediment loads by a factor of three and even more where flushing was not performed annually (as is common practice in China) but less frequently or where sediments were much greater than the 0.1mm threshold. The threefold correction suggested by Atkinson is recommended when conditions differ from those typical in China.

Eq.2 requires the width, W, to be input. Channels formed by flushing in reservoir sediment deposits correlate well with flushing discharge. Fig 2 shows the relationship and the data from which it was derived. The fitted line is described by the equation (in SI units):

$$W = 12.8\, Q_f^{0.5} \qquad (3)$$

In some cases, channel widths may be constrained by the reservoir width.

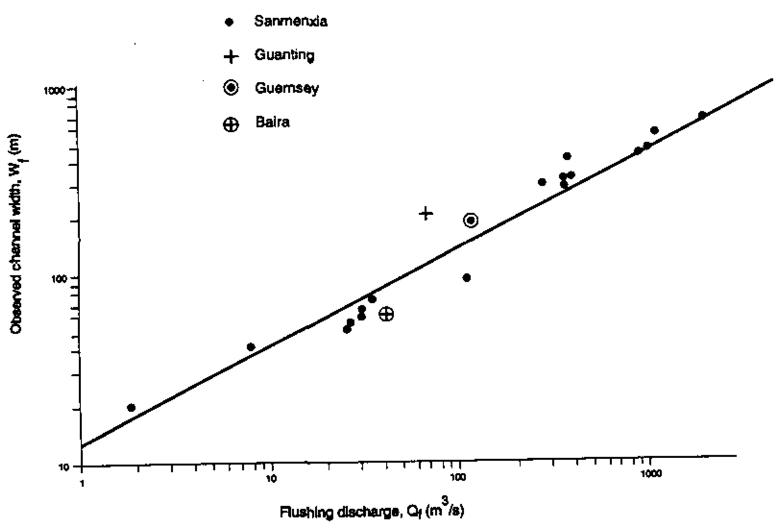


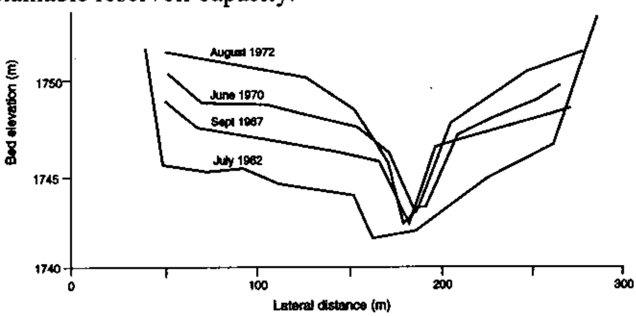
Fig 2 Channel widths formed in reservoir deposits by flushing flows

Eqs.2 & 3, together with an estimate of trapping efficiency taken from Brune's (1953) curves, can be used to derive the sediment balance ratio, SBR. Values of SBR comfortably in excess of unity are required due to uncertainty in the prediction methods and input parameters. Discharge during flushing and the duration of flushing will depend on the reservoir operation chosen, and slope can initially be set at the river slope before impoundment. Predicted values for SBR can be used to guide the choice of the inputs to repeat predictions. For example a low SBR may imply flushing should be performed at a time of higher discharge and a high SBR may imply that flatter slopes can be expected upstream from the dam (as shown on Fig 1c). If SBR comfortably greater than 1 cannot be achieved, then flushing is not feasible.

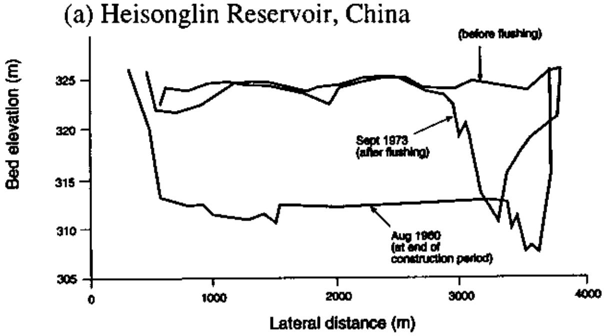
In the comparison with data presented in Table 1 below, SBR was computed for a slope defined as the drawdown depth below maximum elevation divided by reservoir length.

Sustainable Reservoir Capacity

Flushing will cause a channel to be scoured down into the reservoir deposits. In most cases this channel will be narrower than the reservoir and so substantial deposits will remain in the reservoir. In the long term these deposits will rise to an elevation close to the maximum water level, leaving the volume created by the incised flushing channel as the only storage volume remaining in the reservoir. This storage volume is defined as the sustainable reservoir capacity. Fig 3 illustrates the process, it shows cross sections at two reservoirs where flushing has maintained a relatively small sustainable reservoir capacity.



(a) Heisonglin Reservoir, China



(b) Sanmenxia Reservoir, China

Fig 3 Cross sections in flushed reservoirs

A trapezoidal cross sectional shape is assumed for the incised channel. The sustainable reservoir capacity volume is then determined from:

minimum bed elevations at each point in the longitudinal profile, which are shown in Fig 1 and can be determined from the water level during flushing and the sediment balance calculations described above,

the maximum water level,

the bed width of the incised channel, which can be calculated using Eq. 3 and a dominant flushing discharge, (bed width may be constrained by reservoir width) and

the side slope steepness of the incised channel (if side slope is shallower than the reservoir side slope, then the reservoir widths may constrain the width of the sustainable section).

In the list above only the side slope steepness is not known. In well-consolidated sediments near vertical channel sides can occur, while slopes as low as 2.5% have been observed for poorly consolidated material. Therefore a technique to predict this slope is vital to a reliable prediction of sustainable capacity. Atkinson (1996) developed two theoretical methods for predicting side slope and compared them with ten sets of observations from flushed reservoirs. The comparisons were disappointing: uncertainties in both the theoretical method and in the data sets used for comparison caused wide discrepancies between observation and predictions. The simpler method was recommended, as described by:

$$\tan \alpha = 0.63\ \rho^{4.7} \qquad (4)$$

where α is the angle of the side slope (zero is horizontal) and ρ is the density of the deposits expressed as weight of dry material per unit volume (t/m^3). ρ can be predicted from deposit composition and age of the deposits using Lane & Koelzer's (1953) method. Errors as large as a factor of ten may arise from the application of Eq.4. This is clearly an area for further study.

A simple criterion for assessing sustainable reservoir capacity can be developed by fitting a simplified reservoir shape as shown in Fig 4. A cross section just upstream from the dam can be taken as representative of the entire reservoir, and then the area of the trapezoidal flushed section compared to the original cross section area. The ratio of these areas then gives a long term capacity ratio (LTCR), which is an estimate of the reservoir capacity that can be sustained in the long term by flushing.

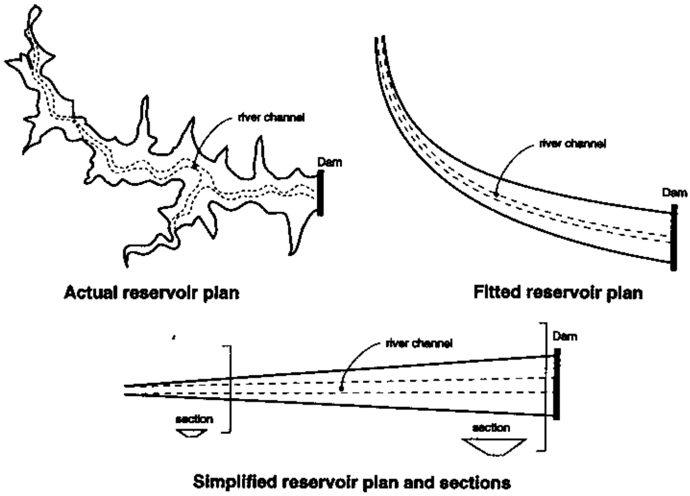


Fig 4 Simplified reservoir geometry for application of the capacity criterion

Usually LTCR>0.5 indicates that flushing is feasible. Atkinson (1996) gives a step-by-step procedure for estimating LTCR.

Evaluation of Flushing Criteria

A review of the literature on reservoir flushing produced information from fourteen reservoirs where flushing had been attempted and sufficient data were available to test the criteria (Atkinson, 1996). The fourteen reservoirs can be divided into two categories: six where observations indicated that flushing would sustain a long term capacity in excess of half the original capacity, and eight where it would fail to do so. Table 1 presents the results of application of the two assessment criteria. The criteria performed very well in distinguishing between the six reservoirs where flushing was successful and the eight where it was not. The predicted LTCR also proved to be a good indicator of the long term capacities that were estimated from the observations. The sediment balance ratio, SBR, was in most cases not a constraint to successful flushing, it was calculated assuming a

Assessing Constraints to Successful Flushing

It is useful to determine the factors that constrain successful flushing at a reservoir, and so assess whether they can be overcome, for example by enlarging outlets in the dam. The four engineering constraints to successful flushing are presented below.

(i) Incomplete drawdown Taking water height as elevation above the base of the dam, a drawdown ratio can be expressed:

Table 1 Application of sediment balance and capacity ratios

Reservoir	Country	Initial capacity Mm^3	Estimated long term capacity (observed % of original)	SBR value	LTCR value	Assessment of Constraints DDR value	SBR_d value	FWR value	TWR value
Reservoirs flushed successfully									
Baira	India	9.6	about 85%	7	0.85	0.68	24	3.4	1.6
Gebidem	Switzerland	9	near 100%	7	0.99	0.93	20	6.7	1.5
Gmund	Austria	0.93	about 86%	21	0.98	0.89	58	5.2	1.3
Hengshan	China	13.3	about 75%	about 3	0.77	0.77	about 4	0.10	7.1
Palagnedra	Switzerland	5.5	100%	33	1.0	1.00	33	1.4	1.0
Santo Domingo	Venezuela	3	97%	11	1.0	1.00	11	1.4	1.8
Reservoirs flushed unsuccessfully									
Guanting	China	2270	low	0.2	0.20	0.81	0.3	0.04	0.5
Guernsey	USA	91	low	1.0	0.26	0.44	3.2	1.4	0.26
Heisonglin	China	8.6	23-35%	about 0.7	0.30	0.77	about 1	0.06	0.8
Ichari	India	11.6	about 35%	7	0.36	0.31	33	9.9	1.4
Ouchi-Kurgan	Former USSR	56	low	7	about 0.1	0.14	110	about 2	about 0.3
Sanmenxia	China	9640	about 31%	3.4	0.39	0.75	4.8	0.26	0.9
Sefid-Rud	Iran	1760	less than 26%	4	0.13	0.96	4.3	0.3	0.1
Shuicaozi	China	9.6	low	4.6	0.39	0.37	15	1.0	2.1

Note that for the Palagnedra and Santo Domingo reservoirs there was some sediment clearance by bulldozer during the flushing operations

$$DDR = 1 - H_{flush}/H_{max} \tag{5}$$

where H_{flush} is water height during flushing and H_{max} is maximum height. DDR less than about 0.7 indicates some degree of constraint due to insufficient drawdown.

(ii) Insufficient flushing flows for a sediment balance A definition of SBR which is not dependent on the extent of drawdown is required to assess flushing flows for a sediment balance. The ratio, SBR_d, is SBR calculated using the original river slope, that is for conditions of full drawdown. $SBR_d<1.0$ indicates constraint.

(iii) Insufficient width of channel formed by flushing A flushing width ratio can be defined:

$$FWR = W / W_{bot} \tag{6}$$

where W is computed from Eq. 3 and W_{bot} is a representative bottom width in the reservoir. FWR>1.0 is required unless the side slopes are shallow (see below – TWR greater than about 2 would indicate that FWR is not a constraint).

(iv) Side slopes too steep If the top width of the section scoured by the flushing channel is not restricted by the reservoir sides, then the side slope steepness is a constraint. A top width ratio is defined:

$$TWR = (W_{bot}' + 2 * H_{max} * \tan \alpha) / W_{top} \tag{7}$$

where W_{bot}' is lesser of W & W_{bot} defined in (iii) and W_{top} is a representative top width in the reservoir. TWR>1.0 is required.

Application of these criteria to the fourteen reservoirs presented above is included in Table 1. Again the criteria have been able to distinguish well between the reservoirs where flushing was successful and those where it was not: all the criteria were met for the six successfully flushed reservoirs and at least one criterion was not met for each of the eight other reservoirs. However, at the Baira reservoir the drawdown criterion was only just not met as DDR=0.68 narrowly fails to meet the DDR>0.7 limit suggested.

The results in the lower half of Table 1 indicate that at two reservoirs, Ichari and Shuicaozi, changes to the outlet structures at the dam could potentially remove all constraints to successful flushing, while at the other reservoirs site conditions constrain the success of flushing. It is noteworthy, but perhaps not surprising, that the six reservoir sites where flushing is successful, and the two further sites where it is feasible, are all relatively small: capacities are all below $14Mm^3$. At these smaller reservoirs, resources may not be available to undertake extensive data collection and modelling programmes. So the criteria developed here may prove especially worthwhile.

EXTENDING THE USEFUL LIFE OF A RESERVOIR

A number of factors enhance the effect of flushing on extending the useful life of a reservoir. Firstly, a large proportion of the annual sediment load may enter the reservoir during the flushing period and so not deposit in the reservoir at all, or expressed another way: 'sluicing' best describes reservoir operation. Secondly, before and after flushing further sediment loads may enter the reservoir while water levels are relatively low, and so will deposit within the incised valley produced by flushing. In these circumstances, all the deposited material will be removed by subsequent flushing. Thirdly, coarser sediments will deposit near the head of the reservoir where the reservoir is relatively narrow and so they will probably be entirely re-eroded by flushing. Finally, if density currents occur in the reservoir they will tend to transport the sediments into the incised valley, which will ensure that they are subsequently flushed. So, even when the sustainable reservoir capacity is relatively low, flushing can greatly extend the useful life of a reservoir. Indeed, this will often be the predominant effect of flushing within the relatively short time horizons that are considered within an economic appraisal. Experience suggests that if LTCR>0.3 then reservoir life can be significantly extended and that further studies should be made.

Most of the factors listed above can be assessed by desk studies. Historical discharge records and an observed or estimated sediment rating curve can be used to derive the proportion of the annual sediment load, P1, that is expected to pass through the reservoir during the proposed period of low water levels. An assumption can be made that all material coarser than 63 microns (usually classified as sands, gravels and coarser) will deposit near the head of the reservoir and so will be entirely re-eroded by flushing. The proportion, P2, comprising those sizes within the material entering the reservoir can be taken. If sediment grading curves for the deposited material are available, but not for the material entering the reservoir, then P2 can be estimated from an assumption that all material passing through the reservoir is finer than sand:

$$P2 = TE\ P_{dsand} \qquad (8)$$

where P_{dsand} is the proportion coarser than 63 microns within the deposited material and trapping efficiency is defined above.

A first estimate for the proportion of the finer sediments that deposit within the flushing channel, P3, is the proportion of the reservoir width lying in the incised channel: P3=FWR, where FWR is defined above (if FWR>1, P3=1).

The proportion of the incoming sediment load that deposits and are not subsequently flushed from the incised channel, TE_{new}, will then be no greater than for the case where no silts or clay pass the dam during the period of higher water levels:

$$TE_{new} = (1 - P1)(1 - P2)(1 - P3) \qquad (9)$$

TE_{new} can be used as an estimate for the reduced trapping efficiency after reservoir operation is changed. The factor TE_{old}/TE_{new} will then be the estimated increase in reservoir life.

NUMERICAL MODELLING

The calculation given in the section above relied on several assumptions and can only provide an approximate estimate for the increase in reservoir life due to changes in reservoir operation. If sufficient input data are available, a more accurate method is numerical modelling.

A one dimensional model is suited to this application. More complex 2D or 3D models will in general require too much computational time because simulations are usually required to cover periods of 50 to 100 years into the future and have a time step of a day or an even shorter period.

For each time step in a one dimensional model, the water levels and flow conditions are predicted from discharges and/or changes in storage, and hence sediment concentrations within each of typically 10 size fractions are routed through the reservoir. Bed level changes are determined, using the concept of sediment continuity, from the changes in concentrations through the reservoir. These changes in bed level are used to update the bed elevations stored in the model. Usually bed elevations are stored as full cross sections rather than single values, so additional rules are required to determine how deposition or erosion is distributed across the sections.

White & Bettess (1984), Basson & Olesen (1997) and other authors present 1D numerical model applications. The White & Bettess (1984) model has now been combined with reservoir survey analysis software, which uses the accurate Stage-Width Modification Method (SWIMM), to form the PC software 'RESSASS' (REServoir Survey Analysis and Sedimentation Simulation).

Two or three dimensional models can be used to assess the localized impact of flushing near low level outlets. Atkinson (1996) briefly presents 3D modelling in an idealised reservoir to investigate the extent of influence achieved by flushing. Such modelling would usually provide little useful information on the feasibility of flushing, but may prove invaluable as a component of the design process.

APPLICATION AT TARBELA RESERVOIR

Another paper at this conference, Attewill et al (1998), provides more comprehensive information on the sediment management studies at Tarbela. An assessment of flushing was a component of these studies.

Tarbela reservoir, situated in Northern Pakistan, is the only large impoundment on the Indus. It was constructed in the early 1970's and had an original total storage volume of 13,600Mm^3. Reservoir life was anticipated to be in the region of 50 years at construction, and surveys have shown this to be reasonable as far as storage is concerned. By 1996 total storage had reduced to about 11,000Mm^3. Of more immediate concern was the delta formed by the deposits, which had reached about 14km from the dam and was at an elevation about 60m above the power and irrigation outlets in the dam. An earlier study had derived locations of the delta for which, in the event of an earthquake, there was a significant danger that the delta would liquefy and slump onto the dam, thus blocking the intakes. In 1996 the delta was close to the region of danger.

A 1D numerical model of the reservoir was established and calibrated using the RESSASS software. Fig 5 shows the results of calibration. Application of the model indicated that the minimum impoundment elevation would need to be progressively raised each year to prevent deposits from threatening the intakes. Thus the live storage available in the reservoir would reduce from 9,000Mm^3 in 1996 to 3,000Mm^3 after 30 years. Economic analysis confirmed that structures to enable the intakes to withdraw flow at higher elevations, and so remove the danger of blockage after an earthquake, would have high net benefits.

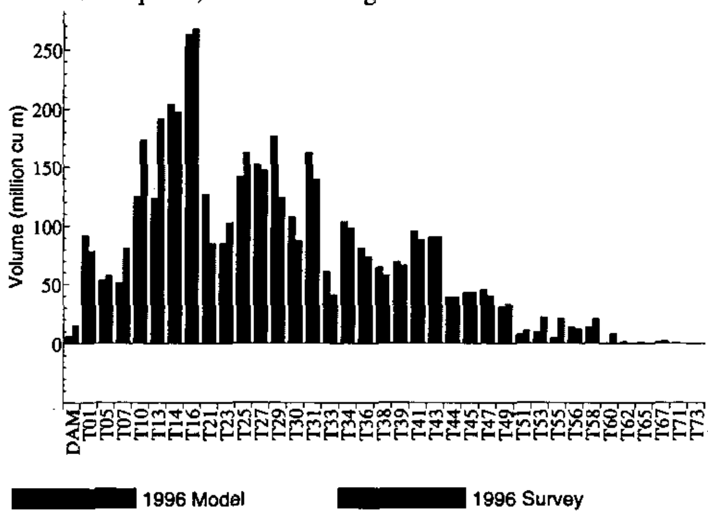
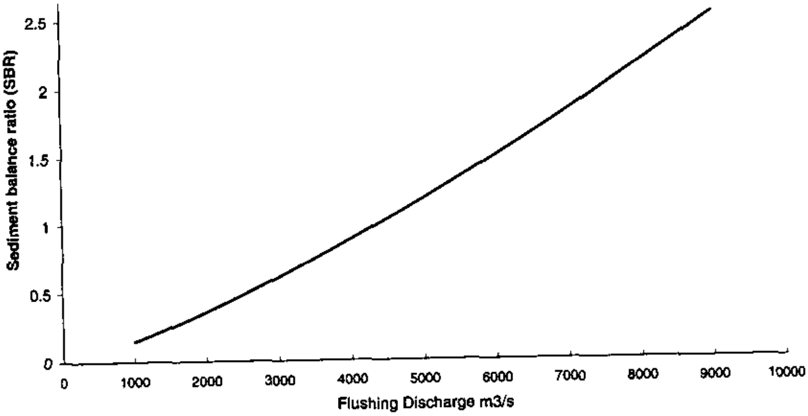
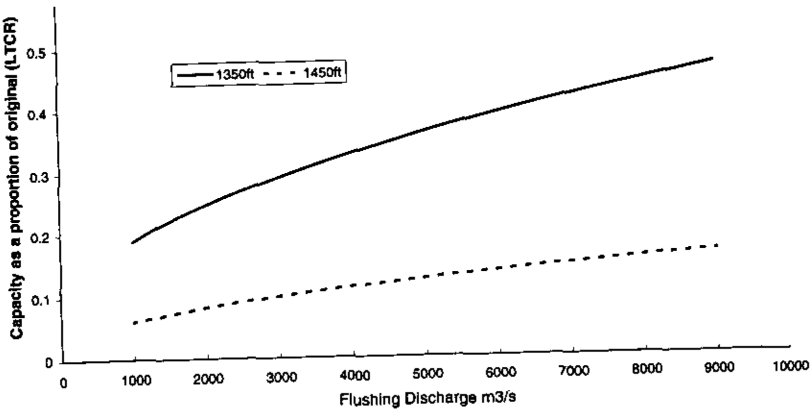


Fig 5 Calibration of numerical model at Tarbela Reservoir

Additional, more expensive, structures at the dam to enable flushing at a water level of 1350ft and 1450ft (411.5m & 442m) were studied (the base of the dam is at 1120ft, 341m, and the maximum level is 1550ft, 472m). Flushing could be performed in May and June. Fig 6 shows the results of applying the simple flushing criteria, discussed earlier in the paper, to Tarbela. A flushing period of 30 days annually has been assumed. The results indicate that flushing at the higher water level (1450ft) will not be effective, and that flushing should be performed when river flows are above about 6,000m^3/s or 7,000m^3/s. Under these circumstances the sediment balance will not be a constraint on flushing, while the long term capacity can be sustained at approaching half of the original capacity. The estimated long term capacities were sufficiently large to suggest that the impact of flushing on extending the life of the reservoir would be large.



(a) Sediment balance ratio



(b) Long term capacity ratio

Fig 6 Application of simple assessment criteria to Tarbela

Application of the 1D model confirmed that the introduction of flushing would greatly extend the life of the reservoir. Even after 60 years (2056), live storage would remain as high as about 8,000Mm^3 and would be reducing at a rate of about 1,000Mm^3 every 50 years. Without flushing, but with civil works to raise the elevation of the intakes, reservoir life is estimated to be at about 70 years.

An important issue relating to the feasibility of flushing at Tarbela was the requirement to provide discharges for peak power production during the flushing period. Peak power supplies can be maintained by daily interruption to flushing and by allowing the sediments in the power discharges to be deposited in a wide channel upstream from the power intakes.

The cost of construction of the flushing structures is very high, but represents a fraction of the cost of providing equivalent benefits by constructing a new reservoir, notwithstanding the environmental and social impact of a new reservoir.

CONCLUSIONS

Reservoir flushing and other changes in operational practice have perhaps not received sufficient attention as a means of restoring and sustaining reservoir capacity, especially during the design of new impoundments. The techniques outlined here enable assessment of operational changes using little data, and then predict their impact on reservoir life. The techniques, especially numerical modelling, provide output suitable for use in an economic appraisal.

Research into the environmental impact of flushing is recommended.

REFERENCES

Ahmed TES & Mamad OET (1986). Sediment transport in relation to reservoirs. International Conf on Water Resorces Needs and Planning in Drought Prone Areas. Khartoum

Atkinson E (1996). Feasibility of flushing sediment from reservoirs. Report OD 137, HR Wallingford, Oxon, UK.

Attewill LJS, White WR & Bilgi A (1998). Sediment management studies of Tarbela Dam, Pakistan. Proc. Conf. British Dam Society, Bangor

Basson GR & Olesen W (1997). Modelling flood flushing. International Water Power & Dam Construction, Vol 49 No.6.

Brune GM (1953). Trap efficiency of reservoirs. Trans of the Am Geophysical Union, Vol 34, No 3, Washington, DC, pp 407-418.

IRTCES (1985). Lecture notes of the training course on reservoir sedimentation. International Research and Training Centre on Erosion and Sedimentation, Tsinghua University, Beijing, PR China.

Jowett I (1984). Sedimentation in New Zealand Hydroelectric schemes. Water International, 9, 1984, pp 172-176.

Lane EW & Koelzer VA (1953). Density of sediments deposited in reservoirs. Report No 9 of *A Study of Methods Used in Measurement and* Analysis of Sediment Loads in Streams, St Paul United States Engineering District, St Paul, Minn, US.

White WR & Bettess R (1984). The feasibility of flushing sediments through reservoirs. Proc of the Harare Symp "Challenges in African Hydrology and Water Resources", July 1984. IAHS Publ No 144

Rehabilitation of Holmestyes Reservoir

T DYKE, Yorkshire Water Services, UK
P J WILLIAMS, Sir William Halcrow & Partners, UK

SYNOPSIS. Holmestyes Reservoir was constructed in 1838 to supply water to the mills above Holmfirth. The failure of Bilberry Reservoir in 1852 lead to an investigation at Holmestyes and the discovery of depressions in the upstream face and silty seepages into the valve shaft. Remedial works in 1857 included placing of an upstream clay blanket despite the existence of a clay core in the original construction. Although the reservoir has been the subject of investigations and remedial works over the years it is still seen as a valuable asset. Following recommendations by the Inspecting Engineer in 1992, and increased seepage into the valve shaft in 1993, Yorkshire Water have carried out investigative studies and rehabilitation works to ensure the continued service of this reservoir into the next century.

INTRODUCTION

Holmestyes Reservoir is owned by Yorkshire Water and is situated some 2.5km south of Holmfirth, West Yorkshire. The reservoir is retained by an embankment dam approximately 25m high and 158m long and was constructed between 1837 and 1840 with a central clay core. The capacity of the reservoir is 313,000 cubic metres and top water level is 266.94m AOD. The slope of the upstream face of the embankment is 1 in 3.1 and is protected by random stone pitching. The downstream face is grass covered and slopes at approximately 1 in 2.1 with a near vertical masonry retaining wall at the toe of height 4m. The dam is founded on shale with Huddersfield White Sandstone of the Millstone Grit Series on both abutments.

The reservoir was one of three dams engineered for the Holme Reservoirs Commissioners by Mr G Leather, one of the other two being Bilberry Reservoir which leaked on first filling and failed in 1852. Following the failure of Bilberry and in response to concerns over leakage through the embankment at Holmestyes Reservoir the dam was modified by the addition of an upstream clay blanket in 1857.

The reservoir was built for the purpose of providing mill owners downstream with water for power during the summer months. The number of mills subsequently declined and now only one operational dye mill remains, using water for the washing process.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

In 1939 contracts were let for repairing and adapting the Holmestyes Reservoir for the purpose of supplying Holmfirth with water for domestic and industrial consumption. This work included the installation of a filtration plant and the laying of distribution mains. New treatment facilities were installed in 1980 and these continued in service until 1993 when raw water supply from Holmestyes was discontinued. However, the reservoir continues to represent a valuable asset and is now used to satisfy minimum discharge requirements downstream (890 m^3/day).

During its 160 year life the reservoir has suffered from a series of leakage problems and has been the subject of a number of investigations. Geotechnical investigations were carried out within the embankment in 1982, 1989 and 1991. This paper summarises the most recent investigations and remedial works arising from the latest statutory inspection under the Reservoirs Act 1975, carried out in 1992.

LEAKAGES

In the first report under the 1930 Reservoirs (Safety Provisions) Act, in 1933, it was noted that there was only slight and insignificant leakage into the culvert downstream of the valve shaft on the combined draw off and scour. By 1938 it was reported that the upper part of the shaft was leaking badly enough to interfere with inspection, and remedial grouting was proposed. It was also reported that the masonry of the culvert downstream of the shaft showed signs of disintegration and the haunches of the culvert were in a state of collapse.

Remedial grouting was undertaken in 1939 in the form of radial grouting from within the valve shaft and was reported as being successful in stemming the leakage. At the same time works were undertaken to strengthen the culvert with steel arches, grouting and a 50mm skin of reinforced mortar. By 1944 reports suggest that leakage into the shaft had re-established in the form of pin hole jets with total leakage in the shaft and culvert at 0.63l/s. The only remedial works undertaken at this stage took the form of caulking at the points of inflow.

GEOTECHNICAL INVESTIGATIONS AT HOLMESTYES DAM

In 1989 a geotechnical investigation of Holmestyes Dam was carried out by the Building Research Establishment. The investigation was instigated as a result of concerns about the build up of pore water pressures in the upstream shoulder fill between the central clay core and the upstream blanket. A major concern was whether the clay blanket would lift during reservoir draw down. The purpose of the investigation was to provide information about pore pressures.

Three boreholes were drilled; one into the downstream edge of the core and two inclined into the upstream fill. The investigation confirmed the existence of a central clay core and that it was significantly less permeable than the upstream fill. The report also concluded that the permeability of the upstream fill was generally high but that piezometers in the upstream fill showed water levels significantly less than reservoir level. It was therefore concluded that the possibility of the upstream clay blanket lifting on reservoir draw down due to excess water pressure was unlikely at that time.

A subsequent investigation was undertaken by the Building Research Establishment in 1992 following the drilling of four more boreholes in 1991 and the installation of additional piezometers. The piezometric data confirmed that the upstream clay blanket remained generally effective and that the piezometric level in the upstream fill was only influenced slightly by changes in reservoir level. As part of the study the Building Research Establishment undertook an investigation into the effects of embankment deformation resulting from changes in reservoir level. The measurements showed that reservoir draw down caused only a small heave along the wave wall and upstream pitching and reservoir refilling caused a small settlement (Tedd et al, 1993). Locations of all piezometers, including those installed in 1997, are shown in the embankment plan on Fig 1.

WATER TRACING STUDY

In his report of 1992 the Inspecting Engineer (J L Beaver) made reference to increased seepage flows into the shaft and culvert which were occurring at higher elevations than previously observed. The report also noted that silt was being deposited in a chamber used for measuring seepage flows. The Inspecting Engineer's report recommended in the interests of safety that works be put in hand to stem the leakage through the intake tower.

In 1993, a water tracing study was undertaken to determine the source of the seepage water into the shaft in order to best determine the nature of the works to stem leakage. As part of this study the records of reservoir water level, rainfall, piezometric levels in the upstream shoulder and leakage flows were reviewed. In general the recorded levels in the piezometers around the valve shaft showed little response to reservoir level changes, with a draw-down in reservoir level of nearly 10m in 1991 producing less than 1m response in a piezometer adjacent to the valve shaft.

On the other hand there did appear to be a correlation between the seepage flows recorded for the shaft and culvert and the reservoir level. Seepage flows were found to decline sharply and immediately when water levels dropped below 0.5m below Top Water Level. However, on refilling to above this level there was a time lag before seepage flows increased to their original levels. This pattern was seen as consistent with a seepage path originating near Top Water Level with connection to the leakage points in the valve shaft. There was no correlation between rainfall, piezometric levels at the shaft or seepage flows.

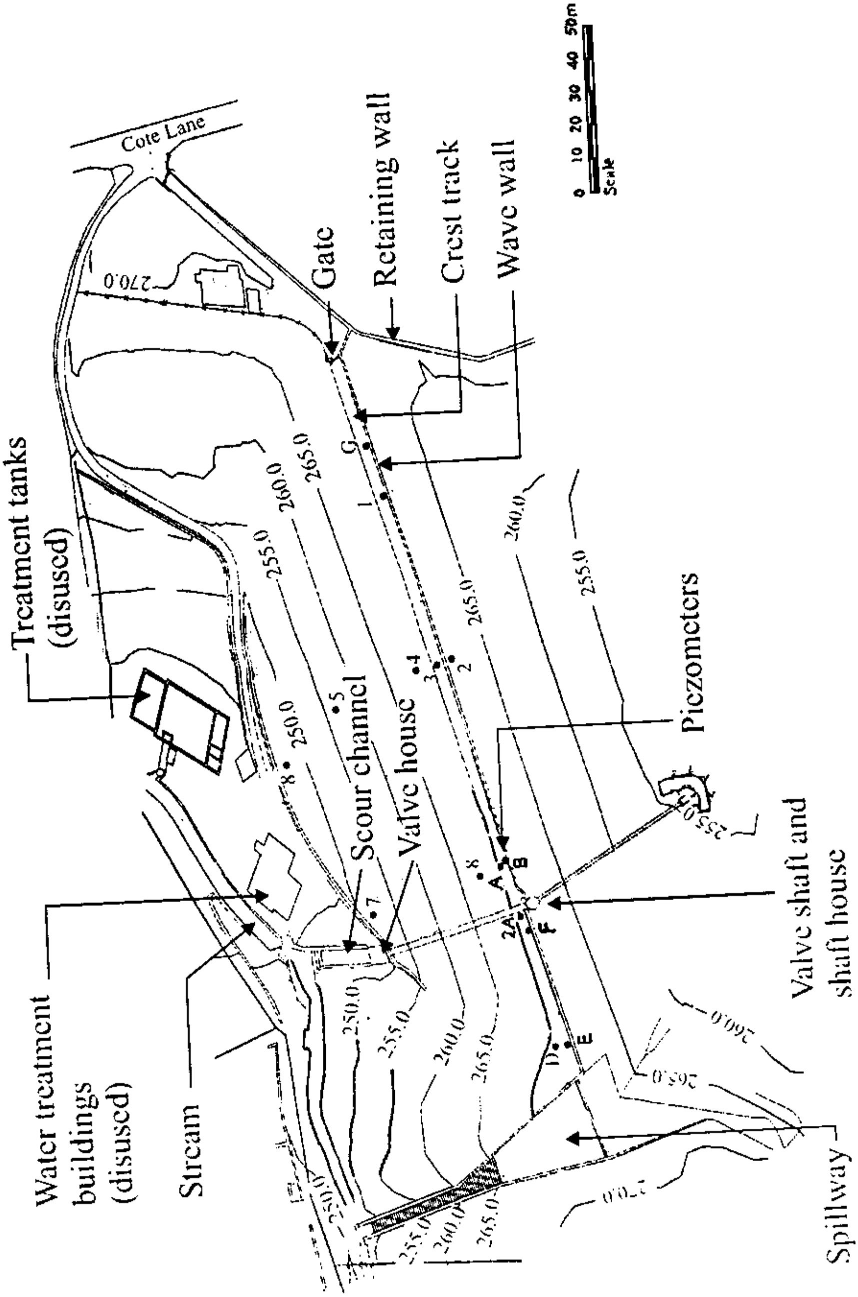


Fig 1 : Plan of Holmestyes Dam

A water tracing study was carried out to identify the source of the discharges into the valve tower by analysis of water samples from a variety of locations around the dam. The study included a hydro-chemical analysis on the two potential sources of leakage water which were seen as the reservoir and ground water from the abutments. Samples from within the embankment were taken from the piezometer boreholes and further samples were taken from seepage flows into the valve shaft.

Testing of the water samples was undertaken at the Scientific Support Laboratory of Yorkshire Water Services in Bradford who analysed the following determinands: alkalinity, chloride, magnesium, sulphate, sodium, potassium, calcium, zinc, manganese and iron in addition to temperature, conductivity, pH, nitrate and TOC.

The study concluded that discharges into the valve house were of the same water quality as the reservoir water and, on available information, different from Huddersfield White Rock ground water. Surface seepage observed on the right abutment at mid height of the dam and immediately downstream was found not to be reservoir water but older ground water.

REMEDIAL GROUTING WORKS

Following the water tracing study and confirmation that seepage water in the shaft was indeed emanating from the reservoir remedial grouting works were proposed around the shaft. The grouting work had two objectives.

- to prevent leakage water washing fine fill material out of the upstream shoulder of the embankment,
- to fill any voids or seepage paths already formed in the embankment.

The grouting took the form of a ring of grout holes around the valve shaft drilled vertically from the surface to the level of the base of the shaft at approximately 1.2m from the outside surface of the shaft. The grouting was carried out by the tube-à-manchette method with alternate holes being treated as primary and secondary grout holes and alternative manchette sleeves within each hole being classified as primary and secondary grouting points. The holes were located at a nominal horizontal spacing of 1.5m and the sleeves at a vertical spacing of 0.5m down each hole. The location of the grout holes relative to the shaft are shown on Fig. 2.

The tube-à-manchette grouting method was chosen as it offered the opportunity to return to re-grout in the event that extensive connections were found or to follow seepages. At the end of the grouting contract the tube-à-manchette holes were flushed out and the ends of the tubes capped to allow for future grouting in the event that leakage resumed.

Grout takes were generally quite modest and there were good connections to the valve shaft. A summary of grout takes by depth zone from the crest is given in Table 1.

Table 1. Summary of grout takes around valve shaft

Depth below crest	Total grout take in 5 primary grout holes	Total grout take in 5 secondary grout holes*
0 - 5 m	80 litres	260 litres
5 - 10 m	1700 litres	1290 litres
10 - 15 m	6360 litres	2360 litres
15 - 20 m	5780 litres	2660 litres
20 - 25 m	3730 litres	1190 litres
* only 4 holes grouted due to connection to culvert for one hole		

The grouting was largely successful and on refilling of the reservoir the seepage into the shaft was significantly reduced, dropping to 40 l/min from the 125 l/min that had been occurring before the grouting. The original grouting design had called for additional radial grouting from within the bottom of the valve shaft. This grouting was not undertaken because of connections between the bottom of two of the vertical grout holes and the culvert upstream of the valve shaft. There was concern that if additional grouting was undertaken in this area it would result in grout entering the culvert upstream of the valve. The decision was therefore taken to stop the grouting after completion of the vertical holes and to carry out further investigations into the condition of the upstream culvert before grouting around the toe of the shaft.

REMEDIAL WORKS TO SCOUR VALVE

The contract to undertake remedial works to the combined scour and draw-off valve and pipe work was let in June 1997. This work was required for two principal reasons.

- firstly problems had been encountered with opening and closing the guard valve and,
- secondly concerns had been raised over the condition of the pipe culvert upstream of the valve shaft following the grouting works in 1996, when positive connections had been found between two of the grout holes and the drain adjacent to the upstream culvert.

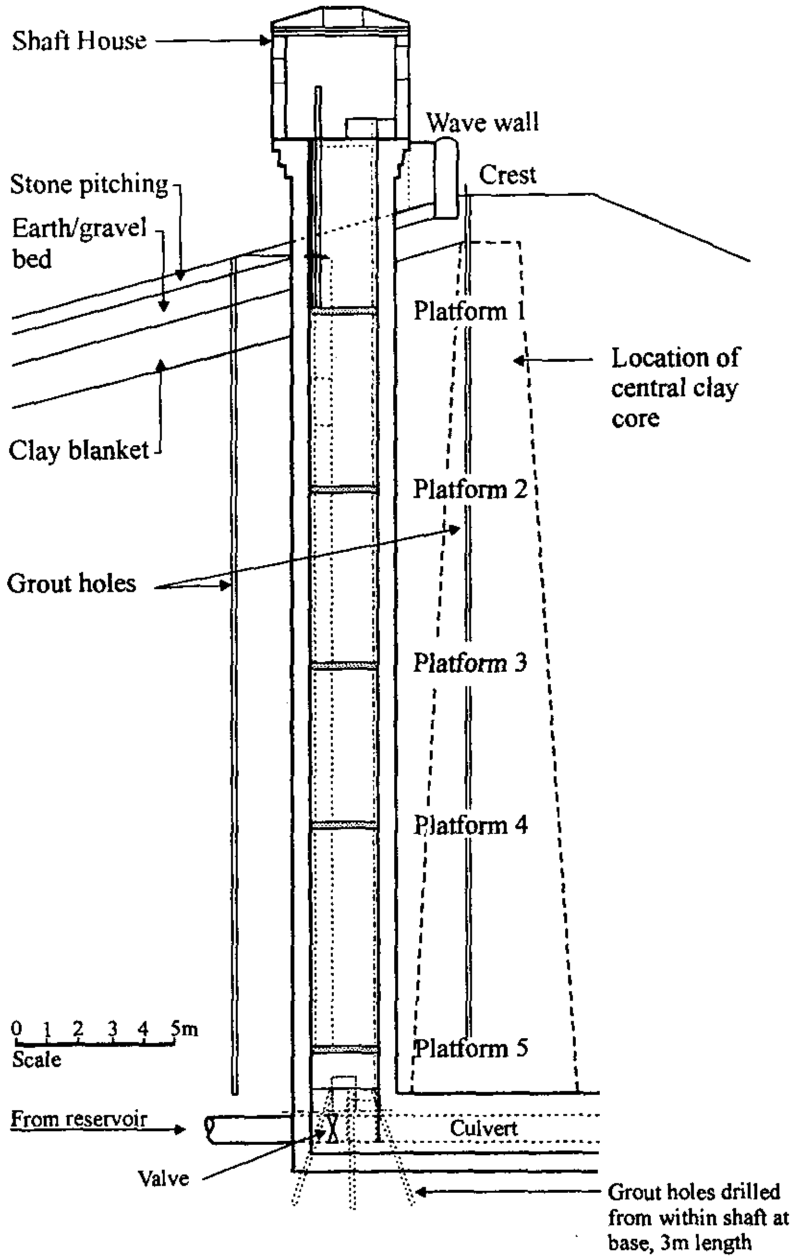


Fig 2 : Valve Shaft Sectional Elevation

The combined draw-off and scour works consist of a pipe through the dam controlled by a guard valve at the base of the shaft located just upstream of the dam core. The valve shaft has an internal diameter of 1.4m and incorporates a concrete floor at the dam crest level and five internal steel landings between the valve house floor and the base. The spindle for the existing valve and an access ladder run up the shaft, the 300mm pipe downstream of the guard valve is laid in a 1.0m diameter culvert leading to a valve house at the downstream toe of the dam. The 300mm pipe connects to a T piece with draw off and scour outlets each controlled by separate 300mm valves housed in the downstream valve house.

The works undertaken comprised dewatering of the reservoir, replacement of the guard valve, investigation of the condition of the pipe culvert upstream of the valve shaft, additional grouting works and other miscellaneous works.

In advance of the contract award the reservoir was drawn down by Yorkshire Water to approximately 25% depth. This draw down was controlled to ensure that excess water pressure in the upstream shoulder did not lift the upstream clay blanket and took place over a two month period at an average rate of 0.3m/day.

It was originally thought, on the basis of the construction drawings, that the pipe upstream of the 24" (600mm) guard valve at the foot of the valve shaft was connected through a concrete plug to an upstream culvert of similar section to the downstream masonry culvert. On removal of the valve and braking out of the concrete thrust block around the valve it was found that the valve was in fact connected directly to an internally flanged cast iron pipe of internal diameter 26"(650mm) with 2"(50mm) flanges and a clear diameter between flanges of 22"(700mm). It is considered likely that the original masonry culvert had been lined with the flanged pipe. This was presumably because of concerns over the stability of the culvert or as a result of leakages into the culvert. It was not possible to determine whether the annulus between the culvert and pipe had been fully grouted.

In order to undertake the valve replacement works it was necessary to overpump the flow entering the reservoir. The lowest point in the reservoir proved to be adjacent to the pipe inlet and was presumably on the line of the original stream. This water was pumped into a cascade of 2 settling ponds set in the spillway on the left abutment. The upper settling pond was constructed upstream of the spillway sill on an area of level stone pitching and the lower settling pond was constructed within the taper part of the spillway channel between the sill and the stepped section. The upper pond had an area of 90m^2 and an average depth of 0.6m whilst the lower pond had an area of 200m^2 and a depth varying between 0.3 and 1.3m. Walls to both weirs were constructed in blockwork to allow them to be quickly removed in a flood event or to fail by overtopping. This cascade settling pond arrangement was designed in consultation with the Environment Agency and proved successful in settling out the majority of silt from the overpumping water of a minimum 890m^3/day and ensuring that the discharge to the Brook remained largely clear.

Overpumping of the reservoir inflows allowed the contractor to excavate the scour pipe inlet, which had been buried in approximately 2m of silt. Excavation of the water laden silt proved extremely difficult. This problem was overcome by constructing a rockfill bund around the inlet working out from the face of the embankment. The rock bund was made partially impermeable by placing a layer of silt on the upstream face. Additional protection against overtopping of the rockfill bund was provided by placing a sandbag wall of 0.9m height along the top of the bund. Temporary pumping arrangements were set up at 2 nearby reservoirs in order to support the compensation releases from Holmestyes during the remedial works to the outlet valve.

The inlet to the pipe was found to consist of a masonry chamber with head and side walls to the inlet pipe founded on rock at the lowest point in the reservoir. The original wrought iron bar screens to the upstream and top faces of the chamber were found to be intact, but largely clogged with stone and other debris. Opening of the inlet and removal of the guard valve permitted cleaning of the upstream pipe and CCTV inspection. The pipe was found to be in good condition with no evidence of deformation of the pipe nor signs of damage to the pipe sections or flanges. Extensive surface deposits were observed in the pipe to an average thickness of 15mm, as would be expected for a reservoir with an upland moorland catchment. These deposits were relatively easily removed and, where cleaned, the cast iron pipe was found to be in good condition with only minor surface pitting and graphitisation. In view of the condition of the pipe section it was not considered necessary nor beneficial to line the pipe, as this would have resulted in a significant loss of area and hydraulic capacity.

There were concerns that the inlet to the culvert would remain prone to silt blockage and it was agreed to amend the inlet arrangement to provide a pipe stack with bulb screen set above the level of the masonry headwall by approximately 2m. This was achieved by connecting a bend and vertical stack in 600mm diameter ductile iron flanged pipework, with the bend surrounded in mass concrete as shown in Fig. 3. A stainless steel screen was set at the inlet to the pipe stack and a buoy attached to the inlet to ensure that it could be easily located in the event that the inlet became silt covered on future dewatering.

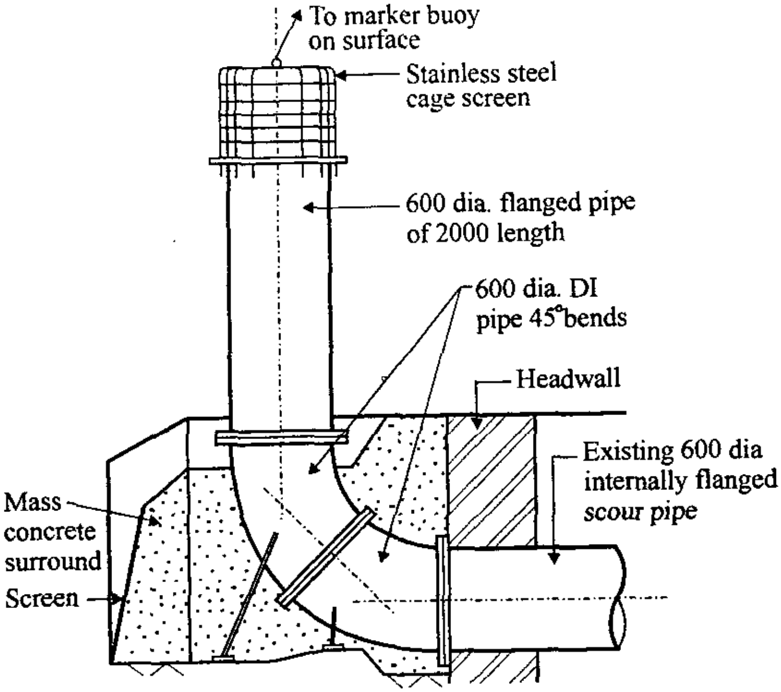


Fig. 3 : Scour Pipe Inlet Arrangement

The remedial works included for removal of the existing 24" (600mm) valve and reducer at the foot of the valve shaft and for its replacement with a 300mm valve and pipe work to connect from a plug in the upstream internally flanged pipe to the 300mm diameter pipe laid in the downstream culvert. Because the contractor opted to make access to the foot of the shaft from the downstream culvert and consequently remove the old 300mm grey cast iron (CI) pipe, the opportunity was taken to replace the pipe within the culvert with a 300mm diameter medium density polyethylene (MDPE) pipe, as preventive maintenance. The transition between the upstream internally flanged pipe and a new 300mm ductile iron (DI) pipe leading to the valve was made by inserting a 3m length of the new externally flanged pipework into the existing internally flanged pipe as shown in Fig. 4 below. The resulting annulus was grouted with a non-shrink grout to form a plug, with grout pipes left in place to allow additional grout sealing after one month. On refilling the reservoir the plug was found to remain watertight.

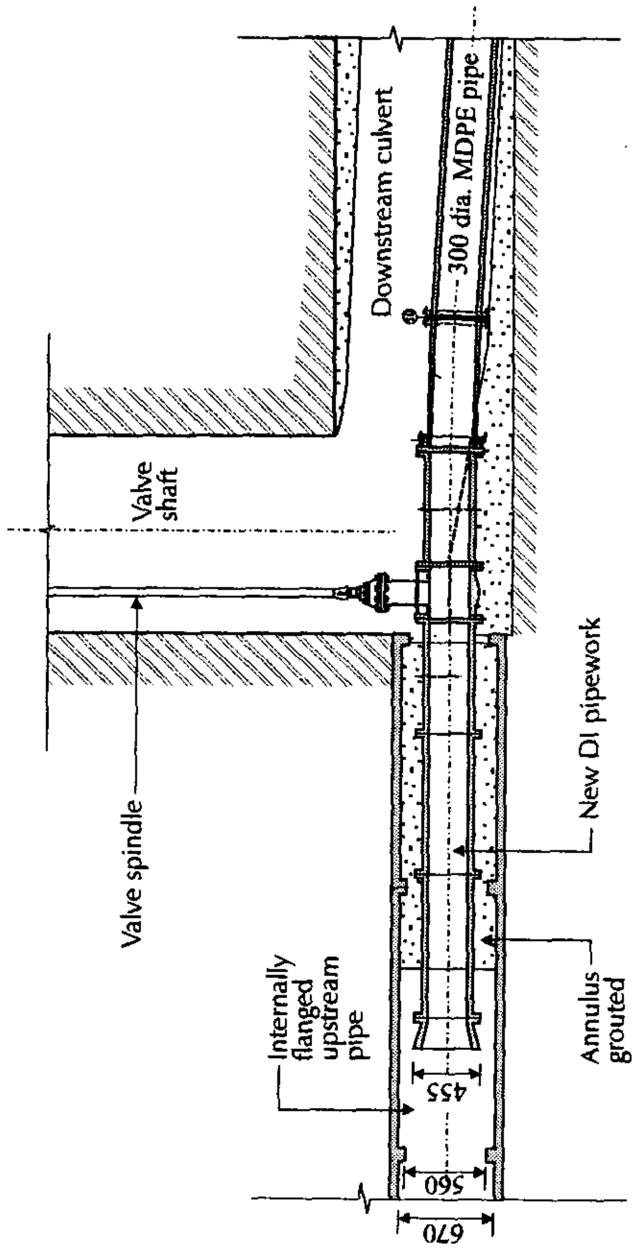


Fig. 4 : Section through base of Valve Shaft

Final grouting of the area around the foot of the shaft was undertaken on completion of the remedial works to the pipework and valve. This grouting took the form of holes drilled from within the shaft. In general the grout takes were small, with only two horizontal holes over the upstream culvert taking more significant grout quantities.

The opportunity was taken to install six new piezometers in three holes on the downstream face of the dam. These piezometers were intended to supplement the existing piezometers and to provide additional information on the phreatic surface through the embankment at its highest part and adjacent to the scour outlet.

SPILLWAY

In addition to the valve and pipe works the opportunity was taken to undertake remedial works to the spillway channel. These works comprised felling trees adjacent to the spillway channel, rebuilding of masonry walls, provision of drainage behind the channel wall, installation of gabion mattresses behind the walls to prevent surface erosion and sealing of invert joints.

The works within the stepped spillway channel included rebuilding of masonry walls where they had been disturbed by tree roots and provision of gabion mattresses of 2m width to the ground behind the walls, where it had been determined that the design flow would overtop. Prior to commencement of the works it was thought that the spillway channel walls were of a single thickness (600mm) of mortared masonry. On dismantling it was found that the walls consisted of a masonry wall on the water side of average thickness 300mm, with a dry stone wall behind to an average thickness of 600mm. Between the two walls the irregular void was filled with sand. On dismantling the masonry wall the drystone section was not disturbed and on rebuilding the wall the sand drainage zone was replaced by pea gravel and drains were provided through the masonry wall to allow drainage of the backfill.

CONCLUSIONS

The investigations and works undertaken at Holmestyes since 1993 were implemented as a result of the Inspecting Engineer's recommendations in 1992 and were completed in early 1998, at a total cost of £500,000. Through the staged studies and investigations Yorkshire Water have gained valuable information relating to the construction and performance of this asset. The remedial works carried out ensure that they have an asset that will continue to serve its purpose into the next century, despite its age and history. Continued monitoring, together with a better understanding of the dam and structures, will allow Holmestyes Reservoir to continue to be maintained.

ACKNOWLEDGEMENTS

The authors are grateful to J L Beaver (Inspecting Engineer), and A C Robertshaw (Yorkshire Water - Reservoir Safety) for their assistance in providing information relating to Holmestyes Reservoir.

REFERENCES

Tedd P, Robertshaw A C & Holton L R (1993). Investigations of an old embankment dam with upstream clay blanket and central clay core. Dams and Reservoirs, vol.3, No.1.

Grouting the puddle clay core at Barrow No 3 Reservoir, Bristol

T St JOHN, Bristol Water Plc, UK
R A NICHOLLS, Sir William Halcrow and Partners, UK
K W SENIOR, Norwest Holst Soil Engineering Ltd, UK

SYNOPSIS. Approximately 370m length of narrow puddle clay core within an earthfill dam was grouted using a single row of tube-à-manchette (TAM) grout pipes; a bentonite-cement grout was injected under strict control of pressure and volume. The grout mix was designed to ensure that the matured grout strength had similar strength to the puddle clay. Falling head permeability tests were conducted in separate boreholes to assess the effectiveness of the grouting operations.

INTRODUCTION

Barrow No. 3 reservoir is situated approximately 7km south west of Bristol and was built between 1887 and 1899. The embankment, which is approximately 2000m long and varies in height between 4m and 20m, has a long history of instability and leakage (Bristol Water Works Co 1982, Swannell, 1996). The reservoir is an earth fill and puddle clay core embankment storage reservoir with a capacity of approximately 2,300ML. Recent inspections resulted in the requirement for Bristol Water Plc (BW) to undertake work to reduce the leakage in the northeastern corner of the reservoir. In late 1995 Sir William Halcrow & Partners (SWHP) was appointed to design the remedial works. A contract was awarded to Norwest Holst Soil Engineering Ltd (NWH) in June 1996, to undertake grouting of the puddle clay core. BW had an Engineering Representative in attendance during the works and SWHP was retained in the capacity of Technical Advisor.

HISTORY OF LEAKAGE

Although several remedial changes were made during the long construction period of the reservoir to its present height, possibly the most interesting remedial works were the core grouting works that were undertaken by Soil Mechanics Ltd in 1971.

As shown in Fig. 1, three sections of core, totalling some 300m in length, were treated using a single row of tube-à-manchette grout tubes installed at a spacing of approximately 1.5m. One section of remedial work required a supplementary row of grout tubes some 2m downstream of the principal row. Each grout tube was fitted with 10 rubber sleeves and a multi stage injection pattern was followed with careful control of grout pressures. Heave tell-tales were installed along the crest of the dam.

The long term monitoring of the dam has confirmed that the 1971 core grouting works were successful.

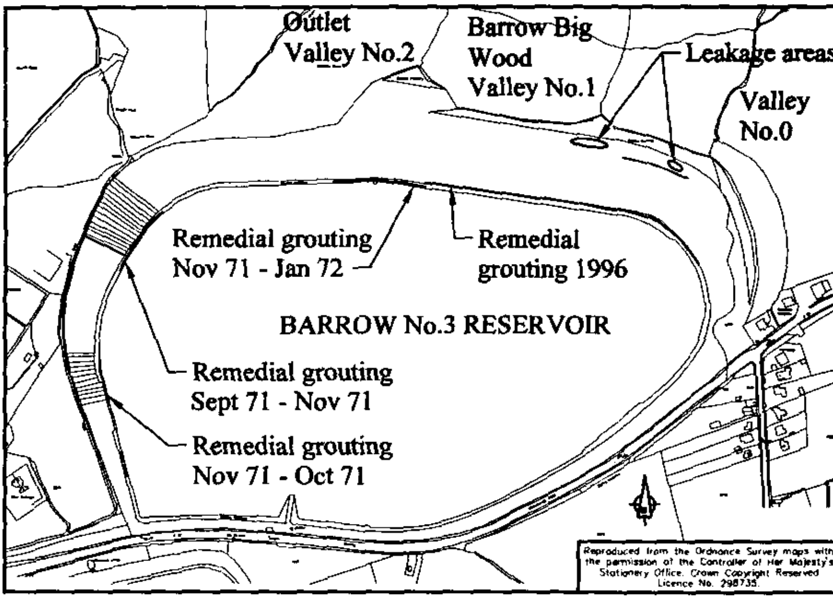


Fig.1. Plan of Barrow No3 Reservoir

Decision to grout

Continued monitoring of the dam resulted in a series of 7 piezometers being installed at Valley No 2, during 1993. The location of the valleys and areas of leakage are shown on Fig. 1. In 1994, an additional 13 piezometers were installed in Valley No 0 and Valley No 1 to assist with understanding the presence of the areas of leakage on the downstream face. The back analysis of a long period of piezometer data culminated in the conclusion that the core was leaking and that remedial works should be scheduled. These remedial works included excavation of the embankment to determine the level of the clay core to eliminate the possibility of reservoir water overtopping the core and the grouting of an extended length of the core.

REMEDIAL DESIGN

From consideration of the observed leakage and piezometer data, together with the history of the dam, it was decided that grouting of the puddle clay core should be undertaken. The initial proposal for grouting some 150m of core was extended to 369m, which would allow the new remedial grouting work to become contiguous with the 1971 remedial grouting work and thus form a considerable length of continuous grouted clay core. The locations of the grouting works are shown on Fig. 1.

Requirements

The Contractor was required to install 246 TAMs at a spacing of 1.5m along the centreline of the puddle clay core. The location of the puddle clay core was to be determined by a series of shallow trial pits along the dam crest access track. The TAMs were required to be installed at a depth corresponding to a level of 1m below the dam foundation; the depth of the TAMs was approximately 18m. The grouting port rubber sleeves were to be spaced at 0.5m vertical spacing.

Stage grouting of the core was required through the TAM ports. The experience gained in the 1971 grouting works and reported in detail by Soil Mechanics Ltd (Bristol Water Works Co. 1982) formed the basis for the Specification for the works, which detailed the sequence of the stage grouting and the upper limit of grouting pressure.

Upon completion of grouting selective testing of the completed grout curtain would be undertaken by *in situ* permeability tests within specific boreholes drilled for such testing.

CDM requirements

The nature and extent of the works required that the works were subject to the requirements of the Construction (Design and Management) Regulations 1994. The Planning Supervisor for the works was Halcrow H & S Limited.

DETAILS OF THE WORK

The width of the puddle clay core at the top is approximately 1.5m, increasing to 3m at its base and only a single row of grout tubes could be installed through the core. The effectiveness of the 1971 work indicated that one single row would be sufficient to tighten the clay core and to reduce the permeability of the core *en masse.*

Installation of TAM tubes

The location of each TAM was marked on the crest track with a wooden peg and each grout tube had a unique number. Even numbers were assigned primary grout tubes and odd numbers were assigned as secondary grout tubes.

The holes for the grout tubes were drilled with a track mounted Hands England drilling rig utilising NWY rods and a 112mm drag bit. Compressed air with water injection was used as the flushing medium. Upon completion of each hole to the correct depth bentonite-cement grout was injected immediately into the hole as the drill rods were withdrawn.

When the complete string had been pulled out the first 3m TAM tube with a blank cap at the base was lowered into the hole and subsequent 3m lengths added until the TAM extended to the required depth. A removable blank cap was attached to the top of the TAM at ground level and the tube was then held in position by means of weighted timber placed across the hole until the annulus grout had set. For the duration of the work the TAM tubes had temporary sealing caps and each tube was labelled with a marked plastic tag collar.

The sealing grout was a weak 2 parts Bentonite/ 1 part Ordinary Portland cement, with a water-cement ratio of 10:1, designed to hold the tube in place but to be weaker than the surrounding clay core and capable of being fractured at low injection pressures.

Upon completion of the grouting work, a small pit was excavated around each TAM tube, the tube was cut down and a sealing cap fitted. The pit was back filled and capped with the same material as used on the crest access track.

Details of TAM tubes

The grouting tubes were UPVC tubes 50mm outside diameter with a wall thickness of 3.6mm. The rubber TAM sleeves were installed over the grouting ports at a vertical spacing of 0.5m and were fitted into machined recesses in the wall of the grout tubes. The assembled tubes were 3m long and joined to subsequent tubes by screw coupling to achieve the full depth of TAM installation. The grout tubes were specified by NWH and fabricated in the UK by Associated Polymers Ltd.

The installed TAM tubes were designed to be used with a grout pipe, or lance, which used a double seal packer system manufactured to a design prepared by NWH. This grout pipe had a 0.5m long perforated section with several seals top and bottom and the perforated section was centred over an individual TAM port during an injection. This system ensured that a specific TAM grout port could be selected and grouted, thus ensuring a fully controlled and flexible stage grouting sequence.

Stage grouting

The specification required that each grout tube be designated as primary or secondary, with each type being spaced at 3m intervals along the core. Each tube was required to have primary and secondary TAM sleeves, with the primary sleeves spaced at 1m vertically and the uppermost sleeve positioned at the top water level of the reservoir.

The required sequence of injections was such as to ensure that alternate sleeves and tubes were covered in each pass; after four passes all sleeves had received one injection. This phase of the work was termed Stage 1 grouting. Those sleeves in which the initial injection did not achieve the appropriate acceptance criterion were scheduled for a repeat injection. Such re-injections constituted passes 5 to 8 and were termed Stage 2 grouting. In some locations where Stage 2 grouting did not reach the acceptance criterion further injections were undertaken in an additional series of injections, passes 9 to 12 and referred to as Stage 3. The sequence of passes and associated ports and TAM tubes are shown on the table below. All stages of grouting were undertaken to the same pressure and acceptance criteria.

Table 1. Grouting sequence

Grout passes			TAM - Port	TAM - Tube
Stage 1	Stage 2	Stage 3		
1	5	9	Primary	Primary
2	6	10	Primary	Secondary
3	7	11	Secondary	Primary
4	8	12	Secondary	Secondary

Injection pressure and grout take criteria

Injecting grout into a thin clay core requires close control over the actual pressures adopted to avoid damaging the core by hydro fracturing. The injection pressure was limited to 10x kPa, where x is the depth in metres from the crest to any particular TAM port.

Early technical meetings on site established the grout acceptance and the method of monitoring surface heave. The upper limit volume to be injected through a TAM port at pressures up to the specified maximum pressure was 45 litres (10gals). If there was minimal acceptance of grout the appropriate maximum limit pressure was maintained for 5 minutes.

The deformation of the ground surface was assessed visually using levelled reference beams upon which were placed a long accurate fluid spirit level. Any movement of the bubble would indicate heave and grouting would be stopped. The 3m long reference beams were levelled by means of miniature screw jacks founded on wooden plinths near the active injection tube. The beams were positioned parallel to the clay core centreline and at distances of 1m and 2m from the dam crest on the downstream slope. This simple system proved very effective in monitoring surface heave.

Equipment used

Grout was mixed with a double drum colloidal mixer and transferred by an in-built pump to an air operated holding agitator tank prior to injection. An air operated GS45 grout pump was used to discharge grout from the holding tank into the injection hose and to maintain a pressure head of grout in the system. The delivery hose incorporated a pressure head consisting of a 'T' piece with a valve operated outlet on the discharge side and a low pressure gauge preceded by a valve operated return to the tank line on the inlet side which acted as a pressure relief valve.

Method of grouting

At the commencement of a grouting session, with the grout pipe set at the specific TAM port, the grout was pumped from the agitator tank with the injection tube inlet valve closed and the relief valve fully opened. The system was considered fully charged when the level in the holding tank had stabilised. Once the system had stabilised the inlet valve was slowly opened at the same time the relief valve was gradually closed. In this condition grout was passing down the grout pipe, through the perforated packer section and forcing grout through the TAM port and annulus grout into the clay core.

Grout was injected into a TAM port until the maximum specified grout pressure was reached and held for 5 minutes or until a volume of 45 litres (10 gals) of grout had been injected at a pressure lower than the specified maximum. If less than 45 litres had been grouted through a port at the maximum pressure held for 5 minutes that portion of the core was considered tight and would not require additional grouting. If any port had taken 45 litres of grout without reaching the specified maximum grout pressure, the grouting was stopped and the records marked so that the port would need additional grouting. At each TAM port a slightly increased initial grout pressure was required to break the annulus sealing grout and lift

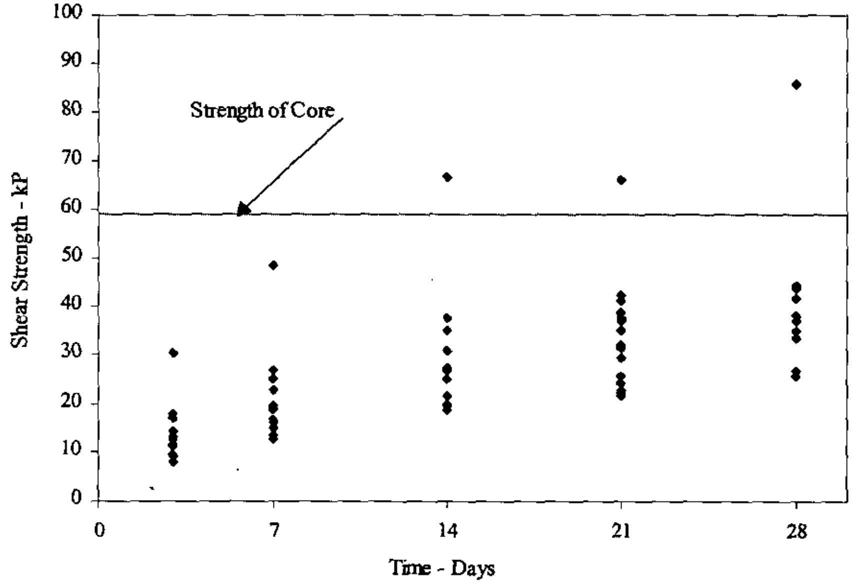


Fig .2. Shear Strength for Various Curing Times the rubber port valve; typical 'cracking pressures' were 20-30kPa. Once the seals are broken the hydraulic pressure drops quickly and the grouting process can be continued with pressure controlled by the return pressure-regulating valve.

Grout mix

Undisturbed soil samples taken from the clay core were subjected to undrained triaxial compression tests in the NWH laboratory and revealed that the average shear strength of the clay core was approximately 60kPa. The injection grout was designed to have a fully set shear strength similar to, but less than, that of the clay core. There is little published information on the strength of bentonite-cement grouts, although Redlands Minerals provided some information. Consequently, trial mixes were undertaken to finalise the grout mix design; samples were taken and tested at various time intervals up to the standard 28-day strength. The final mix design was 1 part bentonite to 1 part cement with a water cement ratio of 10:1. Fig. 2 shows the average shear strength of the grout with various times of curing. The single high values at each specific curing time were from one batch of grout and not representative of the performance of the general mix.

Site control and samples

Samples of the grout were taken weekly and tested at specific time intervals. The samples were kept in plastic tube moulds and strengths were measured using a shear vane. Flow trough and Marsh cone tests were also undertaken daily during the works, average values were 23 and 32 secs respectively.

Control of pressure and grout take

A three-man team controlled the injection operations. One operator controlled the grout pressure, by adjusting the relief valve and observing the grout pressure using a low pressure Bourdon type gauge. A second operator measured the grout take by direct measurement of grout fluid level in the holding tank and a third operator monitored the reference beams on the downstream face.

Sequence of grouting

The grout injection phase of the works was split into sections with each section containing approximately 25 TAMs. The length of a section was the maximum that could be served by the grouting equipment at a particular setup. The grouting in any TAM tube commenced with the lowest primary port and the grouting carried out from the bottom upwards. The sequence of grout passes undertaken is shown in Table 1. The first four passes constituted Stage 1. Additional grout passes were only undertaken at ports that failed to reach the acceptance criteria.

Variability of grout take

It was not possible to identify any specific grout take patterns with depth within the core, but some lengths of the core took considerably more grout than others. In these areas post grouting hydraulic testing was carried out to confirm the work had tightened up the core. The length of the dam between Valley No 1 and the draw off tower, located in Valley No 2, took the highest volumes of grout. In contrast, the grout take immediately opposite the main area of leakage, in the vicinity of Valley No1, were relatively low.

SITE CONTROL

The grout takes for each port were assessed on a daily basis and, based upon the agreed acceptance criteria, the need to re-inject each port was established. The first re-injection of any port constituted Stage 2 and the systematic sequence adopted during the Stage 1 work was followed. The grout takes for the re-injected ports were assessed to establish whether the grout take acceptance criteria during Stage 2 had been achieved, if the takes were still unacceptable then specific ports were scheduled for additional grouting under Stage 3.

Assessment of grout takes

The assessment of grout take in over seven thousand TAM ports requires a reliable and safe site monitoring and recording system to ensure that the grouted TAM ports achieve the specification. A hand drawn graphical system was used; every day the data from the daily grout journals for each

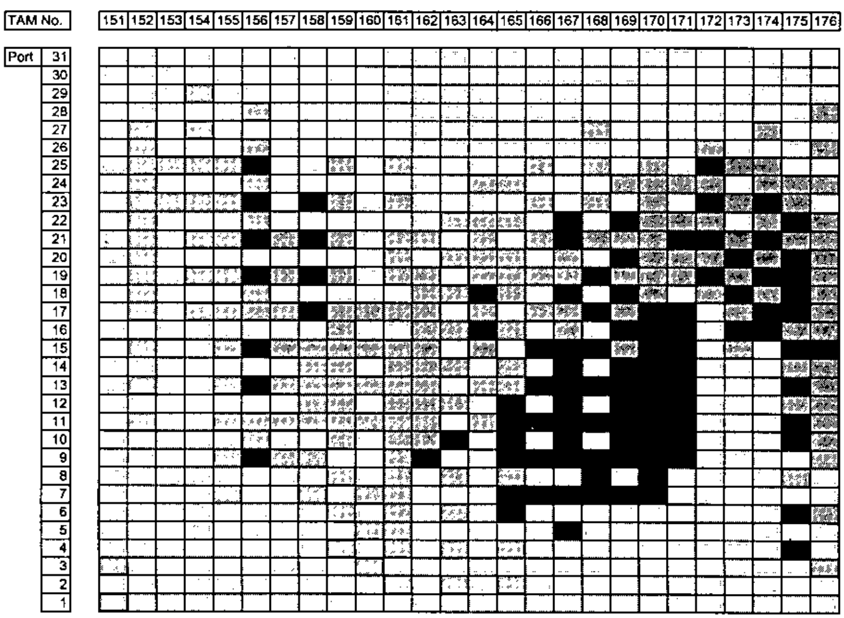


Fig.3. Grout Takes for a Specific Section

grouting team were transferred to the master graphical record which displayed, in colour, the status of each sleeve after each injection. Adopting such a colour coded system clearly showed which sleeves had been treated and which sleeves required additional injections. Fig. 3 shows a typical record for a section that had high grout takes. There are four grey scales on the figure; the lightest shading representing Stage 1 with grout takes up to 45 litres. With increasing darkness, Stage 2 having takes of 45 to 90 litres, Stage 3 having takes of between 90 and 135 litres and the highest takes, in Stage 4, of greater than 135 litres are shown in black.

HYDRAULIC TESTING

The Specification required that, following the completion of the grouting works, *in situ* permeability testing would be undertaken to assess the effectiveness of the works. The tests were undertaken after a minimum of 14 days after grouting. The locations of specific tests were established within areas where the graphical site records showed that the TAM ports had taken the maximum limit volumes of grout in the first and second stages and had tightened up during the Stage 3 grouting.

Borehole permeability tests

Falling head permeability tests were carried out at the base of rotary drilled boreholes. The boreholes were 75mm in diameter and drilled in the centre of the core. The test locations were specified after careful examination of grout takes and were positioned in the nominal centre of areas having the highest grout takes. Tests were undertaken at depths of 5m, 10m and 15m with the initial water level set at the reservoir top water level.

A 1m long test section was drilled below the bottom of the casing without the customary gravel pack. The introduction and subsequent cleaning out of the gravel was considered inappropriate for a thin clay core because of the additional disturbance and possibility of some gravel remaining within the core. Limited deformation of an unlined test section wall is inevitable in a puddle clay core and this affects the absolute evaluation of the core permeability. However, the purpose of the testing was to establish that the grouting works had effectively reduced the permeability of the clay core.

The permeability testing followed the normal procedures for falling head permeability tests. In the majority of the tests the water level within the casing did not drop during the duration of the test. The highest permeability measured was 3.3×10^{-8} m/s, indicating a relatively low permeability and indicative of effective grouting works. Upon completion of a borehole, used for hydraulic testing, it was grouted with the same grout mix as used for the grouting works, as the casing was withdrawn.

PERFORMANCE

Once the hydraulic testing had demonstrated that the grouting work had been effective the temporary restrictions on the reservoir level were lifted and the water level was allowed to reach its normal working level. The routine monitoring of the reservoir following the remedial works has shown a marked reduction in the leakage. The leakages, previously observed in the downstream face, are no longer visible and the remedial works have been considered successful.

CONCLUSIONS

The operation of grouting a puddle clay core needs delicate control of pressures, competent monitoring and good co-operation on site between all the parties. Remedial grouting works using a single row of TAMs within a relatively narrow puddle clay core of an earth fill dam has been shown to have significantly tightened up the core and reduced leakage on the downstream face. Approximately 370m length of core, some 18m deep, was grouted for the sum of £350,000 and the works were completed within 5 months.

ACKNOWLEDGEMENTS

The authors would like to thank Mr J B Straughton, Civil Engineering Manager for Bristol Water Plc, for his support and permission to publish this paper.

REFERENCES

Bristol Water Works Co (1982). Barrow No. 3 Reservoir – Review of the History of the Embankment Dam with the Site Investigation of 1981. Watson Hawksley.

Swannell, N G (1996). Barrow No 3 Reservoir – A Case History of Simplified Seismic Safety Evaluation. Proc. 9th Conf. British Dam Society, York. Pp248-257

The rehabilitation of Luxhay Dam, Somerset

J P MILLMORE, Babtie Group, UK
J F STABLES, Consultant (formerly Wessex Water), UK
F E SHANNON, Babtie Group, UK

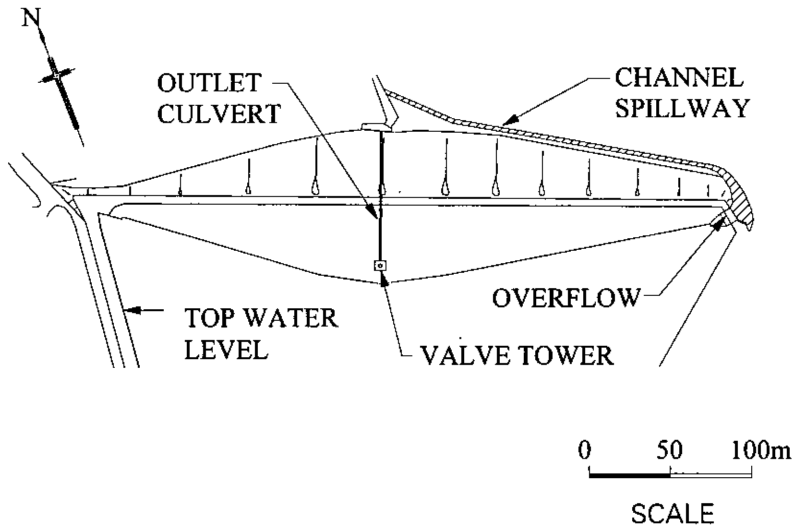
SYNOPSIS. Luxhay Dam is a typical late Victorian earth embankment dam constructed for the public water supply of Taunton, Somerset. The paper reviews the history of the dam in terms of it's construction and performance, the subsequent events, geotechnical investigations, and economic consideration of the options for the control of leakage all of which lead to the decision to replace the puddle clay in the upper part of the core with a cement/bentonite mix material. It also describes the carrying out of the works and the results of monitoring to date.

INTRODUCTION

Luxhay Dam was built in about 1905 by Taunton Corporation in Somerset, UK. The present owners, Wessex Water use the reservoir to supply water to the Taunton area through the recently reconstructed Fulwood Treatment Works. The construction drawings show the 430m long, 19m high embankment dam to have a "puddle wall" surrounded by "selected material" and "outer fill" (Fig 1 & 2). It has been inspected regularly under the two Reservoirs Acts and it was from an inspection in 1969 that there came the first hint of leakage. Inspections in 1977, 1980 and 1990 confirmed a slowly worsening situation which culminated in early 1994 with the formation of two extensive wet areas at the toe of the west bank and the sound of running water. The progressive increase in leakage, its investigation and the subsequent remedial works are all described in this paper.

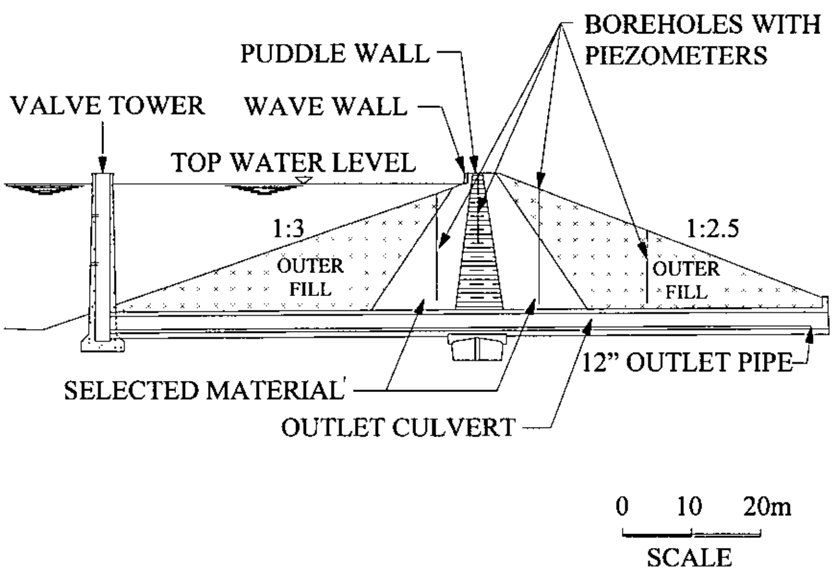
HISTORY OF LEAKAGE

The leakage manifested itself at high reservoir water levels and was observed at the downstream toe. Each succeeding inspection from 1977 onwards called for some form of action. That of 1977 resulted in the installation of 30 shallow observation wells in the downstream shoulder of the dam to observe the phreatic surface should it be located close to the surface of the embankment. The Report on the 1980 Inspection expressed more concern particularly regarding the lower third of the embankment face. Shallow French Drains were constructed a short distance up the face of the dam leading into manholes on the toe drain but there was no accurate method of flow measurement.



Plan on Luxhay Reservoir

Fig. 1



Section of Outlet Culvert and Embankment

Fig. 2

The 1990 inspection under Section 10(1) of the 1975 Act drew together all the data which had been gathered and concluded that the situation warranted a comprehensive geotechnical investigation to try and establish the cause of the leakage. As described below, a further 15 boreholes containing 22 piezometers, were installed during the investigation in 1992. Two V-notch measuring chambers were also constructed, one on each flank toe drain. This instrumentation was to observe water pressures in the body of the embankment and the volume and location of any leakage through the dam.

Wessex staff on a visit one afternoon in early April 1994, heard the sound of running water, which was perfectly audible to the Supervising Engineer at the other end of a mobile telephone connection. Instructions were given to lower the water level quickly by one metre. The reservoir had been full for some weeks. Two extensive wet areas were seen at the toe of the west bank which had not existed when the Supervising Engineer had made his inspection three months previously, the water level at that time had been 2.4m below top water level (TWL). The sound of running water stopped when the water level was reduced by a half metre and the wet areas dried out. This confirmed previously held opinion that leakage was occurring when the reservoir water level was within the top metre. The reservoir water level was kept lower than 1 metre below TWL whilst all available information was reviewed and options for remedial works were assessed.

REVIEW OF GEOTECHNICAL INVESTIGATIONS

Water level had been recorded in the observation wells on the downstream face since their installation in 1977. Fluctuations in level were generally in accordance with those of the reservoir and were observed to be more marked towards the base of the downsteam shoulder with many 'dry' wells located higher up the shoulder. A similar trend of flow within the toe drain was recorded with increased flows at higher reservoir levels. These flows were more evident on the west flank. There appeared to be a 'trigger reservoir level' that caused the drainage flows to increase and produced wet patches on the shoulder close to the toe.

Trial excavations were made into the dam crest in 1994 opposite the larger of the two wet areas on the west flank. The trenches were extended into the core just below the lowered reservoir level but remained dry and there were no detectable signs of seepage, cracks or other defects in the embankment.

Boreholes had been sunk in the embankment during the 1992 investigation on either side of the outfall culvert, samples taken and piezometers installed (Fig 2). Boreholes were drilled in the upstream and downstream shoulders with holes drilled through the core once its precise location was known. This work confirmed the location and properties of the puddle core but no distinction between the "selected material" and the "outer fill" in the shoulder was found. Piezometers were installed at three levels with the lowest level in the foundation. The piezometers either side of the core provided information on the hydraulic profile across the core while those in the downstream shoulder provided information on the embankment and foundation.

The impact of the leakage on stability was considered and analyses were undertaken for a variety of conditions. Soil strength parameters were identified as c'=2kN/m^2 ϕ =31^0 in the shoulder, c'=10kN/m^2 ϕ =30^0 in the core and c'=17kN/m^2 0'=32^0 in the foundation resulting in a minimum factor of safety (FoS) of 1.46. A sensitivity analysis was carried out by varying the shoulder fill strength parameters within the range c' 0-10kN/m^2 and ϕ 28-34^0 and different piezometric conditions. The most conservative assumptions and the introduction of seismic loading of 0.1g produced an FoS greater than 1.1. Stability of the dam was found to be more sensitive to variations in phreatic surface than to variations in strength parameters confirming the importance of seepage control.

An assessment of was made of the likely permeability of the embankment from inflow tests during boring and subsequent falling and rising head tests in the piezometers. This gave the following coefficients confirming that there was not a fundamental problem with permeability

Foundation	k = 7 x 10^{-7} m/s - 2 x 10^{-8} m/s
Shoulder	k = 1 x 10^{-7} m/s - 1 x 10^{-10} m/s
Core	k = 2 x 10^{-8} m/s - 4 x 10^{-9} m/s

The thickness and properties of the core were found to be sufficient to minimise the possibility of hydraulic fracture. The only variations in the core were found at depths of 4m and 7m where slightly higher moisture contents were noted. This may have been caused during construction in wet weather or by leakage through the core. Some soil samples found the core material to be a little courser and this was considered a more likely cause of the local increase in moisture content.

These observations were inconclusive in identifying the cause of the leakage. A review of the findings and the future options by Babtie Group concluded that "the abrupt way in which the leakage flow ceased upon rapidly drawing down the water level by 1 metre suggests that overtopping of the core is the most likely cause of the problem". It added "Further investigations could be undertaken to find the precise location of the leakage but the cost of the exercise is unlikely to reduce the cost of the remedial works required to control the leakage." It was therefore decided to select the most appropriate remedial treatment.

REVIEW OF LEAKAGE CONTROL OPTIONS

Two options were considered the most appropriate to overcome the leakage at Luxhay. These were either to:

a) lower the top water level of the reservoir permanently or
b) reinforce the core.

Lowering the top water level would entail lowering the spillway by 1 to 1.5 metres at an estimated capital cost of £25,000 to ensure the water was always below the level of the weakness. This would result in a loss of at least 15% of the gross storage capacity of the reservoir with a reduction in the reliable yield of 1.5 Ml/d. Wessex Water undertook a review of their resources in the area and concluded that in view of the likely cost of remedial works they wished to retain the storage.

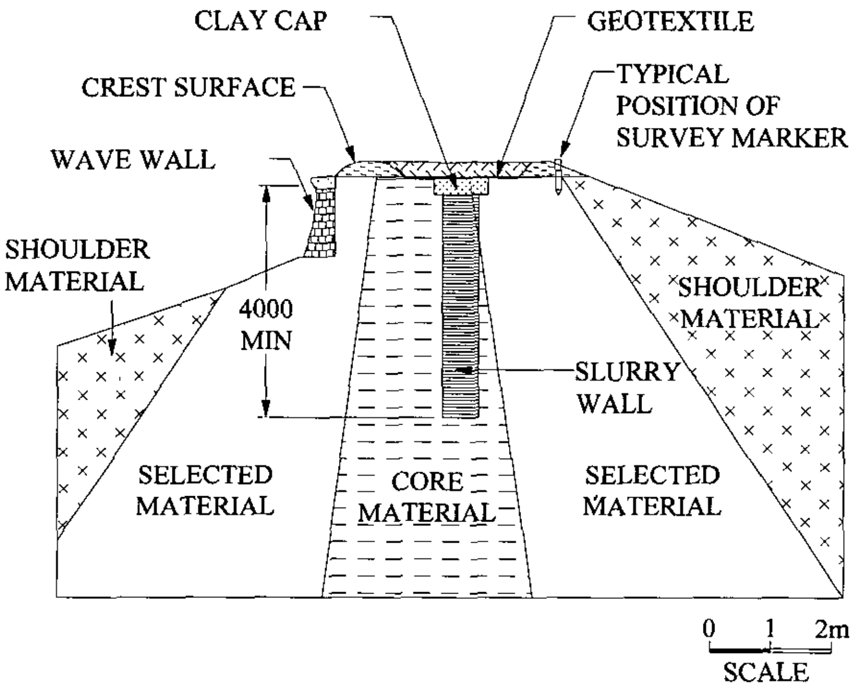
Reinforcing the core would involve remedial works depth enough to ensure the problematic zones within the embankment were adequately intercepted. It was clear that such zones extended to at least one metre below TWL i.e. 2.5m below crest level. A treatment depth of 4m below crest level was considered appropriate. Repairs to this depth could be undertaken using construction plant of a size which would avoid the need for strengthening or widening of the crest since this would increase costs significantly.

Several methods of reinforcing the core were considered in conjunction with the safety and logistical restraints of working on the dam crest adjacent to the wave wall. A cement-bentonite slurry was selected as the most appropriate form of reinforcement. For the 4m depth of slurry wall (Fig. 3) it was envisaged a backactor excavator would excavate the trench to the required depth and samples of the core material removed would be available for continuous visual examination. Replacing the top of the core over the whole length of the embankment with a cement-bentonite wall had an estimated total cost of £169 000. There would be no loss of storage and no reduction in reliable yield.

DESIGN OF DIAPHRAGM WALL

It was envisaged that a single phase, self hardening slurry trench cut-off wall would be constructed by excavating a trench through the centre of the core under a suitable head of cement-bentonite slurry. The hardened barrier was to have low permeability with strength and deformation properties not too dissimilar to the in-situ clay core material. It was recognised that the achievement of all three performance characteristics together is in conflict.

A material with a high strain at failure and a low strength was required. This would be produced from a slurry with low solids content. Low permeability



Section of Slurry Wall

Fig. 3

however would only be achieved by providing a high bentonite and cementitious content. A design compromise was selected by reducing the maximum permeability target from 5 x 10^{-9} m/s to 1 x 10^{-8} m/s at 28 days measured under a hydraulic gradient of 10, increasing the range of unconfined compressive strength from (80 - 120 kN/m^2) to (80 - 220 kN/m^2) and decreasing the strain at failure from a traditional 5% value to 3% at 90 days under a maximum effective confining stress of 150kN/m^2. Although a highly deformable slurry wall is less likely to crack and leak when there is movement of the dam there was no advantage in overspecifying this failure strain criteria. It was considered that the measured coefficient of permeability at 28 days would represent a value significantly higher than would be reached at 90 days by possibly up to one order of magnitude (BRE, 1994). A nominal thickness of slurry wall was set at 300mm and it was to be capped by 300mm of clay core material stockpiled separately to prevent desiccation cracking.

CONSTRUCTION OF DIAPHRAM WALL

In the spring of 1995 the diaphragm wall (Fig 3) was constructed by Keller Colcrete Ltd. The slurry wall was constructed using a continuous bentonite-cement slurry trenching method using a modified backhoe for excavation.

The method of construction had a number of factors to take into consideration. The width of the dam crest was restricted and access was from one end only. Movement of construction traffic was therefore constrained. Pollution of the reservoir water from the cement-bentonite slurry could not be tolerated. Careful working procedures therefore had to be adopted including factors such as the direction of slewing of the backhoe to ensure that there was no risk of a pollution incident. The weight of construction traffic was restricted to 17 tonne for a fully laden dump truck and 13½ tonne for tracked excavation plant to ensure continued stability of the dam crest. No damage was observed at the end of construction.

The mix consisted of bentonite, cement, granulated ground blast-furnace slag (GGBS) and water. The bentonite was hydrated for at least 12 hours before being mixed with cement and GGBS in a high shear colloidal mixer. The mix was stored in agitation tanks before being pumped to the trench. Slurry was generally maintained within 0.5 metre of the ground surface during excavation with a final constructed level near the top of the dam crest. The use of continuous trenching minimised the number of joints to only one per day. The slurry was still viscous after 24 hours therefore the day joint was formed by controlled recommencement of the trench under a head of slurry avoiding displacement of previously placed slurry. Examination of the excavated material was undertaken to check for any variations or inclusions within the core. This was not an easy task due to contamination by slurry but no anomalies were revealed. The top half metre of slurry wall was removed to eliminate any surface cracking. The crest was then reinstated with a clay cap before the stone crest road was replaced.

RESULTS OF TESTS

Routine compliance testing was undertaken on samples of fresh slurry and slurry from the trench producing the following results:-

a) **Permeability** at 28 days ranged from 1×10^{-9}m/s to 5×10^{-11}m/s averaging 2.8×10^{-10} which is 2 orders of magnitude less permeable than specified.
b) **Undrained Shear strength** ranged from 130 - 440 kN/m^2 at 28 days which is approximately twice the specified 80 - 220 kN/m^2 range.
c) **Strains** at failure ranged between 0.40% and 1.76% at 90 days with a mean of 0.83% significantly below the 3% minimum specified.

On the basis of the above strength and deformation results, there was concern that the slurry wall was much more rigid than the surrounding embankment. It was considered that marked differences in properties between the diaphragm wall and the puddle clay could result in cracking at their interface during movement of the embankment. This could connect with existing seepage paths in the shoulders. An additional programme of investigation and testing was proposed by the Contractor. Samples of the top of the slurry wall were subjected to further laboratory testing and he undertook a finite element (FE) analysis of the likely movements involved.

A mini excavator exposed the top of the wall and a chain saw was used to obtain samples of the hardened slurry material. Mottling or marbling due to oxidation of the bluey grey hardened slurry wall was found in the top and sides of the wall with minimal markings within the body of the wall. The material appeared brittle yet when reworked in the hand, it developed a soft consistency. Original tube samples were examined for comparison with the in-situ material. These showed similar physical characteristics but the mottling/marbling was less evident. Laboratory testing of the in-situ samples provided improved results with a mean strain at failure of 4.1% and the lowest value of 2%. The mean undrained shear strength was 418 kN/m^2.

The FE model was developed to assess the interaction of the slurry wall with the body of the dam. The model considered the dam with and without the slurry wall to establish the original dam behaviour as well as that for the specified wall and the as built wall. The shear strains predicted beneath the base of the wall were less than 1% which is less than the lowest failure strain recorded in the in-situ material tests. Concerns relating to cracking due to the different material properties were allayed. It was decided however to enhance the monitoring system with further movement stations along the crest and shallow piezometers at the toe of the embankment.

RECENT MONITORING OF BEHAVIOUR

Following completion of the remedial works the reservoir water level was raised at a controlled rate to permit monitoring of the behaviour of the dam and diaphragm wall. Luxhay Reservoir is fed by it's own direct catchment and three larger indirect catchments. Maintaining a specified reservoir water level can be achieved by controlled pumping to balance the inflow and demand. Throughout construction of the remedial works, the reservoir had been controlled to provide a sustained flow to the treatment works. Post construction, the reservoir level was raised to suit both monitoring of behaviour and operational requirements.

Observation of the flow in the toe drain manholes has been carried out since 1980 but it has not been possible to separate seepage from rainfall. The toe drain is only half a metre below the surface and so mainly collects surface water except when the phreatic surface is close to the ground surface. The V-notches installed in the system are subject to the same uncertainty. The records indicate that generally the flows increased in the past when the reservoir was close to TWL. The most significant such event arose in April 1994 when the toe drain ran full-bore, albeit clear, but it was coupled with the sound of running water.

Since completion of the remedial works the reservoir has been high for only short periods. Between April 1996 and February 1998 the reservoir exceeded the critical levels of "TWL - 1m" and "TWL - 0.5m" for approximately five months and three months respectively. No significant flow has been recorded in the toe drains, including the west flank where the leakage had been most acute. Any flows in the drains were related to surface water run-off after rain.

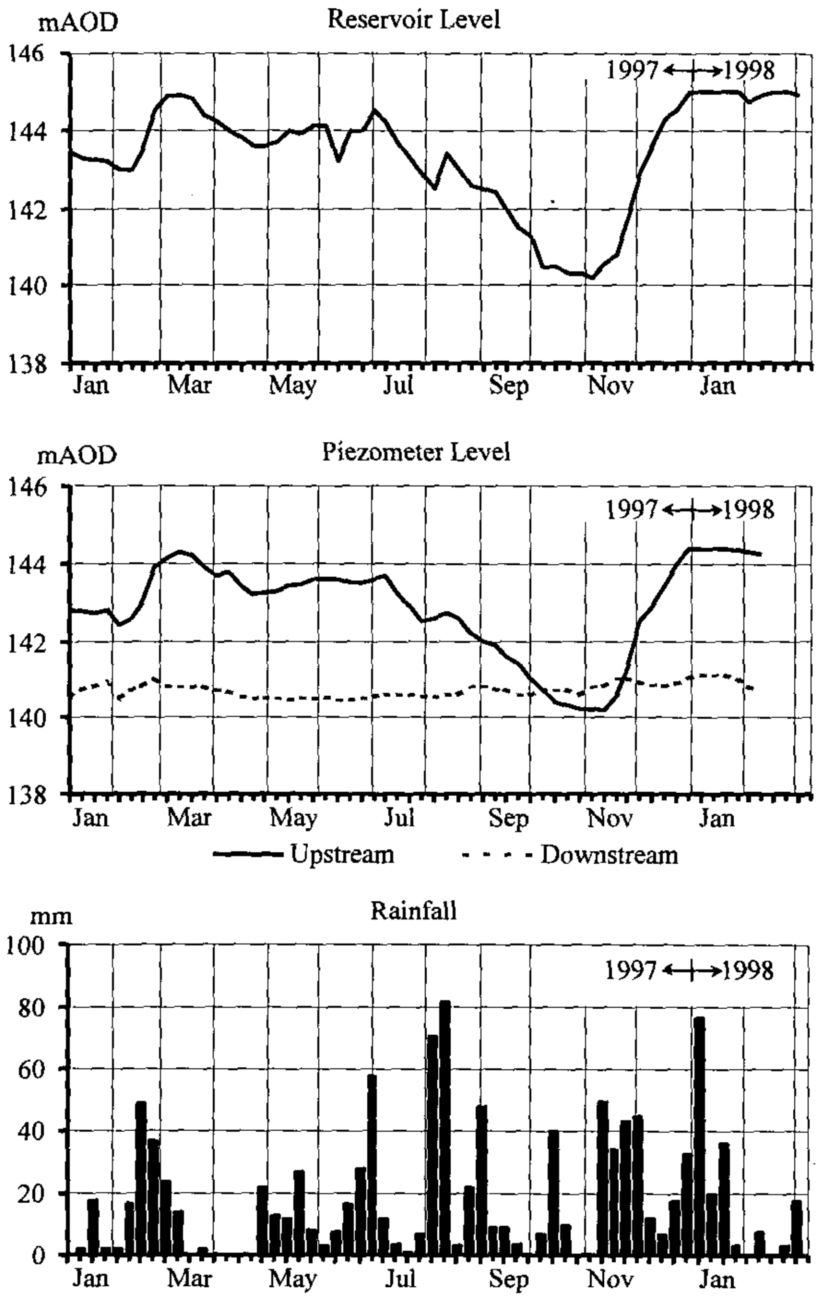
Piezometers installed during the geotechnical investigation have been used to monitor the effectiveness of the diaphragm wall. Fig 4 shows how the upstream and downstream pore pressures have reacted to changes in reservoir water level during 1997 and early 1998. Piezometers upstream of the core reflect reservoir levels while the piezometers immediately downstream of the core exhibit low pressures demonstrating the effectiveness of the core. The shallow piezometers just downstream of the toe of the embankment show water levels well below the surface and do not change greatly with variations in reservoir water level. There are peaks but these are clearly attributable to rainfall.

Movement of the dam, both position and level, is measured using studs set into the downstream edge of the crest. All the readings are within ten millimetres of their datum. The levels taken at each station are all within plus or minus 3 mm of their original reading. Following the remedial works, further stations were set into the top of the diaphragm wall and have been surveyed regularly. The data available shows that no single observation has varied by more than four millimetres and no particular pattern of direction of movement has been established. Since these figures are within the range of survey accuracy, it seems reasonable to conclude that the dam crest is not moving.

CONCLUSIONS

Luxhay dam had suffered from leakage for a number of years at high reservoir levels. Geotechnical Investigations were unable to identify the exact cause of leakage but the most likely cause was considered to be a defect near the top of the puddle core. A significant increase in leakage in 1994 dictating that remedial works be carried out without delay. A cement-bentonite slurry wall at the top of the dam was identified as the most appropriate solution. Although the wall contained less flexible material than was specified, investigation demonstrated that the wall should perform adequately.

Since the completion of the remedial works flows in the drains and piezometric levels appear to indicate the leakage has been controlled. Survey data indicates that there had been no significant movement in the



Luxhay - Typical Piezometric Values

Fig. 4

crest of the dam or in the top of the diaphragm wall. Despite the short period of high reservoir levels there have been sufficient indicators to allow some cautious optimism that the construction of the diaphragm wall has been a success.

The events in April 1994 are a clear reminder of the value of Reservoir Supervision. Visits by the Supervising Engineer are seldom instructed to take place more often than twice a year so routine regular visits by the Undertakers staff cannot be overvalued.

ACKNOWLEDGEMENTS
The authors wish to thank Wessex Water for their assistance with the preparation of the paper and their agreement to publication.

REFERENCES

BRE Digest 395, (1994) - Slurry Trench Cut-off Walls to Contain Contamination.

The use of a composite HDPE membrane/bentonite-cement slurry trench cut-off at Broadwood Loch, Cumbernauld

K M H BARR, W A Fairhurst & Partners, UK
C W BERRY, W A Fairhurst & Partners, UK
P J BARKER, Bachy Soletanche Limited, UK

SYNOPSIS. Broadwood Loch, near Cumbernauld, is a flood storage reservoir, relieving flooding in the Moss Water downstream, and providing an amenity for an adjacent business park. A volume of 350,000m^3 is impounded by an earth embankment dam 6m high. Following a contractor's alternative tender a composite HDPE membrane/single phase bentonite-cement slurry cut-off was adopted. The paper covers the original design of the cut-off, the advantages provided by the alternative and the implementation of the method. The use of composite HDPE/slurry as a cut-off in dam construction utilises technology previously used in landfill containment. The Loch was impounded in 1993.

BACKGROUND

Feasibility Study

As Cumbernauld New Town expanded, progressive urbanisation affected the catchments of several watercourses including the Moss Water, a tributary of the River Kelvin. By 1990, 2.40 km^2 or 60% of the total catchment area of the Moss Water was urbanised or zoned for development, and surface water from an additional 0.52 km^2 had been diverted into the catchment. Peak flows in the Moss Water were substantially increased by discharges from the separate surface water sewer system, and periodic flooding was occurring downstream.

Cumbernauld Development Corporation were requested by the Scottish Office to take action to alleviate the downstream flooding problems. The Development Corporation identified a low lying area of derelict low grade farmland and forestry plantation at Broadwood as a suitable site for a flood storage reservoir. The site was towards the lower end of the Moss Water catchment, and adjacent to areas identified for business, recreation and housing.

W A Fairhurst & Partners was appointed in 1990 to carry out a feasibility study into provision of a flood storage reservoir on the Moss Water. The terms of the brief also required a permanent water feature to provide an

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

attractive waterside location for an adjacent business park, and that the reservoir should be suitable for possible future water-based recreation.

In order to provide suitable waterside locations for the business park, the flood lift had to be restricted, and a permanent water level maintained. The surface area required was consequently large in order to provide sufficient live storage to attenuate flood events. The feasibility study report identified a location where an earth embankment dam 6m high could impound a permanent reservoir of 17 hectare surface area. The proposed reservoir was designed to attenuate events up to 200 years return period back to pre-development peak flows.

W A Fairhurst and Partners was appointed as the designer and Engineer for the project. A bulk earthworks contract to excavate the basin for the Loch was let in July 1991. The contract involved the relocation of 175,000 m3 of peat within the site and construction of the Loch edges.

ORIGINAL DESIGN

Embankment Dam

Detailed design of the embankment dam commenced in March 1992 following receipt of the report on the detailed ground investigation for the dam site. In order to meet the Development Corporation's programme for the business park site only three months were available for design and tender document preparation. The design and construction of the dam was carried out under the statutory provisions of the Reservoirs Act 1975.

The dam to form the reservoir was designed as an earth embankment using locally won boulder clay. It was anticipated that the permeability of the remoulded boulder clay would be less than 1×10^{-9} m/sec which was well within the permeability requirement to retain a permanent water level. The embankment has a modified homogeneous section, with an internal horizontal drainage blanket and associated filters in the downstream side. Integral to the design were the concrete overflow structure founded on piles, cut-off wall and rock grout curtain. A typical cross-section of the embankment is shown on Fig. 1. The dam is approximately 220m long, 6m high and impounds 350,000 m^3 of water.

The average inflow to the Loch from the Moss Water over 90 days in a 25 year return period drought was estimated at 7 l/sec. This is approximately equal to the average summer evaporation from the surface of the Loch. In order to prevent drawdown of the Loch below spillway crest level during dry summers it was necessary to create a highly impermeable barrier in order to meet the client's requirements. Drawdown was considered to be potentially unsightly and undesirable for marketing of the adjacent loch-side business park.

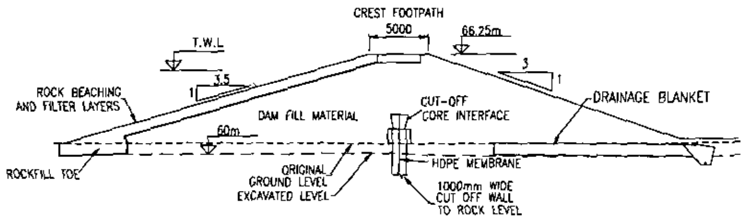


Fig. 1. Cross-section of dam

The area of the dam and reservoir was covered by ground investigations carried out for the Development Corporation in 1968 and 1972 which indicated that boulder clay deposits persisted across the valley floor at relatively shallow depth. It was therefore planned at feasibility study stage to form a rolled clay-filled trench keyed into underlying boulder clay. Detailed site investigation carried out in advance of the design of the embankment showed that the earlier investigation was misleading, and that alluvial deposits existed to a depth of 8m across the valley floor.

The soils underlying the west abutment were found to consist of about 3m of firm to stiff silty, sandy clay (glacial till) overlying rockhead. In the east abutment 3-6m of silty sands and sandy clays (alluvial deposits and possible fill) were found overlying 3m of glacial till. Rockhead was generally at a depth of 7-10m. In the valley floor up to 1.5m of peat was found overlying soft sandy clays and loose to medium dense silty sands and gravels. In some areas glacial till was found at depths ranging from 2-6m overlying rock. Rockhead was at depths of 4-8m in the valley floor. The underlying rocks belong to the Upper Limestone Group and consisted mainly of sandstones and mudstones with some limestones and coals. The rock was found to be highly weathered and jointed in some areas.

Insitu permeability tests carried out during the ground investigation gave a wide spread of permeability results between 2 x 10^{-5} and 2 x 10^{-8} m/sec. A seepage analysis of the dam foundation was carried out. The permeability of the alluvial deposits was considered to be unacceptably high to meet the requirement for no summer drawdown, and it was concluded that some form of low permeability cut-off was necessary. The required permeability of the cut-off was determined to be 1 x 10^{-8} m/sec.

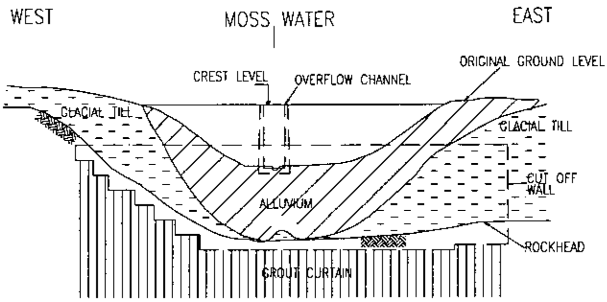


Fig. 2. Schematic cross-section of valley on centre line of embankment

Settlements of 100 to 140mm during and following construction were predicted in the alluvial foundation of the dam. Strains of up to 5% were anticipated in the silty sand layers.

Cut-off Design Options

Various methods were considered for forming the cut-off through the alluvial material:-

- steel sheet pile wall;
- jet-grouted cut-off;
- grouted cut-off;
- single phase bentonite-cement slurry trench cut-off;
- two phase concrete cut-off.

A steel sheet pile wall cut-off was considered. It could be constructed rapidly with readily available plant. However it was concluded that the required permeability could not be guaranteed because of leakage through the clutches of the piles. A jet-grouted cut-off was considered but it was concluded that the risk of openings being left in the cut-off was too high. A grouted cut-off was also considered. This would have been installed by the tube-a-manchette method, using chemical grout. The insitu permeability of the underlying alluvial soils were found to be unsuitable to accept ground readily, and there were in any case problems with toxicity and/or durability of the available chemical grouts.

Use of a single phase bentonite-cement slurry trench cut-off was seriously considered. W A. Fairhurst & Partners had some previous experience of slurry trenches, both in remedial works to dams and in isolation of landfill sites. Design of bentonite-cement slurries is a compromise between low permeability, which requires a high strength, and flexibility (or strain) which requires a low strength. The design requirements for a slurry trench cut-off were:-

- maximum permeability of 1 x 10^{-8} m/sec;
- sufficient strength to support 5m of fill;
- strength to be achieved within 28 days to support compaction plant during construction of the embankment;
- sufficient plasticity to accommodate strains caused by anticipated settlement of the foundation during and following construction.

W A Fairhurst & Partners previous experience with bentonite-cement slurries indicated that it would be difficult to guarantee the simultaneous achievement of all these criteria.

Adopted Design

Following careful consideration of all the identified options it was concluded by the Construction Engineer that the best engineer-designed option available was the two phase concrete cut-off. The design required a 1m thick unreinforced concrete wall to be constructed from 1m above original ground level to a minimum 600mm into rockhead. The wall was to be constructed in two stages. The first, below ground, stage was to be constructed by diaphragm wall techniques using bentonite slurry to support the sides of the excavation and placing concrete by tremie to displace the bentonite. The second, above ground, stage to form a spear into the dam fill was to be constructed by conventional methods. The concrete for both stages was required to be Class 40/20A. There was no permeability specified for the concrete cut-off wall. The first and second stage concrete were required to be connected by dowels. Interface grouting was required at the boundary between the diaphragm wall and rockhead.

A cement grout curtain was provided in the rock below the cut-off. Two rows of holes were to be constructed by descending stage grouting, with primaries at 12m centres and secondary and tertiary holes as required. The target permeability for the grouted cut-off was 1 x 10^{-7} m/sec.

Tenders for the contract to construct the dam were invited in June 1992. An invitation to contractors to submit alternative design proposals for elements of the contract was included in the tender documents. The cut-off wall was anticipated to be the most likely element for alternative design by the contractor.

Three tenderers for the embankment dam contract submitted alternative proposals for the cut-off wall. The two lowest tenderers both submitted the same alternative, using a composite bentonite-cement slurry and high density polyethylene (HDPE) membrane cut-off. The proposal demonstrated worthwhile savings to the Development Corporation and was given a detailed technical investigation.

CONTRACTOR'S ALTERNATIVE

Development of Method

The use of bentonite-cement slurry cut-off walls for groundwater control is a well established technique, and entails using a bentonite-cement slurry, pumped to the trench during excavation, to both support the trench sides during excavation and then to set in the trench to form the permanent cut-off wall. A more recent innovation however is the inclusion, within these walls, of an HDPE membrane resulting in a composite barrier. In general terms an HDPE membrane will be included within the cut off wall in the following situations:-

- where increased wall durability is required (in relation to aggressive leachate);
- where landfill gas is to be retained;
- where significant ground movements are expected post construction (whilst very low permeabilities ($< 1 \times 10^{-9}$ m/sec) can be obtained with bentonite-cement slurries, the resulting material is relatively brittle).

Prior to the Broadwood Loch contract, Bachy had carried out four contracts to construct composite cut-off walls:-

Castle Peak, Hong Kong. The works were carried out in 1988 and consisted of a cut off wall, 800mm thick, 1km long, average depth 12.4m, maximum depth 19m. The wall was constructed through an earth embankment dam and underlying permeable material to penetrate 200mm into rock. HDPE membrane was installed into the top 3.7m of the wall where significant strains were expected.

Irlam, Manchester. The works were carried out in 1991 and consisted of a composite cut off wall, 600mm thick, 7m deep and 180m long, as a barrier to protect domestic housing from landfill gas generated from an unlined waste repository.

M74 Motorway, Glasgow. This composite cut off wall, 1000mm thick, 21m deep and 55m long, was constructed in 1992 across the line of the M74 Motorway during its construction, as a landfill gas and leachate barrier. This was so as to not prejudice further waste infilling operations on adjacent land.

Grouw Aqueduct, Prinses Margriet Canal, Holland. This, the deepest composite wall constructed to date by Bachy, was completed in 1989 and consisted of a 800mm thick slurry wall, 32m deep with an HDPE membrane installed full depth.

In each of the above cases the HDPE membrane is 2mm thick, and panels were joined in the trench using the Geolock method, which is described in more detail below.

Bentonite-cement slurry walls (without HDPE membrane) had previously been utilised a number of times as dam cut-offs notably at Aube Reservoir in France in 1985 for both the Radonvilliers Dyke and the Brevonnes Dyke. In both situations slurry walls were utilised to extend the core of the dam from ground level, through silty clays, and into the underlying Gault clay. The wall thickness was 800mm, maximum depth 12m and total area 37,500m^2.

Proposed Alternative

The alternative, proposed by Bachy, comprised a 1m thick bentonite-cement slurry cut-off wall constructed from ground level. The wall was to include a 2mm thick HDPE membrane to full depth. The membrane was to be placed in "panels" of 2.3m width and joined using the patented "Geolock" system. A 1m length of HDPE was to be left above the top of the guide wall to be connected into the proposed upstand into the embankment to be constructed by others during embankment filling.

It was anticipated that the bentonite-cement slurry would have a permeability of 1×10^{-8} m/sec at 28 days, decreasing in time towards 1×10^{-9} m/sec. The unconfined compressive strength would be between 100 and 300 kPa at 28 days and the strain at least 5% before failure by cracking. This was in line with results from previous slurry wall contracts carried out by Bachy using similar mix design and materials.

The HDPE membrane is durable, strong and flexible, with a typical tensile strength of 34 kN/m, an elongation of 600% before failure (13% at yield) and a permeability of less than 1×10^{-13} m/sec.

It was also stated that the bentonite-cement slurry would penetrate and seal any cracks in the sandstone strata below the wall, and that the interface grouting required in the conforming scheme would not therefore be required.

The proposed alternative was perceived, by Bachy, to have the following advantages:-

- construction of the cut-off wall would be a single phase rather than a two phase process, and therefore circulation, cleaning and storage of digging bentonite would not be required;
- composite wall proposed would be more flexible and better able to accommodate ground movements that were likely to occur during and after embankment construction;
- the permeability would be significantly reduced;
- significant cost savings.

Acceptance Conditions

The alternative wall design was proposed by the contractor on the basis of offering criteria exceeding those specified in the conforming design. The impermeability of the HDPE membrane was accepted on the basis of laboratory test results on the proprietary Geolock system and records of its performance on other projects. The long term target slurry permeability of 1 x 10^{-9} m/sec, together with the advantage of the slurry/membrane plasticity to cope with any ground movement were accepted by the Engineer as providing an advantage.

A further advantage was anticipated in the ability of the slurry during construction to penetrate and seal any fissures in the sandstone strata at the rockhead/wall interface mitigating the need for additional grouting. The alternative was also seen by the Construction Engineer as providing a more effective means of establishing an interface with the clay core of the embankment.

The acceptance criteria for the performance of the wall was set in accordance with Bachy's target permeability of 1 x 10^{-8} m/sec at 28 days of the set slurry, decreasing with time to 1 x 10^{-9} m/sec. The unconfined compressive strength (UCS) was set at between 100-300 kN/m^2 at 28 days. This was considered sufficient to support both the embankment earthworks and construction plant. The membrane was considered by the Engineer to be the principal element providing an impermeable barrier. As such any cracks in the set slurry due to strain caused by foundation settlement would not be detrimental to the integrity of the wall's performance.

The contract for construction of the dam was awarded by Cumbernauld Development Corporation to Raynesway Construction Services Limited in July 1992, with Bachy Limited as specialist sub-contractor for the cut-off wall.

CONSTRUCTION

Guide Walls

Prior to the main trenching operations the contractor constructed the guide walls at the predetermined level of the cut-off installation. The purpose of the walls was to assist in the excavation of the trench and provide temporary support during placement of the membrane. The walls consisted of parallel reinforced concrete walls 1m deep and 0.3m wide.

Slurry Mix Design

Bentonite cement slurry mixes are designed to meet the following criteria:-

- sufficiently fluid to enable mixing, pumping to the trench and excavation through the slurry;
- homogeneous and stable - i.e. there must no segregation and "bleed" must be minimised;
- set slurry properties - permeability, strength and strain.

It is inevitable, when constructing a cut off wall by this method, that some of the ground being excavated will be mixed in with the slurry, with finer particles being held in suspension. This is not normally detrimental to the mix. The mix usually incorporates water, bentonite and cementitious material. The cementitious material is normally a blend of OPC and GGBS, although PFA can also be used as part cement replacement. At Broadwood a blend of OPC and GGBS was used. Additives to assist with the mixing and to retard the set were also included.

Slurry Mixing

The bentonite-cement slurry was mixed in a batching plant set up on site. An idealised plant layout is shown in Fig. 3. Bentonite, cement (to BS 12) and ground granulated blastfurnace slag (to BS 6699) were delivered to site in pressurised tankers in 24 tonne loads. They were stored on site in dry powder silos.

The mixing took place in 3 stages. In the first stage a master mud, comprising approximately 90% of the total mix water and the bentonite powder, was mixed thoroughly (UFB mixer) and stored overnight in tanks to allow hydration. In the second stage the cementitious material (OPC & GGBS) was mixed (FCP mixer) with the remaining 10% of the water. The third stage involved transferring both the hydrated master mud and the freshly mixed cementitious slurry to a large 10 m^3 capacity mixer (BRC) where the final slurry was produced.

During slurry production, regular checks, normally three times per day, were made on the density of both the master mud and the final slurry (by mud balance) to confirm the correctness of the mixing process. These checks were in addition to weekly calibration of the mixer weigh scales. Checks were also carried out on the viscosity (by Marsh Cone) to confirm suitability for both pumping to the trench and the excavation process. The stability of the mix was checked by placing samples in a 1 litre covered measuring cylinder and measuring any bleed water after a period of 24 hours, with a maximum value of 4%.

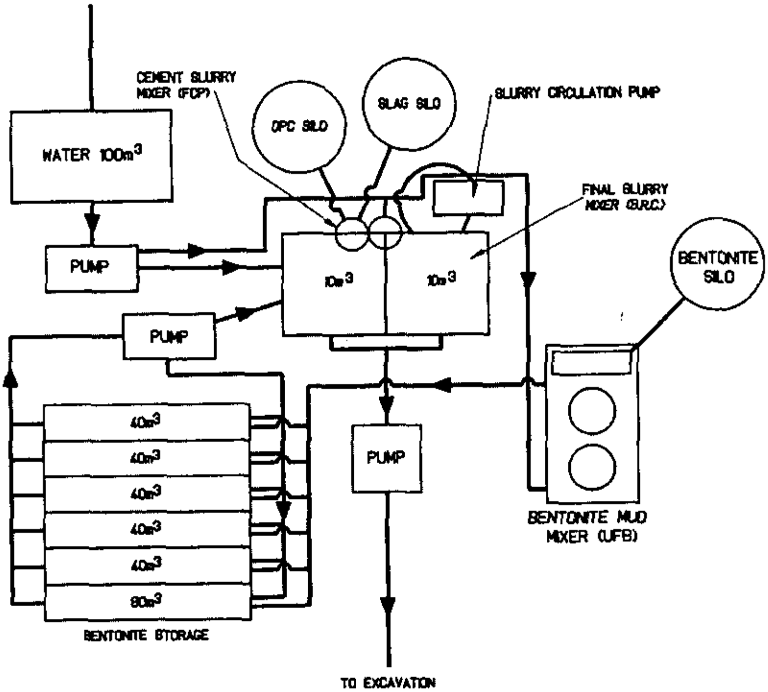


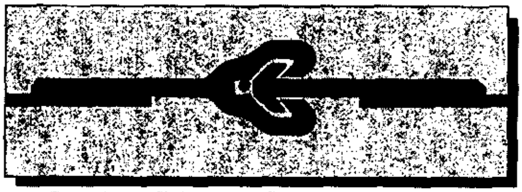
Fig. 3. Idealised slurry batching plant

Wall Construction

Excavation of the cut-off wall was carried out by rope suspended grab. As soon as excavation was commenced, the bentonite-cement slurry was pumped (via 100mm diameter steel pipes) to the trench. Excavation continued under the slurry until the top of the rock was identified in the trench arisings. The depth of the trench was then measured using a weighted tape and excavation continued to achieve the required minimum 600mm penetration into the rock. The depth to the rockhead and final depth of the cut-off wall were recorded for each membrane panel along the whole length of the cut-off wall.

Membrane Placing

When the required depth had been reached, and before the slurry had set, the HDPE membrane was installed. The HDPE membrane was supplied to site in 2.3m wide panels, cut to length in accordance with a schedule of anticipated cut-off wall depths. The Geolock joints (Fig 4) had previously been manufactured and factory fitted by welding them to the HDPE panels. This was carried out by Geotechnics Holland BV who hold the patent for the Geolock joint.



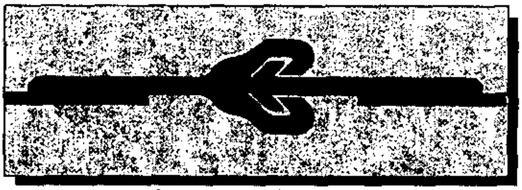


Fig. 4. Geolock joint

Each membrane panel was fitted at its base with small sacrificial plates which hook into locating points at the base of the placing frame. The top of the panel was then attached to tensioning devices and the panel tensioned onto the frame. The panel and frame together were then lifted by crane and placed into the fluid slurry to the base of the trench. A further panel and frame together were then lowered into the trench with the joint sections interlocking, the Hydrotite section being located in the female part of the joint as a slack fit. Once the second panel had been installed the first panel was released from the frame (by releasing the tensioning devices) and the frame withdrawn. The locking and sealing of the joint was then completed by the Hydrotite absorbing water from the slurry mix and swelling to potentially ten times its dry volume. The jointing system has been tested in the laboratory in both the USA and UK and found to be effective up to pressures across the joint exceeding 5 bar, which is far in excess of the likely field conditions.

Construction continued with excavation of the trench closely followed by installation of the membrane. Temporary stopends were installed, attached to the Geolock joint, at the end of each working day in order to protect the membrane from damage during excavation for placing of the adjacent panel.

Interface with Structures

The effectiveness of the interface of the top of the wall with the embankment structures was a critical element to the integrity of the reservoir. There were two structures at this interface, namely the connection with the clay fill of the dam and the connection with the concrete spillway channel through the centre of the embankment.

To establish an impermeable barrier between the concrete spillway and membrane of the cut-off, the top of the HDPE membrane was cast within the concrete of the channel to effect a watertight seal. The key was achieved by adapting the Geolock joint system which was welded to the membrane before casting within the concrete. The solution adopted at the clay/wall interface is considered to provide flexibility to accommodate settlement of the embankment foundations during subsequent construction works, which would not have been achieved with a concrete wall.

The interface with the clay core of the embankment proved to be more problematic and was eventually achieved in two phases of the cut-off wall construction. The initial phase comprised construction of the wall and installation of the HDPE membrane by the contractor. An additional section of membrane, initially 1m in length was left protruding above the construction level of the slurry wall. On completion of the grout curtain and earthworks operations by the main contractor, Bachy returned to site to complete the second phase of wall construction. The phasing of construction adopted allowed the cut-off wall operations to proceed during the poor winter weather while the earthworks were suspended.

The clay core had been extended during the subsequent earthworks operations up to the top of the protruding membrane which was temporarily supported and protected using a system of shuttering. Once a trench had been excavated down either side of the membrane using conventional methods and temporary supports removed, slurry was pumped into the trench forming a key into the clay core. The final earthworks operations proceeded after the slurry had gained sufficient strength to support the plant. The resultant two stage operations and logistical problem of access around the exposed membrane during earthworks and grouting operations had a consequential impact on progress and cost of the contract.

Construction Problems

Failure of Guide Walls. Progress on site was hindered by the failure of the guide walls during excavation of the trench for membrane installation. Two sections of the guide walls and adjacent ground approximately 20-25m in length slipped inwards towards the excavation despite the presence of the slurry. Failure, it was noted, occurred after a prolonged period of wet weather and it was concluded that the higher than expected ground water pressure initiated the failure. It is acknowledged to be normal practice to ensure the top of the slurry in the excavation is at least 1m above the surrounding ground water table. As a consequence of logistical problems during the initial earthwork operations to form a working platform the cut-off wall was constructed at a level lower than expected, approximately 0.5-1m below surrounding ground level in the area of the failed guide walls.

After further investigation the extent of the failure was identified and remedial measures taken to remove the failure plane. The failure plane extended from the level of the guide walls to a depth of 2-3m. Care was taken during construction of the remaining wall to limit the surcharge loading from heavy plant near the excavation and also additional temporary support was provided to the guide walls.

Membrane extension. For ease of construction it is normal practice to leave the HDPE membrane protruding approximately 300mm above the working platform level. Once the slurry has set, the cracked slurry at the top of the wall is then trimmed back by hand, and the membrane trimmed back to the required level and incorporated in a clay capping, which would typically be 0.5 to 1.0m deep. In this case it was required that the membrane be left 1m above the platform level, and during the works this was extended to 2m above platform level in some areas. This caused problems in connecting adjacent panels, especially at "day joints" where it was not possible to leave the previously placed membrane connected to the placing frame, and at "closure panels".

Slurry extension. Upon completion of the construction from original ground level, the slurry batching plant was demobilised and removed from the site. Once the dam core had been filled around the membrane extension it was required that bentonite-cement slurry "backfill" be placed around the membrane. It was therefore necessary, on two occasions, to remobilize slurry mixing plant to the site to complete the backfill operation. Although much smaller plants were used than for the initial installation the cost was significant and placing of the slurry around the suspended membrane was difficult and time consuming.

Sloping sections. When working on sloping sections in the abutments of the dam only a limited number of panels can be installed consecutively before stopping to allow the slurry to take up initial set. It is therefore normal to work on a number of fronts so that the sloping sections can be revisited at intervals of between two and three days. Because of the logistics of the site this was difficult to achieve, and various methods were tried to accelerate this process, the most successful being to use temporary dams, made from a reservoir of set slurry. These were constructed in a series of steps up the abutments to retain the fresh slurry. Progress was consequently slower in these areas.

QUALITY CONTROL

Construction

During construction comprehensive records were maintained to demonstrate that the required depth of cut-off had been constructed, and that each membrane panel had been successfully installed.

Slurry sampling

In addition to the controls described above samples of bentonite-cement slurry were obtained from the trench after completion of excavation and before installation of the membrane, at a rate of a sample set for each day's production. A set comprised 1 sample each from the top, middle and bottom of the wall. Additionally one sample each day was taken from the discharge pipe to the trench. Once the samples had been obtained (with a remote sampler operated from the surface) the fluid slurry was placed in 100mm diameter plastic tubes, 450mm long.

Slurry Testing

The samples were sealed on site and allowed to set for at least 2 weeks before transfer to a specialist laboratory where they were stored in curing tanks. Once the samples had reached the specified age they were extruded from the sample tubes and tested as follows:-

Permeability	-	BS 1377 Part 6, Method 6;
UCS	-	BS 1377 Part 7, Method 7;
Strain	-	BS 1377 Part 8, Method 8.

The test regime implemented during construction of the wall recovered over 150 samples from the wall slurry. The initial text results, at 28 days revealed a number of samples with a permeability greater than the target of 1×10^{-8} m/sec. The UCS results all proved to be satisfactory, within the acceptable range of 100-300 kPa at 28 days.

The Engineer implemented a programme of further testing to monitor the anticipated reduction in permeability over an extended period. Further test results were obtained at 100 days, 300 days and finally 450 days. Permeability, of the set slurry did decrease during this period, however some samples failed to meet the target, the highest result recorded was 1.5 x 10^{-7} m/sec at 450 days on an isolated sample from within the wall. The majority of results achieved the target values with the average permeability being noted as approximately 1 x 10^{-9} m/sec. There was no recognisable pattern to the high permeability results within the line of the wall with each poor result being recorded in isolation.

Review of Performance

The analysis of the test results was based on target figures of 1 x 10^{-8} m/sec and 1 x 10^{-9} m/sec set by the contractor. Following completion of the dam the results were reviewed and it was concluded that the target permeability for the slurry was more onerous than required when compared to the designers original target permeability of the clay core of 1 x 10^{-8} m/sec and the grout curtain of approximately 1 x 10^{-7} m/sec. It was accepted that in such a wall, the prime function of the bentonite-cement slurry is to enable placement of the HDPE membrane. It is the very low permeability of the HDPE membrane (and jointing system) that provides the wall integrity. The wall, as constructed, was considered to be at least as effective as the conforming design.

Following completion of the dam in October 1993 the loch was impounded and the water level monitored to evaluate the performance of the cut-off. The level has remained at or above overflow sill since impounding, except when lowered for maintenance purposes. There has been no indication of summer drawdown, and the cut-off is considered to have fulfilled its design criteria. The loch is currently being monitored with a view to issue of the Final Certificate under the Reservoirs Act.

CONCLUSIONS

Use of a composite bentonite-cement slurry cut-off wall with HDPE membrane in dam engineering gives rise to the following advantages:-

- achievement of low permeability (<1 x 10^{-9} m/sec) while providing sufficient flexibility to accommodate significant ground movements;
- provision of increased reliability where loss of water is a significant issue.

The method carries additional problems which have to be overcome in implementation. Problems experienced at Broadwood Loch include:-

- the slurry has to be prevented from setting until the membrane has been placed and the trench is therefore open for a longer period of time;
- there are practical difficulties associated with working around the membrane and incorporating it into the completed structure;
- the retarded set of the slurry makes it more difficult to deal with sloping ground.

These problems were overcome at Broadwood Loch and the method was successfully implemented. The method is considered to have worthwhile application in dam engineering where it is necessary to create a very reliable low permeability cut-off, or where flexibility to accommodate ground movements must be provided.

ACKNOWLEDGEMENTS
The authors gratefully acknowledge the contribution made to the work described in this paper by Mr A D H Campbell, Construction Engineer for Broadwood Loch from 1992 until his retirement in 1995, and Mr A G Simpson, Engineer for the Contract for construction of the dam.

REFERENCES

Draft specification & notes for guidance for slurry trench cut-off walls. ICE/CIRIA/BRE, April 1996.
Slurry trench cut-off walls to contain contamination. BRE Digest 395, July 1994.
Remedial treatment for contaminated land Volume VI: Containment and hydraulic measures. CIRIA SP106, 1996.
Barrier liners and cover systems for containment and control of land contamination. CIRIA SP124, 1996.

Winscar Dam: Investigations and repairs to asphaltic concrete membrane

A C WILSON, Babtie Group, UK
A C ROBERTSHAW, Yorkshire Water Services Ltd, UK

SYNOPSIS. Winscar Dam, completed in 1975, is a 53 m high rockfill embankment with an asphaltic concrete membrane on the upstream face. A leak through the membrane was repaired in 1980 and a few other defects were noted at that time. Subsequent inspections revealed minor cracks and blisters in the membrane, often associated with debonding of the two upper layers of the membrane. The paper describes the survey, extent and nature of the defects, the repair method and discusses possible causes and likely consequences for the future maintenance of the dam.

INTRODUCTION

Winscar Dam, owned and operated by Yorkshire Services Ltd, is situated close to the village of Dunford Bridge in the Pennines, 24 kms north west of Sheffield and its construction has been described by Collins & Humphreys (1974).

The dam was constructed in 1972-1975 and is formed of sandstone rockfill obtained from a quarry within the reservoir basin. It is 53 m high and 520 m long. Top water level is at an elevation of 343.8 m OD.

The upstream face of the dam is surfaced with an asphaltic concrete membrane connected to a concrete cut-off wall set into bedrock at the toe of the slope. A grout curtain, 70 m deep, beneath the toe wall forms a seal in the foundation rock and into the abutments.

CONSTRUCTION OF MEMBRANE

The dam embankment was constructed (Pattenden & Steffen) of rockfill placed in layers of depth 1.7 m and compacted by six passes of a 13.5 Mg roller. The upstream slope consists of fine granular material shaped and compacted to the required profile with a tolerance varying from +0 to -160mm. A bitumen emulsion spray was then applied and rolled to stabilise the surface.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

Asphalting of the upstream face of the embankment was undertaken in 1974. The sequence of construction was:-

1. A regulating/binder layer placed at a minimum rate of 150 kg/m^2 (It is thought that this would equate to a minimum layer thickness of approximately 60 mm).
2. First layer of dense asphaltic concrete (DAC), with a nominal thickness of 40 mm.
3. Second layer of DAC, with a nominal thickness of 80 mm.
4. Sealing coat of mastic asphalt at a rate of 3.5 kg/m^2, laid in two layers.

The asphaltic concrete layers were laid hot and then compacted by roller. This procedure normally results in each layer adhering strongly to the layer below.

As is normal with an asphaltic concrete membrane, a sealing coat was provided to its surface to protect against evaporation of volatile elements under ultra-violet light. An unusual feature in this instance is a white cement slurry added in a wide band across the upper part of the upstream face of the dam to make the appearance more acceptable to the planning authorities. It also assists in extending the life of the sealing coat by reducing its exposure to sunlight.

Construction of the final coat of DAC progressed from the left towards the right abutment and was completed in early winter 1974.

Contemporary photographs of the construction show that the crest wall was not completed until Spring 1975. This would suggest that the joint between the membrane and the crest wall may not have been properly sealed during the winter of 1974-75.

Comparison with other UK dams

Many dams with bituminous linings had been constructed in Europe and elsewhere prior to construction of Winscar Dam, but only one, Dungonnell Dam, (Poskitt 1972) had been constructed in the UK. Details of the asphaltic membranes on UK dams are shown in Fig 1. The principal elements of the asphaltic membrane on Dungonnell Dam consist of two 50 mm thick layers of DAC above a 125 mm thick asphaltic concrete drainage layer constructed upon a 75 mm thick asphaltic concrete underseal layer i.e. a drained sandwich construction. The membrane at Winscar followed a similar two layer construction for the waterproof seal layer, but dispensed with the "sandwich" drainage feature. It is interesting to note that all four of the UK dams with asphaltic membranes that were constructed after Winscar have only a single course of DAC as their impermeable sealing layer. This type of construction is now frequently followed worldwide on dams with bituminous upstream facings. The slope of the upstream face at Winscar is IV:1.7H, which is common for a rockfill dam. There is no evidence to suggest that this slope, which is the same as at Dungonnell and Sulby dams, was a significant factor causing the defects at Winscar.

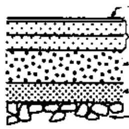
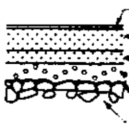
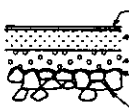
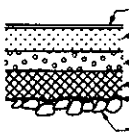
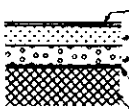
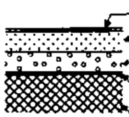
Dam Constructed Reference	Asphaltic Membrane
Dungonnell 1968–70 Poskitt (1972) Slope IV : 1.7 H	 Double seal coat 2 x 50mm dense asphaltic concrete 125mm porous bituminous macadam layer 75mm asphaltic concrete underseal Surface dressing 150mm stone
Winscar 1972–75 Routh (1988) Slope IV : 1.7 H	 Double mastic seal coat 80mm dense asphaltic concrete 40mm dense asphaltic concrete 150kg/m² (60mm?) asphaltic concrete binder Bitumen emulsion spray Fine granular regulating layer
Marchlyn 1976–79 Baines et al (1983) Slope IV : 2 H	 Seal coat 80mm dense asphaltic concrete 60mm min asphaltic concrete binder Cationic bitumen emulsion spray Crushed rockfill 100mm max size
Sulby 1979–82 Eccles & Kaye (1983) Slope IV : 1.7 H	 Hot mastic seal coat 80mm dense asphaltic concrete 60mm asphaltic concrete binder 100mm bituminous drainage layer Cationic bitumen emulsion spray Fine rockfill
Colliford 1981–84 Johnston & Evans (1985) Slope IV : 2 H	 Two coat cold emulsion 80mm dense asphaltic concrete 70mm asphaltic binder Bitumen emulsion spray 350mm granular drainage material
Roadford 1987–89 Johnston et al (1995) Slope IV : 2.25 H	 Hot mastic seal coat 80mm dense asphaltic concrete 70mm asphaltic concrete binder Bitumen emulsion spray Granular drainage material

Fig. 1 Asphaltic membranes on U K dams

HISTORICAL

1975-1985

Seepage flows during impounding

The reservoir level did not rise quickly during the first two years following impounding due to a combination of dry weather and abstraction. During this period Routh (1988) has stated that the underdrainage flows were only marginally higher than expected. However wet weather in January and February 1977 caused the reservoir level to rise and this was accompanied by a rapid increase in underdrainage flow and the emergence of springs in the left abutment downstream of the dam. To stem this flow, installation of Stage I of supplementary grouting in the left abutment was undertaken. The reservoir level was then allowed to rise and marked increases in underdrainage flows were again found to occur at two higher levels. Supplementary grouting Stages II and III were installed in 1979 and 1980 which resulted in some further reduction in underdrainage flows. During this period there was a suspicion that there might be a leak through the asphaltic membrane and this was reinforced by the knowledge that trace element analyses indicated that a significant proportion of the drainage water was passing through or in contact with asphaltic material.

Damage to asphaltic facing

An analysis of the rainfall, underdrainage flows and reservoir level data suggested that, if the seepage on the north side of the culvert was from a single fracture, "it would have a gross area roughly equivalent to a matchbox and would be located at about elevation 308 m AOD" (Routh 1988). It was also considered that the most likely area for such a fracture would be in the area of the connections between the upstream membrane, toe wall and culvert. The reservoir was almost completely emptied to undertake Stage III grouting and during this period cracks were found in the asphaltic concrete membrane vertically above the position where the culvert passes through the toe wall. An area of about 12 m^2 of the asphaltic facing was removed from the location of the fractures and an effective repair undertaken.

Other defects in the asphaltic facing

While the reservoir was drawn down and the fracture in the asphaltic membrane was being repaired, the entire facing was inspected and a number of other defects were found. These included:-

i) small blisters in the mastic coat
ii) a group of cracks over an area of 1-2 m^2 above the toe wall some 200 m from the right abutment and at a higher level two cracks each about 300-450 mm in length.

It was found when repairing these defects that only the upper layer of asphaltic concrete was affected and there was practically no adhesion between the layers.

1985-1995

During a statutory inspection in 1985 the Inspecting Engineer reported the existence of a blister in the asphaltic membrane. The dam is regularly visited by the Supervising Engineer appointed under the Reservoirs Act 1975 and a number of his reports have noted the existence of isolated blisters and other defects in the asphaltic concrete membrane. None of these defects was considered to affect reservoir safety and therefore no action was taken to repair them.

1995-1997

Statutory inspection

A further statutory inspection of the dam, under the Reservoirs Act 1975, was made in 1995. The Inspecting Engineer Mr T A Johnston noted that the bituminous seal in the joint at the junction between the upstream membrane and the crest parapet wall was in a poor condition and in some places was missing. He recommended that the seal be renewed and this was carried out in mid 1996.

Whilst the joint seal replacement was being undertaken by specialist contractor Hesselberg Hydro Ltd the reservoir was at a low level and a number of defects were noticed in the asphaltic concrete membrane on the upstream face of the dam. Closer examination of the membrane found the defects consisted of cracks and blisters in the asphaltic concrete and cracking and peeling of the sealing coat. As a consequence of these findings the Inspecting Engineer recommended that:-

i) the extent of the deterioration of the membrane should be established by detailed visual examination and by site and laboratory testing.
ii) the findings of the investigations should be assessed to determine the nature and extent of repairs that may be necessary.
iii) a programme of remedial works should be implemented.

The dam owner accepted these recommendations and instructed Hesselberg Hydro Ltd to undertake the work under the supervision of the Babtie Group.

Survey of defects

The contractor's survey was limited to areas of the dam face above water level in the reservoir and it found:-

i) twenty cracks within the upper layer of DAC (nominal thickness 80mm), varying in length up to about 1 m.

ii) eight blisters, some of which also had cracks.
iii) de-bonding between layers of the membrane at a number of the cracks and blisters. Two of the larger blisters noted at this time were later found to have moved down the slope, possibly under the effect of internal hydraulic pressure.
iv) five defects in the mastic sealing coat.

A number of the smaller cracks were subjected to vacuum tests and the larger cracks to water tests of up to 1 metre head. All of these tests indicated that there was no loss of vacuum or water through the defect.

INVESTIGATION AND REPAIR OF DEFECTS

Repair of defects in the upstream asphaltic concrete membrane commenced in August 1996. As the work proceeded further cracks and defects in the asphaltic membrane were noted and the repair contract was extended to include the repair of these also.

In total, 47 defects were investigated and repaired. Although defects were found at locations spread over the whole length and breadth of the upstream face of the dam above reservoir water level at the time, it was found that some of the defects occurred in clusters of eight to eleven. Four of the clusters were located within 200 m from the south abutment and another was found close to the spillway channel at the north abutment.

The first stage of the repair work involved breaking out the asphaltic concrete around the defects using jack hammers. During this operation it was noted that at 43 of the 47 locations there was no adherence or bond between the two layers of DAC. In addition water was found to be present between the layers in 22 cases.

During periods of rainfall water gathered in the repair holes and in a number of instances it drained away and it was noted issuing into holes further down the face. This indicated that some of the excavated holes were "connected" below the surface and confirmed that debonding could be present even where the surface was sound. To collect more information on the extent of debonding nine rotary cored holes were drilled through the membrane in three areas where there were clusters of defects. Electrical continuity testing using a method proposed by the contractor was also undertaken.

The electrical testing involved filling all holes in the area with water and leaving it to permeate between the layers of debonded DAC. The test circuit as shown in Fig 2 was set up. Copper plates 1 and 2 connected to the terminals of a battery are inserted into two adjacent holes. A third plate connected via a voltmeter is installed in another hole. If a voltage is recorded, this indicates hole 3 is connected to hole 1. This test enabled the extent of debonding to be established in a number of areas.

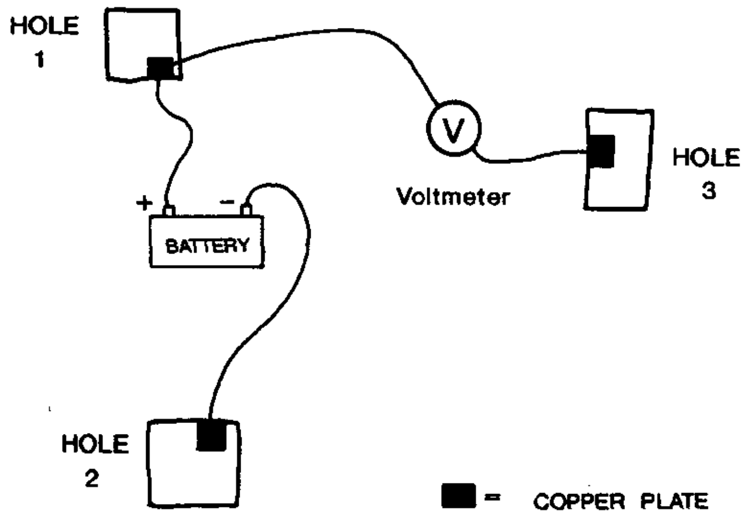


Figure 2 Electrical test circuit

The records indicate that the DAC laid in 1975 was composed of an 80 mm upper course and a 40 mm lower course making an overall thickness of 120 mm. The 1996 remedial works discovered that out of 73 areas that were cut out the thickness of the upper course varied between 90 mm and 215 mm with an average of 117 mm and where the full depth of DAC was measured i.e. in cores, it was found to range from 120 mm to 180 mm. In one exceptional instance a thickness of 270 mm DAC was found. All the evidence therefore indicates that the thickness of DAC in the construction was always in excess of the nominal 120 mm and in many cases by a substantial margin.

In the light of these findings there was some doubt regarding precisely which layers of the membrane had debonded. To examine this and investigate how widespread debonding of the layers within the membrane was, further cored boreholes were drilled in areas of the membrane which did not contain any obvious defects. All the indications from the cores are that debonding occurs between the two courses of DAC. Out of 25 cores located randomly across the upstream face 18 were found to contain a debonded interface. It appears therefore that debonding affects over half of the surface area of the membrane.

At the Contractor's suggestion, primer was introduced into some of the upper holes and allowed to flow between the debonded courses so that it appeared on occasions at lower holes. The hope is that the primer will act as an adhesive between the layers when subjected to water pressure as the reservoir refilled. The effectiveness of this repair procedure has not yet been proved.

MATERIAL TESTING

Samples of material from the top and lower layers of DAC and from the binder layer were tested in the laboratory. These tests indicated that

- the bitumen content in the DAC samples ranged from 6.8% to 7.6% of the total mix and in the binder layer it was 5%. These bitumen contents are within the range commonly used in the asphaltic membranes in other UK dams. The bitumen appeared fresh and in good condition.
- the filler contents present in samples of DAC were 12.9% and 20.6% respectively and in the single sample of binder material it was found to be 6.5%. These filler contents are substantially higher than would normally be expected and could possibly have resulted in a less stable mix but no signs of instability were noted.
- the void content and permeability of two samples of asphalt from the binder layer was measured in the laboratory with the following results:
  Void content 6.5% and 5.2%
  Permeability 2.0×10^{-5} and 2.7×10^{-5} m/s
  these results are considered satisfactory.
- a sample of the DAC used to repair defects in the membrane was tested and it was found to have a void content of 2% and there was no loss of water after applying 2 bar pressure for 65 hours followed by 3 bar for 24 hours. These results are satisfactory.
- no reasons associated with the mix designs were apparent to account for the lack of bond between the layers of DAC.

OTHER FINDINGS

Defects were found in all areas of the asphaltic concrete membrane but a greater concentration was found at the southern end. This concentration of defects is in the same general vicinity as defects noted and repaired in 1980-81 and suggests that the underlying cause of the defects has been present for a considerable time and, possibly, since construction of the membrane.

Where the top layer of DAC was removed to undertake a repair to a defect, no evidence was found at any location of a defect in the lower layer of DAC. The records indicate that the DAC consisted of an 80 mm top layer and a 40 mm lower layer. However, during the 1996/97 work, the thicknesses of the layers were found to vary significantly, but none was thinner than expected.

The Contractor undertook calculations to check the ability of the membrane to withstand the forces imposed by wave action. A range of scenarios was examined varying from 120 mm thickness of DAC to a 40 mm thickness of DAC (i.e. assumes that all the 80 mm top layer of DAC has slipped off). These calculations demonstrated that with a membrane thickness of 120 mm the foundation would be well able to withstand future impacts from waves in excess of the design $H_s = 1.0$ m that is considered to apply. However in the worst case of only 40 mm of DAC being present some damage may occur in the long term.

It is considered that because no defects have been found in the lower course of DAC, it will continue to maintain a satisfactory water barrier provided the upper layer of DAC remains in place and any surface defects are repaired timeously. The lower layer may be considered to be equivalent to a thinner version of the single impermeable layer installed on other UK dams e.g. Marchlyn, Colliford and Roadford and will therefore provide a reasonable degree of security against leakage through the upstream face of the dam. There is therefore no requirement at present to replace the entire upstream facing membrane on the dam.

CAUSE OF DEFECTS

It is not possible to provide a single explanation of the defects at Winscar. A number of possibilities have been postulated and they all involve some process of sequential deterioration e.g. debonding could lead to cracks, which admit water causing blisters which rupture, admit water, and result in further debonding.

Mechanisms which may, either individually or in combination, have contributed to the development of the defects are listed below:-

- Lack of adhesion between upper and lower layers of DAC may have been caused by:
  - insufficient coverage of bituminous emulsion tack coat, if used.
  - deterioration of tack coat effectiveness due to delay in placing second layer of DAC.
  - the upper layer of DAC being too cold when laid.
  - surface of lower layer not completely dry or not properly pre-heated.
- Where the layers of DAC are not bonded, the smooth interface and the possible presence of moisture would result in low friction between the two layers. The top layer would have a tendency to slide down the slope of the dam face and may give rise to tension cracks.
- Once moisture is present in the interface, it is possible that debonding could increase progressively under the hydraulic pressure of the trapped water. The possibility of vapour pressure developing between the layers

during warm or sunny weather may also cause blistering which could result in tension cracking.

Many suggestions have been made as to how moisture could enter the interface between the layers of DAC, these include:-

- rainfall/mist/dew on the surface of the lower layer when placing the upper layer i.e. moisture may have been present since construction.
- rain entering the interface between layers during winter 1974/75 prior to construction of the wave wall and installation of the bituminous seal or through faults which have subsequently developed in the seal against the wave wall.
- rain or reservoir water entering through cracks in the upper layer of the membrane.

CONCLUSIONS

1. All cracks and blisters in the mastic sealing coat or the top layer of the asphaltic membrane which were identified in 1996 have been cut out replaced with fresh material.
2. Debonding between the two layers of DAC is present over substantial areas of the membrane.
3. It has been shown that the thickness of both the upper and lower layers of DAC varies significantly in different locations but in all cases the layer thickness was greater than the specified minimum.
4. The mix proportions of the DAC and binder materials and their physical properties are generally in accordance with normal practice. The materials are in a satisfactory condition and no evidence of ageing was noted.
5. It is known that moisture is present between the layers of DAC in many areas and defects may continue to develop in the surface of the membrane. Regular survelliance of the membrane surface including a detailed examination before 2003 as required by the Inspecting Engineer will be required to locate any further defects and it will be necessary to undertake their prompt repair to prevent deterioration of the upstream face seal.
6. Provided the present thickness of DAC remains in place on the upstream face of the dam the membrane will be able to resist wave impact for many years.
7. If the upper layer of DAC is discounted as an impermeable layer, it is considered that the lower layer will provide a reasonable degree of security against leakage through the dam.

ACKNOWLEDGEMENTS

This paper is published with the permission of Yorkshire Water Services Ltd and the authors acknowledge the assistance provided by Hesselberg Hydro Ltd and the support given by their colleagues.

REFERENCES

Baines J A et al (1983). Dinorwig pumped storage scheme. Proc. ICE vol. 74, pp 635-718

Collins P G M & Humphreys J D (1974). Winscar Reservoir. Journal IWES vol. 28, pp 17-34

Eccles P G & Kaye K A (1984) Sulby Reservoir, Isle of Man: Design and construction. Journal IWES, vol. 38, pp 197-216.

Johnston T A & Evan J D (1985) Colliford Dam sand waste embankment and asphaltic concrete membrane. Proc. ICE vol. 78 pp 689-709.

Johnston T A, Wilson A C & Evans J D (1995) Roadford Reservoir: design, construction and performance. Proc. ICE, Wat. Marit. & Energy, vol. 112, pp 316-325.

Pattenden B & Steffen H The construction of Winscar Dam with particular reference to compaction. Source unknown.

Poskitt F F (1972) The asphaltic lining of Dungonnell Dam Proc ICE vol. 51, pp567-579.

Routh C D (1988). The investigation, identification and repair of the asphaltic concrete facing of Winscar Dam. Trans. 16th Conf. International Commission of Large Dams. San Francisco, pp 655-677

Raising Llysyfran and Brianne dams

R A N HUGHES, Binnie Black & Veatch, UK

SYNOPSIS. In 1990 Binnie & Partners (now Binnie Black & Veatch) (BBV) undertook a feasibility study for Welsh Water into the potential raising of Llysyfran and Llyn Brianne dams. The study concluded that a 1.5m and 2m raising of the respective dams was feasible without affecting the operation of the reservoirs. Llysyfran dam was raised in 1993 to improve water resources in south west Wales; Brianne dam was raised in 1996 to supplement a hydro-electric scheme installed by Hyder Industrial. The paper concentrates on the construction phase of both raisings.

INTRODUCTION

Llysyfran and Llyn Brianne reservoirs are the two principal impounding reservoirs in South West Wales. Llysyfran reservoir lies on the river Syfynwy, a tributary of the River Eastern Cleddau in the centre of Pembrokeshire. Llyn Brianne reservoir lies some 80km to the east on the River Towy. Both reservoirs were originally constructed for river regulation to support abstraction for water supply further downstream.

The Llysyfran scheme supplies water for the Milford Haven/Haverfordwest area while the River Towy scheme supplies the Swansea area. Llysyfran dam, a mass concrete gravity dam, was designed for construction in two phases. Phase 1 was completed in 1972, the reservoir capacity amounting to 9110Ml. Phase 2 enlargement of the dam would increase the storage capacity of the reservoir to 21 250Ml. Brianne dam was also designed to allow future raising. The Stage I reservoir was completed in 1972 and is impounded by a rockfill dam with a clay core. A maximum raising of the dam would increase the reservoir from the original capacity of 62 000Ml to 91 000Ml.

Binnies were appointed in 1990 by Dwr Cymru-Welsh Water to undertake a study of how Llysyfran dam and Brianne dam could be raised by modest amounts to increase water resources within the South Western Division.

The report concluded that a 1.5m raising was feasible for Llysyfran dam without affecting the operation of the reservoir. Raising Brianne dam was also feasible, a raising of up to 2m would be possible. Following the report it was decided that Llysyfran dam would be raised but no work would be carried out on Brianne Dam at that time.

Towards the end of 1995, BBV were appointed by Hyder Industrial, the industrial arm of the Hyder group to undertake design and construction supervision of a 1m raising of Brianne dam. The raising was intended to provide storage for power generation.

LLYSYFRAN DAM

Crest raising

Raising the TWL of the reservoir by 1.69m in 1993 was achieved by raising the spillway crest using suitably anchored precast concrete blocks. The parapet wall was strengthened to resist the higher flood surcharge loads and forces created by wave action.

The original spillway, located at the middle of the dam, had a 76.2m long uncontrolled ogee weir preceded by a portion of the upstream face of the dam which sloped at 1 on 1 over a height of 1.35m. The blocks feature a curved ogee profile along the upper surface and when installed, terminated approximately 800mm above the existing crest.

Precast block production

The blocks were cast on site where a car park adjacent to the eastern abutment of the dam was designated as the site compound. The blocks, each 1.17m wide were formed lying on their side using specially fabricated steel shutters resting on a plywood base. The unevenness of the car park necessitated construction of concrete pads to support the plywood base. A total of 12 bases and four sets of shutters were used in the casting process.

Following the delivery of the steel shutters it was discovered that the curved sections had not been correctly rolled which caused a twist in the shutter. These were returned to the manufacturers for correction. Only 10 plywood bases were constructed initially, but in an attempt to recover some of the time lost from the fabrication error, an additional 2 bases were constructed.

A total of 63 blocks were precast each weighing 6.2 tonnes. The design required the five central blocks to be 20mm lower than the others. To achieve the reduced height, 14mm thick plywood was fixed onto the steel shutter faces corresponding to the faces which would be in contact with the original spillway.

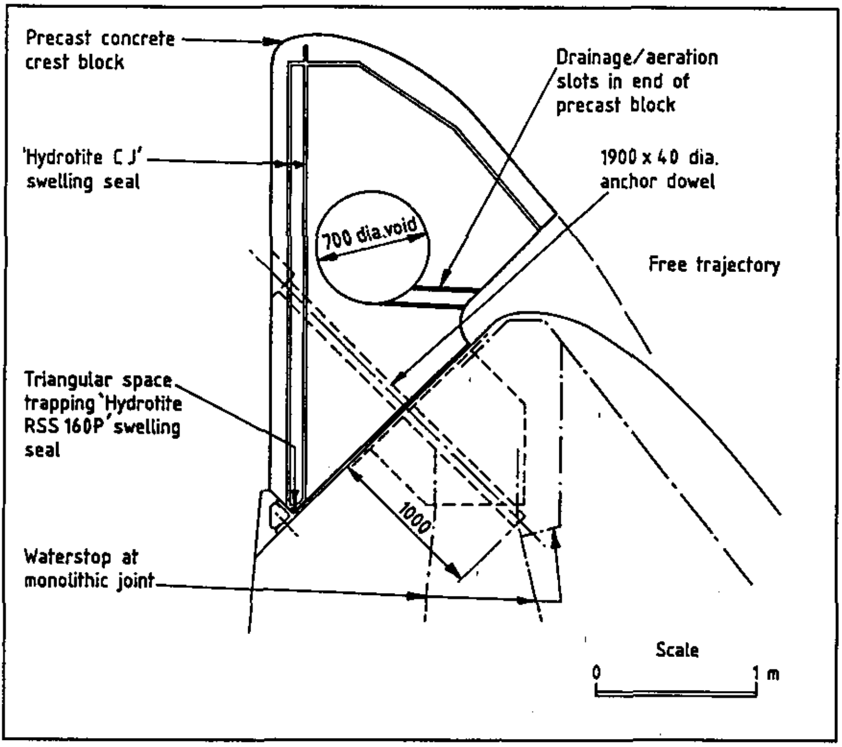


Fig 1. Typical crest block arrangement

The daily sequence of operation during the casting stage involved the preparation and casting of two blocks amounting to one concrete wagon load, striking formwork from the two blocks cast the previous day and the movement of blocks from another two bases into the temporary storage area within the site compound.

As the casting programme progressed the plywood bases deteriorated. The concrete pads supporting the plywood were too widely spaced which allowed the base to deform excessively. The bases were replaced mid-way through the casting programme to ensure the tight tolerances specified in the contract were achieved.

Defective blocks required remedial treatment prior to placement onto the spillway crest. Initial remedial measures effected by the Contractor involved the use of a diamond tipped grinder to remove any surface undulations. It was found that the grinder took too long to remove the deformities and the grinding disks, which were expensive to replace, wore away very quickly. This method was replaced by scabbling after addressing only two defective blocks.

The blocks were moved from the casting area to the temporary storage area by a 12t forklift. Temporary lifting eyes were installed into the blocks during casting. The lifting eyes resembled large eye hooks which were removed after the blocks were moved to the storage area by unscrewing the threaded section from the concrete.

Hydrotite strips, were stuck onto the concrete blocks using the manufacturer's adhesive immediately before they were moved from the site compound onto the crest. This was done to minimise the potential swelling of the material which would have affected the ability to achieve a close joint with the neighbouring blocks.

Block placement

The blocks were placed onto the spillway crest by a 35t crawler crane standing on a floating pontoon. The pontoon was made up of 14 Uniflote units joined together in a tee formation. A 28 feet long workboat was used to move the pontoon on the reservoir.

Up to four blocks were transported on the pontoon at any one time. The blocks were lifted onto the pontoon from the site compound by the crane. A specially fabricated C-shaped lifting frame was used to lift the blocks by crane which utilised the 700mm diameter preformed hole located at the centre of gravity of the block. During block placement the pontoon was anchored in position by ropes tied to bolts which had been drilled into the upstream face of the dam.

To support the blocks during installation a 150 x 150 x 10mm galvanised metal section was fixed to the sloping dam face by 290mm long Hilti 'HVA' adhesive anchors at 300mm centres. Once all blocks had been installed the angle was encased in concrete.

The first block was positioned to correspond with one of the monolith joints of the existing spillway. The location also corresponded to the high point in the upstream face of the original crest. The crest level of this block was then used to fix the other blocks. Packing of the blocks was required to maintain a uniform crest level. This comprised 1mm thick steel shims placed beneath the blocks and against the support angle. The shimming operation was time consuming and required several attempts in most cases to achieve the required level and alignment.

A temporary scaffold walkway was erected along the full width of the weir downstream of the crest to provide access and act as a safety barrier.

Two training walls were constructed, each 500mm wide which were dowelled into the original spillway side walls. These walls were cast in-situ using a concrete skip operated by the crane. A retarder was added to the concrete mix to account for the additional time taken to place the concrete.

The final two blocks being shorter than the standard length were cast in-situ using an adaptation of the special shutters. The blocks were dowelled into the wing walls and debonded from the original crest by two layers of Fosroc Expandite 'Proofex 12' sheeting.

Once all crest blocks were placed the void between the original crest and the placed blocks was filled using a non-shrink cement grout. The hole provided for permanent fixing of the blocks was used to place the grout.

Permanent fixing of each block was by a 40mm diameter reinforcement dowel 1900mm long grouted into a place. A 75 mm diameter hole in the original crest was created by a percussion drill suspended by the crane. The 100mm diameter preformed hole in the new crest blocks acted as a guide to the drill bit during the drilling operation. Some built-in metal work was encountered within the mass concrete and necessitated diamond coring to obtain the necessary dowel hole depth.

Minor grinding of the crest blocks proved necessary after placement to achieve a uniform crest level. The new crest level varies in level by 21mm along its length.

A fixed safety barrier was installed upstream of the crest. The barrier consists of a series of stainless steel frames projecting 3m upstream of the overflow supporting a set of 4 evenly spaced stainless steel cables which act as a fence from 0.2m below to 0.4m above top water level.

<u>Parapet wall strengthening</u>
Strengthening of the parapet wall involved the installation of 20mm diameter, 1000mm long reinforcement at 500mm centres along the full length of the dam. The dowels were installed at an angle of 20° to the vertical.

Drilling comprised a mixture of percussion and diamond coring. The upper 300mm of the dowel hole was drilled using a 40mm diameter diamond corer, the remaining 800mm formed by a 32mm diameter bit percussion drill. Diamond coring was carried out to overcome any reinforcement that would be encountered.

Drilling was carried out from a moveable access platform suspended over the parapet wall. During the strengthening operation a total of 486 dowels were installed an average of 9 dowels per day.

To minimise the risk of parapet wall failure the final three crest blocks were not placed onto the weir until all parapet wall strengthening dowels had been installed.

BRIANNE DAM

Raising the top water level of the reservoir required the raising of the spillway crest and extending the impermeable clay core to road level by means of a mass concrete wall high enough and articulated to allow for future settlement of the embankment.

Embankment modifications.

A nominal raising of the crest road was required to suit the raised maximum stillwater flood level in PMF winter conditions. To make the embankment watertight an extension of the clay core was needed. The core extension was formed by a mass concrete wall extending over the full length of the crest and connected to bedrock at either end of the dam (Fig 3). The wall was constructed in panels no longer than 10m in order to allow for settlement of the embankment. The panels were cast in-situ with their base extending 500mm into the clay core. A pvc waterstop was incorporated into the joints which extended 150mm into the clay core to ensure the joints were watertight. The joint faces were painted with two coats of bitumen to allow the panels to act independently. A 150mm deep rebate was cast into the top of the wall to allow for any future raising works.

The original crest road tarmac was recycled and incorporated into the roadbase layer of the new road. All rockfill material excavated for the crest wall was transported to the site quarry which was used the source of material for the original embankment. A mobile crusher was used by the contractor to recycle the excavated material for backfill to the sides of wall.

Planning approval was obtained by the client for the disposal of surplus material in the quarry. Landscaping of the quarry was carried out by the contractor at the end of the project.

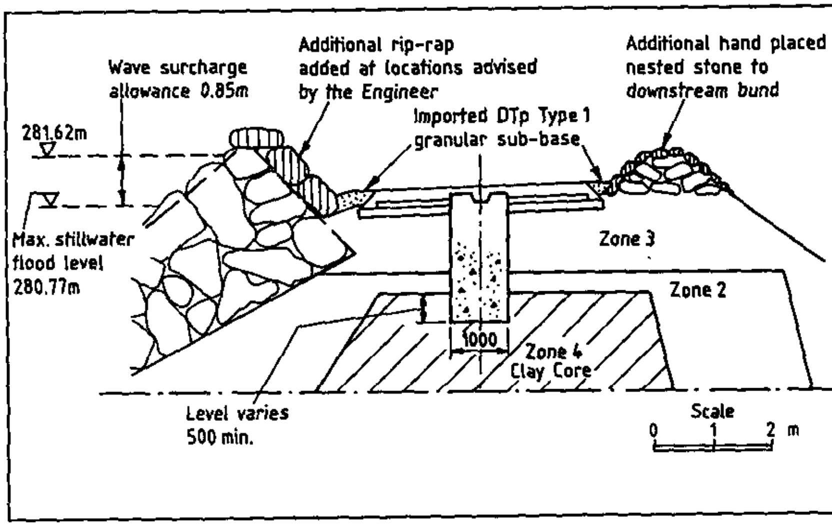


Fig 3. Concrete wall core extension at Llyn Brianne Dam

The existing rip-rap was high enough to give the required protection against wave surcharge coincident with maximum flood level but local deficiencies were made good using stone collected from the site quarry. Sufficient quantity of rock was available at the base of the quarry slopes without the need for any special quarrying operations. The stone bund on the downstream side of the crest road also required localised raising.

Three inclinometers present in the embankment were to be abandoned originally since they were considered to be redundant. Following discussion with the Supervising Engineer it was decided to incorporate them into the new works. To allow the inclinometers to move independently of the wall the tubes were sleeved using 150mm upvc pipe and surrounded by 10mm pea gravel. The upper 800mm of the tubes were sleeved using 600mm diameter pipes to allow access for extending the tubes should the embankment be raised further in the future.

Spillway raising

The spillway was raised using in-situ reinforced concrete with the reinforcement located at the outer face of the concrete to avoid surface cracking.

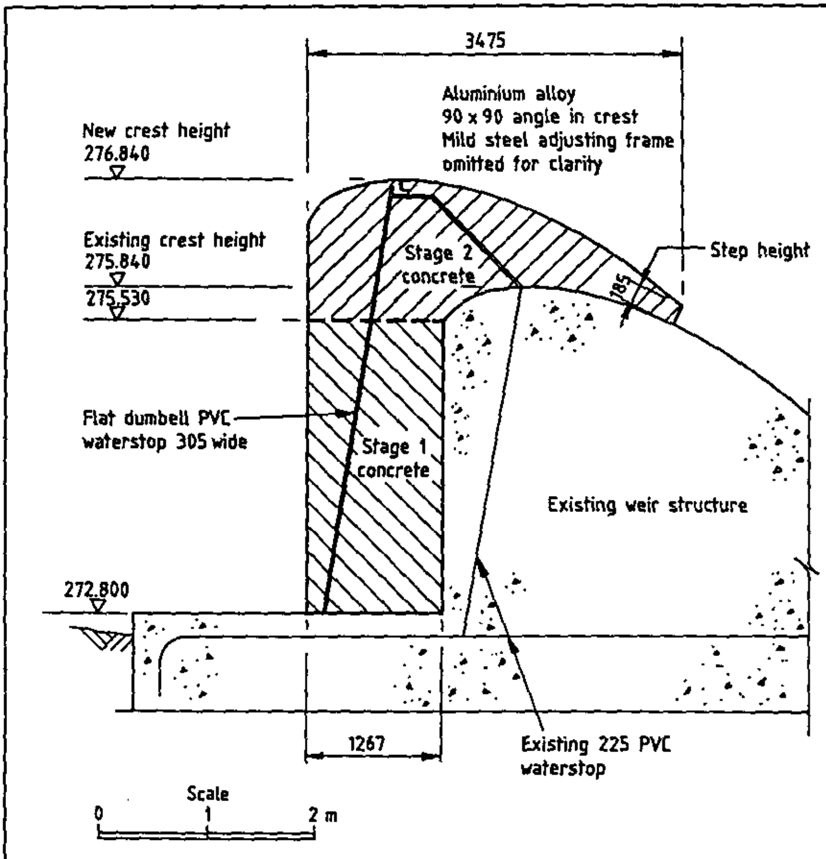


Fig 4. Raised spillway crest profile

The spillway raising was carried out in two stages. Stage I involved placement of concrete up to the level at which the curved section of the original crest started. A total of 5 pours were undertaken in bays up to 12.65m long in order to ensure that the contraction joints in the original spillway were continued in the raised crest.

Stage II of the raising involved the casting of the remaining concrete. The pours were in bays no longer than 3.2m, construction joints were formed between panels except where the joint coincided with contraction joints in the existing crest. The restriction to panel length was considered necessary for two reasons: it would be easier to achieve the required crest curve and there would be less delay should a panel have required recasting due to non-compliance with the specification.

To reduce the risk of seepage through the construction joint on the Stage II works, 48 hours elapsed before an adjoining panel was cast.

A 90mm x 90mm aluminium angle was cast into the crest of the raised spillway to give a true line and level to the work. The angle was connected to adjustment brackets fixed to the top of the stage I concrete. The angle enabled the raised crest to be constructed to the specified tolerance of ±2mm.

Plasterers were employed to finish the surface of the concrete on the crest, plywood templates were used to check the accuracy of the crest profile. The consistency of the concrete was critical in the formation of the crest, a wet mixture would simply flow down the existing crest, a dry mixture would be difficult to place and finish.

Where contraction joints were provided in the raised crest, the adjacent panels were debonded by two coats of bitumen paint. A waterstop was installed between the panels which was joined to the waterstop in the original spillway. The existing waterstop was exposed at the crest, the new waterstop was overlapped and cast into the existing crest using a non-shrink and waterproof polymer concrete. A permanent former was used to ensure the contraction joint was continuous.

Additional contraction joints were constructed in the raised crest approximately 1300mm away from the abutment walls. These panels were bonded to the abutments using a concrete bonding agent. The upstream face of the spillway and floor were bitumen painted to ensure debonding.

The concrete mix used in the raising was the same as that used in the original spillway. Ordovician quartzite aggregate was used, obtained from the Dinas Quarry at Llansawel. This aggregate was chosen due to its better strength properties when compared with other local rock types. Prior to start of construction the owners decided to close Dinas Quarry. Fortunately, the contractor's concrete supplier had arranged for sufficient quantities of the aggregate to be stockpiled at its batching plant at Llandybie before the quarry was closed.

All works associated with raising the spillway and abutments was undertaken from the floor of the spillway approach channel. The water level within the reservoir was lowered in the early stages of the contract to allow access to the forebay. Concrete for the crest raising was placed using a concrete skip fixed to a 360 degree hydraulic excavator. A specially fabricated extension arm was installed to the excavator's jib to allow greater reach.

In conjunction with the raising of the spillway, a 1 metre raising of the abutments was also carried out. Placement of the concrete was carried out by concrete pumps. An initial attempt to pump the concrete from the forebay below was unsuccessful and was attributed to the high flakiness index and angularity of the quartzite aggregate which caused a low workability concrete mix. Finally concrete was pumped from high ground each side of the spillway.

Reservoir safety

To ensure that the safety of the reservoir was not compromised during construction all work involved with extending the embankment was completed before raising of the spillway to its final level (Stage II) was carried out. The two stage construction of the raised spillway was designed to reduce construction time.

CONCLUSIONS

Different methods of raising the spillways were used for the dams, each was deemed to be the most appropriate for the situation. Bearing in mind the ever present possibility of a flood during construction.

Raising of the spillway on Llysyfran dam by in-situ concrete would have been extremely difficult due to the lack of access to the crest for placing concrete. However, the use of precast concrete blocks made it difficult to achieve the required tolerance in crest level since undulations in the original spillway were mirrored in the new crest.

Both dams were designed to accommodate substantial future raising; the nominal raising carried out in these two contracts was achieved at low cost and required minimal alteration to the main structure whilst maintaining reservoir operation.

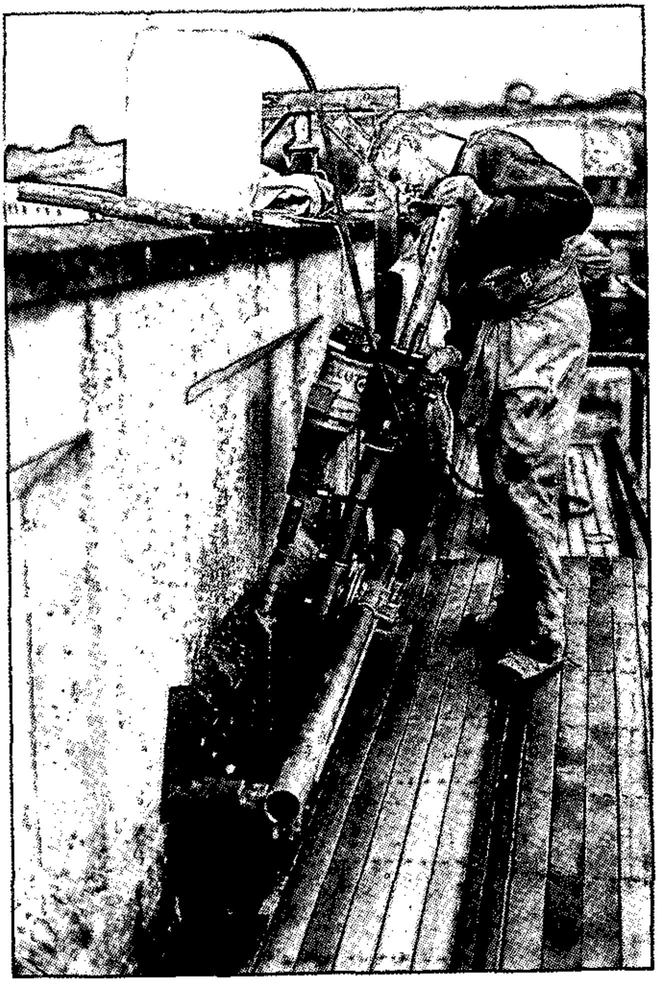


Fig 2. Dowel hole drilling

Prior to drilling a reinforcement cover meter was used to locate existing vertical wall reinforcement. The dowels were positioned to avoid the detected reinforcement. This was to ensure that the structural integrity of the wall was not compromised.

ACKNOWLEDGEMENTS
The author is grateful to Mr L S Davies of Dwr Cymru/Welsh Water for granting approval for the publication of this paper. The encouragement and assistance of Mr D E Evans during the construction of both projects is also appreciated.

The restoration of Rufford Lake

PG DE LANDE LONG, Binnie Black & Veatch, UK
CW SCOTT, Binnie Black & Veatch, UK

SYNPOSIS. Rufford Lake is one of the central features of Rufford Country Park. The lake is formed by a 4 m high dam at the northern end, associated with a listed mill building. The area under Rufford Lake has been subject to deep coal mining for many years. The resulting surface subsidence has meant that the differential levels between the various features have altered and restoration works have been required on a number of occasions. Most recently, a study of the effects of mining on Rufford lake was undertaken on behalf of British Coal. The study determined that restoration work was required for reservoir safety reasons. The works had to balance the needs of reservoir safety with both the maintenance of the amenity aspects of the country park and the building conservation requirements for the listed mill building.

INTRODUCTION

Rufford Lake is one of the central features of Rufford Park, which is owned by Nottinghamshire County Council (Notts CC) and is open to the public as an amenity area. In addition to the lake, the park contains a listed mill structure, the remains of Rufford Abbey and a large area of woodland. The lake is formed by a low dam at the northern end. The dam is about 4 m high, nearly 200 m long and was probably constructed as part of the development of the mill building located at the centre of the dam. The layout of the lake and associated structures is shown on Figure 1.

The area under Rufford has been subject to deep coal mining activity for many years, the resulting surface subsidence has meant that the differential levels between various features, including the dam crest, overflow sill and lake bed have altered . Following mining activities in the early 1970's restoration works were carried out in 1974. These involved raising and sealing of the dam, improving the overflow works and constructing retaining walls upstream of the mill building.

Since 1974 there has been further mining subsidence which reduced the freeboard of the dam in front of the mill. The inspecting engineer recommended the construction of a brick wall in front of the mill building as a temporary measure. This was done in 1988. Continuing settlements further reduced the freeboard and later that year the normal lake level was lowered as an interim measure until permanent restoration works could be carried out.

In October 1989 British Coal (now the Coal Authority) appointed Binnie and Partners to study and make proposals for the restoration of Rufford Lake. The recommended restoration works were constructed between November 1990 and May 1991.

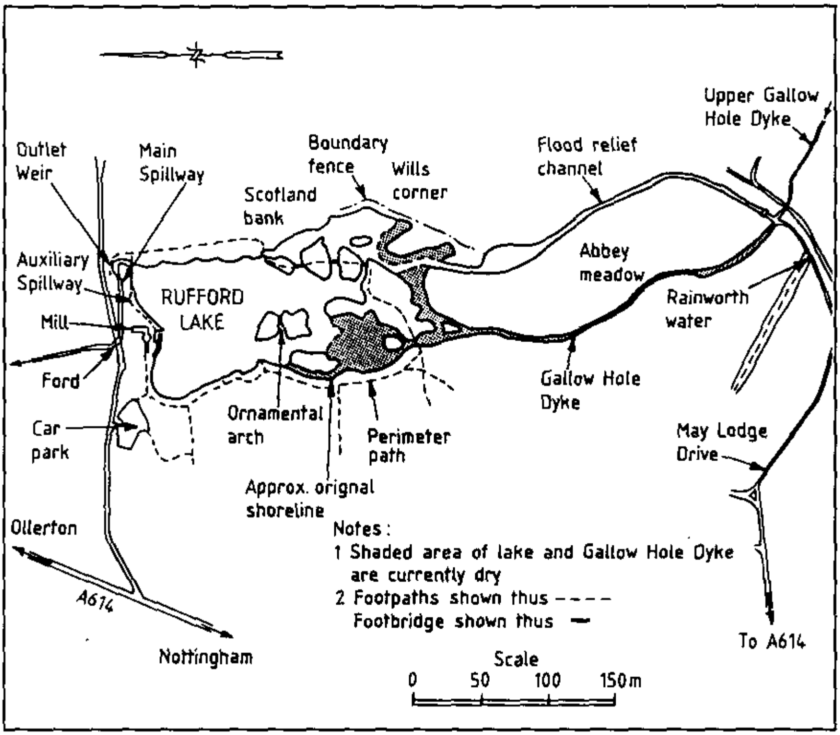


Fig 1. Layout of Rufford Lake.

DESCRIPTION OF RUFFORD LAKE PRIOR TO RESTORATION

The dam, spillway and Mill building are at the northern end of the lake. As a result of the mining subsidence and subsequent lowering of the lake level for safety reasons large areas of the southern end of the lake were dry from 1988 until the completion of the restoration works. The confluence of the two streams feeding the lake is about 300 m upstream of the lake. From the confluence water flowed into the lake along the flood relief channel, as the parallel channel of Gallow Hole Dyke was largely silted up as a result of the subsidence.

The maximum height of the dam was about 4 m. The greater part of the dam to the east of the Mill building was taken up by various outlet and overflow spillway arrangements.

Normal flows were discharged through the curved approach channel at the east end of the dam, over the outlet weir and along the discharge channel. The outlet weir therefore controlled the level in the lake. The weir was equipped with wooden overflow gates which were raised in 1988 to keep the lake level down as required by the Inspecting Engineer.

A masonry wall on the left side of the approach channel was capped with concrete at a uniform level over a length of about 40 m in the 1974 restoration works, in order to act as a weir crest to the main spillway. The 1974 works allowed floods up to the 1 in 100 year event to be discharged over this wall and onto a gently inclined slope below into the discharge channel downstream of the outlet weir. The slope below the weir crest was armoured with earth filled and grassed interlocked hollow concrete blocks (monoslabs).

The 1974 works made further provision for floods in excess of the 1 in 100 year event to be discharged over a low point in the dam between the main spillway and the Mill, known as the auxiliary spillway. The crest of this auxiliary spillway was protected by the tarmac footpath on the dam crest. The downstream slope of the auxiliary spillway was grassed which was considered adequate protection against erosion for the frequency and intensity of flow anticipated.

Since 1974, further mining subsidence occurred altering the relative levels of the spillway structures and outlet weir. As a result, the spillway crests were no longer uniform and the flow which would overtop the auxiliary spillway had been reduced to less than the 1 in 15 year event. The dam freeboard had also been reduced to less than required for reservoir safety reasons.

As part of the 1974 works, reinforced concrete retaining walls were constructed in front of the mill building to act as the water retaining element in the dam and to allow water to be led to the original mill sluice channel. In 1988 brick walls were added to reinstate the necessary freeboard which has subsequently again been reduce by subsidence.

Figure 1 shows the effect of the subsidence on the lake with a significant portion of the southern end dry. There is a perimeter footpath round the lake which links some of the islands at the southern end of the lake by means of five footbridges.

The lake is fed by two streams: Rainworth Water and Gallow Hole Dyke. These streams converge at a point 300 m upstream of the lake from where two parallel channels lead to the lake. These two channels are the continuation of Gallow Hole Dyke which was dry and the Flood Relief Channel which was constructed in 1986 to provide additional capacity to the system and hence reduce upstream water levels.

RESTORATION WORKS

General

The mining subsidence settlements caused two major effects which needed to be considered in the design of the restoration works:

- the dam was left with inadequate freeboard to meet the required reservoir safety standards; and
- the reduction in lake level required to reinstate freeboard had left the southern area of the lake dry.

Restoration work was also required to have no detrimental effect on the flooding situation upstream of the lake which had previously been the subject of a High Court action.

Reservoir safety

Rufford Lake was assessed as a Category C reservoir in accordance with the guidance provided by the Floods and Reservoir safety guide (ICE,1978). For this category of reservoir the design flood is the greater of the 1 in 1000 year flood or 30% of the Probable Maximum Flood (0.3 PMF). A minimum freeboard of 0.4 m is required as an allowance for waves.

Table 1. Estimated peak flood flows

Event/return period	Peak flow at dam
2 year	6.1 m^3/s
10 year	12.6 m^3/s
100 year	22.3 m^3/s
1000 year	38.9 m^3/s
0.3 PMF (winter)	79.4 m^3/s

Estimates of instantaneous peak runoff were made on the basis of the method given by the Flood Studies Report (NERC, 1975) as modified by the Flood Studies Supplementary Report (IH, 1985) for events ranging from the 1 in 2 year flood to 0.3 PMF, which was considerably larger than the 1 in 1000 year flood as can be seen from Table 1. The small size of the lake meant that attenuation of the floods would be negligible and the design peak discharge from the reservoir would be the same as the peak inflow (i.e 79.4 m^3/s)

Spillway

The rating curve for the post subsidence spillway arrangement is shown in Figure 2. For the design flow, the still water level in the lake would have been 48.46 mOD. The minimum level of the brick wall in front of the mill building was 48.5 mOD. The minimum dam crest level was 48.43 mOD. The post subsidence spillway did not therefore satisfy the freeboard requirements of the Floods and Reservoir Safety guide.

To raise the normal lake levels the outlet sluice gates would need to be reinstalled at the required new lake level. The effect on the spillway rating curve of installing the gates at 47.25 mOD is shown on Figure 2. Installing the gates at that level would have raised the lake level by 0.55 m and would have had a small effect on the maximum level during the design flood.

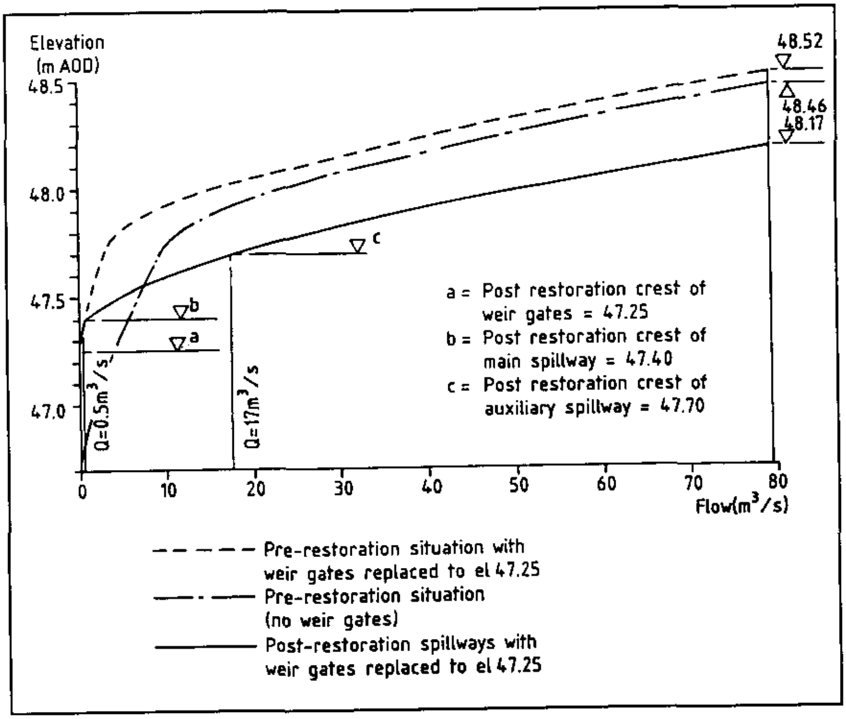


Fig 2. Spillway rating curve.

From these studies, it was clear that obtaining the necessary freeboard involved either:

- raising the crest of the dam by up to 0.5 m; or

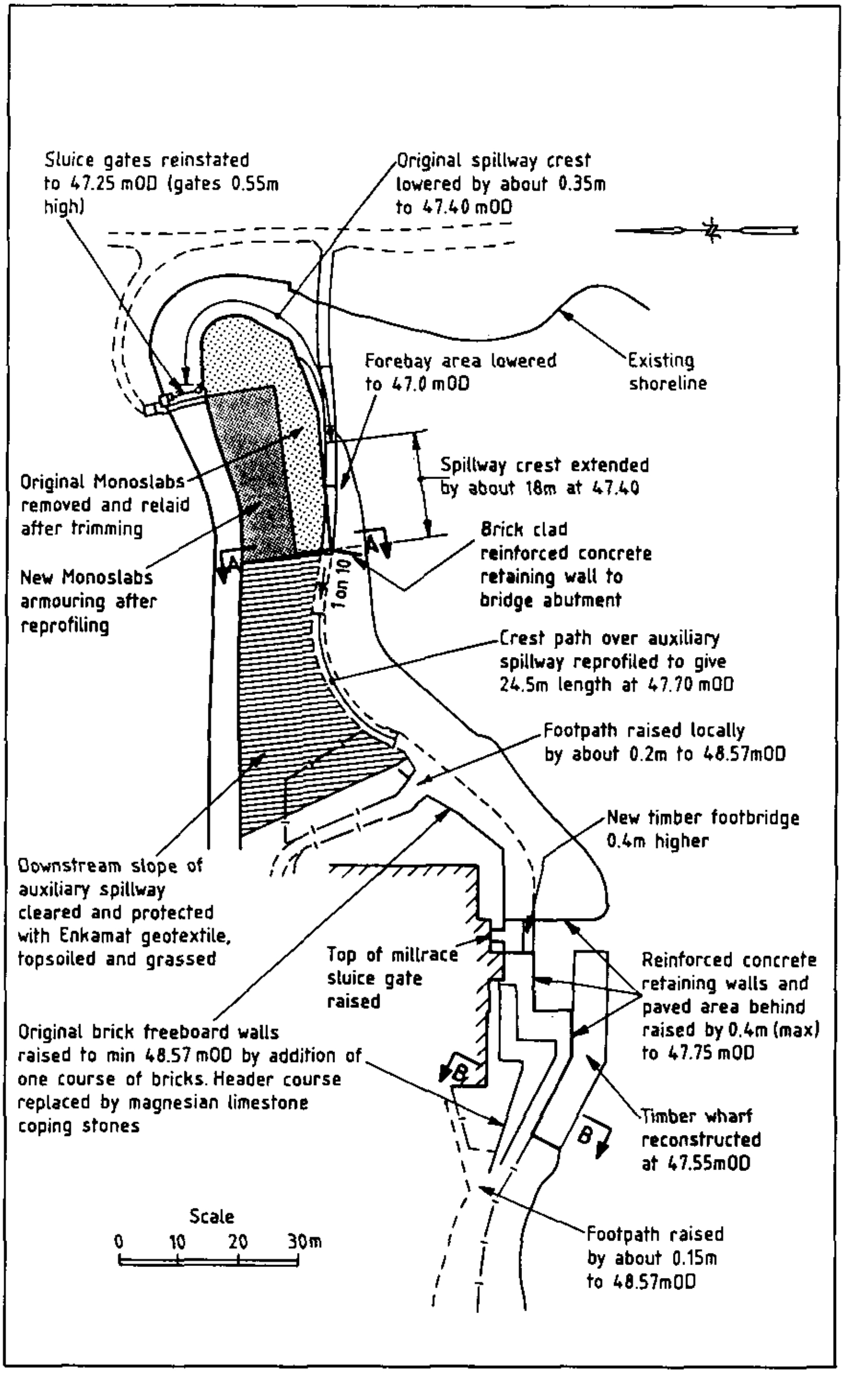


Fig 3. Layout of restored dam and spillway

- modifying the crest of the main and auxiliary spillways to lower the design flood level by the same amount.

In either case some raising of the area in front of the mill building was required although lowering the flood level minimised this requirement. This approach was preferred because:

- further raising the walls in front of the mill building would have adversely affected the aesthetics of the north end of the lake; and
- the existing reinforced concrete walls were cracked by the subsidence settlements and significant raising would have required majorstrengthening work.

Therefore, spillway modification options to minimise the flood rise were studied. The selected option involved lowering and widening the main spillway and widening the auxiliary spillway. Figure 3 shows the layout of the restored spillway.The main spillway was lowered by 0.35 m to 47.4 mOD and widened to utilise the full width available beneath the footbridge. The crest would follow the existing shore line with a concrete crest block. At the bridge abutment a concrete retaining wall was required in place of the earth slope. The lowering of the crest meant that the shallow slope downstream of the spillway crest needed to be reprofiled. This area also had to be extended to accommodate the increased spillway width. Notts CC wished to retain the grassed appearance of the spillway and therefore it was decided to retain and extend the monoslab protection of this slope. Figure 4 shows a cross-section through the restored spillway.

The outlet weir gates were set 0.15 m below the spillway crest to ensure that

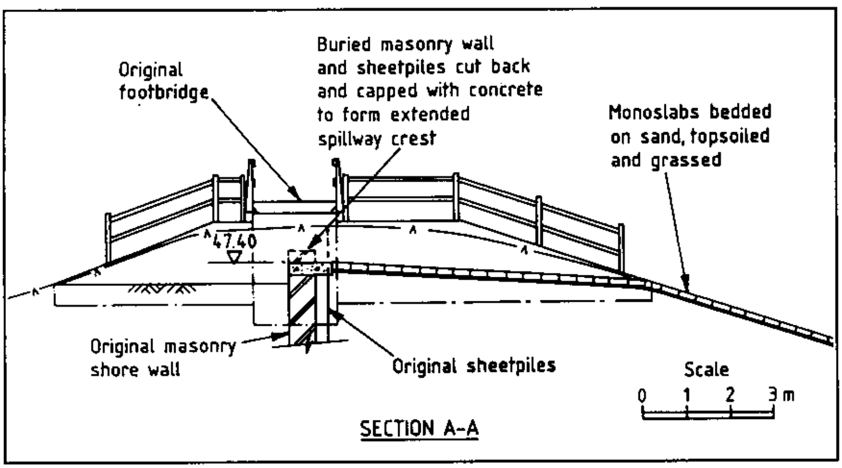


Fig 4. Cross-section of restored spillway.

only flood flows passed over the spillway. In normal conditions there is at least 50 mm flow over the gates giving a nominal lake level of 47.3 mOD.The footpath forming the crest of the auxiliary spillway had to be suitable for wheelchair access which limited acceptable gradients to 1 in 10. The crest level of the auxiliary spillway was lowered to 47.7 mOD over a 24.5 m length with 1 on 10 slopes to the normal crest level at either side. Unfortunately, this meant that part of a tree planted area had to be cleared to allow proper operation of the spillway. Geotextile slope protection was considered to be necessary on the slope downstream of the auxiliary spillway crest to limit erosion. The protection was designed in accordance with guidance given in the Construction Industry Research and Information Association design guide (CIRIA, 1987).

The rating curve for this arrangement is shown on Figure 2. The main spillway is overtopped by flows in excess of 0.5 m^3/s, which occur a number of times in any normal year. The auxiliary spillway is overtopped by flows in excess of 17.5 m^3/s which represents about a 1 in 35 year event. This return period is rather shorter than that adopted for the 1974 restoration which is why slope protection was considered necessary on the downstream slope.

The flood level for the 0.3 PMF event was estimated to be 48.17 mOD. A dam crest level of 48.57 mOD is therefore required to maintain the required 0.4 m freeboard. Some minor raising work was therefore required.

<u>Dam and mill area</u>

The area in front of the mill building including the paved area, the footbridge, the timber wharf and the freeboard wall required raising to accommodate the raised lake level. Short lengths of the dam crest on either side of the freeboard wall also required raising to the minimum freeboard level. The existing mill race sluice gate and frame required modification to raise the top of the gate. The freeboard wall required raising to maintain the minimum freeboard of 0.4 m above maximum flood level during the 0.3 PMF. Figure 3 shows the layout of the restoration works for the dam in front of the mill building.

The reinforced concrete retaining walls in front of the mill building form the waterproof element of the dam in front of the mill. The walls needed to be raised to 47.75 m which represented the flood level for the 1 in 100 year event. The paved area behind the walls also needed raising to the same level. The timber wharf in front of the mill needed to be raised to reinstate its original clearance to the normal water level. Figure 5 shows a cross-section through the restoration works in front of the mill building.

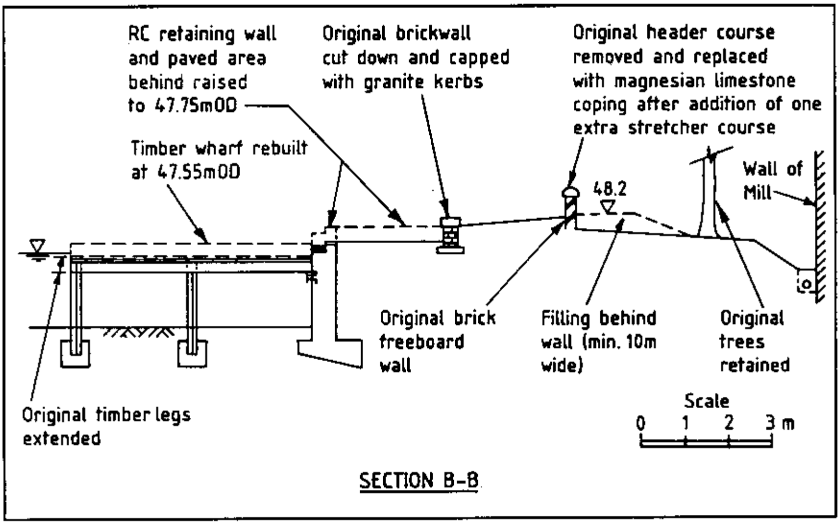


Fig 5. Cross-section through restoration work in front of mill building.

The reinforced concrete walls had been cracked and tilted by the settlements and a thorough inspection could not be undertaken until the lake was drawn down further for construction. Therefore, it was possible that repair works could be necessary following inspection of the wall during construction of the restoration works.

The footbridge across the mill race channel had been damaged by the wall movements. Notts CC requested a hardwood bridge constructed of Ekki timber to replace the existing bridge. Replacement handrails around the paved area and the raising of the timber wharf were also done with Ekki.

The mill is a listed building and listed buildings consent was required for the work in front of the building from Newark & Sherwood District Council (NSDC). Following consultation with the conservation architect at NSDC it was decided to raise the height of the brick freeboard wall using a magnesian limestone coping to co-ordinate with the detailing of the mill building.

Lake, margins & islands

Notts CC required a minimum water depth throughout the restored lake of 1 m to minimise weed growth. To achieve this some excavation of the southern end of the lake was required. From previous restoration work, the lake bed was known to be covered by between 0.3 and 0.4 m thickness of black silty material thought to be derived from colliery washings upstream. This material overlies sandy gravel. The silty material forms an effective seal for the bed of the lake.

The required excavation was to be done in the sandy gravel material with the silt removed and replaced following excavation to maintain the seal on the lake bed. This method also limited any potential disposal problems with the silty material.

A similar approach was adopted for the excavation of the dry length of Gallow Hole Dyke downstream of the confluence with Rainworth Water.

Notts CC also took the opportunity presented by the contract to replace the existing footbridges between the islands in the southern part of the lake with bridges fabricated from Ekki.

CONSTRUCTION

A contract for construction of the restoration works was let in late 1990. Work was to be completed over a five month period up to Easter 1991. The majority of the work proceeded without major incident.

The investigation of the retaining walls in front of the mill building found that the wall on the west abutment of the footbridge was extensively cracked. Even with the lake drawdown it was not possible to sufficiently drain this

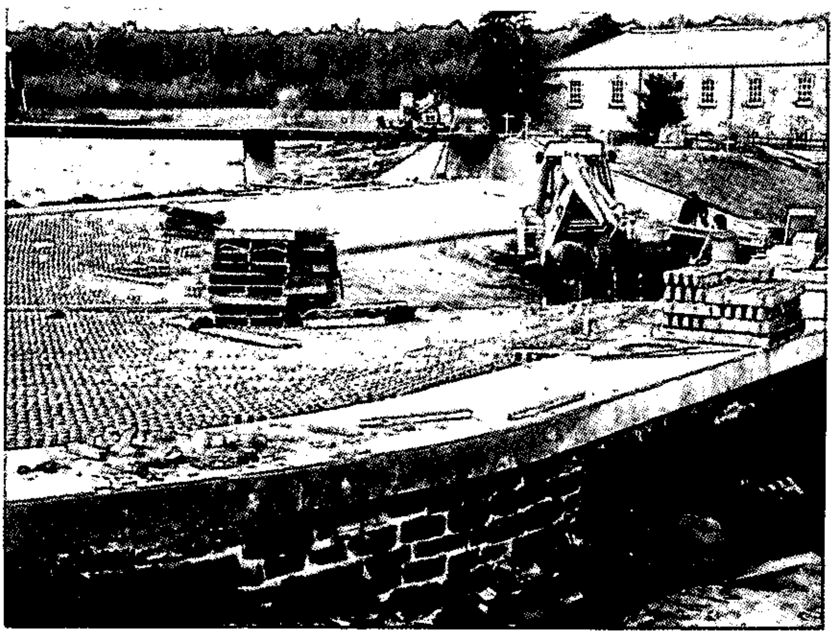


Fig 6. Modification of main spillway.

area to allow remedial work to be carried out. Therefore a sheet pile cofferdam was constructed in front of the mill building to allow remedial work to be completed. This involved the construction of a strengthening wall behind the existing damaged wall.

The modification of the spillway proceeded without incident although it was not possible to purchase identical monoslabs to those installed previously as sizes had changed to metric dimensions. The two types of slab were easily accommodated by ensuring that the new blocks were installed in a regularly shaped area. Figure 6 shows the modification of the spillway underway.

Close liaison was maintained with the conservation architect from the local planning authority throughout the construction period. The suitability of samples of brick and stone were discussed to ensure that the restoration work blended with the listed mill building.

Public access to the country park was maintained during the works. Close liaison with Notts CC was maintained with regular meetings to ensure that the impact of the works on the usage of the park was minimised. The Supervising Engineer was an employee of Notts CC and he was involved in all technical discussions related to the contract.

POST CONSTRUCTION PERFORMANCE

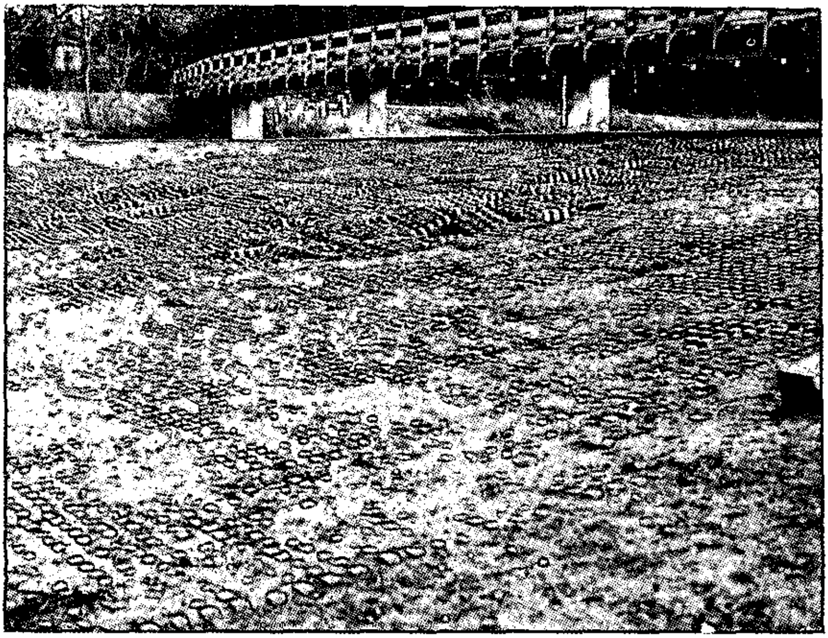


Fig 7. Damage to main spillway.

The construction of the works was completed in May 1991 and, following a statutory inspection, the lake was filled. Filling proceeded slowly because of the dry weather. Notts CC paved the upstream face of the dam between the mill building and the spillway with textured concrete paving. A mining subsidence fissure was identified in the lake bed during 1991 and the lake had to be drained to allow British Coal to carry out sealing works. Filling recommenced in March 1992 and the lake was full to top water level by early May. Over the summer the lake level was steady and no overtopping of the spillway occurred. Grass cover on the monoslab spillway was growing despite the dry weather although a full covering of grass had not been achieved. Grass cover was poor on the monoslabs close to the spillway crest beam as waves were slopping over the crest of the spillway, washing out some of the soil within the blocks.

In November 1992 the main spillway was overtopped and disruption of the monoslabs was noted with some local settlement. Following further heavy rain the monoslab protection was quite badly disrupted as can be seen from Figure 7. It was clear that material was being eroded from beneath the monoslabs.

The failure appeared to have been initiated by the overtopping water

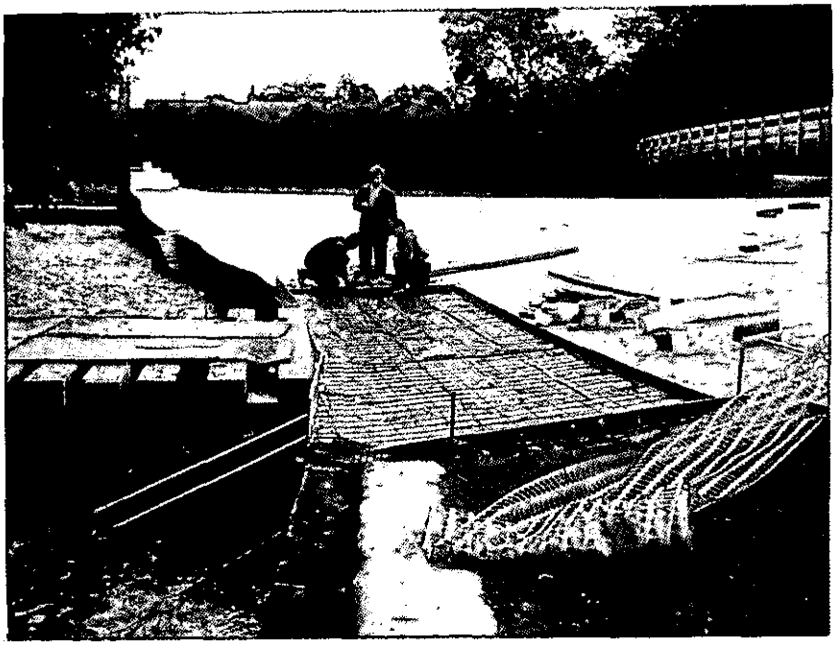


Fig 8. Construction of textured concrete spillway protection.

penetrating through the monoslabs downstream of the spillway crest. Once the water had penetrated the monoslab layer it was able to cause uplift lower down the spillway slope even where grass cover was well established. This resulted in displacement of the monoslabs exposing the underlying material to erosion. The consequent loss of this underlying material led to the monoslabs settling.

In many areas, the topsoil and grass remained undamaged in the cells of the monoslabs despite the fact that the monoslabs themselves had become displaced. This suggests that, if the grass cover had become established over the whole spillway, the system would have provided adequate slope protection. The frequent wave slop resulting from the low freeboard of the spillway crest prevented grass becoming established close to the crest.

Following discussions with British Coal and Notts CC it was decided that attempting to retain the grassed appearance of the spillway was impracticable. This approach was adopted as it was apparent that, if monoslab protection was to be retained, the lake would need to be kept drawndown until adequate grass cover was established over the whole spillway area. An alternative textured concrete paving option was selected as Notts CC wished to ensure that the spillway had an appearance appropriate for a country park.

A remedial works contract was let and the work was completed in early 1993. Figure 8 shows the construction of the spillway surface.

The new spillway protection has performed well since its construction and no problems have been reported.

CONCLUSIONS

The restoration of Rufford Lake provides a case history of the implementation of substantial civil works required for reservoir safety at a popular public amenity, while causing minimum disruption and environmental damage to a sensitive area in close proximity to a listed building..

ACKNOWLEDGEMENTS

The authors are grateful to Mr Steve Shakespeare of the Coal Authority and the management of Binnie Black & Veatch for permission to publish this paper.

REFERENCES

Binnie & Partners (1990), Rufford Lake - Proposed Restoration Works, unpublished report for British Coal.

Construction Industry Research and Information Association (1987). Design of reinforced grass waterways. CIRIA Report 116. London. CIRIA.

Institution of Civil Engineers (1978). Floods and reservoir safety: an engineering guide. London. ICE.

Institute of Hydrology (1985). The FSR rainfall-runoff model parameter estimation equations updated. Flood Studies Supplementary Report No 16. London. IH.

Natural Environment Research Council (1975). Flood studies report. London. NERC.

Ireland Colliery Reservoir: A reservoir created by deep mining subsidence

A K HUGHES, RKL-ARUP, Director of Ove Arup & Partners and AR Panel Engineer.
C J BEECH, Derbyshire County Council, Derbyshire Consulting Engineers.

SYNOPSIS. During reclamation of the former Ireland Colliery near Staveley, Derbyshire, a lake created by deep mining subsidence was identified as a 'large raised reservoir' under the Reservoirs Act 1975 because of the volume of water which had become impounded behind a former railway embankment. Derbyshire County Council, who were reclaiming the area to encourage its future redevelopment, appointed an AR Panel Engineer to inspect and assess the lake as a reservoir and to fulfill the role of 'Construction Engineer' during reconstruction work necessary for its enlargement and continued safety as a reservoir under the terms of the Reservoirs Act 1975. This paper describes the inspection and assessment, the site investigation work, the determination of the flood outflow, the design and construction of the new works and the subsequent performance of the 'new' reservoir.

INTRODUCTION.

Derbyshire County Council began to reclaim spoil tips 11/038 and 11/039 in 1989 when Ireland Colliery, Staveley closed after 100 years of coal production. The tips were to be regraded and two open areas of water dredged to encourage recreation and redevelopment of the area. Figure 1 indicates the layout of the site. In 1991 deep mining caused differential surface subsidence of up to 2 metres within the site, along the line of Erin Road and beneath part of the tips, drying out much of the water area, introducing a backfall within a section of gabion lined channel and partially submerging two culverts on the line of Pools Brook. Figure 2 indicates levels of subsidence recorded on the site up to June 1993.

To restore open water levels an enlarged lake, with a volume of 200,000 cubic metres and a maximum depth of 5 metres, needed to be created upstream of Erin Road, the alignment of which followed the crest of the former Markham Colliery Branch Line railway embankment. A second smaller lake, with a volume of 19,000 cubic metres, was proposed downstream of the railway embankment. The volume of water to be impounded, and the likelihood that existing culvert structures within both proposed 'reservoir' and 'lower pond' embankments would require replacement, made it essential that construction satisfied all aspects of the Reservoir Act 1975.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

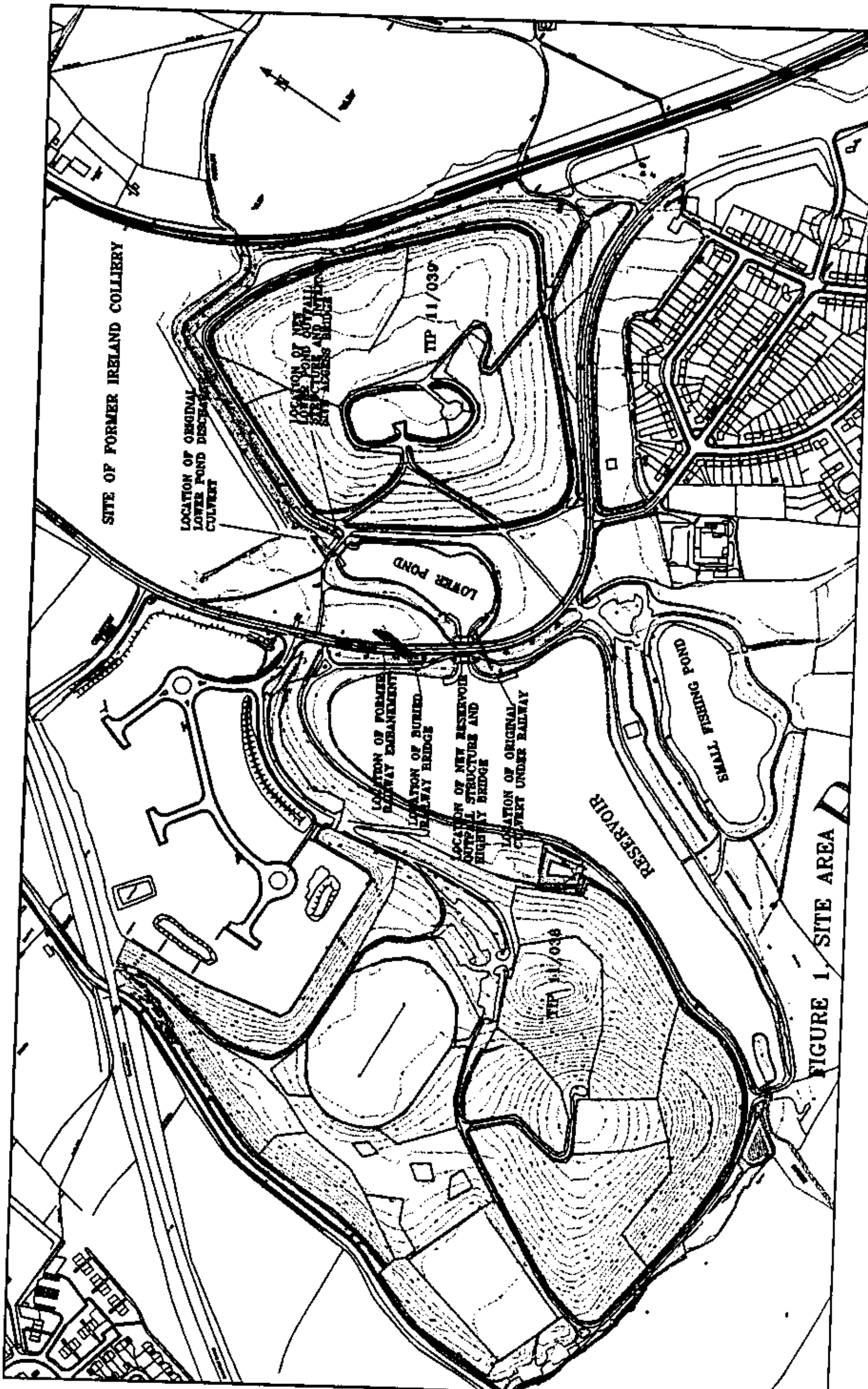


FIGURE 1. SITE AREA D

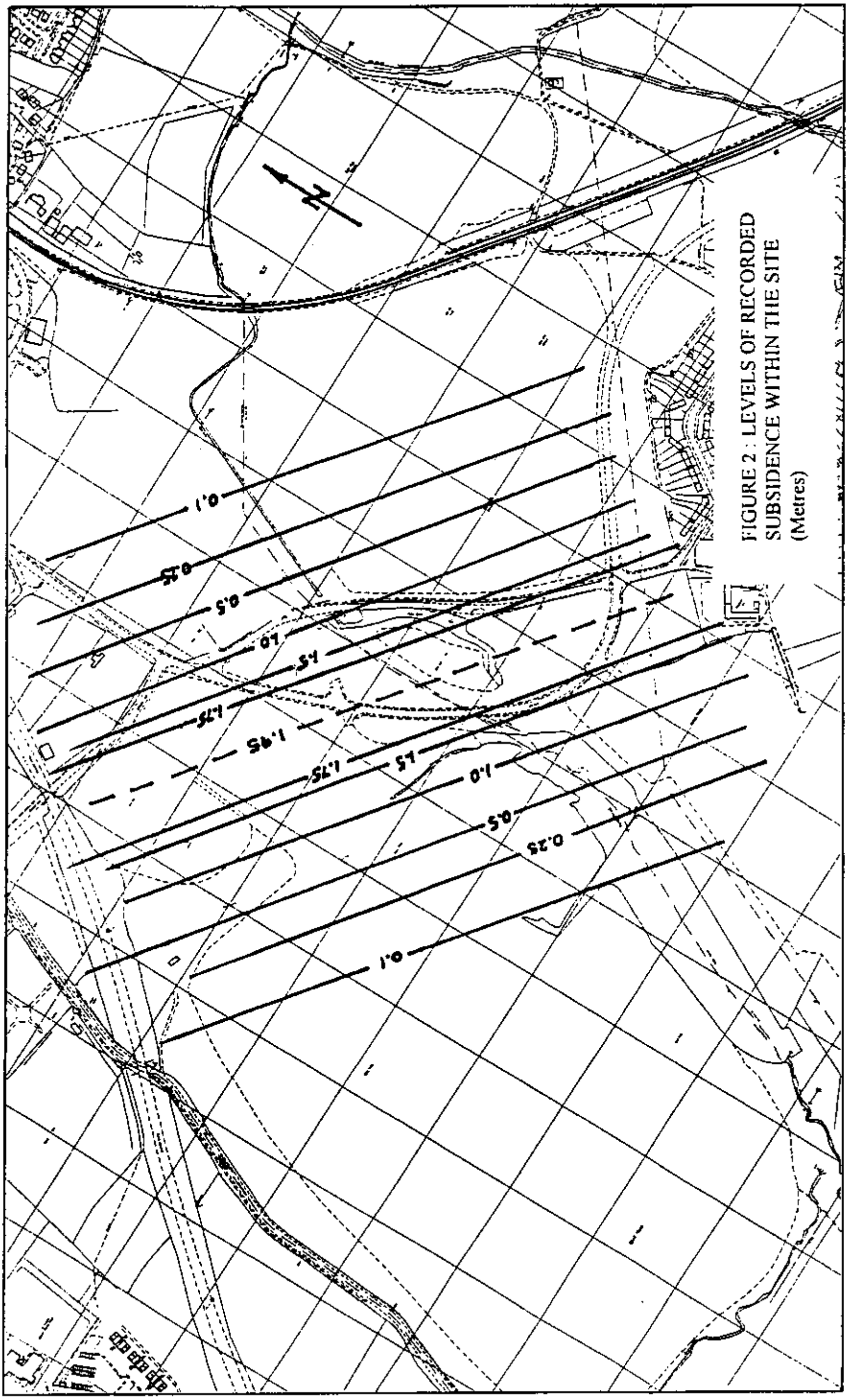


FIGURE 2 : LEVELS OF RECORDED SUBSIDENCE WITHIN THE SITE (Metres)

INSPECTION AND ASSESSMENT.

A member of the All–Reservoirs Panel of qualified engineers was appointed by Derbyshire County Council to assist in the design and construction of this 'new' reservoir under the Act. After confirming that a large raised reservoir already existed, the enlarged reservoir was considered to fall within Category C of the ICE 'Floods and Reservoir Safety' Guide in that a breach would pose negligible risk to life and cause limited damage. This was confirmed in the 1994 report which identified several main items for consideration:

i) to establish the stability of the old railway embankment;

ii) to establish likely seepage values and hydraulic gradients, consequent upon raising the water levels, to ensure that excessive volumes of water would not pass through the embankment and cause erosion;

iii) to investigate the location and condition of a buried bridge within the embankment, to investigate the nature of the backfill within the bridge structure, and to consider options for its removal.

iv) to establish the magnitude of the design flood;

v) to consider the need for downstream slope protection;

vi) to establish a method for sealing the existing culverts.

SITE INVESTIGATION.

On completion of the tip–reclamation contract in 1993, a site investigation was carried out to investigate the potential for redevelopment of the buried railway embankment as an earth fill dam. A desk study reviewed site history and development, and a borehole investigation determined information about the levels and structure of the embankment and complemented information gathered during the reclamation contract.

In–situ testing included Standard Penetration Testing (SPT) and falling head permeability testing at each metre of depth at 30 metre intervals alongside Erin Road, i.e. on the line of the former railway track bed. The construction material for the former railway embankment consisted mainly of slightly sandy silty clay, very similar to the natural sub–surface clay of the surrounding area. There was no evidence of topsoil or drainage at the base of the embankment. In–situ test results for the embankment material indicated generally low bearing capacity in the region of 55 $kN.m^{-2}$ and permeability in the range $5x10^{-4}$ to $1x10^{-8}$ m/s. Laboratory testing included both drained and undrained triaxial testing of 'undisturbed' samples retrieved during the borehole investigation. A selection of samples were tested to determine soil coefficients for both short and long term stability analyses. Initial applied confining pressures were related to sample depth beneath proposed water levels. Short term coefficients of Cu–35 $kN.m^{-2}$ and $\Phi=0$ were determined alongside long term coefficients of Cu=0 and $\Phi=28^{\circ}$. Trial pit investigation established the line and level of the embankment crest, confirmed the nature of the embankment fill and enabled accurate definition of the location, condition and nature of backfill within the buried former railway bridge identified previously on old Ordnance Survey maps.

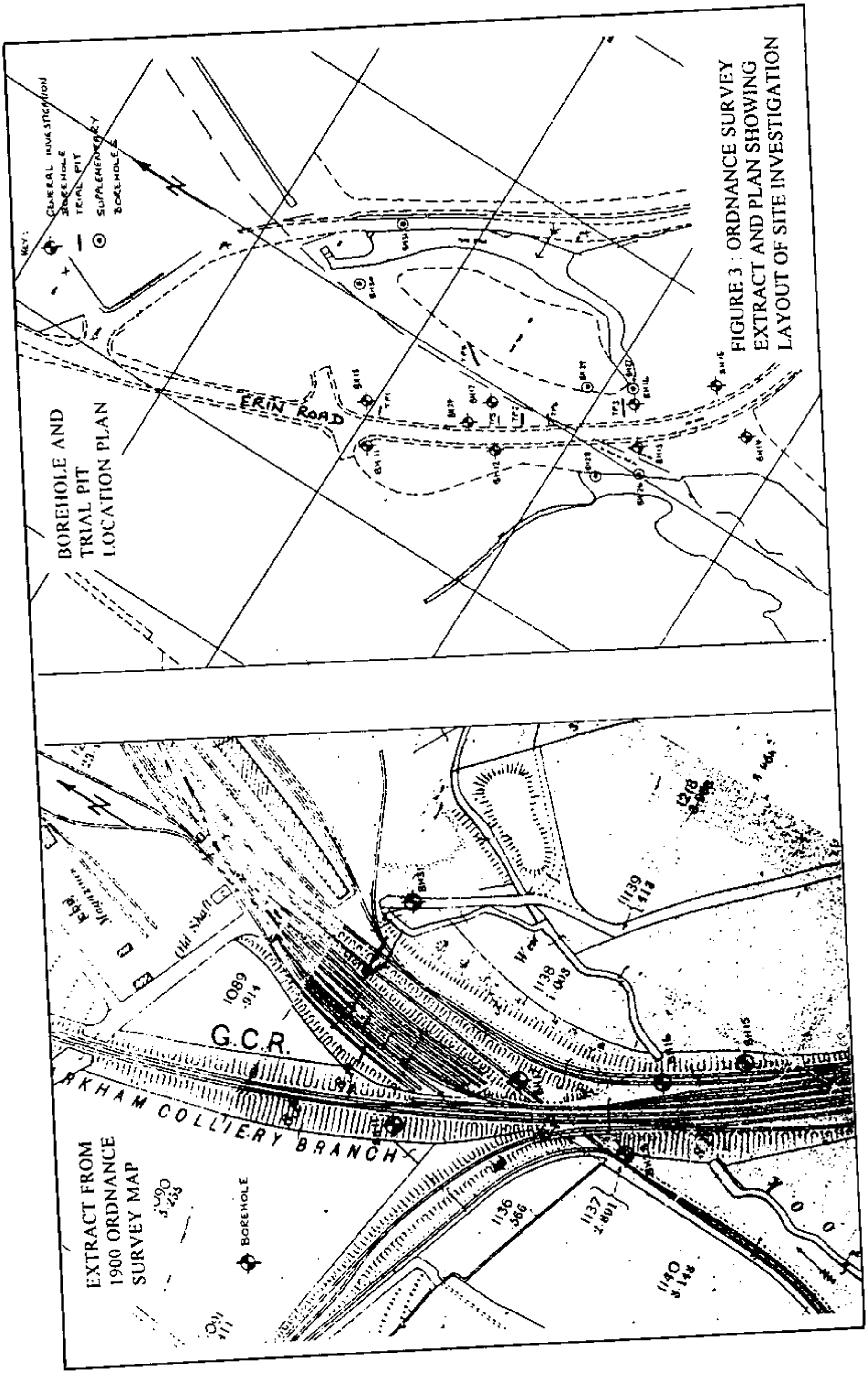


FIGURE 3 : ORDNANCE SURVEY EXTRACT AND PLAN SHOWING LAYOUT OF SITE INVESTIGATION

During the design phase, a supplementary borehole investigation established bearing capacity and ground conditions immediately adjacent to and beneath proposed structures with SPT tests at one metre depth intervals in each borehole to a depth at least five metres below the deepest proposed foundation level. Figure 3 shows locations of boreholes and trial pits for each investigation phase and includes and extract from the 1900 Ordnance Survey map.

FLOOD ROUTING.

Reservoir flood and wave standards set out in Table 1 of the 1978 ICE publication 'Floods and Reservoir Safety' for a category 'C' dam suggest that as a general standard the greater of the 1000-year flood or 30% of the Probable Maximum Flood (0.3 PMF) inflow should be routed through a reservoir and the outfall structure designed to accommodate sufficient discharge in order to prevent any overtopping of the dam. In cases where rare overtopping is tolerable a minimum standard is that the greater of the 150-year flood or the 0.2 PMF inflow should be routed through the reservoir. Although overtopping of the embankment dam was considered tolerable in this case, the extent of the earthworks necessary to contain the raised water levels during the 'flood' event at other locations around the periphery of the reservoir rendered this option unsuitable. It was decided that the general standard should be adopted in order to minimise the physical disruption likely to be caused by the necessary engineering works. Taking into account the existing crest level, and the wave surcharge allowance of 0.4 metres, adoption of the general standard effectively fixed the maximum water level and storage capacity of the reservoir.

Using methods identified in the Flood Studies Report, 1975, inflow figures were established for the 1000 year flood of 19.75 cumec and for the PMF, establishing a figure for the 0.3 PMF flow of 28.8 cumec. Flood routing calculations gave an attenuated discharge figure of 24.6 cumec. The capacity of the existing open channel downstream of the reservoir was confirmed using Mannings Equation and was found to be well in excess of the attenuated discharge. A longitudinal section through this watercourse is shown in Figure 4.

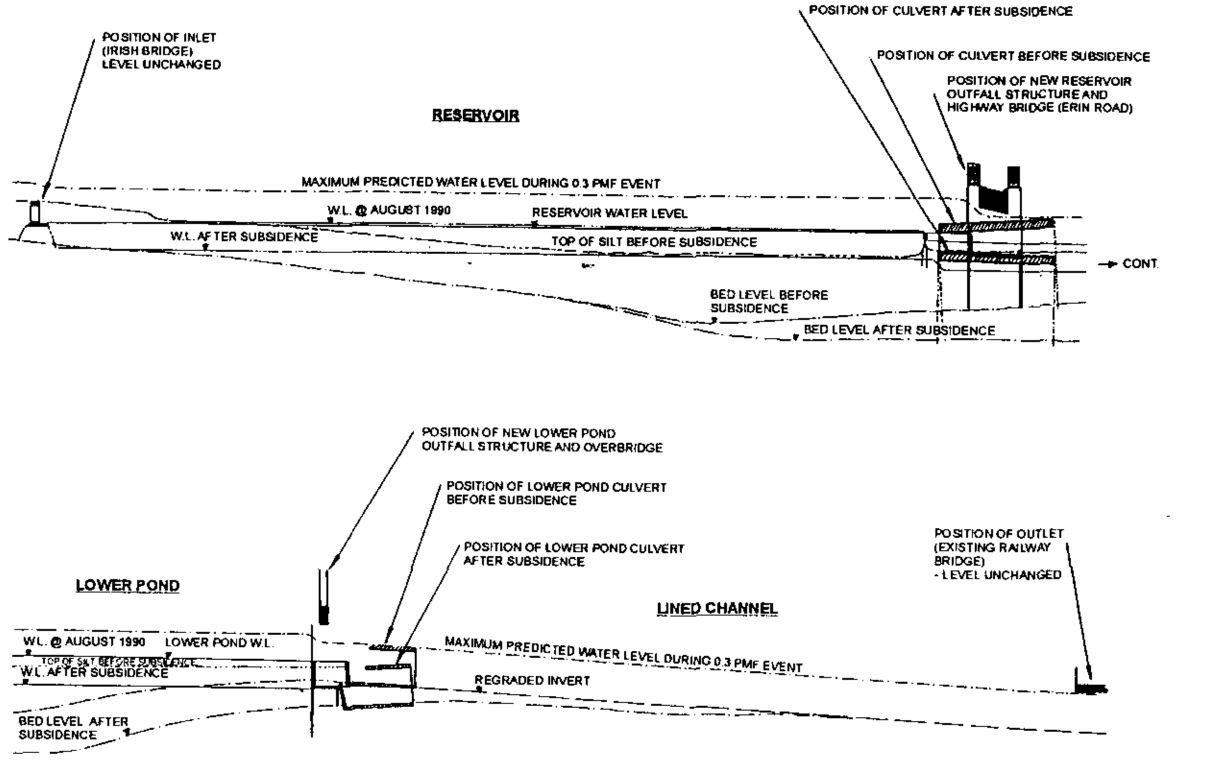


FIGURE 4. LONGITUDINAL SECTION THROUGH POOLSBROOK

ENGINEERING DESIGN.

Prior to detailed structural design it was vital to assess the suitability of the existing railway embankment for use as an earth dam. A typical cross section through the embankment and new structure location is shown in Figure 5. Using soil parameters identified during the site investigation the factors of safety against both circular and non-circular slip failure of the downstream embankment face were calculated at 3.3 for the steady state condition and 2.4 when simulating overtopping of the reservoir, with a fully saturated embankment and an empty lower pond; a condition unlikely to occur in practice. These figures were considered acceptable. Utilising a permeability of $1x10^{-5}$ m/s, flow net analysis of the embankment indicated likely seepage per metre run at between 0.07 and 0.08 l/s. These seepage rates were not considered high enough to present any threat to overall stability. To prevent piping failure and unravelling of the downstream face, a carefully selected geotextile membrane, anchored by a placement of 300mm thick stone rip-rap layer, was proposed to ensure retention of soil particles through the creation of a natural graded filter at each newly engineered embankment face.

This technique additionally provided protection against wave erosion and was adopted for the protection of both upstream and downstream embankment faces.

Assessment of the two existing culverts confirmed that replacement structures would be needed as both had been installed without measures to prevent seepage at the soil / structure interface and both were considerably undersize to accommodate the attenuated 0.3 PMF discharge of 24.6 cumec. The buried bridge structure beneath the railway embankment was found to be filled with uncompacted railway ash, brick and clay pipe rubble. To ensure the integrity of the new dam, the decision was made to remove both the existing bridge and culvert structures to allow construction of new 'core' elements to link existing structurally adequate sections of the former railway embankment. It was proposed to clean out, backfill and seal up the existing concrete culvert within the lower pond embankment.

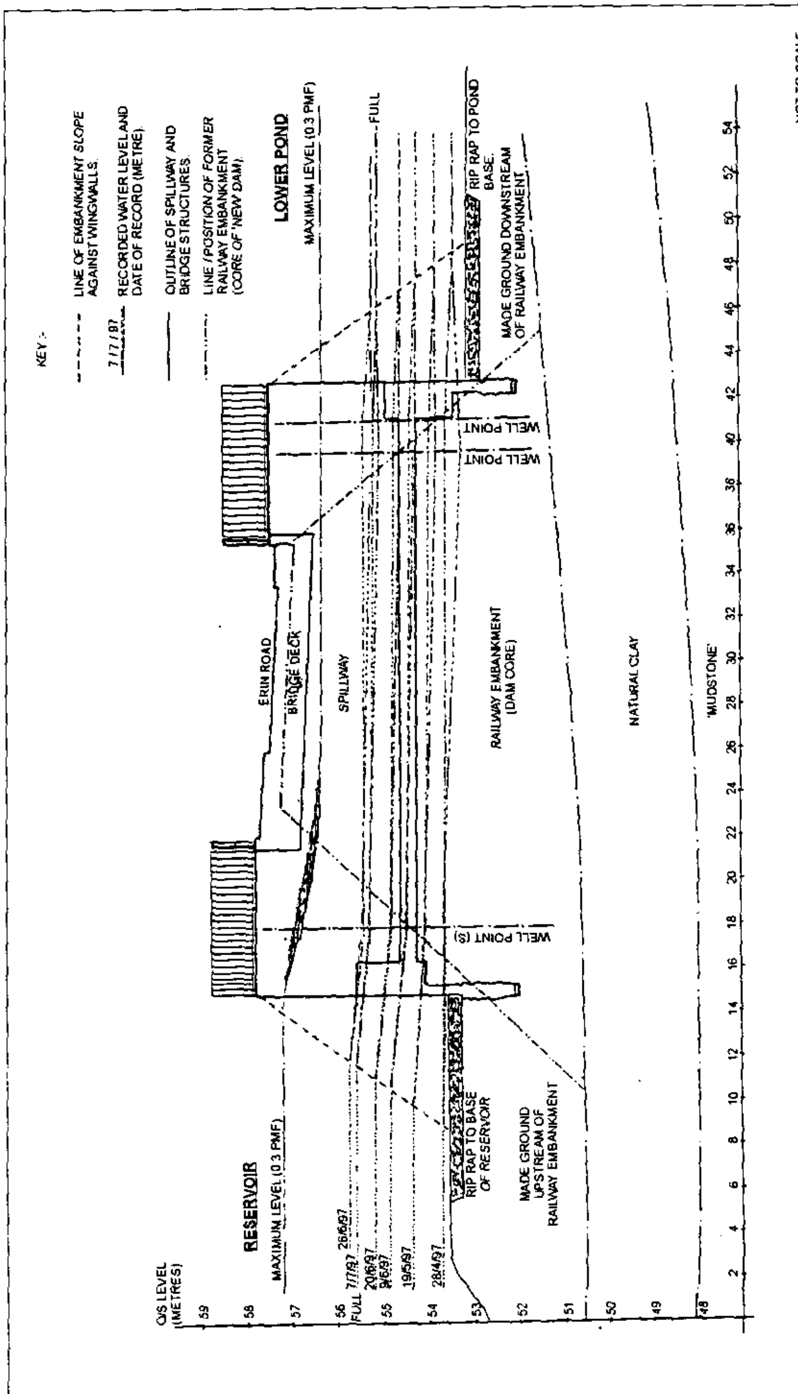


FIGURE 5. TYPICAL SECTION THROUGH EMBANKMENT AND NEW RESERVOIR SPILLWAY

Both reservoir and lower pond outfall structures were designed in-house by Derbyshire County Council. Constructing a simple box culvert through the embankment beneath Erin Road was not practicable due to the soffit level restrictions beneath Erin Road, which were likely to cause problems of inlet submergence and flow reduction during flood conditions. Limiting upstream water level rise to avoid submergence would have reduced the temporary reservoir storage capacity and therefore increased the discharge required to prevent overtopping during the 0.3 PMF event. To utilise temporary storage capacity of the reservoir whilst restricting water level rise beneath the road, the decision was made to provide a simple broad crested weir at the upstream end of a new spillway. Potential erosion downstream of the weir was overcome by the provision of a shallow stilling basin within the new spillway structure to ensure the redevelopment of linear flow in a controlled environment, whilst cut-off collars were proposed at each end of the spillway structure to control seepage. Similar principles were to be adopted for design of the lower pond outfall structure.

Successive iteration enabled a balance to be achieved between static water levels, reservoir storage capacity and the head of water upstream of each weir during the 0.3 PMF event. Both weirs were designed to accommodate in excess of 24.6 cumec to ensure that water levels would not exceed the previously determined maximum levels. Wing-walls were provided at each weir to retain a raised embankment height to enable the creation of viewing platforms and to improve seepage control alongside each new structure.

Construction of each cut-off was completed by the addition of curtain walls beneath each weir.

It was apparent that excavation and reconstruction of the embankment to allow construction of bridge abutments beside and beneath the spillway would not be consistent with the need for minimum disturbance to take advantage of the long term consolidation that had taken place within the embankment. An alternative which avoided disturbance of the embankment below the spillway base slab was the use of cast in-situ reinforced concrete piles socketed into the underlying 'mudstone' strata. This method was also adopted at the lower pond spillway to limit ground disturbance by excavation. Since bridge bearings would not be accessible, separate 'integral deck' bridge decks were designed to span each spillway in order to reduce the need for future maintenance. A significant advantage of this method was the substantial reduction in both dead load and highway live load which would otherwise have been transferred to each spillway base slab necessitating a significant increase in concrete section. Although further residual settlement was thought to be unlikely each bridge was designed to accommodate up to 10mm of differential settlement at any location within the structure, in addition to other highway loading requirements.

Approval in Principle for each of the structures was obtained from the Construction Engineer in accordance with requirements of the Reservoirs Act 1975. An additional Approval in Principle was required from the Highway Authority to ensure future adoption of the new bridge which was to carry Erin Road above the reservoir spillway. Licences and consents were obtained from both the Environment Agency and the Local Planning Authority and included a licence to impound water, consent for the construction of permanent structures within the watercourse and planning approval for the detailed construction proposal. The project was funded by investment obtained from English Partnerships under the Leasehold Reform, Housing and Urban Development Act 1993 and the Derelict Land Act 1982.

CONSTRUCTION.

A construction contract was awarded in August 1996 to Alfred McAlpine Construction Ltd of Retford, Nottinghamshire. Work began on site in September 1996. Early in the contract, water levels in both the reservoir and lower pond were reduced by pumping. Each new structure was located off the line of the existing watercourse and there was little problem with water in the construction areas. The buried bridge structure was excavated in its entirety. Strip footings beneath the abutments were found to consist of thick layers of sintered ash and clinker, were obviously highly permeable and were also removed. The void was backfilled in layers using local 'mudstone', compacted to 100% dry density or less than 5% air voids. Regular testing was carried out to ensure that adequate compaction was achieved. Construction of curtain walls was carried out in open trench excavations without shutters at each weir location. This method was used to help restrict development of preferential seepage paths beneath each structure by ensuring good contact between the new reinforced concrete walls and the existing embankment material. Once 'out of the ground', construction of each spillway was rapid, the majority of work being completed by December 1996. Installation of cast in-situ concrete piles was carried out in January 1997 and was closely followed by the construction of cast in-situ concrete integral bridge decks. The original culvert beneath Erin Road was removed and the voids carefully backfilled by placement and compaction of mudstone and previously excavated embankment material. Again regular testing was carried out to ensure that the necessary degree of compaction was achieved. Reconstruction of Erin Road along the crest of the 'new' dam, backfilling of the original downstream culvert, works to protect the open channel and exposed embankment faces against erosion and the installation of survey monitoring stations and well points completed the construction works in May 1997.

PERFORMANCE MONITORING.

Regular monitoring of open water levels and of water levels within the embankment was carried out following completion of construction works. Inspections were carried out twice weekly during initial re-filling of the reservoir and lower pond to ensure close monitoring of each of the new structures and embankments in case of seepage, settlement or cracking. Close inspection of water level records shows that changes in levels within the embankment follow but lag level changes within both the reservoir and lower pond. Results obtained over the eight months prior to preparation of this paper, in February 1998, indicate a significant head loss occurs within the upstream embankment section. This is entirely consistent with the theoretical flow net analysis carried out during the design phase. Within the downstream embankment section, head loss is less predictable. It appears that there is some development of seepage paths of different hydraulic gradient within the partially layered construction beyond or below that section of the embankment which was re-constructed immediately adjacent to the new spillway structure. Low hydraulic gradient within the embankment and the low seepage rates apparent when comparing inflow against water level changes during filling of the reservoir are unlikely to lead to unravelling of the protected downstream embankment face. Recent survey results indicate that minor settlement is occurring at each structure although level reduction is consistent throughout each structure. Since bridge and spillway elements are not structurally connected and rely on different foundations, the results could indicate that there is still some residual mining subsidence occurring seven years after coal extraction was completed. Future monitoring is necessary to establish a cause for the recorded changes in level of between three and six millimetres to date.

DISCUSSION.

Investigation of the site at Ireland Colliery during design of a reclamation scheme gave little indication of the extent of the flooded area alongside tip 11/038. Maximum water depth was measured at 1 metre and it was estimated that a further 1.5 metres of silt would require removal from the pond in order to create a lake suitable for water sports. There was similarly little indication of the scale of the existing railway embankment which had become buried over the majority of its length by tipping alongside each embankment face. The existing culvert did not appear to offer any restriction to discharge beneath Erin Road from the flooded area. The full extent and depth of 'flooding' was not established until dredging work began in 1992 when up to 3 metres of silt had to be removed to allow access by construction traffic during the dredging operation.

In this already heavily affected mining subsidence area, the magnitude of subsidence which affected the site in 1991, added to similar previous effects, led to gross and uneven variation of invert levels along the line of the watercourse. The need to reinstate water levels, in order to create open waters for recreational use, resulted in a requirement to impound an increased depth of water against the former railway embankment. Results obtained during monitoring following construction works in 1996/7 indicate that performance of the former railway embankment as an earth fill dam is satisfactory. The well consolidated, clay based, embankment 'core' combined with the volume of clay and tipped shale in place against much of the upstream embankment face, has resulted in a sufficiently stable and impermeable structure to allow its use as an earth fill dam. To date water levels have risen to only 200 millimetres above the reservoir weir crest level, and behaviour of the weirs, spillways and embankment has been as predicted. There is little evidence of seepage through the embankment structure and there is no visible evidence of soil particles being washed out at the downstream 'protected' embankment face into the lower pond. It is however likely that it will be necessary to continue monitoring for a further 18 months before a Final Certificate can be issued by the Construction Engineer as required by the Reservoirs Act 1975.

It is extremely unlikely that this is the only instance where an artificial embankment bisects the line of a natural watercourse. In the event of mining subsidence resulting in gross and uneven changes in ground levels, subsequent storage of water above the natural discharge invert level may, as in this case, result in large volumes of water and silt being artificially impounded. There are likely to be other instances where fluctuation in ground level has resulted in storage of water and which meet the criteria for registration under the Reservoirs Act 1975.

ACKNOWLEDGEMENTS
The authors would like to thank Derbyshire County Council for their agreement to the publication of this paper.

REFERENCES

Reservoirs Act 1975, HMSO, LONDON, 1975 Chapter 23.

The Institution of Civil Engineers, 1989, Floods and Reservoir safety: an engineering guide, I.C.E., LONDON, Second Edition.

Remedial works to upstream face protection, Megget Reservoir

D GALLACHER, R M DOAKE, D HAY-SMITH, Robert H Cuthbertson & Partners, UK

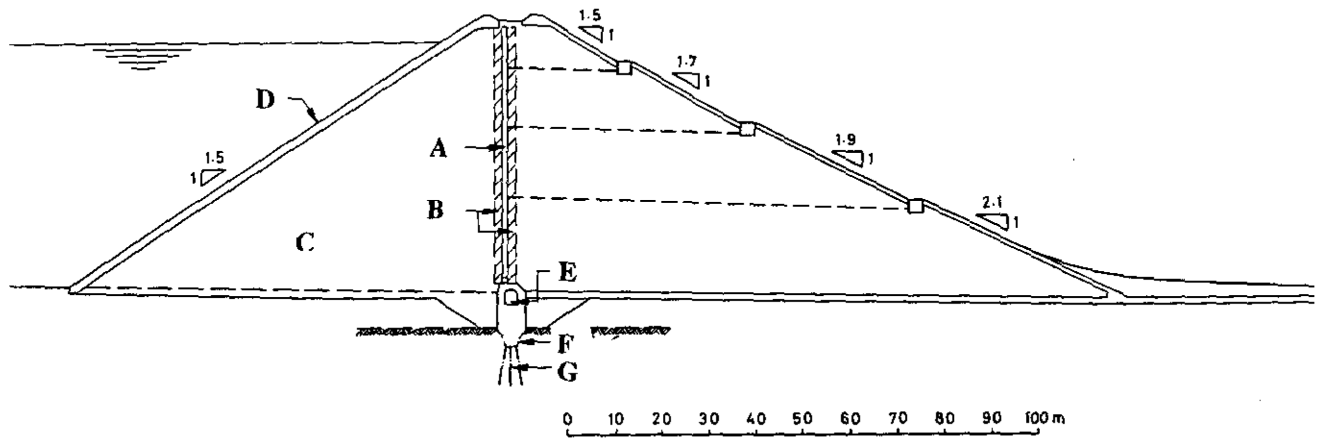
SYNOPSIS Damage to the rip-rap protection on the upstream face of Megget Dam has occurred since first filling in 1983 and independent wind-wave investigations have demonstrated that waves exceeded anticipated wave heights. Value Planning Studies for alternative schemes to upgrade the rip-rap protection indicated that bituminous grouting was the preferred option and its satisfactory performance was proved by site trials during May 1997. The bituminous grouting works for the upper part of the face was completed over a 12 week period (September-early November 1997) and the grouting works for the remaining area to be upgraded are programmed for May-September 1998.

INTRODUCTION

Megget Reservoir is situated in the Borders region of Scotland about 75 km from Edinburgh. The dam is a gravel fill embankment, 570 m long and 56 m high with a central asphaltic core. The area lies within a U-shaped glaciated valley about 4 km long. A cross-section of the dam is shown in Fig. 1. Damage to the rip-rap protecting the upstream face has occurred at unacceptable frequency since first filling of the reservoir. The long fetch generates substantial waves in the reservoir and independent wind-wave investigations have demonstrated that waves have periodically exceeded the design wave height.

Repairs were carried out by different methods on an ongoing basis to provide interim protection and information on performance. A *Report on Design and Performance of Rip-rap Protection on Upstream Face of Megget Dam with Recommendations for Upgrading and Repairs,* March 1995, was prepared by W P McLeish, Construction Engineer for the reservoir. His Report included recommendations in the interests of safety with regard to upgrading of the rip-rap protection covering interim measures and long term service. East of Scotland Water (ESW) appointed Robert H Cuthbertson & Partners in October 1996 to prepare a Value Planning Study for the upgrading of the rip-rap protection to ensure minimal maintenance in the long term. D Gallacher was appointed as Construction Engineer for the upgrading works and final certification of the reservoir.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998



(A) Asphaltic core
(B) Transition zones
(C) Gravel fill
(D) Rip-rap
(E) Control gallery
(F) Cut-off plug
(G) Grout curtain

Fig. 1 Cross-Section of Dam

VALUE PLANNING STUDY

General

Hydraulics Research Wallingford Ltd (HR) was included in the design team to utilise their particular knowledge of Megget conditions, having carried out wind and wave measurements at the reservoir (HR, 1986). T A Johnston, Babtie Group, was appointed by ESW as their Independent Reservoir Panel Engineer (IRPE) for the Value Planning (VP) Study and the follow-up works covering the design and construction of the preferred scheme to upgrade the rip-rap protection.

The VP Study considered alternative schemes for upgrading and repair of rip-rap protection and other schemes to reduce the effect of waves such as the installation of a floating breakwater to meet the technical requirements of the Construction Engineer (CE) and to give consideration to ESW's operational requirements and overall financial implications.

Two meetings were held during the study at which the CE acted as "facilitator". The meetings were attended by representatives of the design team including HR, ESW and the IRPE. The first meeting took place early in the concept phase when various options were reviewed and decisions were taken on which schemes should be retained for more detailed consideration. The second meeting took place towards the end of the concept phase when the retained schemes were considered in detail to determine the preferred scheme.

Design Studies

Wind-wave studies carried out by HR resulted in adopted wind speeds for consideration of alternative schemes being based on extrapolated figures produced by the Meteorological Office in 1989. It was also found that the simplified Donelan/JONSWAP wave prediction method provided consistently conservative results that compared well with data recorded on site and consequently this method was adopted for wave prediction.

Analyses of damage caused by winds of different durations together with the standard Met Office 7 hour storm show that the most damage is caused by winds of about 6 to 12 hours duration which is also comparable to the damage caused by the Met Office storm. It was also found that the damage can be conveniently modelled by taking the wave height generated by the mean hourly wind and applying it with three times the nominal number of waves generated in one hour which is about 3000 waves.

The standard of protection considered for new rip-rap at Megget embraced two elements: the design wind event and the corresponding acceptable damage level. The standard of protection was selected such that rip-rap would only sustain "onset of damage" (S_d=2 (Van der Meer, 1987)) during the 1:200 year event as recommended by the previous CE. It was

Decision criteria	Buildability	Construction staging	Net present costs	Routine Maintenance	Incidental Maintenance	Reliability	Drawdown required	Effect on water quality	Visual impact	Impact on recreation	Impact during construction	Total rating
Criteria reference	A	B	C	D	E	F	G	H	I	J	K	
Criteria importance	0.105	0.07	0.175	0.1	0.1	0.2	0.075	0.025	0.0525	0.0225	0.075	1.0
1. Upgrading of Existing Rip-rap	3 0.315	6 0.42	6 1.05	8 0.8	3 0.3	9 1.8	4 0.3	9 0.225	10 0.525	10 0.225	3 0.225	6.2
2. Flattening of Upstream Slope	1 0.105	2 0.14	2 0.35	8 0.8	6 0.6	9 1.8	5 0.375	4 0.1	10 0.525	10 0.225	1 0.075	5.1
3. Grouting Reinforcement of Existing Rip-rap	8 0.84	8 0.56	8 1.4	6 0.6	5 0.5	7 1.4	6 0.45	9 0.225	7 0.3675	10 0.225	6 0.45	7.0
4. Provision of Blockwork Protection	2 0.21	2 0.14	4 0.7	8 0.8	3 0.3	9 1.8	4 0.3	8 0.2	5 0.2625	10 0.225	5 0.375	5.3
5. Provision of Floating Breakwater	6 0.63	2 0.14	10 1.75	2 0.2	7 0.7	4 0.8	9 0.675	9 0.225	4 0.21	3 0.0675	7 0.525	5.9

Note: Upper figures in boxes are marks allocated from 1 to 10 in increasing order of merit.
Lower figures are scheme ratings, derived from product of marks and Criteria Importance

Fig. 2 Decision Matrix

considered that the adopted design return period event of 1 in 200 years would minimise maintenance requirements. The influence of a drought event on the reservoir level was considered together with the probability of wind events and this combined probability approach was adopted for embankment levels below 317 m OD, where a change in rip-rap type occurs. The latter level is 17 m below the Top Water Level and represents about the 50% storage level. Above this level, a uniform standard was adopted, i.e. the 1 in 200 year wind event.

It is estimated that the reservoir will reach a level of about 317 m during a 1 in 10 year drought under the Stage 2 development when the scheme is augmented with pumping from St Mary's Loch. Designing the rip-rap to a 1 in 20 year wind event below this level, therefore, gives a combined probability in excess of 1 in 200 years. The 1 in 20 year wind event was adopted for levels from 317 m down to 307 m and below 307 m OD no improvement to the rip-rap is considered to be necessary. The dam freeboard, taking account of the design flood (PMF) and the concurrent wave surcharge, is satisfactory for the existing rip-rap with regard to the standards given in the latest FRS guide, and it is also satisfactory if the rip-rap is grouted.

Alternative Schemes and Value Management Comparisons

Nine schemes were considered at the initial VP meeting and they included a *Do Nothing* option. The latter was rejected by the CE as he considered that this would prevent issue of a Final Certificate for the reservoir as there would be an unacceptable risk of damage which would be difficult to repair, particularly lower down the slope during extended periods of bad weather. The following five alternatives were taken through to the second VP stage:

Alternative 1 - Upgrading of Existing Rip-rap
Alternative 2 - Flattening of Upstream Slope
Alternative 3 - Grouting Reinforcement of Existing Rip-rap
Alternative 4 - Provision of Blockwork Protection
Alternative 5 - Floating Breakwater

For each alternative, the characteristics, construction methods and potential problems, were reviewed and the capital and maintenance costs and whole life costs were prepared. The value planning process identified the grouted rip-rap as the best value option by a significant margin, followed in turn by the new rip-rap layer and floating breakwater alternatives. The *Initial Decision Matrix* is shown in Fig.2 which includes values of criteria importance from the *Weighted Value Tree*. The ratings in the matrix of the alternatives covering (a) flattening of the upstream slope with the addition of a new rip-rap layer and (b) placing new blockwork on the upper section and removed rip-rap on lower section are substantially lower on account of their high capital cost.

The indirect cost effects from construction of the new rip-rap layer and grouted rip-rap alternatives was not taken into account in the assessment of the *Net Present Costs* for each scheme. This was an additional factor which favoured the grouted rip-rap alternative as the cost impact arising from reservoir operation would be substantially greater in the case of the construction of a new rip-rap layer from the longer construction period required for the latter.

The CE was satisfied that all alternatives except the floating breakwater met his requirements regarding issue of the Final Certificate under the Reservoirs Act. It was proposed in the VP Study that grouting trials be carried out for the grouted rip-rap alternative to confirm its viability and to provide information on grouting techniques for tender preparation. The concern with the floating breakwater alternative was the design of a mooring system which would require to be effective over the full reservoir range, and substantial maintenance would be required during the life of the structure.

Allowance was made in the recommendations of the VP Study for the adoption of Alternative 1 - Upgrading of Existing Rip-rap if the site trials for the grouted rip-rap showed that control of the grouting operation could not be depended on to provide adequate improved stability to the existing rip-rap to meet the recommended design criteria without a significant requirement for maintenance.

GROUTING TRIALS

Introduction

A contract for the grouting trials was negotiated with Hesselberg Hydro because of the specialised nature of the work and the short time available to meet the overall programme.

The upstream face of the embankment is constructed on a slope of 1V:1.5H (Fig. 1) and the rip-rap layer contains rock of two different specifications. Type A rip-rap refers to the rip-rap layer of 1.2 m thickness laid on the slope above 317 m AOD to the top of the upstream slope at 338 m OD. Type B rip-rap is laid below 317 m OD in a layer of the same thickness. Type C rip-rap refers to the first filter/bedding layer of 300 mm thickness, below which lies another 300 mm of filter material referred to in the original specification as Filter Type A. The original rip-rap specifications are given in Table 1. The weight W_{50} is calculated using a density of 2850 kg/m^3.

Filling of voids in rip-rap using bituminous grout binds together adjacent stones and increases the effective size (D_{50}) of the rip-rap, and the overall mass of the layer thereby increasing its resistance to wave attack. However, the porosity will be reduced and this can have a detrimental effect on the stability since part of the stability of rock armour derives from the dissipation of wave energy in the voids between the stones. Uplift pressures behind the grouted rip-rap layer can also be increased if adequate drainage through the layer is not maintained.

Table 1 Original rip-rap specifications

Rip-rap Type	Spherical diameter (m)				W_{50} (tonnes)
	Max	D_{50}	D_{15}	Min	
A	0.90	0.60	0.225-0.30	-	0.32
B	0.60	0.40	0.15-0.225	-	0.10
C	0.15	-	-	0.10	-
A (filter type)	0.075	0.0375-0.05	0.01-0.02	0.002	-

Pattern grouting of the existing face was adopted to obtain the benefits of void filling while mitigating the detrimental effects of grouting referred to above, by providing sufficient residual drainage areas to allow uplift pressures to be adequately dissipated. The method involves grout penetrating a whole layer of stone in a predetermined pattern, effectively forming lumps of stone of greater mass.

Two design methods were used to design the appropriate grouting pattern including minimum areas of grouting, degree of penetration and drainage requirements:

- Modified Hudson Method (see below)
- Blockwork stability method, HR Wallingford Report SR459

The Hudson formula is a much used empirical method to design the required size of rock armour to withstand wave attack which includes a damage coefficient factor (K_D). A modification to account for the effect of pattern-grouted rock is described in "The use of asphalt in hydraulic engineering" by the Dutch Technical Advisory Committee on Water Defences, Report 37/1985. The report recommends that the damage coefficient K_D may be increased by a factor of 5 - 7 for pattern grouted stone where approximately 60% of the surface of the layer is grouted. The modified Hudson formula was used as a primary check on the size of stone required within the grouted layer. Adopting a K_D increased by a factor of 5, the size of the existing rip-rap was found to be comfortably large enough to withstand the design wind-wave event, and therefore suitable for pattern grouting.

The design method for blockwork protection described in the HR Report SR 459, 1996, was used as a check on the adequacy of the proposed grouting grid pattern. The HR method identifies the minimum area of drainage, and the depth of penetration of the grout required for a given block size.

Fig. 3 shows the proposed 2.5 m nominal grid where the grouted strips represent 64% of the total surface area. Using the HR method, it was calculated that for the proposed grouted area, 70-75% penetration of the rip-rap layer would be necessary while the required drainage area would be 0.09 m^2 per panel. It was anticipated that effectively the full 1.2 m depth of the rip-rap layer would be grouted and additional grouting carried out within the main strips, either by adding supplementary strips or spot grouting, to ensure all significant stones were locked.

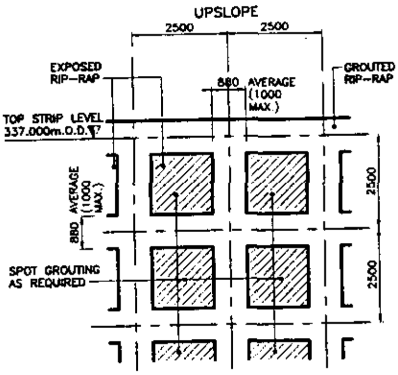


Fig. 3 Bituminous Grouting Grid Pattern

<u>Grouting Trials</u>

Grouting was carried out in three main panels, each 5.0 m x 5.0 m, to form three main strips vertically and three horizontally in each area. Four additional horizontal strips were also grouted to give a clearer indication of the spread of grout within the strips and at the rip-rap/cobble bedding interface.

Sand/stone mastic was used in the trials with bitumen of 100 Pen, limestone filler, sand and 10 mm aggregate. Two mixes were tried initially, with 10 mm stone added to the basic sand mastic in different proportions (Mixes 1 and 2). A third, stiffer mix (Mix 3) was used towards the end of the trial. The details of the mixes are scheduled in Table 2. The temperature of the grout when applied was in the range of 150 to 160°C.

Table 2 Asphaltic grout mixes on site

Mix	Bitumen %	Filler %	Sand %	10 mm Agg %	Grout viscosity Poise (140°C)
1	16.0	20.5	48.5	15	1500
2	14.5	19.0	44.0	22.5	2000
3	15.3	19.0	50.7	15.0	3000

The grout was applied to the trial panels using the specially designed mastic hopper which held a maximum of 2 tonnes. The hopper was loaded from a truck mounted boiler on the crest and positioned over the grid using a crawler crane. The following investigations were carried out during the trials:

(a) Different grout quantities and application rates were investigated including the effect of applying consecutive passes of grout "hot on hot" compared to staging the grouting where passes were applied "hot on cold".

(b) Grouted panels and adjacent ungrouted areas were excavated to observe the grout behaviour (penetration, spread and effectiveness of void filling).

(c) Pull-out tests were performed on both grouted and ungrouted stones to obtain a measure of the increase in strength due to grouting (rock anchors grouted into selected stones and loaded up to 2 tonnes, and different loading periods applied). Any movement of stones or uplift of the grouted mass was noted.

(d) Porosity of existing rip-rap was estimated by measuring excavated volume local to the test panels and weighing the stone.

(e) Residual porosity of grouted sections was estimated by taking account of grout spread and quantity placed, and original porosity.

The main findings from the grouting trials on the Type A rip-rap were:

(a) Penetration of the full 1.2 m rip-rap layer was obtained from all grout mixes, including from the first pass. The upper filter layer of cobbles below the rip-rap layer was not penetrated.

(b) Limited lateral spread of grout (about 0.5 m total) occurred in vertical panels, except where the arrangement of stones guided the grout flow in a greater lateral direction. The spread of grout down the slope was about 2.5 m from the lowest application point.

(c) Vertical spread from grouting horizontal panels was more substantial, although it could be controlled to leave adequate drainage areas.

(d) Resistance to uplift was increased by grouting by factor of at least 10 to 20. A stone weighing 0.1 tonne was pulled out after applying 2 tonnes for 10-20 seconds and once removed it was found that the stone was only bonded in 3 small areas.

(e) The grid of 2.5 m by 2.5 m was satisfactory for effective grouting of the rip-rap layer thickness.

(f) Target void filling of 80% for vertical strips, 67% for horizontal strips was obtained by staged grouting while maintaining adequate drainage. Void filling of 10% for ungrouted areas was adequate to lock-in stones between the main vertical and horizontal strips. The overall average quantity of grout was 0.475 t/m^2 of total surface.

(g) An application rate of 0.7 t/m of strip appeared to be appropriate (0.3 t/m in pass 1, 0.3 t/m in pass 2 ("hot on cold"), and 0.1 t/m in pass 3 ("hot on cold")) which represents 80% and 67% void filling for vertical and horizontal strips respectively, based on observed grout spread.

The grouting trials showed that the conditions required by both design approaches for stability of the grouted mass could be met by the proposed pattern grouting method. The general design of the pattern developed in the trial was satisfactory for grouting of the Type A rip-rap. The use of intermediate vertical strips or spot grouting with a stiffer mix or limited application rates for locking stones in the "ungrouted" areas required further development during the main works.

It was considered that the results of the trials on the rip-rap Type A may not be fully applicable for rip-rap Type B lower down the face, and it was planned that further trials be carried out during the main grouting works contract to determine any adjustments required to application techniques and rates for bituminous grouting.

On completion of the grouting trials, the CE recommended that Alternative 3 - Grouting Reinforcement of Existing Rip-rap be adopted as the selected scheme design to be carried forward to the Value Engineering (VE) Stage as it provided best value and met his requirements in terms of certification under the Reservoirs Act. The estimated contract value for the upgrading works based on information obtained during the grouting trials was £1.75M.

It was also recommended that as much as possible of the upgrading works be carried out in 1997 with the balance in 1998 after a break over the winter period. It was considered that the latter approach would provide additional protection on the upper part of the dam from further wave attack under extreme events over the winter period and give greater flexibility with regard to reservoir operation when extensive drawdown would be required in 1998. The above recommendations were accepted by ESW.

GROUTING WORKS

General

The Contract for the Rip-rap Upgrading Works was let in August 1997 to MJ Gleeson Group Ltd who are carrying out the work through their subsidiary Hesselberg Hydro 1991 Ltd. The Works were divided into two Stages (Stage 1 in 1997 and Stage 2 in 1998).

Stage 1 works comprise the upper zone of the face, from 337 m OD (1 m below the crest) down to a nominal 325 m OD (-13m). The centreline of the lowest horizontal grouted strip is 325.9 m OD and the gross surface area of the Stage 1 Works is about 9750 m^2. The Stage 1 works were subdivided into two phases of grouting, covering the nominal height ranges -1 to -7 m and -7 to -13 m. Stage 1 Works commenced on 18 August 1997 and were substantially completed on 8 November 1997.

Stage 2 works comprise the middle zone of the face, between 325 m OD and a nominal 307 m OD (-13 to -31 m). There is a change in rip-rap type at 317 m OD (-21 m) from the larger size grading Type A above this level to the smaller Type B. The gross surface area of the Stage 2 Works is about 16000 m^2. Stage 2 works are scheduled to commence at the beginning of May 1998 and continue for 22 weeks. This Stage was originally planned to be grouted in two phases, corresponding to the rip-rap types. However, the second phase has been further subdivided into two parts to postpone the final drawdown until as late as possible to assist with maintenance of supplies.

The principal items of Contractor's plant employed for the Stage 1 Works were two hot storage bays for half-mix, mixing plant, two wagon-mounted boilers, a track-mounted tower crane and two skips. A site laboratory was also established. The site staff comprised up to 5 engineers and technicians, including the Site Agent, and up to 8 operators, finishers, etc. The mixing plant, material storage and laboratory were located in and around the turning area at the north end of the dam crest, Fig. 4.

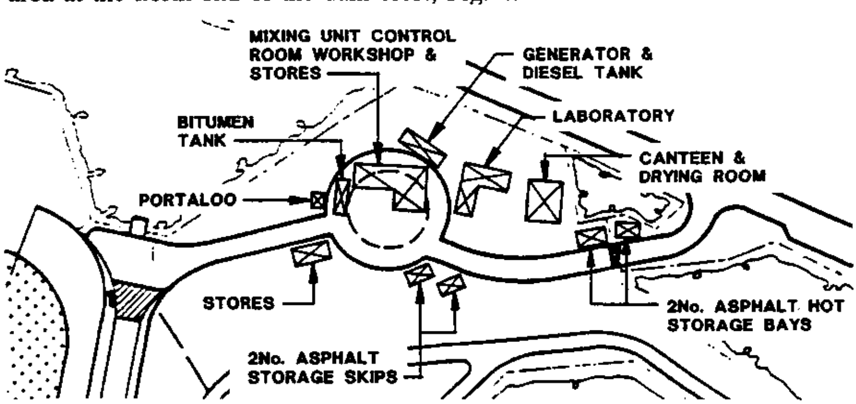


Fig. 4 Plan of Site Establishment

Preliminary Works

The bituminous grout is composed of a basic mastic asphalt with bitumen of 100 Pen, limestone filler, and sand, to which 10 mm aggregate is added to form the grout mix. The mastic asphalt was designed such that the viscosity at 140°C would lie between 800 and 1200 poise. When tested, the viscosity of the mastic was found to be 1150 poise.

The grout mix design is based on the mastic asphalt with the addition of 10mm aggregate. Two grout mixes were produced for the permanent works: Mix 1 is a "thin" mix used for the vertical strips of the main grid, Mix 2 is a "thicker" mix designed for use in the horizontal strips and spot grouting, Table 3.

Table 3 Composition of bituminous grout

Grout composition	Mix 1	Mix 2
Bitumen	14.8%	14.3%
Filler	16.3%	16.4%
Sand	48.9%	49.3%
10 mm Aggregate	20.0%	20.0%

Part production of the grout took place on site, with delivered "half product" being mixed with additional bitumen to form the grout mix. The half product contained all the filler, sand and aggregate required, bound together with a proportion of the bitumen (4-5% of half mix) and the remaining bitumen was added at the mixing plant on site. The layout plan for the Stage 1 grouting works for half the length of the dam is shown in Fig. 5.

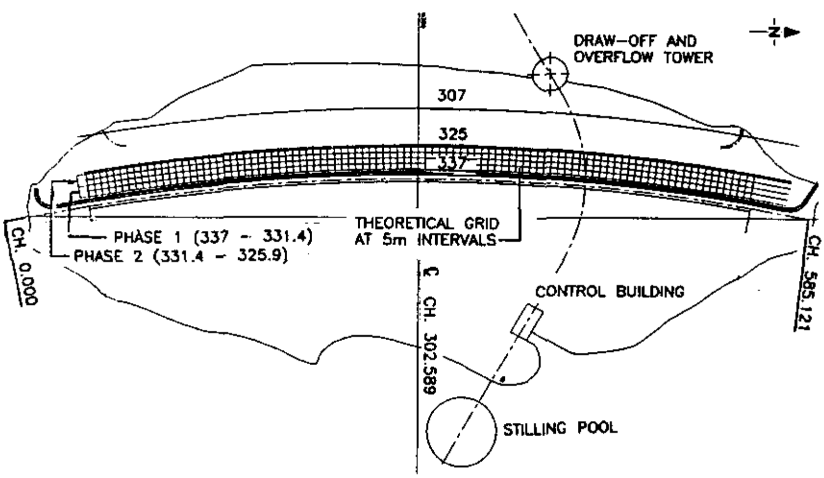


Fig. 5 Stage 1 Works - Layout Plan of Grid for Pattern Grouting

Vertical strips were made radial to the curved dam crest with the spacing set to be the specified width at the top level of the pattern (nominal level of 337.0 m OD). The radial layout gives minor divergences from the specified spacing of 1.2% and 3.2% at the lower ends of the Stage 1 and Stage 2 works respectively. Horizontal strips were set out by taking horizontal offsets from the centreline of the dam taking no account of either profile changes from the camber to the crest or from disturbance to the rip-rap.

Over part of the Stage 1 Phase 1 Works (337 to 331 m OD), small stones had previously been blinded into the surface to reinforce the rip-rap layer. They were mainly removed using an industrial vacuum unit with some additional hand removal as required to allow good grout penetration. There was also a considerable overlay of disturbed stones from the upper part of the face over areas on the upstream face within the Phase 2 Works (331 to 326 m OD). The objective in the latter areas was to effectively grout the original rip-rap layer with limited grouting of the overlying stones and some stones were moved to achieve good grout penetration. Areas of the face below the original profile were filled by placement of overlying stones from other areas where possible or by placement of imported rock.

Initial application trials were carried out to check that the grout mixed on site from delivered half product and bitumen behaved in a similar way to that observed during the earlier site trials. The trials covered vertical and horizontal strips, and spot grouting. Three 10 m long vertical strips were grouted in 2 passes on consecutive days, with a total average application rate of between 600 and 700 kg/m. Excavation of a trench adjacent to the strips showed that full penetration of grout to the cobble layer had occurred after the first pass. A further trench was excavated following the second pass and good void filling of approximately 80 to 85% was observed. It was concluded for the vertical strips that the grout mixed on site behaved similarly to the mixes used in the grouting trials, and target application rates set after the trials were satisfactory for the vertical strips.

A trench was also excavated to expose part of a previous horizontal strip grouted during the earlier site trials to check the spread of grout down the slope. This revealed that it was excessive and had completely sealed the drainage layer. Further trials were therefore carried out to test application rates lower than the original target of 700 kg/m and using a stiffer mix. This required use of a grout hopper with a wider throat to ease delivery and application rates of about 200 to 400 kg/m over two passes were tested. Investigations showed that full penetration of the rip-rap layer was still achieved and the maximum spread from the centre line of the strip was about 1 to 1.5 m. Void filling was observed to be slightly higher in the upper section of the layer, but on average was estimated to be 50%. A reduced target application rate for the horizontal strips of 300 kg/m using the stiffer Mix 2 was adopted.

Spot grouting was investigated by grouting two strips with about 50 and 100 kg/m of Mix 2 and excavating through the strips. This showed limited penetration of about 0.5 - 1.0 m and low void filling. The grout coated the stones rather than filling voids between them and the trials proved that penetration could be restricted, preventing sealing of the drainage layer.

Grouting Works

Vertical strips were grouted in three passes on separate days using the thinner Mix 1 with target production rates of 300 kg/m for passes 1 and 2 to provide full penetration and void filling and a third "top-up" pass of 100 kg/m. Horizontal strips were grouted using the thicker Mix 2 in 2 passes and initially these were placed on separate days. Later, to increase production rates, the second pass was placed on the same day with a delay between passes to allow adequate cooling of the grout in the first pass.

The general approach adopted to production spot grouting was to identify critical loose stones, and apply limited quantities of grout to contacts between these stones and adjacent grouted or ungrouted stones, to form a stable mass without sealing the surface. Thermocouples were inserted into selected panels prior to grouting to check that excessive penetration did not occur to full depth. Selected groups of grouted panels were also tested by applying water at around 10 l/s to check that adequate drainage had been maintained.

The average application rates for grouting are given in Table 4:

Table 4 Average Grout Application Rates

Location	**Rate**	**Remarks**
Vertical strips	680 kg/m	Including top-up of grid during spot grouting
Horizontal strips	340 kg/m	Including top-up of grid during spot grouting
Spot grouting	50 kg/m^2	50% of panels with average of 100 kg/m^2
Total	400 kg/m^2	Overall grouting rate - kg/m^2 of surface area

Testing

Grout performance tests in addition to standard material production conformance tests were carried out as follows:

(a) Small Inclined Plate Test: Standard flat plate tests were carried out to investigate the behaviour of the grout, and in particular, its stability on a 1:1.5 slope. This test was developed to measure stability of asphaltic concrete and, as such, gives only a guide to the behaviour of asphaltic grout. Two sets of tests were carried out, with two samples being tested in each case, all containing 20% stone.

Two samples were placed or cast on a steel plate, and the other two on a cobble layer to simulate site conditions. The sample which was cast on a cobble layer fixed rigidly to a steel plate was considered to most closely represent actual site conditions and its results in terms of stability were considered to be satisfactory (0.5 mm after 7 days at 30°C).

(b) Large Box Test: A sample box was designed to allow a large scale model of the rip-rap protection to be tested for stability. The rip-rap was grouted and the stability of the grout under various ambient temperatures was investigated. Details of the test layout are shown in Fig. 6. The temperature within the box is controlled thermostatically by an element heater feeding warm air into the upper section. Grout containing 20% stone was poured over the 400 mm thick rip-rap layer in a 1 m wide strip, to simulate the vertical strips of the design grid. Movement of grout local to the cobble layer was measured from the ends of 2 metal rods set transversely in the grout by extending the rods through the box sides. Particular care was taken to ensure that rod movement was not restricted directly by rip-rap stones.

The air temperature inside the box was established before the steel rods were released and any displacement caused by movement of the grout recorded every 24 hours. The temperature of the grout and fill below slabs was also recorded daily along with the air temperature within the box and the results of the test are shown on Fig. 7. The results indicate that the grout within the depth of the rip-rap will be stable under the wide range of ambient temperatures anticipated at the dam. It is accepted that there may be some creep on the top surface of the grout giving the appearance of an elephant's skin.

(c) Adherence Test: A test was designed to investigate the adherence between the rip-rap and the grout, and the effect on the adherence of placing the grout over wet and dry stones. A steel box 750 x 750 x 750 mm was fabricated to enclose the test sample and restrain the stones not being tested. Three layers of selected stones approximately 250 mm deep were placed in the box with the test stone (about 250 mm dia) placed centrally in the top layer and weighing about 15 kg. Grout was applied to dry and wet stones to grout in the test stones for individual tests. The test stones were subjected to cyclic loading to simulate uplift forces exerted on the rip-rap layer during wave attack. Good adherence was obtained between the rip-rap and the grout for both the dry grouted (27 kN) and wet grouted (23 kN) stones, and failure in both cases resulted from splitting of the test stones rather than by adherence failure of the surrounding grout.

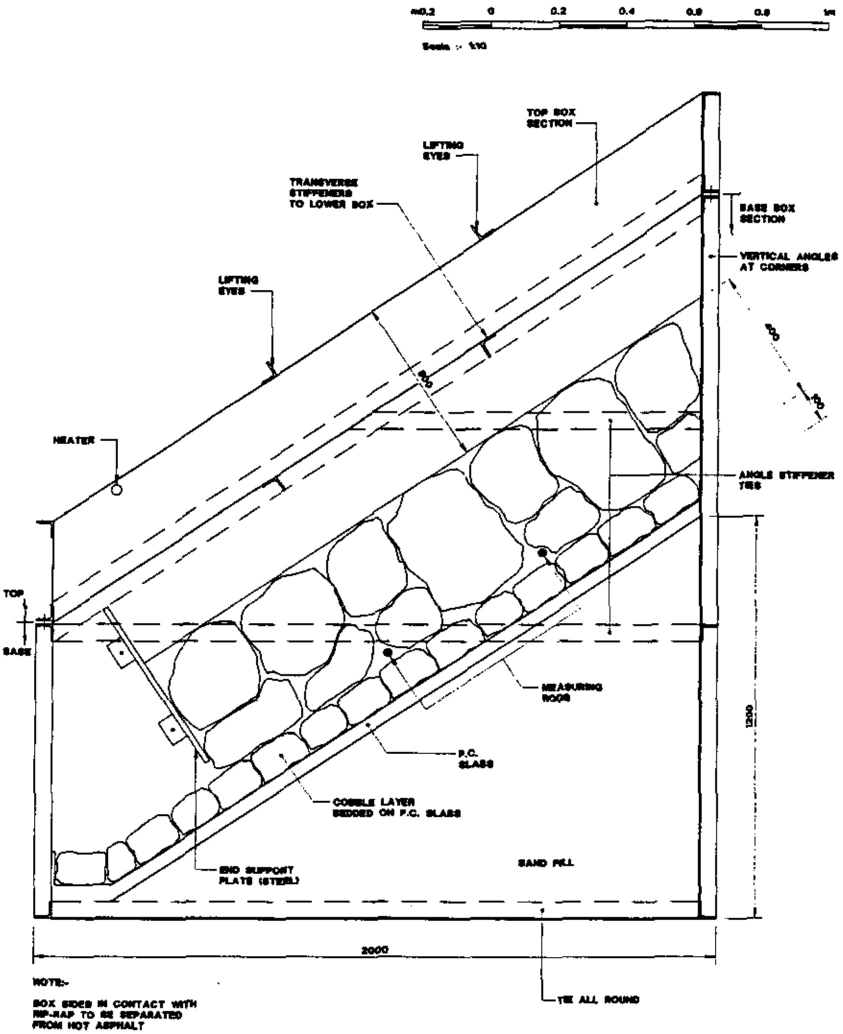


Fig. 6 Layout of Large Scale Stability Test Box

(d) Grout Penetration, Void Filling and Spread: Trenches were excavated to the full depth of the rip-rap layer to expose grouted strips and allow visual inspection of penetration and void filling. Assessments of porosity were also made from the excavations which gave slightly lower percentages (23% and 20%) compared with the average 27% found during the earlier grouting trials. Attempts to measure porosity by non-destructive means by taking conductivity measurements were too variable to give useful information. The spread of grout over the cobble bedding layer was monitored using thermocouples attached to stainless steel tubes fed through the rip-rap layer. About 330 thermocouple readings were taken and grout was detected in only about 15% of the tests showing that drainage through the rip-rap layer had been maintained. The adequacy of drainage through the grouted rip-rap layer was further proved by water application tests involving discharge of relatively large quantities of water onto the rip-rap surface.

Progress of Works

The Stage 1 Works were completed on programme in early November 1997 and the total amount of grout placed was 3,693 tonnes compared with 7,000 tonnes allowed for in the contract documents. This reduction is due mainly to 14% reduction in overall grouted area due to gravel deposits at the mitres between the upstream face and the abutments, and lower grout quantities placed in the strips (principally horizontal) from the target rate of application identified by the earlier site trials. The value of the Stage 1 works at the tender, excluding contingencies, was estimated to be about £675,000 and this was reduced to about £480,000 due to the above savings.

The Construction Engineer was satisfied with the Stage 1 works and a revised Preliminary Certificate was issued under the Reservoirs Act to cover control of water level and supervision of the upstream face until issue of the Final Certificate for the reservoir. The reservoir level will be lowered to allow the Stage 2 works to commence in early May 1998 and completion of the works is programmed for October 1998.

ACKNOWLEDGEMENTS

The paper has been written with the kind permission of East of Scotland Water. The cooperation of Hesselberg Hydro in the development and construction of the works is also acknowledged.

REFERENCES

HR WALLINGFORD, 1986. Wind and wave measurements at Megget Reservoir, HR Report No EX1477.

VAN DER MEER, 1987. Stability of Breakwater Armour Layers - Design Formulae, Coastal Engineering, 11, 219-239.

A drained synthetic geomembrane system for rehabilitation and construction of dams

A M SCUERO, Carpi Tech, Switzerland
G L VASCHETTI, Geosynthetics Hydro, Italy

SYNOPSIS. The use of synthetic geomembranes as a remedial measure to restore imperviousness on deteriorated dams has been pioneered and developed mostly in Europe. The paper compares drained and undrained systems, and illustrates the state of the art installing a drained PolyVinylChloride (PVC) composite membrane on the upstream face of the dam. The system has been adapted to construction of embankment and RCC dams: exposed and protected configurations are compared. The case history of Bovilla embankment dam in Albania reports advantages obtained by changing the original CFRD design to an impervious geomembrane system.

INTRODUCTION

The use of synthetic geomembranes as impermeable elements on the upstream faces of dams has been pioneered in Europe. The first installations were undertaken on embankment dams, with placement on a gentle slope representing generally an easier task than on near vertical faces. Different materials were adopted, the first installation employed a butyl rubber geomembrane (Contrada Sabetta, a rubble masonry rockfill dam in the south of Italy, 1959), followed by PolyVinylChloride (PVC) membranes (on the upstream face of Dobsina earthfill dam, Slovakia 1960), bituminous membranes (Banegon earthfill dam, France, 1973), and impregnated geotextiles (Trnavka earthfill dam, Czech Republic, 1981).

With improvements in quality of material and techniques of installation geosynthetics have been applied to near vertical faces. Starting with concrete gravity dams in the early 1970s (Scuero & Vaschetti, 1997), the system has now been applied to all types of dams. Today, Europe is the leader in the field of geosynthetic application for waterproofing hydraulic structures, for quantity of materials installed, for sophistication of the design and for efficiency of the solutions. Of the 70 dams incorporating a geomembrane reported by ICOLD (Corda et al., 1991), 52 are located in the Europe. Since 1993, the European Working Group on Geomembrane and Geosynthetics as Facing Materials has further investigated installations in Europe. Table 1 summarises some of the information collected.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

Table 1. Application of waterproofing geomembranes on European dams (1997)

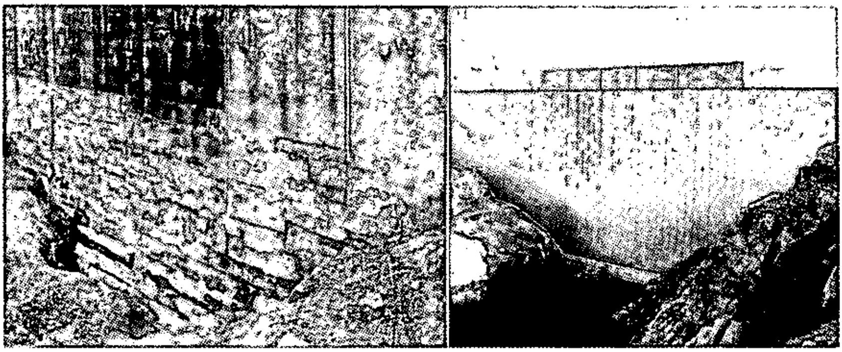
Country	Number of dams	Type of dam	Type of geomembrane	Quantity installed (m²)
Albania	1	1 earthfill	1 PVC	9,000 PVC **9,000** total
Czech Republic	5	2 earthfill 2 rockfill 1 earth/rock	3 PVC 2 im. GT	103,250 PVC 9,860 im. GT **113,110** total
France	25	11 rockfill 9 earthfill 2 gravity 1 arch 1 mult. arch 1 RCC	13 bitum. 9 PVC 2 butyl 1 im. GT	59,366 bitum. 51,310 PVC 5,940 butyl 16,000 im. GT **132,616** total
Germany	3	1 earthfill 1 rockfill 1 gravity	2 PVC 1 HDPE	2,500 PVC n. a. HDPE **2,500** total
Italy	24	10 gravity 8 earthfill 2 arch 2 mult. arch 1 buttress 1 rockfill	23 PVC 1 bitum.	216,329 PVC 30,000 bitum. **246,329** total
Portugal	2	1 rockfill 1 buttress	1 im. GT 1 PVC	8,000 PVC 73,000 im. GT **81,000** total
Slovakia	2	2 earthfill	2 PVC	10,850 PVC **10,850** total
Switzerland	2	2 gravity	2 PVC	4,450 PVC **4,450** total
United Kingdom	6	5 rockfill 1 earthfill	6 bitum.	

bitum. = bituminous
im. GT = impregnated geotextile
PVC = PolyVinylChloride

UNDRAINED AND DRAINED MEMBRANE SYSTEMS

Undrained membrane systems attach the impermeable liner to the upstream face of the dam by gluing it to the substrate. The major disadvantage of this method is the lifting action exerted by trapped water which has migrated from the dam body towards the warmer upstream facing. Especially in case of formation of ice or vapour, this phenomenon causes excessive stresses which can lead to detachment and failure of the liner, with subsequent water infiltration.

In drained membrane systems, the liner is an independent layer secured to the upstream face by mechanical anchorage. All water present behind the liner can flow by gravity in a drainage gap at atmospheric pressure towards a drainage collector and discharge at the heel of the dam. Pore pressure is thus relieved and stresses do not build up behind the liner in the event of rapid drawdown of the reservoir. Progressive dehydration of the dam can be achieved with this system and monitoring the drain water can indicate the efficiency of the system. A drained system is considered superior to an undrained system in modern practice, and 55 out of the 65 European dams recorded by the cited Working Group incorporate a drained system.



Figs. 1 and 2. Lago Nero dam pictured before and after installation of a drained 2.0 mm PVC composite membrane over a deteriorated facing, in 1979. Samples tested by the Owner after 16 years of exposure at 2031 m of elevation assessed <10 % deterioration, insuring long service life

THE STATE OF THE ART DRAINED MEMBRANE SYSTEM

A low permeability synthetic geomembrane constitutes the barrier to water infiltration. The geomembrane covers the entire upstream face of the dam, bridging expansion and construction joints, existing fissures and discontinuities. It is good practice to connect the system waterproofing the upstream face with the system waterproofing the foundations. The concept and a possible construction configuration are illustrated in Fig. 3.

Watertightness can be obtained by a very thin liner, of the order of mm thick. The most commonly used material is PVC due to its flexibility, resistance and elasticity which enable it to resist opening of cracks caused by alkali-aggregate reactions, settlements, seismic events. Its durability has been proven on concrete gravity dams at high altitude with high UV exposure for more than 20 years (Cazzuffi, 1996).

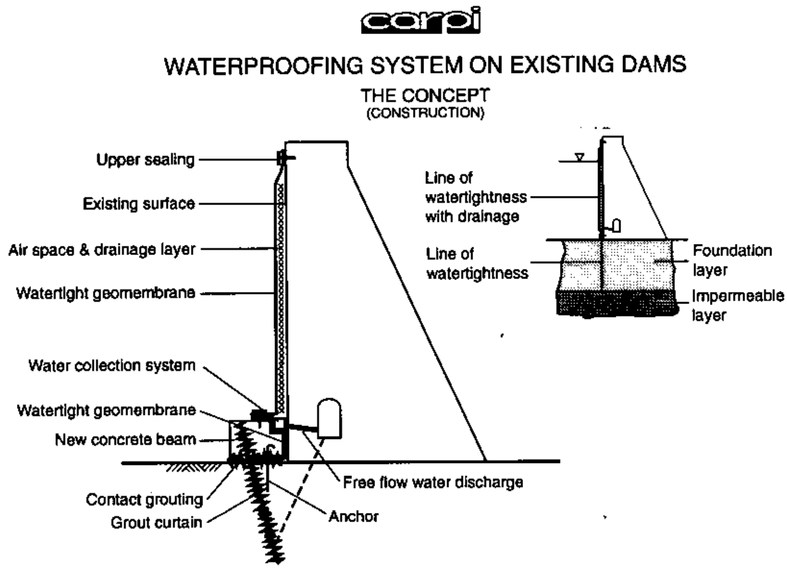


Fig. 3. The state of the art drained system

Rehabilitation of deteriorated dams

Deterioration due to ageing affects all types of dams either as a "natural" phenomenon due to environment aggression and poor resistance of the materials used for construction (temperature changes associated with wetting-dehydrating and freeze-thaw cycles, impact by waves, ice, debris, chemical action of water), or as a "pathological situation" (problems at foundations, differential settlements, expansive concrete). With deterioration, permeability increases, water infiltrates the structure, washing of fines and carbonation can cause clogging of the drainage system. When the drains do not perform efficiently, saturation can extend to the whole structure, pore pressure increase in the dam body and the stability of the structure can be jeopardised. Rehabilitation aims to stop water infiltration and to restore the dam's initial low permeability and stability.

Traditional rehabilitation materials and methods have some drawbacks: relevant construction times and costs (concrete, metal sheets), poor resistance and susceptibility to rapid start of new deterioration (concrete, shotcrete), lack of constant quality and difficulty of installation on steep slopes. A drained synthetic membrane system, properly designed and installed, can constitute a viable alternative, improving the stability of the dam by its dehydration capability.

The state of the art system, conceived and further developed in Italy, by CARPI, and illustrated in ICOLD Bulletin 78 (Corda et al., 1991), was developed for application on demanding near vertical faces. Of the 23 concrete dams recorded in Table 1, 21 were waterproofed with this system using a PVC membrane.

The solution includes a drainage collection and discharge system, an anchorage system, an impermeable liner and a perimeter seal. Preliminary operations include removal of debris and any unstable parts of the facing to expose a sound substrate for anchorage. No extensive surface preparation is required.

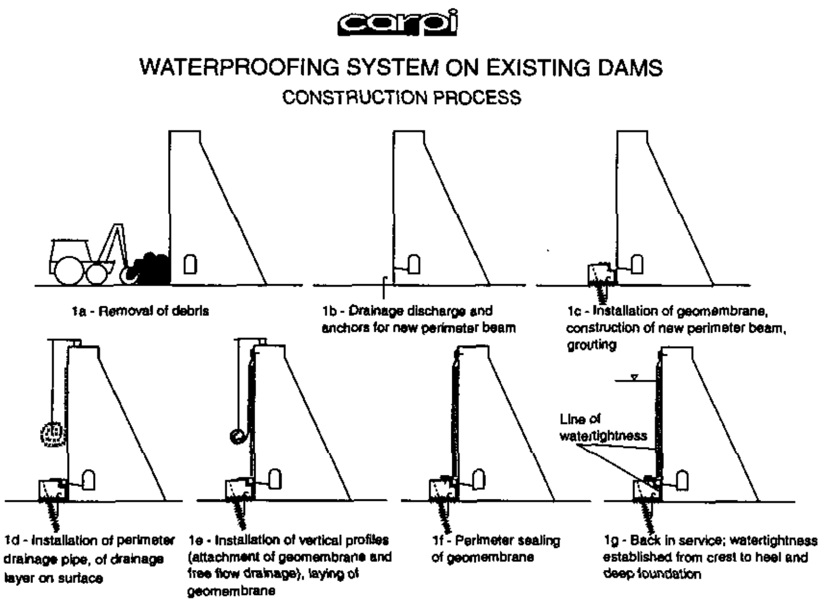


Fig. 4. Construction process. Simple sequential steps minimise interference with dam operation and schedule risks

The bottom connection upstream face/foundation waterproofing and the drainage collection and discharge system are installed first at the heel of the dam. One among the possible configurations is illustrated in Fig. 4, b-d: transverse pipes are constructed to discharge either in the gallery or downstream, the perimeter beam is waterproofed and prepared to accept the perimeter seal, pipes for longitudinal drainage collection are installed along the bottom perimeter over location of the perimeter seal.

A synthetic material having high in-plane transmissivity is typically installed over the face of the dam to facilitate water flow towards bottom collection and to provide planarity over small irregularities. The drainage layer which can be a geotextile or a geonet, conveys water towards vertical drainage conduits constructed with the anchorage system.

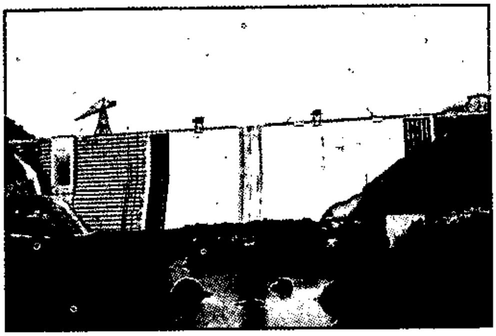


Fig. 5. Pracana dam showing installation of anchorage profile/drainage conduits, drainage geonet and PVC liner from left to right. The waterproofing system was installed to mitigate the effects of AAR

The anchorage system typically consists of two vertical stainless steel strips (Corda et al., 1991) and performs three functions. It secures the liner to the dam face, it tensions it to avoid slack and folds that could affect its stability and it provide vertical drainage. The internal profile of assembly (3) is secured first to the dam facing (1), as shown in Fig. 6.

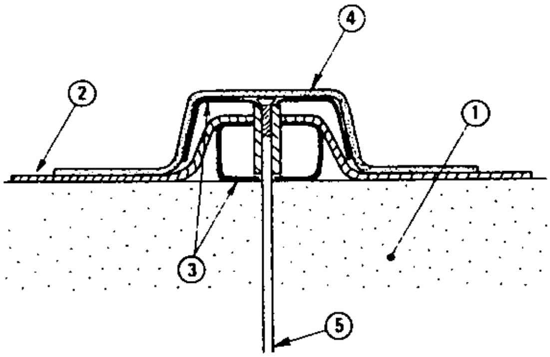


Fig. 6. Configuration of vertical anchorage and tensioning system as reported in ICOLD Bulletin 78

This anchorage system has been applied to all types of concrete and fill dams. On fill dams, the anchorage has been installed when the substrate had the necessary soundness to allow securing of the batten strips. The Bovilla case history describes how anchorage was accomplished on a different project.

The PVC liner is supplied in sheets, manufactured to cover the face of the dam avoiding transverse junctions. Modern practice prefers the use of composite PVC liners, coupling the impermeable PVC geomembrane to a heat-coupled, nonwoven, needle punched geotextile. The geotextile provides additional dimensional stability and anti puncture protection for the geomembrane so that it can accommodate rough substrates, and facilitates water drainage by its high in-plane transmissivity. In case of very aggressive substrates, such as on sharp rock masonry, additional geotextiles have been installed for extra antipuncture protection to avoid surface preparation.

PVC geocomposites are engineered materials: they are designed site specific in respect to type and thickness of geomembrane, and to type and weight of the coupled geotextile. It is important that a homogeneous material is used, providing constant resistance to UV and environment aggression throughout its whole thickness.

The PVC sheets are installed vertically or horizontally, depending on site requirements. Adjacent sheets (2) overlap over the internal profile already installed, are heat-welded along their edges, and are covered the external profile of assembly (3). A three components device (5) clamps the two profiles of the assembly, to accomplish anchorage and tensioning. To avoid any water intrusion through the threads of the fastening device (5), a PVC cover strip (4) is welded over the profiles assembly.

PVC membranes are thermoplastic materials characterised by reliable and easy field welding. Design can be based on rolls of reduced width, easier to transport, handle and place, and welding can be quickly accomplished and controlled, also in adverse weather conditions.

The perimeter seal is watertight at all peripheries which can be submerged by water. Typically watertightness is obtained by uniform compression of the liner against the perimeter beam, or the dam face, by stainless steel profiles and gaskets on a perfectly plane substrate.

The system has been used to control alkali aggregate reaction at a number of dams. At Pracana dam, in Portugal, the system was part of a major rehabilitation project. It was adopted as a safety precaution to prevent water infiltration in the event of new cracks forming. The rate of concrete expansion was high and the dam was built in a seismic area. At Chambon dam, in France, installation of the membrane was meant to increase stability of the upper part of the dam in the event of an earthquake as it avoided the risk of uplift and avoided leakage near the slots cut in subsequent years to relieve stresses (De Beauchamp, 1995). At Illsee dam in Switzerland the PVC membrane was preferred to shotcrete or reinforced concrete solutions for ease of construction and performance.

PVC geomembranes have now be installed underwater with the reservoir in operation. Recently CARPI completed the first project of installation of a geomembrane underwater, on Lost Creek dam, in California. This project demonstrated that leaking problems on dams can be solved by installing a geomembrane without affecting the operation of the reservoir and potential loss of resource in terms of power generation or water supply (for drinking or irrigation purposes.

New construction

Cofferdams, embankment dams and RCC dams incorporate PVC membranes as the only waterproofing element. Exposed and covered systems are available. Exposed membranes can be inspected and any accidental damage easily repaired. Covered membranes are protected against damage or deterioration and a ballast or protection layer entails additional design for stability of the layer, longer construction times and costs, and higher risks of damage during construction.

Embankment dams. In this type of dam, a PVC membrane on the upstream face has some additional advantages:

- a flexible material can maintain watertightness when differential settlements of the fill occur and can prevent migration of fines, erosion and piping .
- a synthetic material can make construction possible when natural construction materials are not available at reasonable cost on site.

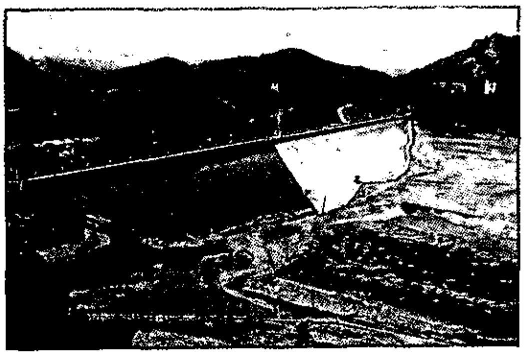


Fig. 7. At Sa Forada embankment dam (Italy) the exposed membrane system was installed to rehabilitate a bituminous concrete facing which had deteriorated after 20 years of service, despite repairs carried out every 10 years

To date, the highest embankment dam in the world using a geomembrane as the only waterproofing element is Bovilla dam in Albania (Sembenelli, 1996). Bovilla dam is 91 m high from crest to the lowest point in the foundation and is located in a very narrow and irregular valley north of Tirana. The upstream face of the dam has a very steep slope, 1.6 horizontal to 1 vertical. The initial purpose of the dam was to supply irrigation water but it was later required for potable water and hydropower supply. A CFRD was the original design type: the fill consisting of alluvial materials of 5 different gradings was to have a heavily reinforced (130-150 kg/m^3) concrete revetment on the upstream face, consisting of cast-in-place concrete slabs over a web of concrete beams on the upstream fill.

As differential settlements and seismic events were a possibility in the area of the dam, the rigidity of the watertight element and the reliability of waterstops was a major concern. Moreover, the steep slope of the dam entailed significant construction difficulties, an extended time (works were already behind schedule), a great amount of labour and, even if continuous concrete placement would have been used, possible discontinuities in the waterproofing revetment could jeopardise watertightness. The Albanian government had had problems with the construction of Komani dam, a 110 m high CFRD, located on the same river. 300 people worked for 2 years on 3 shifts to construct only the waterproofing element, consisting of cast in place concrete slabs. The Albanian contracting party sought a more reliable alternative solution which could also reduce construction time and simplify problems connected with site organisation and labour.

Alternative options included bituminous concrete facing and prefabricated bituminous reinforced liner. The owner had had unsatisfactory experience with bituminous concrete facing. Its use was rejected because it involved high cost, the need to import heavy equipment, long construction time and the possibility of rapid ageing and localised deterioration. The proposed bituminous reinforced geomembrane was very heavy which prevented manufacture of a single roll, long enough to go from the crest to the toe (about 110 m). Transverse joining of several rolls to obtain the required length was not acceptable because the seams would not be strong enough. Furthermore it was considered that bituminous geomembranes did not have sufficient flexibility to accommodate differential settlements of the rockfill. It also found that in some applications bituminous geomembranes showed signs of rapid ageing despite being covered.

The synthetic membrane option was finally chosen because of similar successful applications and satisfactory extensive testing programme carried out by the waterproofing contractor and the designer. A PVC based geocomposite was installed over a stabilised gravel layer on the upstream face of the dam and ballasted by unreinforced concrete slabs, 30 to 20 cm in thickness, cast in place over a transition non-woven polypropylene geotextile, 800 g/m^2. The membrane was installed on the upper 56 m of the dam face.

The waterproofing geo-composite consists of a 3 mm thick PVC geo-membrane that is heat-coupled during manufacture to a 700 g/m^2 non-woven PET geo-textile. The geo-composite was installed over a stabilised gravel (porous concrete) layer which has the following functions: i) to prevent erosion of the fill on the more than 100 m long face during the rain season; ii) to provide a fairly regular support for the membrane; iii) to provide a drainage layer immediately behind the liner. The friction between the geotextile and the underlying drainage layer is sufficient to support the weight of the PVC sheets.

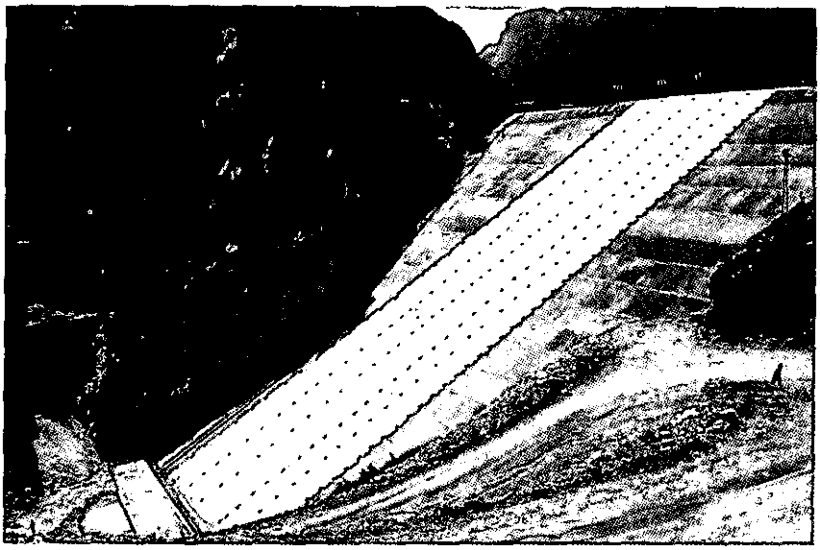


Fig. 8. The waterproofing PVC sheets installed on the 91 m high Bovilla earth dam and afterwards covered by concrete slabs

The perimeter anchorage of the liner is fixed mechanically at the crest into a trench, and a watertight fixing is made at the abutments and the foundation. The watertight fastening is positioned over the concrete perimeter beam, and consists of a stainless steel batten strips compressing the geomembrane against the concrete. Bedding resin mortar and neoprene gaskets are used at the concrete contact surface to obtain an even compression. A fold in the geocomposite with the sand/synthetic sandwich permits easy unfolding and allows it to accommodate differential settlements at the rockfill - plinth - geomembrane border of the order of 0.2 to 0.3 m.

The geocomposite was secured to the face by ballasting or weighing it down with concrete slabs, 3 x 6 m, thickness 0.3 m in the lower part and 0.2 m on the upper part of the face. The decision to ballast the liner was taken because of possibility damage caused by high winds, vandalism and wave action.

The slabs were cast in-situ over a 800 g/m² polypropylene transition geotextile, and connected and reinforced only at locations where stresses are high. The geotextile protects the geocomposite during concrete casting and, by adhering to it, constitutes also a light reinforcement for the slab. The geometry and coupling of the slabs allows relative movements to occur with acceptable tear stresses on the liner. The possible displacements in the central zone of the facing will be in the order of 1 m. Vertical joints are continuous, horizontal ones are alternated, with a 90° angle against the perimeter beam.

The introduction of a continuous geomembrane covering the entire upstream face produced substantial benefits:

- it enabled only two sizes of gravel material to be used; 20/50 mm aggregates for the embankment and 20 mm granular material for the transition-drainage layer.
- it was quickly installed (total construction time of the revetment was less than 8 months, including casting the concrete slabs);
- the project was completed on time.
- substantial saving was achieved (about 15 % of the expected cost of the entire project).
- the major cost represented by the use of the waterproofing geocomposite was compensated by the lower costs due to reduction of number of classes of aggregates, less stringent requirements for their installation, and reduced costs of construction of the simplified covering concrete slabs.

The Bovilla project characterises the modern waterproofing geomembrane system: a light, elastic waterproofing revetment that freely accommodates deformations and differential settlements without failure, providing long lasting imperviousness.

<u>RCC dams</u>. On RCC dams, the static function is performed by concrete but lower cement content and the presence of lift joints do not guarantee sufficient imperviousness. Water seepage especially at lift joints can affect the stability of the dam. Provision of a low permeability barrier allows a more economic mix of lower cement content to be used. Significant reduction of time and cost can be achieved, similar to the philosophy of RCC dams. A protected membrane solution (Winchester system) was developed and is

mainly used in the USA, an exposed membrane solution (CARPI system) was developed in Europe and its application has been extended to Central America and South East Asia.

The Winchester system constructs the waterproofing barrier by means of prefabricated concrete panels embedding the PVC geomembrane. The concrete side of the panels faces the reservoir, the PVC side is in contact with the dam body. As the panels are the permanent formworks against which RCC lifts are placed, installation of the panels proceeds with construction of the dam. The panels are anchored by metal anchor bars embedded in the RCC, the watertight perimeter seal is made at the heel by a PVC strip welded on the first line of panels and laid horizontally in the dam body, to intercept water coming from foundations. Joints between the panels are covered by welded PVC cover strips. The system has the disadvantage that any damage or fault during construction, or in subsequent service, cannot be easily located and repaired. The main advantage is the possibility of limiting ageing due to UV and damage due to vandalism or an aggressive environment.

The CARPI system is conceptually similar to the one already described for the rehabilitation of existing dams. The exposed PVC membrane forms a continuous waterproof barrier from the crest to the foundation beam where the perimeter seal is placed. The internal profile of assembly (3) in these installations is embedded in the RCC lifts as they are placed. Configuration of the assembly is very similar to the one used for rehabilitation. The PVC membrane is deployed on the dam face after all RCC layers have been placed, or during placement, then it is fastened with the profiles assembly (3), and PVC cover strips guarantee impermeability over the profiles. The irregularities in the RCC surface usually provide the necessary drainage so that no drainage layer is usually necessary.

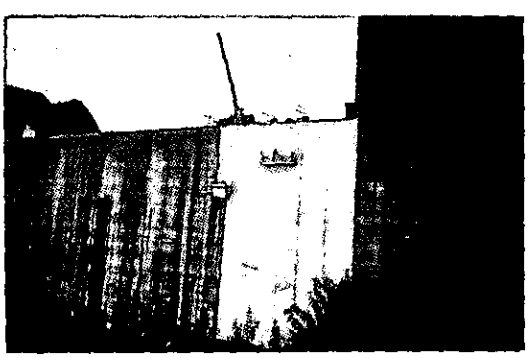


Fig. 9. Installation of the exposed membrane system at Nacaome dam.

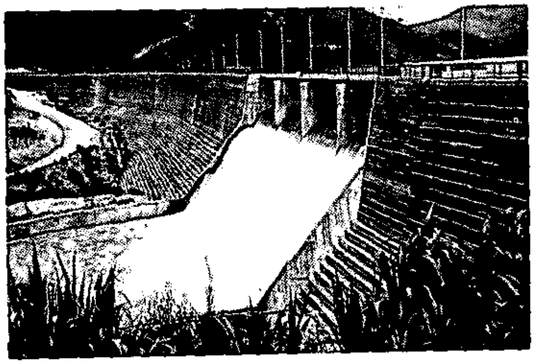


Fig. 10. Concepcion dam, 8 years after installation of the exposed membrane system. Leaks from the liner amount to 0.01 l/s.

Both the above dams are in Honduras where there high UV exposure.

PERFORMANCE

The drained PVC membrane system provides several substantial benefits:

- it forms a continuous impervious liner, bridging construction joints and existing cracks
- it forms a continuous impervious liner, capable of resisting opening of new cracks (differential settlements, seismic events, concrete swelling)
- it forms a continuous impervious liner from the crest to the foundations
- it is capable of resisting high water heads
- it can dehydrate the dam body, relieving pore pressure
- it can be applied on rough substrates and accommodate complicated geometry
- its efficiency can be controlled by drainage monitoring
- installation is quick, easy, independent of weather conditions
- it has a more than 20 years maintenance free history
- it is cost effective
- repairs are done by simple patching
- it can be installed underwater.

World records for waterproofing with geomembranes were all attained with the described system:

Concrete dams - 174 m high Alpe Gera, Italy
Embankment dams - Bovilla dam
RCC dams - 71 m Concepcion dam will be surpassed by the 93 m Balambano dam whose construction is now starting in Indonesia.

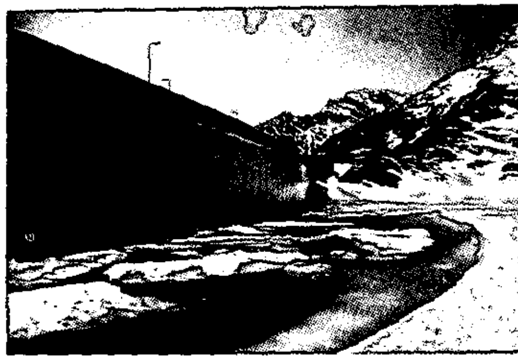


Fig. 11. Camposecco, a masonry dam, altitude 2337 m, where ice blocks driven by winds impact on the exposed PVC membrane installed in 1993

The system has been applied to all types of dams in different climates and latitudes, and at very high elevation, and it has been indicated by the Research Institute of Hydro Québec (Durand et al., 1995), that it is the most suitable for repair of dams in cold climates. The survey conducted among owners by the mentioned ICOLD Working Group reported more than 95% satisfaction concerning the technical aspect of the solution, and more than 97% satisfaction concerning the economical aspect.

REFERENCES

Cazzuffi D (1996). Italian experience for the use of geomembranes in rehabilitation of concrete and masonry dams. Proc. Colloque Tecnique, Paris, pp 14/1-12.

Corda R, Degoutte G, Bernhard C, Timblin L, Morrisson W & Cazzuffi D (1991). Watertight geomembranes for dams - The state of the art, pp 92-97. Paris, ICOLD.

De Beauchamp T (1995). The progress of remedial measures at Chambon dam. Proc. 2nd USCOLD International Conference on Alkali-Aggregate Reactions in Hydroelectric Plants and Dams, Chattanooga, USA, pp 209-220.

Durand B, Tremblay S, Germain Y & Houde J (1995). Etudes des revêtments étanches pour la face amont des barrages en béton, pp 160-167. Varennes (Canada), Hydro Québec - IREQ Institute.

Scuero A M & Vaschetti G L (1997). Italy's role in developing geosynthetics for hydraulic works. The International Journal on Hydropower & Dams vol. 4, Issue 2, pp 86-93.

Sembenelli P (1996). Nova barragem de aterro de Bovilla, na Albânia. Proc. LNEC, Utilização de geomembranas em obras hidráulicas e subterraneas, Lisbon, pp 1-4.

The International Journal on Hydropower & Dams, vol. 3, Issue 5, pp 86-87.

Stabilisation of Tai Tam Tuk Dam, Hong Kong

D GALLACHER Senior Partner, Robert H Cuthbertson & Partners, UK
R J MANN, Senior Engineer, Robert H Cuthbertson & Partners, UK

SYNOPSIS The stability of Tai Tam Tuk Dam, a mass concrete gravity dam in Hong Kong constructed in 1917, was found to be inadequate under extreme earthquake loadings using contemporary design standards. A study of pore pressures within the dam and its foundations and of the effects of drainage within a part of the dam indicated that significant improvement in stability could be achieved by internal drainage. The above study was described in a paper presented to the 1994 BDS Conference. This paper describes the implementation of Drainage Stabilisation Works at the outlet and overflow sections of the dam and the impact which they have had on the performance of the dam with particular regard to stability and internal stresses under earthquake loading.

INTRODUCTION

Tai Tam Tuk Dam is a masonry faced mass concrete gravity structure, about 49 m high and 364 m long, constructed in 1917. A layout plan of the dam and a cross-section showing the dam construction are given in Fig. 1. It is built in two main lengths each side of a central knoll with the Right part incorporating the outlet works and the Left part incorporating the overflow.

The stability of the dam under extreme loadings was questioned during early independent inspections and a number of proposals for remedial measures, some of which were implemented, were put forward. A trial piezometer installation and drainage scheme was carried out, being completed under the Fourth Independent Reservoir Inspection Programme (IIR4), to assess whether or not the stability of the dam could be improved to an acceptable level by the provision of drainage. Reference to this work is made in a paper to the 1994 BDS Conference (Gallacher et al, 1994).

Results of stability analyses carried out for the above inspection showed that the stability and internal stresses were acceptable under static loading, but unacceptable for sections of the dam with deeper foundation levels under earthquake loading. Analyses also showed that provision of drainage with an efficiency equal to that obtained in the trial drainage would achieve sufficient reduction in uplift to bring stability and internal stresses under earthquake loading to within acceptable limits.

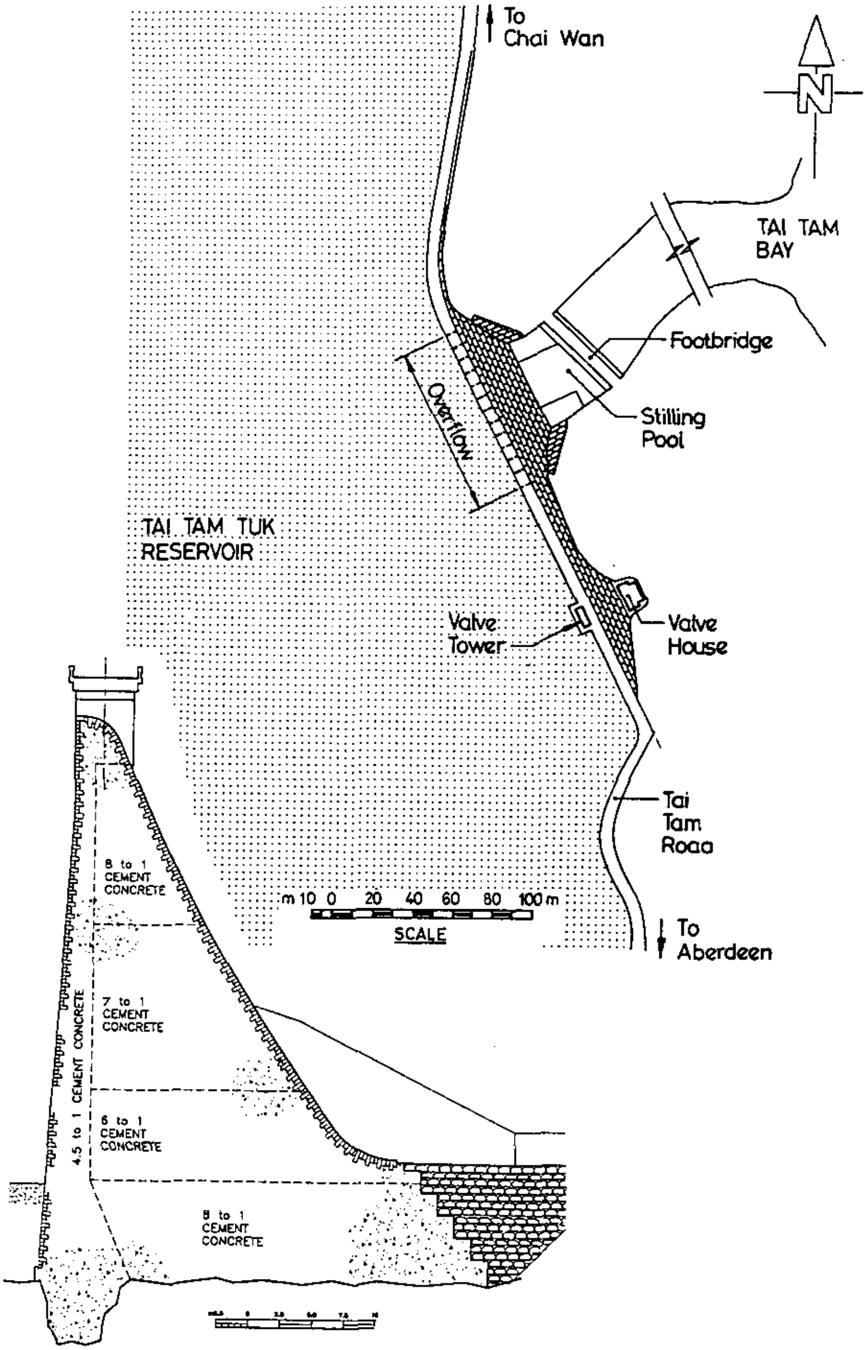


Fig. 1 Layout plan of dam and cross-section showing dam construction

It was recommended that Drainage Stabilisation Works be carried out to reduce pore water pressures and uplift within the lower part of the dam and its foundation at both the outlet and overflow sections. The proposed works were to install piezometers and drains by drilling from within the culverts at the Outlet Section and from the lower downstream face at the Overflow Section into the body of the dam and its foundation. Three Stages were proposed:

- Stage 1 Installation of piezometers in the dam and foundation to measure pore pressures in relation to reservoir water level in advance of drainage works, to allow the effects of the drainage works on pore water pressures to be monitored.
- Stage 2 Drilling of primary drainage holes in the body of the dam and its foundation, and shallow drainage holes through the masonry facing on the lower part of the downstream face of the dam, to relieve any pore pressures.
- Stage 3 Drilling of secondary drainage holes in the body of the dam and its foundation to provide further relief of pore pressures where adequate reduction was not achieved in Stage 2.

Construction work commenced in July 1996 and was completed in May 1997. Readings of piezometers installed in the dam and its foundation and of drainage discharges were taken during and after the works to monitor performance of the dam and the effect of drainage. The performance of the dam since the completion of the works has been reviewed and a certificate has been issued generally following procedures set out under Section 10(6) of the UK reservoirs legislation which confirms that the recommended works have been completed to the satisfaction of the Advisor to WSD, Hong Kong.

CONTRACT WORKS

The contract allowed for a number of piezometers and drainage holes to be provisional, with review of results of piezometer readings and drainage discharges stages as the works proceeded to determine whether or not further piezometers and drainage holes were required. It was specified that the overflow and outlet sections could be considered separately and the works phased independently for review purposes. Provision was also made for separate reviews at the Left and Right abutments of the overflow section.

It was planned that the works be carried out with the reservoir at least two thirds full, to allow piezometer readings to be related to reservoir full conditions.

Piezometer and drainage installations

The maximum scope of the piezometer installations (Section I of the Works) envisaged at the outlet and overflow sections are shown in Fig. 2. There were to be five arrays each of four piezometers at the overflow section and a single array of three piezometers at the outlet section, all connected to a data logger located in the valve house at the dam crest.

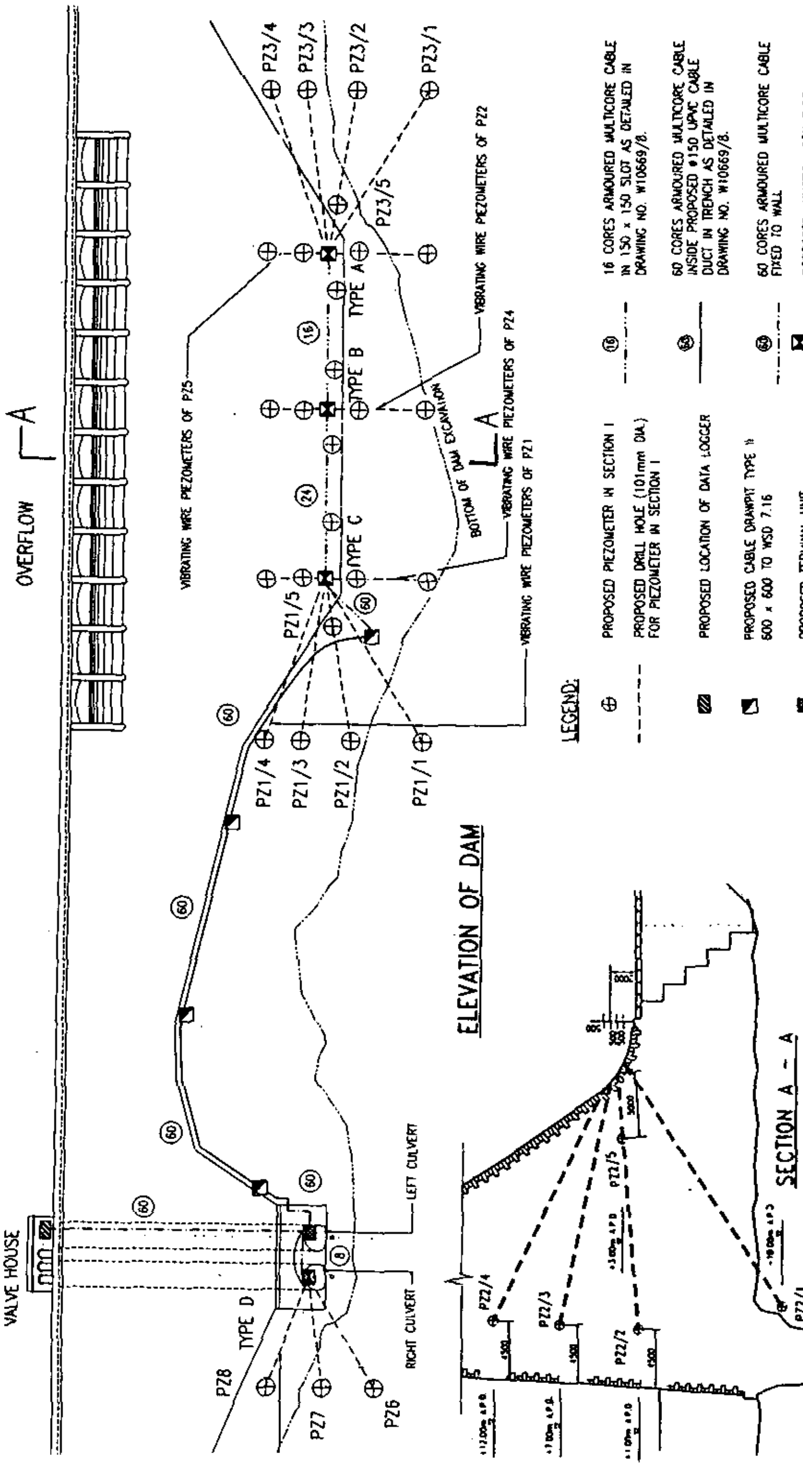


Fig. 2 Elevation showing maximum scope of piezometer installations (Section I of the Works)

Drainage installations (Section II of the Works) envisaged at the overflow section are shown in Fig. 3. Provision was made for ten arrays (six primary and four secondary) each of five drainage holes at locations 1 to 5 at the overflow section. No additional drainage holes were allowed for at the outlet section.

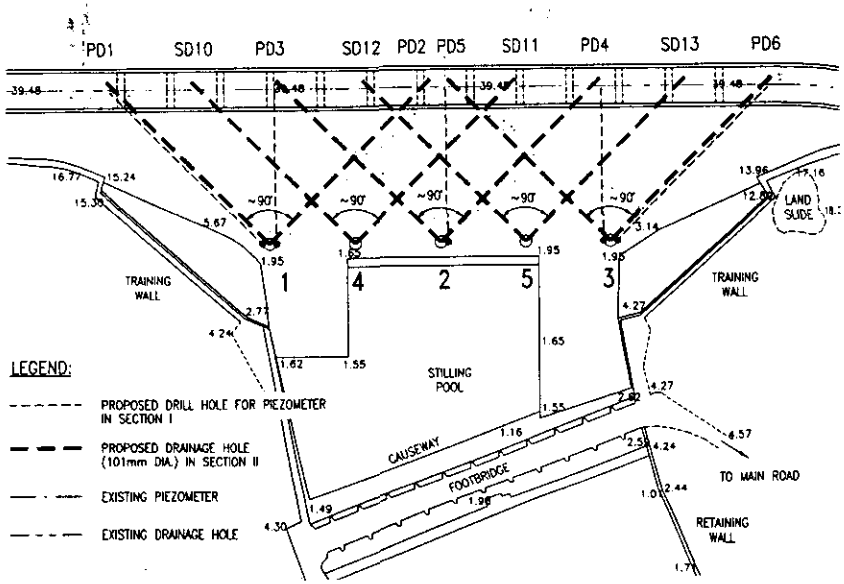


Fig.3 Plan showing maximum scope of drainage installation (Section II)

The piezometers at the overflow section were installed starting with the lowest in each array, followed by a review to determine whether the higher piezometers were required. The same procedure was adopted for the drains. Provision was made for a valve to be installed at the downstream face in the event of the upstream face being penetrated during drilling, to allow any resulting discharge to be controlled.

REVIEW OF PROGRESS OF WORKS

Drilling works commenced in mid June 1996 at both the outlet and overflow sections but the work was stopped at the overflow works when the reservoir overflowed on 22-23 June 1996 and scaffolding and drilling rigs were washed away. The reservoir level had fallen sufficiently in early December 1996 to allow work at the overflow section to resume.

The Advisor carried out inspection visits to review progress over the period 3 October 1996 to 5 May 1997. Reviews of monitoring results were made as they became available, including results from the original piezometers at the outlet section where faults in the wiring were investigated and repaired.

A review of progress was made on 18 March 1997 giving recommendations for completion of the Stage 1 works and review of results. Progress of the works required that piezometers and drainage be installed concurrently in some areas at the overflow section and, therefore, the improvement in stability achieved by drainage could not be assessed in this area.

The review included recommendations for installation of two additional piezometers at each of locations 1 and 3 at the overflow section, drilling of two drain holes from the Right culvert at the outlet section, flushing out of drain holes at the lower part of the Left culvert, and connection of original piezometers to the new Geoscan Unit.

Further review of all piezometer and drainage information was made during stages of the above works. The main outstanding question was whether or not additional drainage would be required at the overflow section following installation of the additional piezometers referred to above. This depended on results of monitoring, and another factor was the ability to carry out the work before the reservoir level rose to require removal of the drilling equipment. At a meeting on 23 April 1997 it was agreed to drill four drainage holes at the overflow section from the quarter-points towards the abutments, as these locations showed minor seepage and greater pore pressures and drainage flows than the centre section. A decision on a further four holes, from the quarter-points towards the central part of the overflow, was deferred and these latter four holes were subsequently omitted on the basis of available information which indicated that sufficient drainage had been provided to control pore water pressures. This was confirmed by final review of overall stability of the dam structure when piezometer readings had stabilised under reservoir full conditions.

WORKS AS EXECUTED

Outlet section

At the outlet section, the three piezometers (PZ6, PZ7 and PZ8) at the Right side of the outlet section were provided as originally envisaged, and the operable original piezometers were connected to the new data-logger unit. The installation arrangement as executed, including the original piezometers, is shown in Fig. 4.

Provision for drains in the Right side of the outlet section was not included in the Contract Works, although some were included in earlier draft documents. Review of the pore water pressures recorded in the above piezometers indicated that some drainage was required and an instruction was given to install drains R1 and R2 between the piezometers as shown in Fig. 4. The layout of the drains installed from the Left culvert at the outlet section in the earlier trial drainage scheme are also shown in the figure.

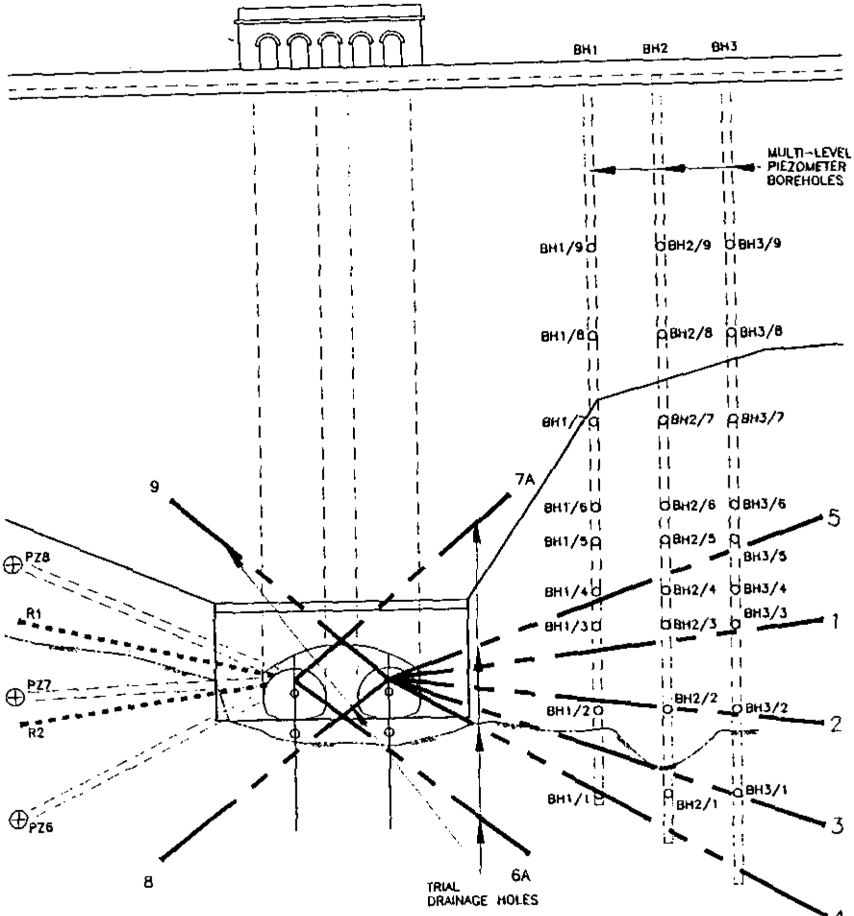


Fig. 4 Installation at Outlet as executed, including original piezometers

Overflow section

The final piezometer installation were the lowest three instruments at locations 1, 2 and 3 (Right, centre and Left) and the further two instruments at each of locations 1 and 3 obliquely into the Right and Left abutments respectively following review. The arrangement amounts to eight piezometers in the foundation and five in the dam as shown in Fig. 5, giving a total of thirteen piezometers installed out of the twenty piezometers allowed for in the contract at this section.

The final drainage arrangement at the overflow was three arrays each of three primary drains at locations 1, 2 and 3, followed by two further arrays each of two secondary drains at locations 4 and 5 obliquely into the Right and Left abutments respectively as determined following the review described above. The arrangement is shown in Fig. 5.

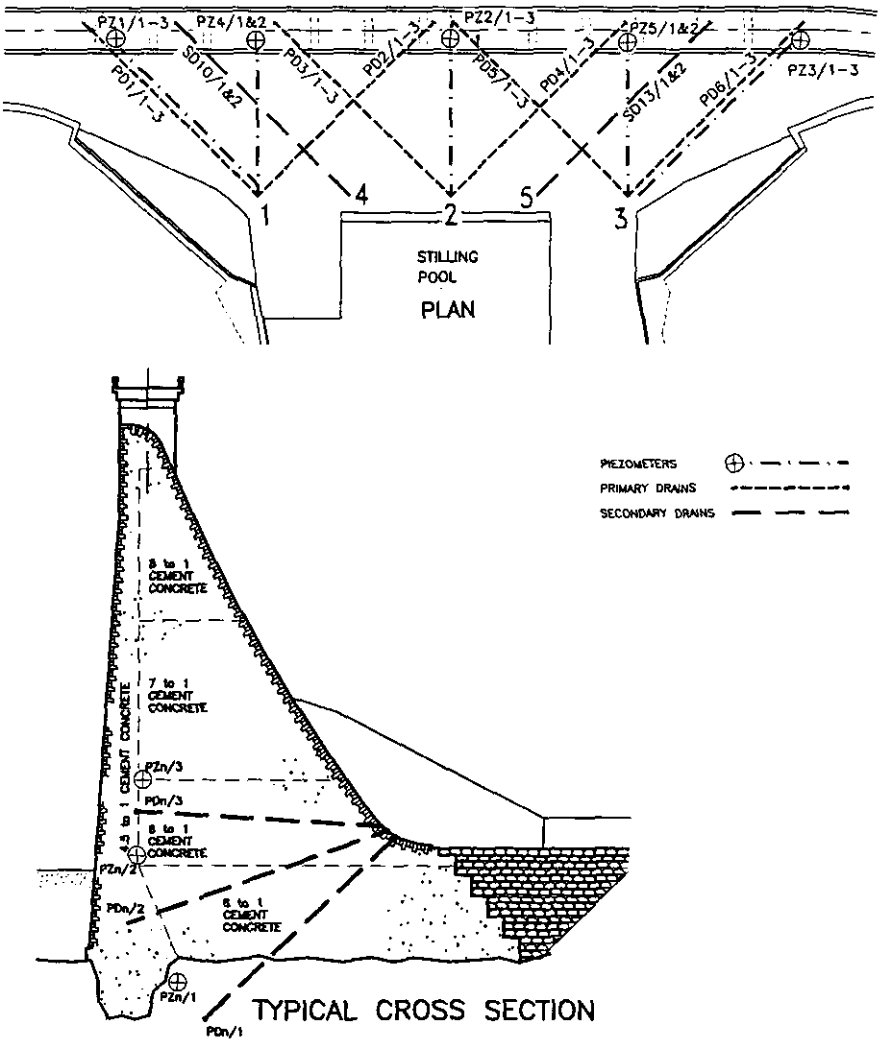


Fig. 5 Installation at Overflow as executed

This amounts to a total of twenty-two drain holes at the overflow out of the forty drain holes allowed for in the contract.

MONITORING OF RESULTS

Pore water pressures (pwp) in the dam and its foundation were monitored against reservoir level from the piezometers installed under the Contract and the original instruments from the earlier Trial Drainage Scheme. Drainage discharges were monitored from the various drain holes formed in the dam at both the outlet and overflow sections under the above works.

A summary of the review of pwps from piezometer measurements and drainage discharges is made below at the outlet and overflow sections.

Review of piezometer and drainage results - Outlet section

Plots of pwp against reservoir level in piezometers at the Right side of the outlet section are shown in Fig. 6 from installation in early March 1997 to February 1998 including a period at Top Water Level. The levels of the piezometers are included in the Figure. Review of the results of indicates that a substantial reduction in pwp occurred below the dam in PZ7 and a minor reduction in PZ6 after installation of drainage. Relief of pwps is being provided by drain R2 at foundation level and the pwps at the most critical horizon just below the foundation are about 50% and 18% driving head before and after installation of drains respectively.

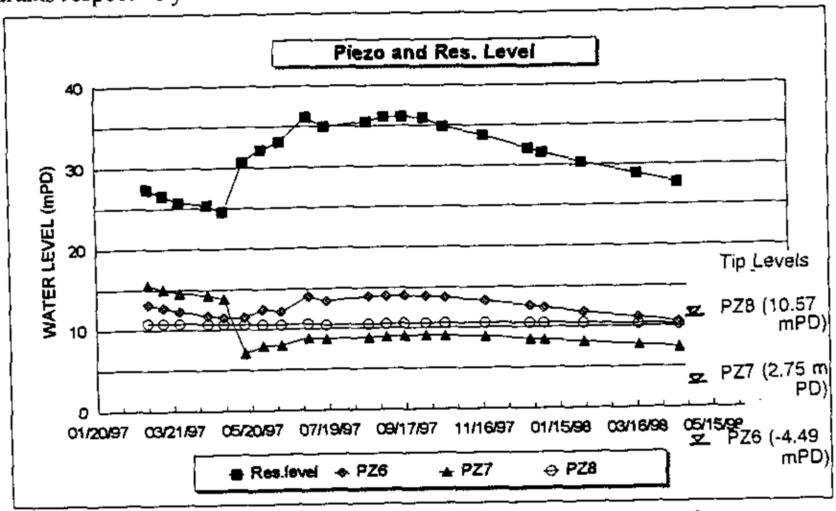


Fig. 6 Plots of pwp in piezometers at Right of outlet section

The original piezometers installed under the earlier trial drainage scheme indicate significant pwps averaging about 49% of driving head just below foundation level under reservoir full conditions, compared with 72% before the trial drainage was installed. Pwp has apparently increased since completion of the trial drainage scheme. Piezometers at higher elevations in the dam indicate low pwps decreasing to nil in the upper part of the dam.

Discharges from drains in the Left abutment have been fairly consistent since completion of the trial drainage scheme, although the rise in pwps referred to above indicates that there has been some decrease in their efficiency. Discharges from trial scheme drains above and below the culverts have been very minor indicating that the foundation rock is sound with an effective upstream cut-off, and are consistent with no significant pwps in the dam downstream of the rich upstream concrete section. Discharges from the Right abutment drains installed under the Contract are also minor.

Review of piezometer and drainage results - Overflow section
Piezometer installations were not completed before drainage was provided in this area and therefore the pwps reflect fully drained conditions apart from some early readings which may reflect partial drainage.

At the Right end, plots of piezometer readings indicate consistent pwps in the range of 26% to 35% of driving head under reservoir full conditions in the foundation, and very low pwps in the lower part of the dam. The pwp results are likely influenced by adjacent drains installed from Location 1 where records indicate consistent flow from the lowest drain, a small discharge from the middle drain, and the upper drain dry except during a period of heavy rainfall. These discharges vary with reservoir level and can increase sharply due to rainfall. There are other drainage discharges at the lower part of the Right mitre from previous installations. The two secondary drains installed towards the Right abutment foundation from Location 4 (Right quarter-point) have no discharge.

At the centre and quarter-points of the overflow section, piezometer readings indicate very low pwps in both the foundation and the lower part of the dam. Readings indicate a malfunction with one piezometer (PZ4/1) which requires to be investigated, and inconsistent performance of another (PZ4/2) which shows erroneous readings on several occasions. There are only minor discharges from the drain holes in bedrock. Readings from the reliable piezometers indicate very low pwps, in the range 8 to 10% in the foundation and lower in the base of the dam.

The central area of the dam and foundation is drained by arrays of three oblique drain holes drilled from each of locations 1 and 3 at the Right and Left ends respectively. The bottom drain of each array is in the foundation and all other drains are in the dam. The Right and Left quarter-points are drained by two arrays of three drains drilled from the centre to the dam and foundation. Discharges from all the above areas have been generally very low since the installation of drainage. This is indicative of sound rock conditions and an effective upstream cut-off. The extent of drainage provided in both areas may explain the low pwps recorded.

At the Left end, plots of piezometer readings indicate consistent pwps in the range of about 15%-16% under reservoir full conditions. The results for one piezometer (PZ3/3 in the lower part of the dam) are inconsistent, showing a rise from to about 27% of driving head with reservoir level falling from full, and this may be due either to a malfunction of the piezometer or the influence of ground water from the abutment. The Left mitre area is drained by an array of three oblique drain holes drilled from location 3, all in the foundation which rises steeply at the abutment. There have been minor flows from the two drain holes in the bedrock and negligible or zero flows from the other drains located in the dam. There are some minor seepages at the Right mitre. The two secondary drains in the Right abutment foundation from Location 5 (Left quarter-point) have little or no discharge.

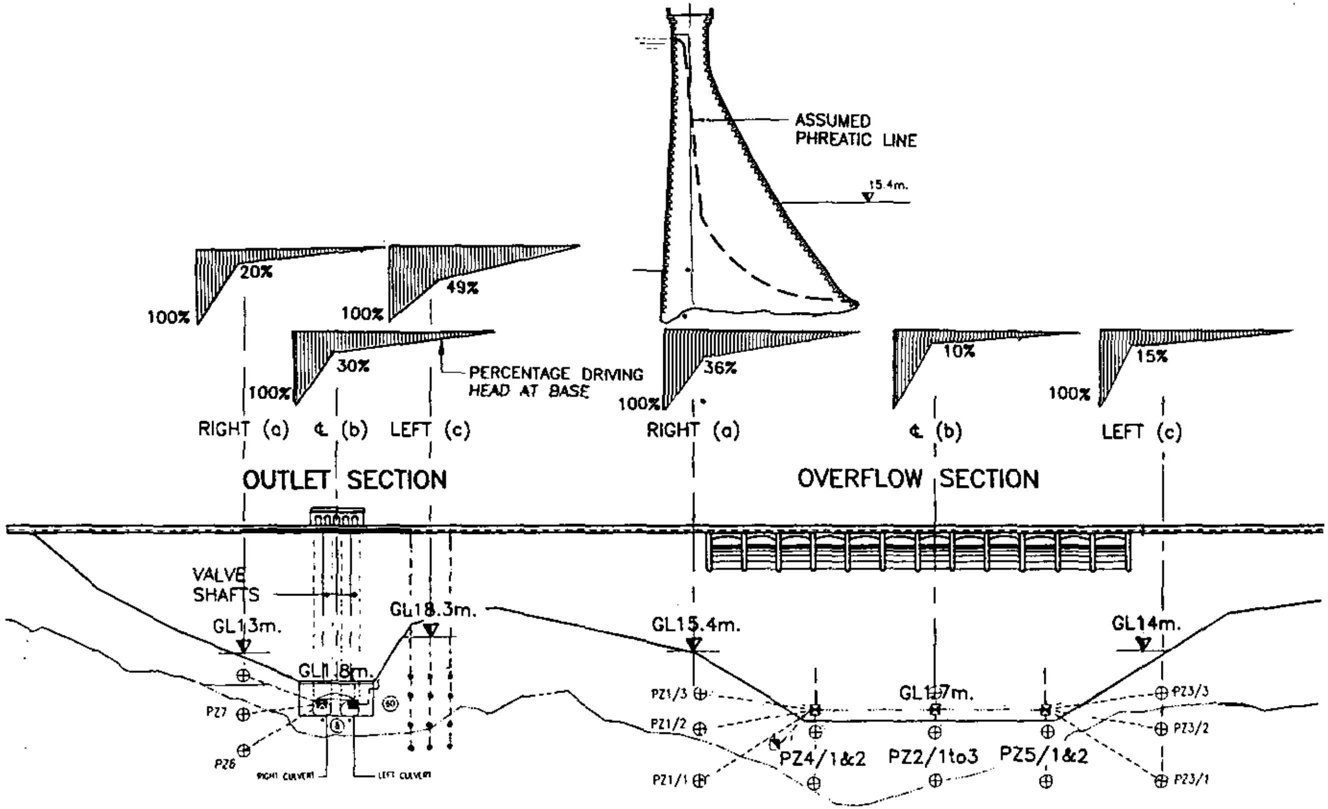


Fig. 7 Elevation showing cross-sections of the dam analysed, and assumed pore pressures at base

REVIEW OF STABILITY

Three cross-sections of the dam have been considered at (a) Right, (b) centre and (c) Left of both the outlet and overflow sections as shown in Fig. 7. Pore water pressures adopted for the stability analyses are based on the results described above, and the assumed pore pressure distribution across the dam base is indicated on the Figure for each cross-section. A typical assumed phreatic surface is also shown in the figure.

A conservative approach has been adopted for the typical cross-section (b) analysed at the overflow section by adopting uplift pressures based on the highest pwps found at the three instrumented sections within the overflow width. Account is taken in the analyses of the depth of backfill, notably at cross-sections (a) and (c) for both the outlet and overflow sections.

Stability analyses have been carried out for both the Operational Basis and Safety Evaluation earthquake cases (OBE and SEE) for each of the above cross-sections using a two-dimensional pseudo-dynamic method of analysis (Fenves and Chopra, University of California, 1986). The 84 percentile value of the Peak Ground Acceleration (PGA) values adopted are 0.10 g and 0.17 g for the OBE and SEE respectively.

Criteria for acceptance under earthquake loading are detailed below. The Factors of Safety relate to the dam/foundation interface and lower minimum Factors of Safety are acceptable within the body of the dam for these extreme loading conditions, provided that corresponding tensile stresses are not excessive. Stability against sliding at the base is considered in terms of the Limit-Equilibrium Factor of Safety. Acceptance values for tensile stress in concrete under earthquake loadings conditions (84 percentile) are assessed on the basis of a characteristic tensile strength of 0.85 N/mm^2, with appropriate factors of safety and increases for transient loads. This is regarded as conservative because the tension generally applies in the richer concrete zone which has higher strength.

Load case	*FoS Overturning*	*FoS Sliding*	*Tensile stress (N/mm^2)*
Reservoir full, OBE	*1.25*	*1.3*	*0.62*
Reservoir full, SEE	*1.0*	*1.0*	*1.23*

The allowable stress generally does not apply at or below the dam/ foundation interface. However, the interface extends into the foundation in a substantial cut-off key which can tolerate tensile stress, and cores indicate that the foundation rock is of very good quality and there is good bond at the interface. This will allow some relaxation of the above Factors of Safety at the general foundation level.

Review of stability - Outlet section

Summaries of the results at each cross-section for load cases (OBE and SEE) are given in Table 1. The resulting Factors of Safety against overturning, stresses at the upstream and downstream faces, and the line of thrust at selected elevations on the dam for the OBE and SEE load cases are given on Fig. 8 for cross-section (a).

Table 1 Outlet section - Reservoir full, OBE and SEE

Cross-section		FoS - Overturning		FoS - Sliding		Tensile Stress (N/mm^2)	
		OBE	SEE	OBE	SEE	OBE	SEE
BEFORE DRAINAGE INSTALLATION							
Overflow -	*Base*	*1.01*	*0.8*	*2.48*	*1.96*	*0.49*	*0.79*
Typical	*Body*	*1.00*	*0.70*	*-*	*-*	*0.52*	*0.83*
AFTER DRAINAGE INSTALLATION							
(a)	Base	1.15	0.97	2.78	2.26	0.47	0.74
Right	Body	1.06	0.71	-	-	0.44	0.76
(b)	Base	1.16	0.99	2.46	2.05	0.40	0.66
Centre	Body	1.06	0.71	-	-	0.44	0.76
(c)	Base	1.19	1.01	3.08	2.47	0.47	0.74
Left	Body	1.06	0.71	-	-	0.47	0.76

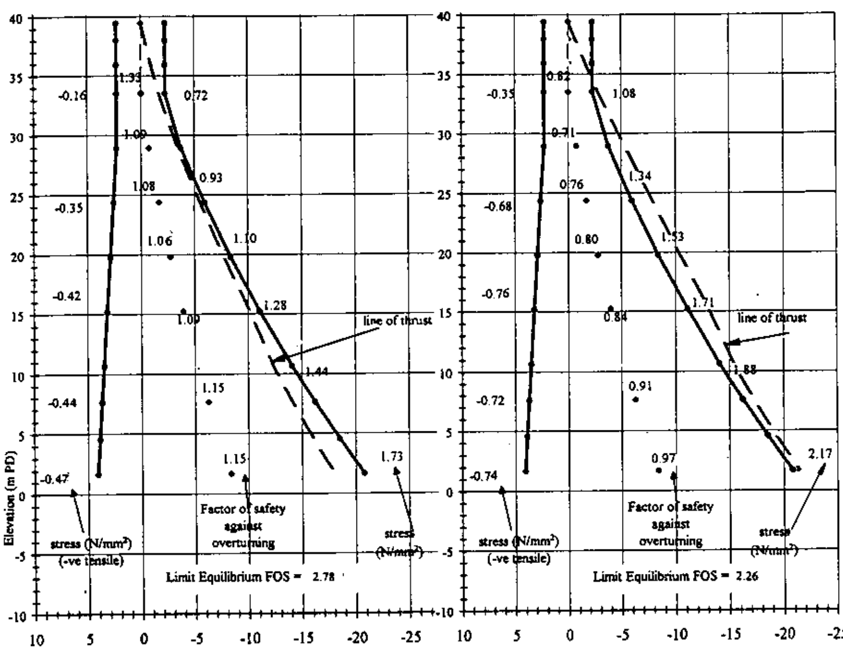


Fig. 8 Results of stability analysis for OBE and SEE load cases - Outlet Section (a) (Right abutment)

The results indicate a substantial improvement in stability as a result of the drainage works. The most critical criterion is the Factor of Safety against overturning at the base for extreme earthquake load cases, and this has been improved in the range 14 to 21%. This corresponds to a substantial increase in the return period of the earthquake which the structure can withstand, up to about the required 1,000 year return period for the SEE earthquake.

The results are based on conservative estimates of pwp at the foundation/dam interface and within the lower part of the dam. Furthermore, the analysis for the outlet section does not take account of the stabilising effects of the upstream valve shaft and of the abutments beyond cross-sections (a) and (c) where there is a higher percentage embedment and a rising foundation level. It is considered, therefore, that the dam now has adequate stability in this area under earthquake loading.

Review of stability - Overflow section

Summaries of the results at each at three cross-sections of the overflow section for OBE and SEE load cases are given in Table 2. In this case the pwp distribution was not obtained before drainage but it is likely that similar reductions have been achieved to reach the low levels recorded. The acceptance criteria have been met overall. The results are based on conservative estimates of pwp at the foundation/dam interface and within the lower part of the dam. The dam now has adequate stability in this area under earthquake loading.

Table 2 Overflow section - Reservoir full, OBE and SEE

Cross-section		FoS - Overturning		FoS - Sliding		Tensile Stress (N/mm^2)	
		OBE	SEE	OBE	SEE	OBE	SEE
(a)	Base	1.19	1.00	2.91	2.35	0.46	0.73
Right	Body	1.06	0.71	-	-	0.47	0.76
(b)	Base	1.23	1.07	1.92	1.63	0.35	0.59
Centre	Body	1.00	0.65	-	-	0.44	0.75
(c)	Base	1.29	1.07	2.90	2.35	0.40	0.67
Left	Body	1.06	0.71	-	-	0.44	0.75

Stability at both the outlet and overflow sections is dependent on effective drainage being maintained at the outlet section to control pore water pressures at the foundation/dam interface and in the lower part of the dam.

CONCLUSION

The Drainage Stabilisation Works consisting of the installation of vibrating wire piezometers and the drilling of drainage holes at both the outlet and overflow sections combined with information from previous piezometer and drainage installations have provided adequate additional information on the performance of the dam both before and after installation of drainage for assessment of stability under earthquake loadings.

Three sections have been analysed at both the outlet and overflow sections for the OBE and SEE load cases under reservoir full conditions adopting pore water pressures from a review of monitoring of the piezometer readings against reservoir level. Review of the combined results at each of the outlet and overflow sections shows that the dam has currently adequate stability in both areas under earthquake loading. Stability is dependent on effective drainage being maintained to control pore water pressures at the foundation/dam interface and in the lower part of the dam.

ACKNOWLEDGEMENTS

The paper has been written with the kind permission of the Government of the Hong Kong Special Administrative Region, Water Supplies Department. The cooperation of Hyder Consulting Ltd as associated consultants is also acknowledged.

REFERENCES

(a) Fenves G & Chopra A K, *Simplified Analysis for Earthquake Resistant Design of Concrete Gravity Dams*, Report No. UCB/EERC-85/10, University of California, June 1986

(b) *An engineering guide to seismic risk to dams in the United Kingdom*, Building Research Establishment, UK, 1991

(c) *Engineering guide to the safety of concrete and masonry dam structures in UK*, CIRIA, UK, 1995

The ICOLD Committee on Rehabilitation of Dams

G P SIMS Chairman, ICOLD Committee on Rehabilitation of Dams
P TEDD Building Research Establishment

SYNOPSIS. The Committee on Rehabilitation of Dams was established by ICOLD in 1994 with the aim of completing the Bulletin in 2000. The purpose of this paper is to describe the objectives of the Bulletin and outline how it is being prepared. A database of case studies of rehabilitation is being established for the Committee by BRE to provide statistical data on rehabilitation and selected case histories of innovative work. The Bulletin will deal with aspects of the management of rehabilitation with particular attention being paid to control of cost and programme when the scope of the work is not always known in detail at the start of the project. Proposals will be made for research for the more important aspects of dam rehabilitation.

INTRODUCTION

The Committee on Rehabilitation of Dams was established by ICOLD in 1994 following the publication of the comprehensive **Bulletin 93 'Ageing of Dams and Appurtenant Works'** (ICOLD 1994). The final Bulletin or report will be completed in 2000. The purpose of this paper is to describe the way the committee is carrying out its work, and by referring to a small number of examples to illustrate the approach to be taken in preparing the Bulletin. The Bulletin will provide a reference to practising engineers concerned with the design, planning or construction of rehabilitation works for dams. It will seek to identify and present the state of the art of dam rehabilitation.

Inevitably there is some overlap with the work of other ICOLD Technical Committees. A particular requirement for this Committee is to co-operate with the Committee on Dam Safety who will publish a Bulletin in the next year or so on the related topic of Safety Improvement of Existing Dams. There will be the opportunity to use some of the same case studies. However, they will have selected their examples to illustrate their focus on dam safety and the rehabilitation case histories must also reflect the economic choice in the rehabilitation equation. The Bulletin will also deal with the management of rehabilitation projects and include the instrumentation and monitoring of ageing dams as well as encouraging a better understanding of the research needed to improve the understanding of the fundamental processes involved in ageing.

The distinction is often unclear between many terms used in rehabilitation of dams including repair, remedial works, rehabilitation and upgrading. The following definitions are to be used in the Bulletin:

Maintenance: the work required to keep the installation in working order. It includes repair.

Rehabilitation: is synonymous with remedial measures. The limited work needed to restore to the installation the reliable life expectancy it had when it was new. New construction or equipment is provided only where the cost of such new work does not significantly exceed that of the original. Rehabilitation work is not planned or designed to enhance performance except as a concomitant of meeting the specific goal outlined above.

Upgrading: is synonymous with uprating. It is the work considered necessary to maximise the benefit of the existing installation. New construction or equipment is installed where it can be justified economically.

THE PROBLEM

Deteriorating dams and their related structures can threaten life and much attention has been given to the safety of dams and reservoirs. Even where safety is not the major issue, economic loss can be severe. Irrigated crop production or hydropower may be reduced to the extent that a small country's economy can be at risk. Flood control might be less effective than it should. Civil engineering works are only permanent if they are regularly maintained.

Regard must be taken of the urgent need in some developing countries for institutional strengthening to overcome the lack of interest in this activity by politicians and policy makers. It is often more appropriate to rehabilitate the organisation responsible for the infrastructure than to provide expensive aid funds to undertake a one-off rehabilitation project that immediately starts to deteriorate.

Rehabilitation is needed for at least one of two major reasons. First to counter the effects of ageing, and second to strengthen the dam to overcome deficiencies in the design or construction. It is important to realise that this represents no criticism of our ancestors. They were working at the current state of the art. Our present understanding of the physics of dam behaviour is now improved and rehabilitation is sometimes necessary to achieve acceptable safety in the light of technical advances in design. The present work is more concerned with countering the effects of ageing but does not ignore the other aspects.

Examples of earlier designs not meeting modern criteria include flood prediction and the required spillway capacity, and the effects of uplift pressure on the stability of gravity dams. Masonry gravity dams in Germany designed by Otto Intze around the turn of the century depended on having good contact between the bottom of the dam and the foundation rock which was achieved by careful cleaning of the rock. The upstream face of the dam was covered in a cement-based waterproofing element behind which was a drainage system designed to remove water seeping through the face. Excavated material was placed at the upstream toe and served to increase the length of the seepage path. No account was taken of uplift within the dam body or at the foundation. Despite this many of these dams continue to stand after nearly 100 years. By modern criteria they are unstable. Current rehabilitation works includes adding weight, installing a grout curtain and improving drainage sometimes by drilling and blasting a drainage gallery under the dam.

DATABASE

There exists a database of over 1000 case studies of deterioration of dams and appurtenant works recorded up to 1975 and published by ICOLD (ICOLD 1983). A much smaller database of good quality case histories of rehabilitation is being assembled. They only include rehabilitation work carried out after 1975. They include innovative techniques and well established methods. As far as possible the case histories are well recorded in published work.

The database is an extension of the BRE Dams Database which was established as part of the Building Research Establishment's work on the safety of dams in Britain (Tedd et al, 1992). The database includes basic details of dams as listed in the World Register, problems, incidents and rehabilitation case histories. A report for a particular dam can be obtained from the database. Most importantly the database includes a bibliography of references which gives more detailed descriptions of the rehabilitation. The database is currently limited to the following countries:

	Number of case histories	
Country	Embankment	Concrete/masonry
France	8	21
Finland	7	0
Japan	2	7
Sweden	11	0
Switzerland	2	12
Portugal	9	14
United kingdom	500	40

The type of rehabilitation for a particular country is reflected by the type and age of dam and the current legislation and guidance. In Britain the majority of the dams are embankment dams and most of the remedial works have been associated with internal erosion other than those required to take design floods.

STRUCTURE OF THE BULLETIN

The 31 major ageing scenarios identified in Bulletin 93 have been used for the basis of structure of the Bulletin under preparation. Appendix 1 sets out the proposed structure. Appurtenant structures include power stations, intakes, pressure pipelines, protection devices, hydro-mechanical equipment, spillways, bottom outlets and the like. It is intended that the Bulletin should deal with each ageing scenario individually. Appropriate case histories will be used to illustrate the problem, how it has been detected and what remedial measures are currently available and degree of success.

Following the introduction the Bulletin contains five chapters. The first of these introduces aspects of the management of rehabilitation work.

MANAGEMENT OF REHABILITATION

There is much activity in the period before the decision is made to invest in rehabilitation; the initial reconnaissance, the feasibility study and the implementation. The management of the process will seek to ensure that a pragmatic approach is followed in which decisions are made on sound information and at an acceptable cost.

It is important to link the level of funds committed to a rehabilitation project strictly to the quality of the estimate of its final cost. The table below illustrates how the confidence level, defined as 100% minus the claimed accuracy of the estimate increases as the work increases through the project.

Stage of development	Potential error in the cost estimate, %	Confidence level
Initial realisation of the problem	+/- 100	0
Initial reconnaissance	+/-50	50
Feasibility study	+/-30	70
Rehabilitation contract estimate	+/-10	90
Completion of the work	0	100

The table shows how rapidly and at what proportionately low cost it is possible to increase the confidence level in the cost of the project. Thus the reconnaissance study gives perhaps 50% confidence for say 1% of the project cost.

Practical matters that require the detailed attention of the manager of rehabilitation include pre-qualification of tenderers. This is an important consideration particularly in the developing world where it is not so readily appreciated that highly developed technical and management skills are needed by the contractor and the temptation is to use a local firm at a low price. The selection of the most appropriate Form of Contract is important in control of programme and budget.

Rehabilitation is like other construction activities in the sense that we are seeking the economic optimum solution. This is achieved by comparing the costs and benefits of alternative solutions, including the one of doing nothing. Of fundamental importance here is knowledge of the accuracy of the estimates of cost of the rehabilitation work (UMIST, 1989).

Detailed discussion will be required to reduce the upset caused by the rehabilitation. It will be important for the Owner to make clear what camps, plant and local labour he is making available. The requirement for training and the desirable institutional strengthening will be clarified. Organisational changes might be appropriate.

REHABILITATION TECHNIQUES

The following three chapters of the Bulletin deal with the practice of rehabilitation of concrete and masonry dams, embankment dams and appurtenant structures. In each chapter, sub-sections relate to the ageing scenarios in Bulletin 93. Generally each sub-section describes the mechanism of the ageing scenario, the method of detection and monitoring leading to the need to rehabilitate, the consequences on safety and operation if deterioration is not halted and finally the rehabilitation measures that have been used. The rehabilitation measures are illustrated with case histories that describe established techniques and more recent concepts and materials. Some typical examples for each of these chapters are given briefly below. Ageing and remedial works to the foundation as well as to the structure are described.

Concrete and masonry dams

Section 3.3.1 - Swelling due to chemical reaction

The introduction describes the mechanisms of alkali aggregate reaction and sulphate attack of concrete and mortar which are seen to be the most common forms of swelling and the conditions under which they are likely to occur. References giving more detail are provided. Bulletin 93 lists 45 case histories where this scenario has occurred in concrete and masonry structures and appurtenant works.

Detection methods include displacement and strain monitoring. The effects on safety and performance depends on the type and characteristics of the dam and these are described for gravity, arch and buttress dams.

The exclusion of water from the vulnerable concrete has been attempted as a remedial measure against the swelling caused by AAR , but its success is not known. Mitigation of the effects of the swelling by cutting slots and post-stressing are illustrated with examples including Chambon dam, Kamburu dam and Val de la Mare dam.

Earth and rockfill dams

Section 4.3.4 - Internal erosion. Internal erosion is one of the most common causes of deterioration and failure of embankment dams. The introduction describes the mechanisms and causes of internal erosion. The difficulties of detecting internal erosion are emphasised. Detection and investigation methods are covered briefly including the use of geophysical methods.

The rehabilitation methods described include:

- Grouting of the erosion path
- Impervious cut-off (diaphragm walls, steel sheet piling etc)
- Addition of toe drain and downstream filter
- Replacement of core section

Each of these methods is described and illustrated with case histories. Analysis of data from Britain and Sweden show that grouting is the most widely used remedial method to control leakage and internal erosion of embankment dam cores. The Bulletin will mention the various types of grouting. Porjus dam in Sweden is used as an example to illustrate the occurrence of internal erosion, its investigation and successful remediation using grouting (Johansson et al, 1996).

The use of the various types of impervious cut-off wall will be covered. The advantages of single phase and double phase methods of constructing diaphragm walls are discussed. Until recently the double phase method using plastic concrete has been more commonly used in Britain, however there has been an increase in the use of the single phase method for repair of some small dams. The technique was first used in Britain in 1985 (Connery, 1985).

Appurtenant works

Section 5.6 – Problems with gates and other discharge equipment. One of the six scenarios identified by ICOLD in Bulletin 93 for appurtenant works is problems with gates and other discharge equipment. The limited life of control equipment such as gates is emphasised and therefore its rehabilitation plays an important part in the rehabilitation of dams. The problems experienced by hydraulic equipment that limit its life is caused by the combined effects of corrosion and poor maintenance. Low quality corrosion protection systems have proved inadequate in may instances. The need for rehabilitation is frequently indicated by poor performance, gates becoming jammed or vibrating badly.

Failure of gates and discharge equipment will not as a rule lead to failure of the dam. It will however make it impossible for the structure to fulfil its designed role. It may not be possible to divert water for irrigation, power or water supply. The reservoir may not be able to be filled or more likely emptied under control.

The rehabilitation of Kotri barrage on the Indus River in Pakistan (Padgett & Dahar B A, 1997) and the replacement of the low level outlet valves at Moehne dam (Campen & Mantwill, 1994) have been used to illustrate rehabilitation of gates and valves.

RESEARCH NEEDS

Cost effective rehabilitation measures require a sound understanding of dam behaviour. Research relevant to the rehabilitation of concrete and masonry dams will relate to the generally rock foundations and to the concrete and masonry of the dam body. For embankment dams further research is required into the mechanism of internal erosion of clay cores and the long term effectiveness of grouting, slurry walls and other rehabilitation techniques. Mechanical, hydraulic, and thermal behaviour of the materials is of interest. The effects of ageing and the need for rehabilitation may be detected first in the appurtenant structures particularly for modern dams Thus research into the performance of gates in older dams is of interest.

Monitoring equipment is used to detect abnormal behaviour perhaps as a result of ageing or following exceptional events including earthquake, floods or landslides. Improvements to the design of instrumentation are envisaged as are means by which inactive instruments may be reliably reactivated. Post construction in situ measurement of the properties of the foundation or dam body may call for the development of special techniques and devices.

Developments in the physico-chemical behaviour of construction materials including rock, soil and cementitious materials is a field that will benefit from research.

Further development of mathematical models for the simulation condition of behaviour can assist with the assessment of the safety of old dams. Models investigate the effectiveness of some rehabilitation works. Further improvement will be helpful in this field to investigate the effects of the loads, strains and stresses caused by the rehabilitation itself in, for example high pressure grouting, blasting and post stressing. The effect of making repairs with material that is different from the original also needs detailed study. These simulations will be helpful in assessing the deterioration of structures following exceptional events such as earthquakes and floods. Coupled models to simulate the combined effect of different actions will be the subject of further research.

Research topics in the field of design and construction include study of new high performance materials including resins, and geo-membranes. New construction methods to reduce the cost and time needed for rehabilitation will be useful. Such techniques will include underwater construction and the temporary diversion of the river to repair low level outlets. This latter may be through the reuse of sealed tunnels or outlet works.

More enlightened operators carry out preventative maintenance and a rehabilitation schedule that reduces the risk to them of unexpected failures. To plan these works risk evaluation has been found to be a useful approach. Research into the effectiveness of the procedures currently available would be helpful.

CONCLUSIONS

This paper has sought to explain the way in which the ICOLD Committee as the Rehabilitation of Dams has approached its task. The Bulletin under preparation will assist designers, constructors and operators to care for ageing structures so that they continue to provide valuable service.

REFERENCES

Campen F & Mantwill (1994). Preventative overhaul and refurbishment of the bottom outlet works of an 80 year old masonry dam performed without emptying the reservoir. Transactions of 18th International Congress on large Dams. Durban Q71 R3.

Charles J A, Tedd P, Hughes A K & Lovenbury H T (1996). Investigating embankment dams: A guide to the identification and repair of defects. Building Research Establishment Report BR 303. Construction Research Communications, London

Connery (1995). Reported in BNCOLD News and Views, No. 28, pp13.

ICOLD (1994). Ageing of dams and appurtenant works. Bulletin 93.

Johansson B, Johansson S & Nilsson R (1996). Investigations and repair of the embankment dam at Porjus power station. Proc. of the symposium on Repair and Upgrading of Dams, Stockholm, Sweden. pp218-227

Padgett I E & Dahar B A (1997) Removal of Head Limitation at Kotri Barrage Pakistan. Q75, R19. 1 Transactions of the 9th International Congress on Large Dams. Florence.

Tedd P, Charles J A & Holton I R (1992). The BRE dams database. 7th Conf. of the British Dam Society, Stirling, pp403-410

UMIST (1989) A Guide to Cost Estimating for Overseas Construction Projects. ODA funded study.

Appendix 1

REHABILITATION OF DAMS

Foreword

1 GENERAL CONSIDERATIONS

2 MANAGEMENT OF REHABILITATION

2.1 GENERAL OBSERVATIONS

2.2 PROJECT DEFINITION
1. Introduction
    2.2.1 Preliminary Optimisation Studies
    2.2.2 Data Collection
    2.2.3 Technical and Economic Feasibility
    2.2.4 Project Plan

2.3 CONTRACTUAL ASPECTS
    2.3.1.Contract Packaging
    2.3.2 Contractor Identification
    2.3.3 Construction Management
    2.3.4 Operation, Maintenance & Handover
    2.3.5 Training

2.4 RISK MANAGEMENT

3 CONCRETE AND MASONRY DAMS

3.1 INTRODUCTION

3.2 REHABILITATION OF THE FOUNDATION
    3.2.1 Loss of Strength due to Repeated Actions
    3.2.2 Erosion and Solution
    3.2.3 Grout Curtains and Drains

3.3 REHABILITATION OF THE DAM BODY
    3.3.1 Swelling due to Chemical Reaction
    3.3.2 Shrinkage and Creep
    3.3.3 Degradation due to Chemical Reaction
    3.3.4 Loss of Strength due to Repeated Actions
    3.3.5 Freezing and Thawing
    3.3.6 Structural Joints
    3.3.7 Upstream and Downstream Facings
    3.3.8 Prestressed Structures

Recent developments in the seismic analysis of concrete gravity dams

C A TAYLOR, University of Bristol, UK
W E DANIELL, University of Bristol, UK

SYNOPSIS. It is shown that standard finite element packages with appropriate concrete material models can be configured to handle the non-linear behaviour of the dam, its reservoir and foundation. Important issues controlling the effectiveness and validity of non-linear dynamic analyses are raised. Examples of the use of such analyses to assess the seismic performance of typical UK dams are given. These analyses yield useful qualitative information. However, it is concluded that until reliable material property data are available, non-linear seismic analysis of gravity dams should not be regarded as a robust assessment tool.

INTRODUCTION

Even in regions with low to moderate seismicity, such as the United Kingdom, the extreme safety assessment earthquake for critical structures is often of a magnitude that has caused significant damage to a wide range of structures in many events world-wide, especially where seismic loading has not been considered in the initial design. For example, a typical UK extreme event, having an annual probability of exceedance of 10^{-4}, is about Richter magnitude 6.0. Earthquakes of this magnitude and lower have caused significant damage in southern Italy, Greece, Armenia, Central America, and Australia. While prevention of loss of life should always be the primary concern of safety assessment, recent earthquakes in developed countries have highlighted the potentially wide ranging and costly economic consequences of major events. The insurance losses in the recent earthquakes in Northridge, California (1994 - $35 billion) and in Kobe, Japan (1995 - $85 billion) are likely to prompt an increase in the priority given to the prevention of enterprise losses. These factors are leading to an increased awareness of the need to account for extreme seismic hazards in the safety assessment of all aspects of existing and new dams and their related works, in the UK as well as abroad.

Associated with these changing political and financial demands for the protection of both life and enterprise is the growing expectation that seismic safety assessments are rigorous and use the latest techniques. The ready availability of cheap computing power has prompted the development of sophisticated, and potentially powerful, non-linear dynamic analysis tools.

The prospect for reservoirs in the 21st century. Thomas Telford, London, 1998

These tools are often very easy to use and offer excellent facilities for the presentation of results. They have been used with a reasonable degree of success in the analysis of conventional building structures, for which there is a large body of experimental and prototype observations to aid validation. Their ease of use is leading these tools to be applied to concrete dams, for which the supporting evidence of their validity is scant. It is essential that the dam engineer recognises this, so that the use of these tools does not mislead the seismic safety assessment process. Nevertheless, such tools, if properly used, can give a valuable insight into the potential seismic behaviour of a dam, and thereby inform engineering judgement.

The aim of this paper is to present an overview of some recent developments in the non-linear seismic analysis of concrete gravity dams resulting from ongoing research at the University of Bristol. The research is focusing on how standard non-linear analysis tools can be used both to improve understanding of the fundamental seismic behaviour of dams and to play a valid part in seismic safety assessments. It follows a three-faceted approach that integrates analysis with laboratory experiments and observation of prototype behaviour. The research has concentrated on dams typical of the UK in terms of size and earthquake loading characteristics, with the aim of improving the knowledge base underpinning dam safety assessments in this country. The paper highlights some important issues that the engineer should be aware of. It does not go into technical details; these may be found in the quoted source references.

DYNAMIC BEHAVIOUR OF CONCRETE GRAVITY DAMS

Meaningful dynamic analysis must reflect the principal mechanical aspects of the problem. In the context of gravity dams, these aspects can be broadly categorised as; description of the input motion, structural response, material behaviour, fluid-structure interaction, and foundation-structure interaction. The following sections briefly discuss some key issues associated with these aspects.

Description of the input motion

It is worthwhile outlining, for the benefit of the non-specialist, the common ways of specifying the input seismic motion.

The most basic characterisation of a ground motion is an acceleration time history such as that recorded in the 1966 Parkfield, California earthquake shown in Fig. 1. This is a measure of the ground movement in a single component. In practice, three orthogonal components will be measured by a strong motion instrument placed in the field. The acceleration time history can be integrated to give velocity and displacement time histories.

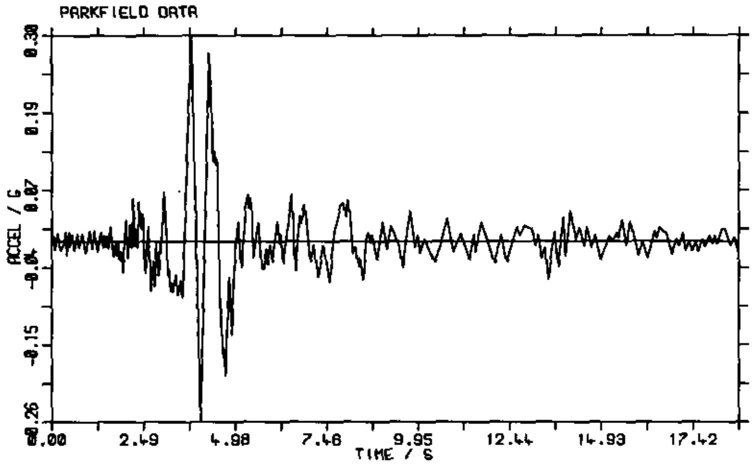


Fig. 1 Parkfield earthquake - acceleration time history

The random looking time history results from the interaction of several different types of stress waves that pass through the ground, causing the latter to vibrate. The early part of the time history tends to be dominated by the compression waves, which have a higher transmission velocity than the higher amplitude shear waves that arrive later. The Parkfield record is representative of a short duration event that might occur in intra-plate regions (i.e. far away from the plate boundaries) such as the UK, as a result of local fault movements. Inter-plate earthquakes that occur on plate boundaries tend to be of longer duration and greater magnitude.

The most common way of specifying a design ground motion is through a response spectrum. A full description of what follows can be found in the excellent textbook by Clough & Penzien (1993). When a structure is shaken by an earthquake, it will vibrate. If the structure remains elastic it is relatively straight forward to calculate how it will respond. For a simple, elastic, single degree of freedom structure having a given viscous damping, the response time history can be calculated by solving the Duhamel integral. A response spectrum for a given damping is constructed by taking the computed absolute maximum response acceleration and plotting it on a graph as the ordinate with the frequency of the structure as the abscissa. By repeating this procedure over a range of frequencies, a curve is created, as shown in Fig. 2 for the Parkfield motion. When analysing the seismic response of an elastic structure, if the natural frequency and damping of the structure can be estimated, then the maximum response of the structure to that earthquake can simply be read from the response spectrum curve. Response spectra can also be constructed in terms of velocity and displacement responses. Techniques are available to extend this approach to calculate the maximum responses of multi-degree of freedom structures; they are to be found in most general purpose finite element analysis packages with a dynamics capability.

Design response spectra are usually smoothed envelopes derived from a statistical analysis of appropriate recorded acceleration time histories, and increasingly, numerical models of the fault mechanisms and wave transmission paths. Fig. 3 shows a hard ground response spectrum for the UK derived in a study on behalf of the nuclear industry (Principia Mechanica, 1981). The spectrum is normalised to a peak ground acceleration of 1g.

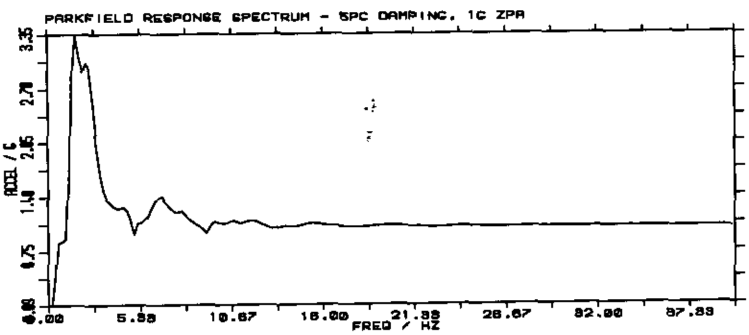


Fig. 2 Parkfield earthquake - response spectrum for 5% damping

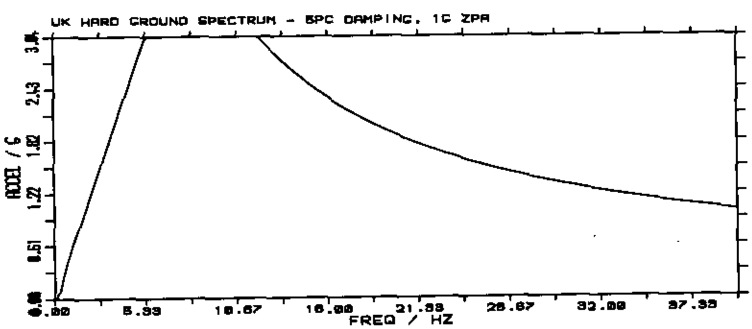


Fig. 3 UK hard ground response spectrum for 5% damping

Structural response and material behaviour

Prior to being shaken by an earthquake, a gravity dam will be subjected to the usual static loads of hydrostatic pressure from the reservoir, uplift due to seepage through the foundation rock (and to a small extent through the dam body), thermal stresses, and the dead weight of the dam body. When an earthquake strikes, this system is subjected to additional transient, dynamic loads as the incoming compression and shear waves generated by the seismic event travel through, past and within the dam site and dam. The dynamic loads cause the whole dam system, including the foundation rock, to vibrate. Stress waves transfer between the dam, its reservoir and the foundation in a complex manner that is dependent on the geometry of the system and its material properties. The different components of the dam system will modify, amplify or attenuate the stress waves before the latter exit the system by radiating away through the effectively semi-infinite extent of the foundation rock.

The seismic waves lead to fluctuating stresses that are superimposed on the static stresses. If the combined static and dynamic stresses stay within the elastic limits of the materials, then the structure will respond elastically, exhibiting several natural frequencies and associated mode shapes. The extent to which each mode responds will depend on the energy content of the incoming earthquake around the frequency of the mode, as well as the damping associated with that mode. In a purely elastic case, the total dynamic response of the system can be obtained by superimposing the individual modal responses, taking into account the phase differences between the modes. Results can be obtained either in the form of time histories of response quantities, such as acceleration, displacement or stress, or as estimates of the maximum values of the response quantities using a response spectrum analysis. The latter can be obtained by taking the square root of the sum of the squares of the individual modal responses, or through a more rigorous procedure such as the complete quadratic combination (CQC) method (Clough & Penzien, 1993).

An extreme seismic event is likely to excite the dam system beyond the linear elastic range. There are several sources of non-linearity in the dam system, but the two most important are cracking of the dam concrete and movement of joints within the foundation rock.

Dam concrete usually has a compressive strength (> 30MPa) that is several times greater than the actual compressive stresses (~10MPa) that are typically induced in a gravity dam. Crushing of the concrete is, therefore, highly unlikely. Furthermore, the stress-strain behaviour of the concrete up to the amplitude of the actual stresses is, in most cases, nearly linear elastic. The tensile behaviour of dam concrete, however, does not benefit from such a large margin between the ultimate strength (typically taken to be about 10% of the ultimate compressive strength) and the induced stresses. In many cases, elastic and inelastic analyses show that the tensile strength can be exceeded at the upstream heel of the dam adjacent to the foundation rock, and near to the crest, the latter especially if the dam has a neck with a discontinuous downstream face slope (e.g. Koyna dam, India, which was damaged by an earthquake in 1967).

If the tensile strength is exceeded, a crack will form that will propagate into the dam body or foundation. The propagation path will depend on the geometry of the dam and foundation, and on the relative material stiffnesses and strengths. The development of cracks, which can open and close, leads to non-linear stiffnesses and energy losses that can have a marked effect on the dam's dynamic response. Usually, the onset of cracking will increase energy losses (i.e. damping), which will tend to attenuate the dynamic response. Cracking will also tend to reduce stiffness on at least one side of a dynamic stress cycle. This may reduce the equivalent natural frequency of the dam system (bearing in mind that a non-linear system does not have a true natural frequency) and in turn affect the amount of dynamic energy that the dam 'sees' in the earthquake. This could be advantageous or disadvantageous depending on the frequency content of the earthquake. The non-linear nature of the problem can be further complicated by the presence of water in the cracks, the effects of which are poorly understood.

Similar mechanisms are present in the foundation rock, especially that portion immediately beneath the dam. This is the part of the system that carries all the loading from the dam body, yet it is the most imprecisely defined and most poorly understood. Existing joints will be present due to previous geological processes, and new joints may be created by the dynamic loads imposed on the rock mass. These intersecting fluid-filled joints will not only open and close, but will tend to slide relative to each other. This will produce additional, highly non-linear, stiffness and damping mechanisms.

Mao (1996) and Mao & Taylor (1997, 1998) reviewed various constitutive models for plain concrete. They concluded that the model proposed by Bathe et al (1989) was an acceptable representation of the behaviour of dam concrete, given the paucity of laboratory test data. The model is based on a parabolic uniaxial stress-strain curve in the compression zone (Fig. 5). In the tensile zone, the stress-strain behaviour follows a triangular path that represents the tensile loading up to the initiation of fracture, followed by a linear decent as the fracture propagates and relieves the tensile stress. Recent materials testing by Trunk & Wittman (1998) has confirmed this pattern of behaviour for dam concrete and the effectiveness of the concepts on which the Bathe model is based. A full description and critical review of the model and its application to gravity dams are given by Mao (1996). He draws attention to important details, such as the possible mesh dependency of predicted crack patterns, that must be considered when developing a numerical model.

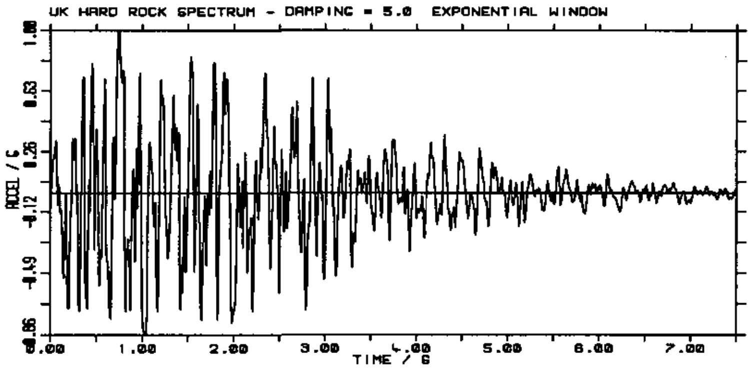


Fig. 4 Synthetic UK hard ground acceleration time history

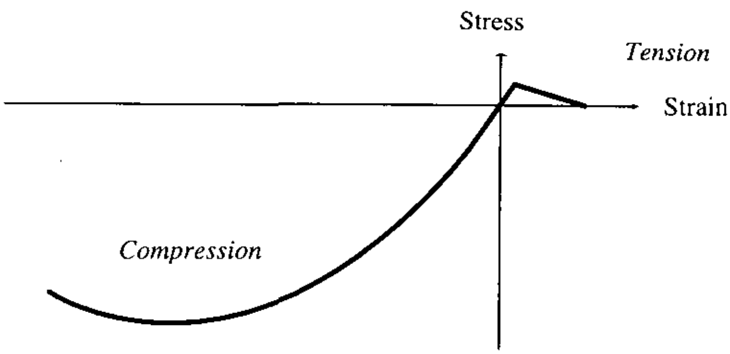


Fig. 5 Parabolic stress-strain model for concrete

The key issue that must be recognised when interpreting the results of a non-linear analysis of a concrete dam is the lack of experimental data on the mechanical behaviour of dam concrete and typical jointed rock, especially for seismic loading. This contrasts with the vast body of data that is available for conventional structural concretes. The work by Trunk & Wittman (1998) has demonstrated clearly the important influence that aggregate size has on the fracture characteristics of concrete and the need for testing of correctly sized specimens. Material parameters for conventional concretes might not be appropriate for dam concrete. As will be discussed later, the resulting uncertainties can have a profound effect on the use, interpretation, and reliability of non-linear dynamic analysis.

Fluid-structure coupling

The way in which the reservoir will act dynamically with the dam is primarily controlled by their respective natural frequencies. Westergaard (1933) gave the first rigorous analysis of dam-reservoir hydrodynamic interaction. He assumed a rigid vertical dam face vibrating harmonically into the reservoir and derived harmonic (i.e. Fourier) solutions to the governing differential equations. He showed that the hydrodynamic pressure distribution could be represented for practical purposes by a much simpler parabolic distribution. He also showed that if the compressibility of the water is neglected, then the pressure waves remain close to the dam face and do not radiate away. In this case, the hydrodynamic effects can be represented by a parabolic zone of water whose mass acts in unison with the dam. This so-called 'added mass' has become a standard simplified way of handling hydrodynamic effects. It has its limitations, in that it is now known that the flexibility of the dam changes the hydrodynamic pressure distribution, and that in some cases the compressibility of the water has a significant effect. Nevertheless, the Westergaard distribution, and its extension by Zangar (1952) to deal with inclined dam faces, give a reasonable estimate of the additional hydrodynamic forces that act on a dam.

Chopra (1970) adapted Westergaard's approach to cater for a flexible dam and this was subsequently incorporated in the well known computer program EAGD84 (Fenves & Chopra, 1984). Being of harmonic form, the solution is performed in the frequency domain, with the consequence that it is only suitable for linear elastic analyses. Chopra (1970) showed that if the ratio of the natural frequency of the reservoir to that of the dry dam is less than 1.2, then energy is transferred between the two, leading to radiation of energy away from the dam through the reservoir, and hence increased damping. For higher ratios, the compressibility of the reservoir can be ignored and the added mass assumption made. A good review of these issues is given by Greeves (1991).

If non-linear effects are of interest, they can only be modelled by solving the equations of motion in the time domain using step-by-step integration methods (Clough & Penzien, 1993). The EAGD84 type of frequency domain approach to modelling hydrodynamic effects is inappropriate in this case. Instead, the fluid must be modelled explicitly in such a way that the fluid response is coupled to the displacement response of the dam and foundation. Various techniques are possible, including finite differences, boundary elements and finite elements (Greeves, 1991). The easiest approach is to define the fluid domain in terms of displacement variables in the same way as the structure. The fluid is given a bulk modulus and no (or a very small) shear modulus. A major problem arises, however, in the form of spurious 'zero energy' modes in the fluid, which can seriously pollute the results. Greeves (1991) and Greeves & Taylor (1992) developed a special displacement-based fluid finite element that overcomes many of the zero energy problems, and can be readily incorporated in standard finite element codes. They demonstrated its use in the dynamic analysis of concrete dams. An attempt has been made to collect data from forced vibration testing of a prototype dam (Daniell & Taylor, 1998) in order to validate this element. Further work on this is proceeding. The use of displacement based fluid elements needs further refinement before they can be used regularly in dam analyses. Greeves (1991) showed that the Westergaard added mass approach can still give a useful estimate of the fluid-structure interaction effects for use in non-linear analyses when compared with more rigorous methods.

Foundation-structure interaction

Foundation-structure interaction can play a significant role in the overall dynamic response of concrete dams. The interaction arises from the flexibility of the foundation material and from its effectively semi-infinite extent. Foundation flexibility affects the overall stiffness of the dam-foundation-reservoir system, which in turn influences the system's natural frequencies and, hence, its overall dynamic response. The semi-infinite extent of the foundation allows energy to escape from the system in the form of radiation damping, whereby stress waves are able to propagate away from the dam without undue hindrance.

In general, allowing for foundation flexibility will reduce the natural frequency of the dam system. Allowing for radiation damping will normally significantly reduce the overall response amplitude of the dam system, leading, in some cases, to effective overall viscous damping of between 10-15%.

Foundation-structure interaction is similar to fluid-structure interaction and can be modelled in similar ways. Frequency domain solutions, which regard the foundation rock as a visco-elastic half-space, are particularly effective in linear elastic analyses, and are incorporated in EAGD84. They are not suitable for non-linear analyses.

Simic (1995) reviewed the modelling of foundation-structure interaction effects for gravity dams, with particular reference to enabling non-linear analysis. He concluded that the foundation could be modelled with standard solid finite elements, and that simple viscous boundaries to the foundation zone were adequate for modelling radiation damping effects. He also extended a method given by Clough & Penzien (1993) for inputting the ground motion into a standard finite element analysis in a manner compatible with foundation-structure interaction requirements, including radiation damping effects. The method uses the motion defined at the rock surface and eliminates the need for this motion to be deconvolved to obtain the bedrock motion for use as the input. In addition, it allows the substructuring of the dam system into linear and non-linear zones.

A fundamental issue that has so far tended to be avoided in most research is the jointed nature of real rock and the non-linear behaviour that this can produce. Current techniques tend to take a broad brush approach. They assume that the rock is contiguous and that energy losses due to local non-linearities can be represented by a simplified global viscous damping model. Such approaches are unable to take into account the potential changes to the structural mechanisms (including failure mechanisms) caused by the joints. In reality, the foundation rock is not contiguous. Clearly, the analysis of a jointed rock mass is complicated. However, the recent development of distinct and discrete element analysis computer programs, such as UDEC (Itasca Consulting Group, 1996), has produced tools which make possible detailed analysis of such problems. The dam, foundation and reservoir can be modelled as a complicated set of interacting deformable blocks. The block interfaces cater for joint movements, tension, friction and cohesion, as well as fluid flow. Each block is modelled internally by a finite difference zone that can include non-linear material property effects.

An ongoing research project at Bristol is investigating the use of distinct element tools, and is producing encouraging results. It has been shown that it is possible to model explicitly the whole dam system, including lift joints in the dam, jointed foundation rock, fluid-flow through the joints, hydrodynamic interaction and radiation damping in a single analysis on a fast personal computer. However, these tools need further rigorous assessment and validation before they can be used reliably in practice for dam analysis.

<u>Non-linear dynamics and chaos</u>

Previous sections have highlighted the main sources of non-linear and inelastic behaviour in dam systems. Such behaviour has a profound effect on the dynamic response. It causes the extreme response to be highly sensitive to slight variations in material properties, geometry and the features of the input loads. It is the basis of the science of chaos. This is a real phenomenon that manifests itself widely, especially in earthquake engineering. Chaos research has so far confined itself mainly to relatively simple, often contrived problems that are subjected to harmonic (i.e. sinusoidal) loading. The reason for this is simple: chaotic dynamics is very difficult to characterise and model, even for simple systems! It is essential that engineers dealing with non-linear dynamical problems recognise this fact, as it implies that a single, or even a few, non-linear dynamic analyses cannot give a definitive representation of the extreme response of a system.

It is vital that the philosophy of seismic safety assessment properly caters for the key physical phenomenon of chaos when non-linear behaviour is expected. The current state of knowledge tends to lead to the adoption of probabilistic assessment methods. For common structures, such as buildings, probabilistic issues are implicitly dealt with within the simplified procedures and safety factors defined in codes of practice (e.g. Eurocode 8). For dams, the knowledge and experience base is not yet broad enough to permit similar simplified seismic assessment procedures to be implemented. There are, however, many aspects that are common between the dam and buildings problems and thereby provide scope for adaptation of procedures for the latter to the former.

TYPICAL RESULTS FROM NON-LINEAR DYNAMIC ANALYSES

Mao & Taylor (1998) carried out a parametric study of a 30m high dam having a base width of 20.76m. The purpose of the study was to develop a conceptual understanding of the effects of foundation deformability and cracking on the overall seismic response. The dam-foundation system was represented by a 2-D, plane strain, non-linear finite element model, using four-noded isoparametric elements. The rock foundation was modelled as a contiguous rectangular block extending 90m below the dam and 90m upstream and downstream of the dam. Dead weight and hydrostatic loads were applied to the dam and foundation. Hydrodynamic loads were approximated using Westergaard added masses. Hydrostatic and hydrodynamic loads in any cracks that formed were not modelled.

In all the analyses, the concrete was assumed to have a tensile strength of 2MPa, an elastic modulus of 20GPa, and a fracture energy of 200N/m. Subsequent material tests reported by Trunk & Wittman (1998) suggest that the fracture energy for dam concrete should be around 400N/m. Foundation elastic moduli of 5GPa, 10GPa, 20GPa, 40GPa, and rigid were considered, with the tensile strength being taken as 1/10,000th of these values.

The dam was subjected to a horizontal synthetic ground acceleration time history (Fig. 4) that was generated to envelope the UK hard ground spectrum shown in Fig. 3. The history was scaled incrementally to determine the peak accelerations at which cracking initiated and propagated deep into the dam or foundation for each set of material properties.

Figure 6 shows the final cracked zones for each of the foundation moduli. Table 1 summarises the cracking accelerations. The softest and weakest 5GPa foundation produces nearly vertical cracks into the foundation from the upstream heel. As the foundation modulus and strength increase, the crack zone rotates towards the horizontal, becoming horizontal for the 40GPa case, in which the rock is considerably stronger than the concrete. Cracking in the neck region of the dam only occurred for the 20GPa, 40GPa, and rigid cases. This is probably because the stiffer rock inhibits the radiation of energy away from the dam body, thereby inducing higher stresses in this zone. Scale model shaking table tests of a similar dam configuration on a rigid foundation showed similar cracking along the base of the dam, but showed no signs of cracking near the neck (Mir & Taylor, 1994).

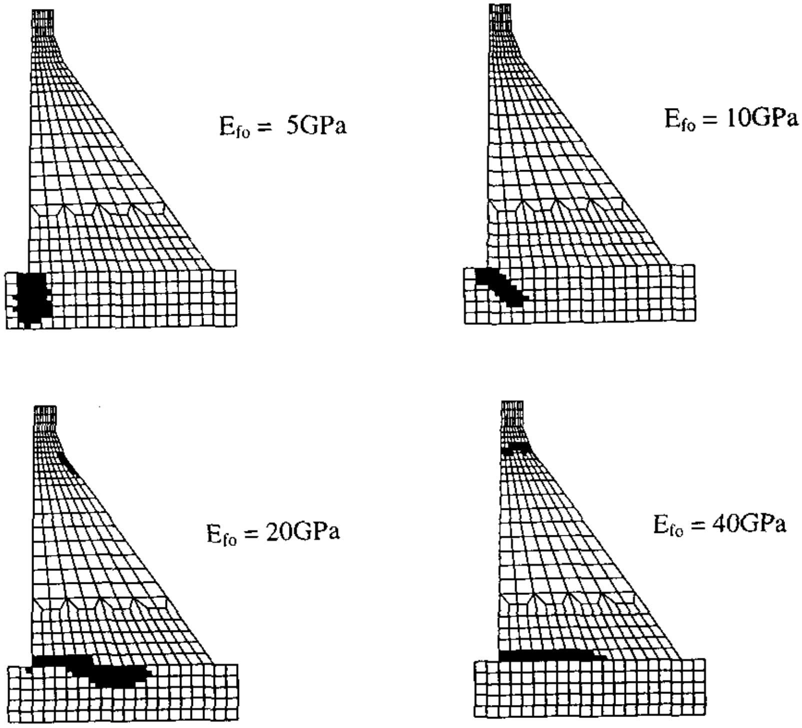


Fig. 6 Crack patterns for various foundation parameters

Table 1. Summary of peak ground accelerations causing cracking

Foundation Condition	Initial Base Cracking	Initial Neck Cracking	Deep Base Cracking	Deep Neck Cracking
Rigid	0.105g	0.210g	0.190g	0.210g
E_{fo} = 40 GPa	0.125g	0.250g	0.190g	0.250g
E_{fo} = 20 GPa	0.120g	0.240g	0.190g	-
E_{fo} = 10 GPa	0.070g	No cracking	0.160g	No cracking
E_{fo} = 5 GPa	0.040g	No cracking	0.120g	No cracking

The 5GPa and 10GPa cases are almost certainly unrealistic, especially with respect to the assumed low tensile strengths, which lead to the very low cracking accelerations. The other cases are more realistic, although probably conservative, bearing in mind that significant cracking is predicted for peak accelerations of about 0.2g. Observed behaviour of real dams suggests that they can withstand such shaking without suffering serious damage.

The study showed the potential of this type of analysis, but it also showed the need for improved knowledge of material properties before non-linear analysis of concrete gravity dams can be considered a reliable tool.

CONCLUSIONS

Non-linear analysis of the seismic performance of concrete gravity dams is still an immature science. It shows good potential in the longer term provided that progress is made in measuring appropriate dynamic material properties of dam concrete and jointed rock.

Given the chaotic nature of non-linear dynamic problems (and their consequent sensitivity to material parameters, initial conditions and the characteristics of input loadings), non-linear analysis procedures for dams will only be of substantial engineering value when they are robust enough to enable the associated uncertainties to be assessed within acceptable confidence limits. Much more research and development is needed before this stage can be reached.

ACKNOWLEDGEMENTS

The Authors gratefully acknowledge the financial support of the Engineering and Physical Sciences Research Council for much of the research on which this paper is based.

REFERENCES

Bathe K J, Walczak J, Welch A & Mistry N (1989). Non-linear analysis of concrete structures. Computers & Structures vol.32, No. 3/4, pp563-590.

Chopra A K (1970). Earthquake response of concrete gravity dams. Proc. ASCE, vol.96, No.EM4, pp443-454.

Clough R W & Penzien J (1993). Dynamics of Structures. 2nd ed. New York. McGraw-Hill, Inc.

Daniell W E & Taylor C A (1998). Forced structural and hydrodynamic responses of a concrete gravity dam. Dam Engineering vol.9, No.1, pp27-55.

Fenves G & Chopra A K (1984) EAGD-84 A computer program for the earthquake analysis of concrete gravity dams. Earthquake Enginering Research Centre, University of California, Berkeley, report no. UCB/EERC-84/11.

Greeves E J & Taylor C A (1992). The use of displacement type fluid finite elements for the analysis of dam-reservoir interaction. Dam Engineering vol.3, No.2.

Greeves E J (1991). The Modelling and Analysis of Linear and Nonlinear Fluid-Structure Systems with Particular Reference to Concrete Dams. PhD thesis. Department of Civil Engineering. University of Bristol, UK.

Itasca Consulting Group, Inc. (1996). UDEC - Universal Distinct Element Code, Version 3.0, Users manual, Minneapolis, Minnesota, USA.

Mao M & Taylor C A (1997). Non-linear seismic cracking analysis of medium height concrete gravity dams. Computers & Structures vol.64, No.5/6, pp1197-1204.

Mao M & Taylor C A (1998). Non-linear seismic analysis of concrete gravity dams. Accepted for publication, Proceedings of the Institution of Civil Engineeers, Water, Maritime and Energy Journal.

Mao M (1996). Non-linear Seismic Cracking Analysis of Concrete Gravity Dams. PhD thesis. Department of Civil Engineering. University of Bristol, UK.

Mir R A & Taylor C A (1996). An investigation into the base sliding response of concrete gravity dams to dynamic loading. Int. Journal of Earthquake Engineering and Structural Dynamics, vol.25, No.1, pp79-98.

Principia Mechanica (1981). Seismic ground motions for UK design. Report prepared for British Nuclear Fuels Ltd. and the Central Electricity Generating Board, UK.

Simic M (1994). Earthquake Analysis of Concrete Gravity Dam-Foundation Systems. PhD thesis. Department of Civil Engineering. University of Bristol, UK.

Trunk B & Wittmann F H (1998). Influence of element size on fracture mechanics parameters of concrete. Dam Engineering vol.9, No.1, pp3-24.

Westergaard H M (1933). Water pressures on dams during earthquakes. Trans. ASCE, vol.98, pp418-472.

Zangar C. (1952). Hydrodynamic pressures on dams due to horizontal earthquakes. Engineering monograph no.11, U.S. Bureau of Reclamation.