SAFETY
MANAGEMENT AND
RISK ANALYSIS

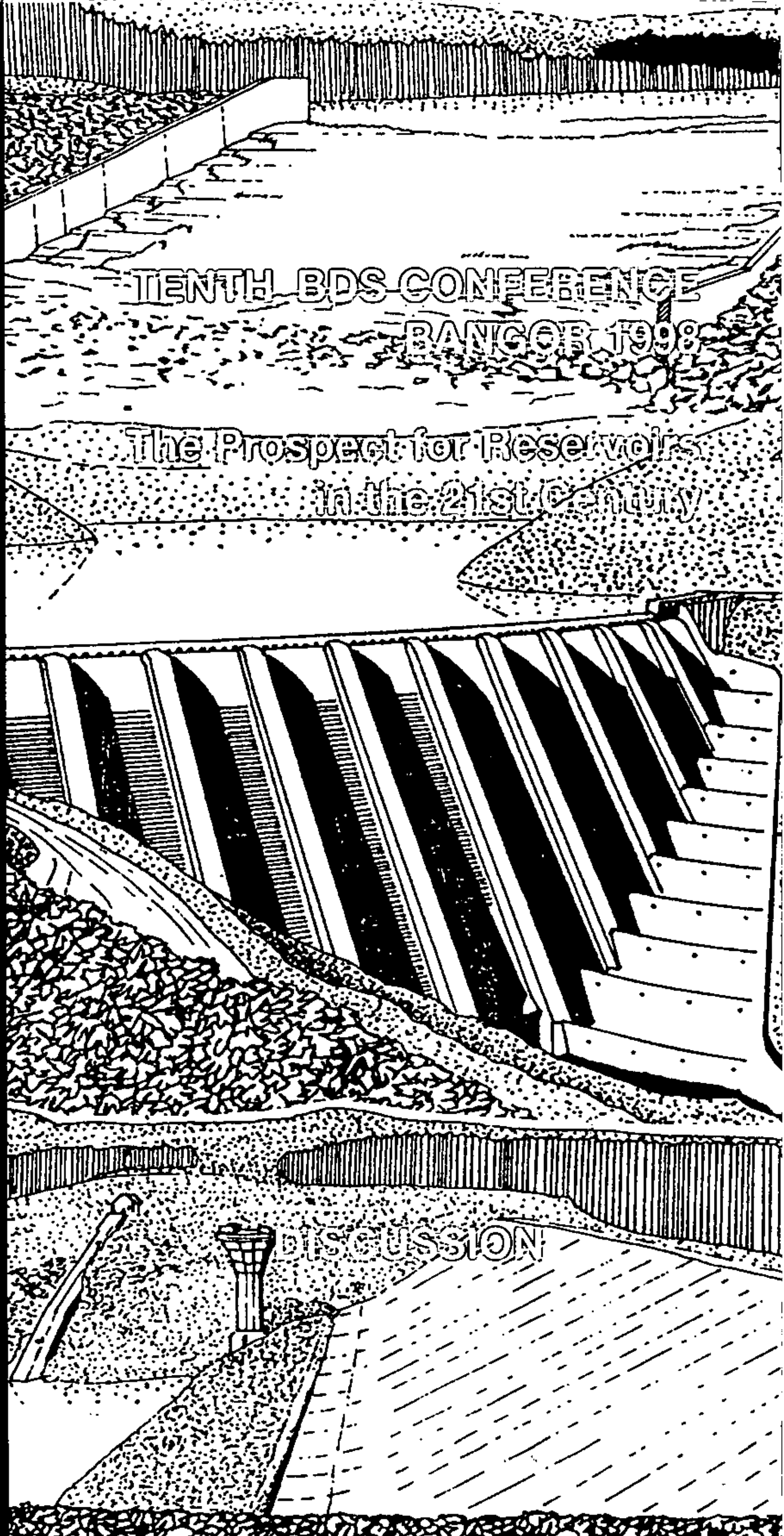
DEVELOPMENTS IN
LEGISLATION AND
PRACTICE

ASSESSMENT OF
DAM PERFORMANCE

RESERVOIR
OPERATION AND
MANAGEMENT

CONSTRUCTION AND
REHABILITATION
CASE HISTORIES

GEOFFREY BINNIE
LECTURE 1998



TENTH BDS CONFERENCE
BANGOR 1998

The Prospect for Reservoirs in the 21st Century

DISCUSSION

THE BRITISH DAM SOCIETY

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The Prospect for Reservoirs in the 21st Century

DISCUSSION

Edited by Ian Hay

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CONTENTS

PREFACE

This volume contains the complete record of discussions from the Tenth Conference of the British Dam Society entitled 'The Prospect for Reservoirs in the 21st Century' which was held at the University of Wales, Bangor, on 9-12 September 1998.

The Conference papers are in a separate volume published by Thomas Telford Services Ltd and available from the bookshop at the Institution of Civil Engineers, Great George Street, London SW1P 3AA.

TENTH BRITISH DAM SOCIETY CONFERENCE
UNIVERSITY OF WALES, BANGOR, 9-12 SEPTEMBER 1998
'THE PROSPECT FOR RESERVOIRS IN THE 21st CENTURY'

The Tenth Conference of the British Dam Society was convened at the University of Wales, Bangor, between the 9 and 12 September 1998. A total of 168 persons registered for the event. The attendance of one third of the membership at the Conference suggests that the Society remains in a healthy condition.

The theme of the Conference was explored via five topics: 'Safety management and risk analysis', 'Developments in legislation and practice', 'Assessment of dam performance', 'Reservoir operation and management' and 'Construction and rehabilitation case histories'. Thirty-three papers were published in the Proceedings and many were presented by their authors at the Conference. A technical exhibition ran alongside the Conference and eighteen organisations were represented. The well-balanced mix of exhibitors included specialist contractors, the technical press, specialist equipment suppliers and consulting engineers.

The keynote "Geoffrey Binnie" lecture was presented by Dr Andrew Charles of the Building Research Establishment. It was titled "Lives of embankment dams: construction to old age" and presented an account of British embankment dams in the form of an analogy with human life. As Dr Charles remarked, there is no period during life, from birth to death, which is exempt from troubles. This highly original paper was delivered with great style and dry humour. A transcribed version is included later in this document.

The technical programme of the Conference was structured to allow a full day of field excursions. Delegates visited concrete and embankment dams within the Snowdonia National Park. In addition, about 60 delegates attended a pre-Conference visit to several other dams in North Wales. Each of the visits was instructive and thought provoking, but none more so than the visit to the broken dam at Eigiau above Dolgarrog village, the scene of the last major dam disaster in this country. As ever, the visits were hugely enjoyable and our thanks go out to the many host organisations involved, notably First Hydro Company, BNFL, Magnox, Dwr Cymru Welsh Water and National Power.

The social programme organised by Mair Williams included an excellent programme of visits to local cultural attractions. Another highlight of the

Conference was a light-hearted quiz night with questions on dam-related subjects. The Conference Dinner was an extremely pleasant affair, with delegates enjoying traditional Welsh fare to the strains of harp music. The principal guest was Iddon Jones, retired dam engineer and local 'grandee', whose response to Chairman James Martin's toast was one of the most humorous after-dinner speeches ever received by the Society.

The general consensus of the delegates was that the Conference was a highly successful event. The success can be attributed to the contributions of a great number of people. Without wishing to make this sound like the 'Oscar' awards ceremony, I would like to give special thanks not only to the authors, presenters, session chairmen and technical reporters but also to those delegates who contributed to the lively discussions at the end of each technical session. I also wish to single out Karen Morgan Tallents and Jenny Marshall of the University's Conference Office, who provided a highly professional service in managing the day to day arrangements. Last but not least, I must express my gratitude to my colleagues on the Organising Committee - Ian Hay, Jim Millmore, Paul Tedd and Owen Williams, whose assistance and support were immeasurable.

Ian Carter
Chairman of the Conference Organising Committee

OPENING REMARKS

by Professor Roy Evans, Vice Chancellor, University of Wales, Bangor

It's a very great pleasure for me to welcome the Dam Society Conference to Bangor for its 10th Biennial Conference. Croeso in Brifysgol Cymru Bangor. In my role here I welcome many conferences to the University. I'm always pleased to do so but today I'm particularly pleased because I have an opportunity of welcoming people from my own profession.

I started work in a design office for Freeman Fox & Partners in 1966 and ever since then I've had a huge respect for the profession of civil engineering. Civil engineers strive to meet the very real needs of their communities and to do so, of course, we have to contend, according to the old cliché, with the forces of nature. The cliché is very true and I think in this area more than most the contribution of civil engineers is very much valued and has been for a long time. I hope that some of you have had a view from the Halls of Residence of the two major bridges crossing the Menai Straits. As a result of the work of Telford and Stevenson towards the beginning and the middle of the last century, civil engineers have had a huge impact in this area. Although my own experience is mainly in bridge design and later in bridge research, I spent enough of my early years plotting seepage lines through gravity earth dams, and later trying to fit finite elements to the walls of curved arch dams, to know something of the very real challenge of dam engineering.

I found the themes of the Conference and the titles of the papers fascinating. One of the things that has impressed me most is the interdisciplinary nature of much of the work that is going on here today. We have no civil engineering department here at Bangor although we have a department of electronic engineering and computer systems and also a major institute of environmental sciences. We have a school of biological sciences and a school of ocean sciences. We have a school of agriculture and forest sciences and we have commercial units as well in biocomposites in environmental chemistry. We have an institute for terrestrial ecology and also we have a centre for arid zone studies. Now if anything brought home the international significance of university research, it must be to have a centre for arid zone studies at Bangor! But it's part of this interdisciplinary emphasis that we place on environmental matters.

I remember once hearing, when I was heading a school of engineering elsewhere, that where the engineering industry had problems, universities had departments, and if the problems did not align particularly well with the remit of the university department then they tended to go unsolved, at least on the academic side. I think that that is why I am very keen to emphasise the importance of interdisciplinarity between our various disciplines within the university. When you subdivide an academic activity you're really trying to subdivide what is essentially a continuum and wherever you put the line you always put it in the wrong place. I think the interdisciplinarity, the theme of this Conference, is something that I find very attractive and I see that you're covering issues not only on dam construction but also on the very important environmental consequences of that construction.

I'm very pleased to welcome you to Bangor. I apologise for the weather but it is the sort of weather that keeps reservoirs full! It also keeps Conference sessions full because delegates tend to attend more to their academic matters during this sort of weather than normally. We'll do all we can here to make you comfortable and I hope you have a very successful Conference. Welcome to Bangor.

SESSION 1
SAFETY MANAGEMENT AND RISK ANALYSIS

Chairman	James Martin
Technical Reporter	Andrew Robertshaw

Papers presented

1. The role of risk analysis in the safety management of embankment dams.
   J A Charles, P Tedd and H D Skinner
2. Risk assessment strategies for dam based hydro schemes.
   N M Sandilands, M Noble and J W Findlay
3. Hazard and reliability of hydraulic equipment for dams.
   J Lewin

Presentation

1. CIRIA Research Project 568 "Risk management for UK reservoirs".
   H W M Hewlett.

Paper not presented

1. A programme of risk assessments for flood gates on hydro electric reservoirs
   N M Sandilands and M Noble

SESSION 1

Chair

Welcome from the British Dam Society. It is good to see the large numbers and the spread of ages here today which ties in well with the theme of the Geoffrey Binnie Lecture this year - 'Lives of embankment dams: construction to old age'. There is over a third of the membership here which I think is very good for a conference. The number and quality of the papers also reflect the lively position of the Society at the moment.

We are delighted to see several members from the European Community here, strengthening the links between the British Dam Society and what is becoming known as EuroCOLD. There is a lot of interaction these days between the members of the Community which is very healthy and so we welcome particularly members from other European countries.

The first session is on safety management and risk analysis for dams. This whole subject is becoming of considerable interest to practitioners, owners and consultants alike and is an area that potentially could bring great difficulties and could take the dam industry into areas that the nuclear industry already finds itself in. My personal view is that that may not be entirely healthy for our profession. So it's an important subject and I'm very glad it's the focus of the first session of our Conference this year. All of the people speaking this morning are what I would call real engineers - those who are in the business of making judgements and doing real engineering. That's where I personally believe dam engineers should stay and not get too wound up in the detailed "ten-to-the-minus" numbers.

Presentation

Henry Hewlett (RKL-Arup)
CIRIA Research Project 568 "Risk management for UK reservoirs"
This is a good opportunity to tell everybody what is going on with the CIRIA project because it's referred to in several of the other papers.

The objective is to produce a guidance document which will provide owners, operators, regulators and all those concerned with reservoir safety with guidance on risk management of UK reservoirs. The project is funded by DETR, British Waterways, North West Water, Severn Trent Water, Thames Water, Yorkshire Water, Scottish Hydro-Electric, Scottish Water Authorities and the Environment Agency. They're all involved in the Steering Group so we're getting a wide range of views as we progress with the study.

The contract was awarded nearly a year ago to a consortium led by RKL-Arup. Andy Hughes is the Project Director, assisted by myself, and we've got several other organisations on board. HR Wallingford are assisting on dam break modelling and contingency planning using knowledge gained from a European-wide initiative called CADAM (Concerted Action on Dam Break Modelling). HR Wallingford will also undertake risk assessments for various reservoirs using different methods as the study progresses.

EQE are a firm of risk analysis specialists who haven't done very much work in the dam industry. They're going to be comparing risks in the dam industry with other sectors of the UK economy such as the nuclear and North Sea oil industries.

Iain Moffat of the University of Newcastle is helping us with advice on failure mechanisms and providing information from his database on overtopping and other incidents.

Andrew Charles and Paul Tedd of BRE are making available their extensive database of dam incidents, which will be used to try to get some idea of the probabilities of certain events. We're a bit dubious about allocating probabilities to failures or major incidents but I think we can put an upper and lower band on some of the them.

The study is in four stages. An assessment of the industry requirements and prioritising tasks have been completed and we're now into Phase 2, the Analysis of Risks posed by Reservoirs. That will be followed up by examining the role of risk assessment, which will involve undertaking some trial risk assessments, and finally preparing a guidance document.

Stage 1 involved a literature review and consultation with those involved in UK reservoir safety. We sent out a questionnaire to over 100 people - many of you here responded and I would like to thank you for that. We also held a workshop at HR Wallingford in January attended by an invited audience of about 30 people, a cross-section of the industry, to get a good idea of where they felt we should be going. In April/May this year we produced a position report setting out the conclusions of our Phase 1 work and how the rest of the study should proceed.

Conclusions for Stage 1 were broadly as follows:

The application of risk assessment should help improve reservoir safety and should therefore be welcomed. A relatively simple and cheap and easily understood risk assessment methodology would be preferred. Full probabilistic risk assessments using fault trees, event trees etc are not desired although a simplified approach using such methods may be appropriate in some cases, particularly for larger dams.

Hazard indexing, which is a parameter based on the value of life and property at risk in combination with engineering matters may be useful in identifying a potential consequence of failure and in classifying reservoirs particularly as far as remedial works are concerned.

There's concern about the lack of data on dam incidents. Unfortunately, our dams are too safe, which is good news, of course! Owners of reservoirs should be encouraged to provide information on incidents and near misses on a confidential basis to incorporate on the database. I'm pleased to see some papers in this conference are being fairly open about incidents and that will be helpful.

We have also studied the international approaches to risk analyses. Basically each country seems to be doing its own thing and we feel we should do the same because our stock of dams are mainly embankment dams, older than most of the other countries', and merit special treatment.

The other thing people felt very strongly about was inundation maps and contingency planning. There are a lot of people undertaking dam break analyses at the moment but there doesn't seem to be a consistent approach. Some people just file away the results of their analyses, others are preparing contingency plans and discussing them with emergency services. We feel a constructive and consistent approach is desirable. The question of whether the preparation of contingency plans should be mandatory also needs to be considered.

The final report should be prepared in about a year's time and it will include a summary of incidents and failures at UK reservoirs, a discussion on risk assessment methodologies, a proposed methodology for use in the UK, mitigation of risks and a section on emergency planning issues.

I should emphasise that forms of risk assessment are already undertaken on our reservoirs - Panel Engineers' reports being the most obvious one. The CIRIA report is meant to complement these, not replace them. Unfortunately it will be another guide to carry around in our briefcases but it should ensure our good record of dam safety is maintained as dams get older.

Discussion

Geoff Sims (Brown & Root)
I have three questions.

For Neil Sandilands: You mentioned, in connection with a gate that had failed, that the problem lay in a design fault. You said that it was a design fault that was not apparent in any way until failure. I am intrigued by this and wonder whether you could elaborate a little. You have referred to the friction factor at the trunnion bearings being 0.3. How did this come about? Was the bearing designed to have this coefficient of friction, or was it to do with corrosion or some other sort of ageing process? Was there noise from the bearing during operation? Was the hydraulic oil pressure high? It is the concept of a design fault that gives no evidence of its existence that is intriguing.

For Andrew Charles: What is the purpose of an F-N chart? Is it used in connection with a risk assessment at an existing dam for example, to demonstrate that the anticipated probability of failure lies within a range that the public will find acceptable? Does its successful use depend on a reasonable assessment of the probability of failure?

For Jack Lewin: We are seeking to identify spillway gates that are designed to open automatically. You have referred to two in your presentation, but you did not name them. I know of Jindabyne in Australia, designed in about 1960. I also believe Gibb designed gates at Victoria in Sri Lanka to open automatically. Could you please name any other examples of which you are aware?

Neil Sandilands (Scottish Hydro-Electric)
Yes, the design fault was that friction at the bearings was not taken into account. When a friction factor of around 0.3 was realised, the gate failed. This was caused by overloading and failure of the strut bracing, followed by buckling failure of the main struts. The friction factor increased due to the corrosion of the upstream faces of the hinge pins. Due to the large size of the gates and the extreme pressure on the film of lubricant, the lubricant was compressed and water penetrated the film by capillary action. Research carried out by the United States Bureau of Reclamation (USBR) indicated that the effectiveness of lubrication was reduced only forty minutes after application. The gates were lubricated weekly and operated normally until immediately before failure. The remedial work undertaken included the installation of "Lubron" self-lubricating bearings on the new gate and the installation of an automatic system on the remaining gates which lubricates them during operation.

Andrew Charles (Building Research Establishment)
I started off my presentation by saying that we had to look at the dam reservoir system and the downstream valley system, and certainly to get something you'd plot on an F-N chart you've clearly got to look at both. In other words, you've got to come to some evaluation of the likelihood of failure and also the consequence of that failure, and the results will only be as good as those two analyses. Having done that, you can then look and see how they compare

with these criteria of acceptability that people are suggesting. If you're saying that you feel a lot of the numbers are dubious, I'm sure they are, but doubtless they're dubious in all the other industries where these things are used. Whether ours are much more dubious than theirs is another matter, but it does give a way of comparing different situations and, indeed, different dams.

Jack Lewin (Independent Consultant)
In response to Geoff Sims, one example was quoted in New Civil Engineer of an event in France: The paper mentions the self-induced opening of two radial spillway gates on the Mavcice Dam in Slovenia which resulted in a discharge of $1192m^3/s$. My Australian clients are very aware of the possibility of inadvertent opening of gates. All automatic control systems should be designed so that the operation of a gate is for a limited period only, followed by a dwell interval. This is important to prevent surges both in the reservoir and downstream in the river, as well as prevention of inadvertent automatic operation. It can be easily incorporated in the design of electrical control systems.

Jack O'Keeffe (Electricity Supply Board, Ireland)
During the management of a routine flood at one of the dams owned by the Electricity Supply Board, Ireland, all three spillway gates were almost fully opened, inadvertently, due to a gate location indication system fault on DC power supplies. The gates were given a command to open, but the indication to the operator was absent, so he assumed that the gates were not moving. The situation was noticed within 20 minutes, and the gates were immediately closed. However, flood damage was caused to property for a distance of approximately 3 miles downstream from the dam.

It would be easy to blame the operator but the incident showed up a number of other flaws in the system apart from the immediate cause viz. the DC fault. There were no "Dead Man" switches, no tailrace alarm, no back up alarms and no motor-running indication. A number of fail-safe systems have since been installed at all spillway gates including physical observation of gates during movement - either at the gate or by camera.

Geoff Sims (Brown & Root)
The F-N curve is a conventional way of illustrating aspects of grand policy. For an individual structure it is vital, in my opinion, to include the outcome of skilled engineering judgement based on detailed and informed inspection.

Chris Binnie (Independent Consultant)
Neil Sandilands mentioned that in the North of Scotland there were several hydro systems with reservoirs in cascade. Neil and I visited Eigiau and Coedty dams yesterday. Eigiau dam had failed in 1925, the resulting flood wave overtopping the Coedty dam downstream leading to a flood wave and 16 deaths in Dolgarrog. I am interested to know whether in the Scottish schemes the dam break flood wave exceeded the PMF design condition and, if so, whether the dam had been analysed for it?

I am interested in Andrew Charles's statements about dam break inundation mapping being in the public domain. I have been involved in a number of dam break inundation mapping studies. My experience is that dam owners are very concerned about the possible adverse reactions from the public if the public became aware of the extent of dam break inundation. I know of no owner who is yet prepared to allow the release of plans to the public. Most owners are only prepared to let the emergency services have a copy. I know of only two owners who have even allowed it be known that such studies have been done. Does Andrew consider that the inundation maps should be in the public domain?

Regarding Henry Hewlett's points about contingency plans. When inspecting dams, I discuss with each owner his contingency plans and generally include some reference in my report. For instance, on one occasion where there was no bottom outlet but some houses downstream, I required the owner to enter in the Reservoirs Book where major pumps could be obtained in the event of an incident requiring water level lowering in the reservoir. Does Henry consider that contingency planning should be mandatory and become an item in the Inspecting Engineer's report?

Mark Noble (Scottish Hydro-Electric)
In reply to Chris Binnie, Scottish Hydro-Electric have just commenced a programme of inundation studies and are currently carrying out a pilot study on the Glenmoriston catchment which will be used to establish policies for future studies. Information on the size of flood waves caused by failure of upstream dams will be available from this study and from the programme of future studies. The studies will consider the worst case scenario where a cascade failure occurs. When the upstream dam complies with current engineering standards on floods, earthquakes and stability under static loading, as is generally the case with Scottish Hydro-Electric dams, then the flood from an upstream failure would not be used for design purposes unless risk criteria indicated otherwise. At the current stage of risk and

inundation studies this situation has not arisen.

Jack O'Keeffe (Electricity Supply Board, Ireland)
With regard to the distribution of Inundation Reports, the Electricity Supply Board, Ireland, have carried out inundation studies for all of their "Category A" dams (Floods & Reservoirs Safety Guide). This information has been passed on to the relevant local authorities, following meetings where the process and findings were explained to representatives of the local authorities. Initially, there was a reluctance by local authorities to have possession of the information. However, all information is now with local authorities for use in emergency planning. The information is not used for normal planning purposes.

Ken Shave (Babtie Group Consultant)
I draw the parallel between the work of the Environment Agency and the flood maps to which you refer. I am Supervising Engineer for a large flood storage reservoir on the River Medway and we are currently undertaking a dam break analysis to determine the effect this would have on the existing flood maps which the Environment Agency hold.

These maps are in the public domain, and are regularly used for reference before commenting on planning applications received from Local Authorities and the County Councils.

The dam break analysis is likely, under PMF conditions, to extend the published areas liable to flooding which currently represent the 1:100 year event (being based on an historic flood of that intensity in 1968). While the maps will be revised to take the dam break into account, the existing maps are already an integral part of flood warning procedures which incorporate a system for activating the emergency services, and the emergency plans of the Local Authorities, in times of significant flood. This highlights the current difference between the consequences of overtopping/failure from a managed flood storage embankment and an impounding dam in that public safety is already taken into account in the operating procedures of flood storage reservoirs.

I would anticipate that planning matters will continue to be considered under the 1:100 year condition but that the PMF flood extent will also be shown on these plans, and I imagine the Environment Agency will continue to place the 'updated' flood maps in the public domain.

Arthur Penman (Geotechnical Engineering Consultant)
Release of water from spillways can sometimes cause alarm. At the 85m high Matahina dam in New Zealand, some water was released to lower the reservoir level as a precaution following the 6.6 magnitude Edgecumbe earthquake of 1987, which shook the dam. People on a bridge in Edgecumbe, fearing the worst, saw the front wave of the released water coming down the river and wasted no time in getting into their cars and making for higher ground. Fortunately those on foot soon realised that this was not the flood from dam failure and the panic quickly passed.

At the 19m high Cwmwernderi puddled clay core dam, built in 1901, retaining 277 x 10^3m^3, we (BRE) looked at the condition of the old core, taking samples for testing, installing total pressure cells, piezometers and a system of very accurate topographical surveying to check on any horizontal or vertical movements. The reservoir had been completely emptied as part of a comprehensive inspection, and to cater for the PMF, a 1.8m diameter steel pipe was installed alongside the wide weir spillway with its soffit at weir rim level, so as to lower the TWL by 1.8m, to give greater capacity at time of flood. When the reservoir was refilled, we needed it to go to its old TWL, so we made up (in BRE workshops) a stoutly framed plywood disc containing a sliding gate, which we fitted into the outer end of the steel pipe. It worked very well and brought the reservoir level up to and overflowing the weir. We needed to take readings over a few weeks to allow conditions to reach equilibrium and it was all very satisfactory. But before we could effect the intended controlled lowering through the sliding gate, some weekend vandals attacked the disc, loosening it from the pipe so that the water pressure blew it out, rapidly lowering the reservoir by 1.8m, releasing 51 x 10^3m^3 of water. This produced a bit of a flood in the little stream leading from the reservoir which caused a little excitement but no damage.

Jonathan Hinks (Halcrow Group)
I have found the various papers presented this morning very interesting and useful. It may therefore seem a bit churlish if I pick up on a single sentence in the paper presented by Mr Sandilands. I do so because there is a very similar sentence in the paper which Dr Sims will present after the break.

What is bothering me is the suggestion on page 21 that some low risk reservoirs should be removed from the legislation as a result of risk analyses. I have no problem with introducing a less intensive surveillance regime for such reservoirs but I do question the wisdom of taking them permanently out of the ambit of the Act. I imagine that Mr Sandilands may be thinking mainly of structures owned by major and responsible owners. I am concerned about the effect such a move would have on privately owned dams.

At present, the rules as to which reservoirs fall within the Act are very simple and there is only very limited scope for argument. I do not think that it would be helpful to thrust the decision onto the Inspecting Engineers who already have to withstand pressures put on them by owners keen to minimise expenditure. Furthermore, opinions between Inspecting Engineers can vary and circumstances can change. A new superstore had been built downstream of one allegedly Category D dam which I inspected this year. If the previous Inspecting Engineer had taken it out of the Act it would presumably never have been inspected again. In another case, I inspected an allegedly Category D dam and found a group of houses downstream. If an Inspecting Engineer does get it wrong at least under the present system the decision is reviewed ten years later.

I would ask that as little opinion as possible should go into deciding on whether a reservoir comes within the Act. In other words let's keep all reservoirs holding more than 25,000m^3 within the Act.

Ian Gowans (Cuthbertson)
In Scotland we have a large number of low risk Category D dams, many in very remote parts of Scotland. These reservoirs are retained by low weirs, less than 1m in height, but exceed the 25,000 cu m capacity because of their large surface area. They are owned by Private Estates with limited funds. Inspection of low weirs, often only 0.3, 0.5 or 1m high, is in my opinion imposing unnecessary high costs on these private owners. If these weirs were washed away, it would not be noticed, no damage would occur and to impose the full rigours of the Reservoirs Act cannot be supported in these instances.

Andy Hughes (RKL-Arup)
In reply to Mr Hinks' contribution, I believe that our legislation should move away from one based on capacity to one based on hazard. There are many dams in this country with a capacity of more than 25,000 cubic metres which should not be subject to the Act. They could fail and no one would notice. However, there are many dams with a capacity of less than 25,000 cubic metres which would cause loss of life and extensive damage and which should be subject to legislation.

I also question the removal from the Act of dams subject to heavy siltation. I know of two owners who are currently removing silt from reservoirs that contained less than 25,000 cubic metres of water and which, following desilting, will contain much more than 25,000 cubic metres of water.

Written Contributions

Iain Moffat (University of Newcastle)
In reviewing hazards to dams in the context of risk assessment it is imperative that a precise and consistent terminology is employed. Reference to '*dam failure*', in particular, must be based on a clear and unambiguous understanding of what constitutes 'failure'. A precise and rigorous definition is preferable to the use of 'softer' definitions which lead to quite erroneous conclusions as to the historical frequency and/or probability of failure.

Failure events should be distinguished from other major incidents as a unique characterisation, eg.:

> *Failure:* A major uncontrolled and unintended release of retained water, or an occurrence whereby a dam is rendered unfit to safely retain water due to loss of integrity.

A complementary characterisation of serious non-failure events as major incidents of differing degrees of severity provides a logical and consistent basis for the analysis of long-term performance trends, thus:

> *Major Incident:* A serious occurrence which necessitates an immediate reaction and/or drawdown and restriction on impoundment level to obviate a significant risk of subsequent progressive deterioration leading to catastrophic failure and a major uncontrolled release of water.

Major incidents may be further classified by the perceived threat level, eg. 1, 2, 3 etc. where level 1 would rank as an incident stopping just short of failure.

A further characterisation should be used to account for any major uprating and improvement of a dam which is not the consequence of a major incident, thus:

> *Preventive remedial works:* Major works carried out with a view to correcting recognised deficiencies in design or construction, or works carried out to remedy less urgent but potentially serious problems. (Examples might include reconstruction and uprating of spillways, a major increase in or restoration of freeboard, or the provision of a toe berm etc.)

The initiating mechanism for a specific incident (or failure) is frequently masked by a progression from one mechanism to others, with the latter being ultimately responsible for the final outcome, eg. internal erosion → local depression → overtopping by an extreme flood event, etc. Published statistics on the relative frequency of occurrence of different mechanisms must consequently be treated with circumspection.

Published and frequently quoted statistics on frequency or probability of failures etc. almost invariably relate to the minority of 'Large Dams' qualified for inclusion on the World Register. The c. 85-90% majority of lesser dams are, as a generalisation, likely to be older, less capably engineered and subject to less rigorous surveillance, all factors which will have implications with respect to incident and/or failure rates.

A generic characterisation of major incidents and failures based on extensive study of UK dams subject to safety legislation has suggested (Moffat 1982):

<u>Primary mechanism:</u>	
seepage/internal erosion	c. 35%(±5%)
overtopping	c. 35%(±5%)
instability/overstress	c. 15%
settlement/deformation	c. 10%
other/uncertain	c. 10%

The foregoing analysis is compatible with published figures from overseas. Great care is required, however, when comparing national statistics in the light of differing national circumstances relating, eg. the nature and size of the dam population at risk in terms of distribution by age, type etc.

Many upland valleys in the UK accommodate chains of reservoirs, where the *potential* for a cascade failure must be recognised. Two serious events of this nature are on record in the UK:

Upper → Lower Creggan	(1908)	N. Ireland
Eigiau → Coedty	(1925)	N. Wales (the Dolgarrog failures)

In parallel with generic primary mechanisms, recognition must also be given to major incidents which derive from critical component failure, ie. failure of gates, valves, or other ancillary equipment or structures.

There are two further hazards to which it is suggested we pay insufficient regard. Firstly, there is the very real possibility of serious damage resulting from deliberate action by subversive organisations. While stopping short of generating a catastrophic failure such activities, directed against valves and pipework, would result in a serious release of water and, at least, very considerable embarrassment. The second is the probability that for some UK dams it is at least as appropriate to consider the hazard represented by a crashing military aircraft as it is to consider seismic or other very low probability hazards.

In conclusion, in any statistical assessment it should be recognised that:

- forensic investigation of incidents and failures, particularly those dating from earlier years, is commonly less than conclusive and isolation of the initiating mechanism, is generally difficult;
- historical records of incidents/failures are less than exhaustive, generally inadequate, and variable in consistency of reporting standard;
- published data generally relates to that 10% minority of 'Large Dams' qualifying for inclusion on the world Register. (In the case of the UK only some 530 dams out of c. 2500 subject to the 1975 Act so qualify;)
- the great majority of incidents can be traced to design and/or construction inadequacy. There is therefore an associated 'date of construction' (and, let it be said, 'designer') dimension to be considered in any analysis.

Reference:

Moffat, AIB: (1982) *'Dam deterioration - a British perspective'* 2nd BNCOLD Conference, Keele, pp.103-115

Jack Lewin (Independent Consultant)
Spillway gates can be designed to open automatically under power, which can be the reservoir water or an external power source, or they can be arranged to open under gravity. At the Victoria Dam in Sri Lanka, the gates can open automatically without a power source.

The first group includes water level controlled gates such as drum and sector gates, as well as radial automatic gates (Lewin 1984 and 1995). Motorised gates are actuated under automatic control by programmable logic controllers operating feed back level control systems (Lewin 1986, July 1995 and 1995). Electromechanical automatic control systems are still in use, although they do not provide the flexibility of electronic systems.

Radial or vertical lift gates which open under gravity are counterbalanced to open. They require a power source such as a winch or oil hydraulic rams for closure. One example of radial spillway gates of this type is in the Ivory Coast in Africa.

The eight 12.5m wide counterbalanced spillway gates at the Victoria Dam in Sri Lanka (Back and Wilden 1988) open under gravity and the gate openings are automatically controlled by floats. The floats actuate oil hydraulic poppet valves. They direct the oil from the piston side of the gate closure cylinders to the tank. When the gates have reached their appropriate opening step, an actuator on the gate closes another poppet valve which stops the flow of oil and locks the cylinder in position. Other floats are connected over pulleys to electric limit switches which energise relays for solenoid operation of oil hydraulic, directional control valves. This provides a standby system for opening the gates.

References:

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Rod Bridle (Independent Civil Engineer)
I was asked if the Reservoirs Committee had any action in view with respect to the implementation of some kind of national standard in relation to Contingency Planning against dam breaks. At the Conference, I said that there was no action being taken. However, I will take advantage of the written discussion to correct myself and explain that the Reservoirs Committee are awaiting the outcome of the CIRIA project on Reservoirs and Risk, before taking any action. This is because it will include discussion and make recommendations on what should be done about Contingency Planning. That report will be completed towards the end of 1999.

SESSION 2
DEVELOPMENTS IN LEGISLATION AND PRACTICE

Chairman	Rod Bridle
Technical Reporter	Howard Lovenbury

Presentations

1. Seismic risk to dams in the UK: application note to the 1991 BRE guide.
   N Reilly
2. Seismic behaviour of ancillary structures.
   C A Taylor
3. Video - Matahina Dam after the Edgecumbe earthquake, New Zealand 1987

Papers presented

1. Should reservoir control systems and structures be designed to withstand the dynamic effects of earthquakes?
   G M Ballard and J Lewin
2. The review of the Reservoirs Act 1975.
   G P Sims and N M Parr

SESSION 2

Chair

This session starts with three presentations not included in the Conference Proceedings. The first speaker is Nick Reilly, who is speaking to a new publication which will be coming out soon - the "Application Note" to the Seismic Guide. This will be giving a lot of additional information and reassurance to reservoir engineers regarding seismic analysis of dams. He will be followed by Colin Taylor from Bristol University who has been working on the effects of seismic events on the ancillary structures to dams. He is not going to speak to the paper he's written in the Proceedings but about seismic analysis of ancillary structures. Finally, Geoff Ballard and Jack Lewin will introduce and commentate on a video about the Matahina Dam following the Edgecumbe earthquake in New Zealand in 1987.

Presentations

Nick Reilly (Consulting Engineer)

Seismic risk to dams in the UK: application note to the 1991 BRE guide

Most of you will be aware of the Engineering Guide to Seismic Risk to Dams in the UK, which was published in 1991. It is an excellent and useful document which brought together a great deal of information in the area of seismic risk to dams in the UK which was not available before. It aimed to be consistent with the ICE floods guide, at least in terms of return periods, but it immediately ran into some criticism in respect of the peak ground accelerations proposed for the highest categories of dams. In particular, engineers were being asked to assess dams for peak ground accelerations of up to 0.375g when they could well have been simultaneously designing dams in seismic areas overseas in, say, Indonesia for only 0.18g. Dam owners and panel engineers alike queried the justification for this although it has to be said that little adverse comment on this aspect had been received during the draft stage of the Guide.

A working party was set up under the auspices of the ICE Reservoirs Committee and chaired by Roy Coxon. The working party included representatives of dam owners together with panel engineers and Dr Charles, one of the Guide's authors, and operated on a voluntary basis. It met many times over the period to 1997 and collected a great deal of useful information including results of recent research. But the work pressures on the individual members were such that progress on pulling the information together into a document was slow.

Towards the end of 1997, thanks to the efforts of Dr Charles and Roy Coxon, funding was obtained from DETR's reservoir research budget and I was appointed under subcontract to the ICE to pull together a draft of the document which we are calling "An Application Note to the Guide". The aim of the Note is to bring to the attention of the dams community the results of recent research and developments as well as to review the thorny issue of risk and return periods. This work is now complete and the Note will be published shortly by the ICE at a charge in the region of £10. It will be complementary to the Guide.

There are three main changes of emphasis in the Application Note and it is on these that I would like to concentrate. These changes are:

- A staged approach to the seismic assessment of existing dams
- The results of recent seismic hazard assessment for the UK
- Results of a re-assessment of the return period proposed for Category IV dams.

The first area is relatively minor and probably reflects what many engineers were already doing. Tables 5 (reproduced below) and 6 of the Guide identified the levels of analysis likely to be adopted for given categories of embankment and concrete dams, showing that the more sophisticated methods were suited to the higher hazard categories. There is no disagreement with this but the Note will emphasise that it will often be appropriate to proceed through stages of simpler and less expensive analysis before embarking, for example, on a full 3-D dynamic analysis.

Guide Table 5

Proposed levels of safety evaluation for embankment dams

Dam Category	**Dam Height**	
	<15m	**>15m**
I	E_a	E_b
II	E_b	E_b
III	E_b	E_c
IV	E_d	E_d

E_a in general no seismic safety evaluation is required

E_b look for features particularly vulnerable to earthquake damage and undertake seismic analysis only if such features are found

E_c in addition to E_b, some relatively simple analysis will usually suffice

E_d in addition to E_b, a full dynamic analysis will often be appropriate

Since the Guide was drafted important work has been done on UK seismicity. In 1996 the results of a DTI funded study "Seismic Hazard of the UK" carried out by Musson and Winter of BGS and AEA Technology was published (see reference). This presents the findings of a comprehensive and detailed examination of seismic hazard over the whole of the UK. The main result of this work is Fig 23 of the report which will be reproduced in the Note. This Figure gives contours of PGA at 10,000 years return period and may be used in place of the seismic zone map (Figure 5) of the Guide, which divided the UK into three zones. This is considered to be an improvement on that map. Of particular note in this Figure are the high localised peaks or hot spots in areas where many of the UK dams are situated - in Scotland, down the Pennines and in one or two other places.

To give an idea of the effect of using this contour map in place of the zone map, I have prepared the following simplistic comparison;

Comparison of Musson & Winter and Guide PGAs for 10,000 Year Event

Zone	M and W		BRE Zone Map
	Max PGA	"Typical PGA"	PGA
A	~0.3g	<0.2g	0.25g
B	~0.3g	<0.15g	0.2g
C	~0.2g	<0.1g	0.15g

The maximum values in the hot spots are rather higher than the values in the BRE zone map but the "typical" values over the extent of each zone tend to be rather lower. The conclusion to be drawn from this is that the zone map tends to be conservative for the bulk of each zone but may be unconservative for the hot spots where the PGA contours are very closely spaced.

Two other interesting pieces of investigation have been carried out in the last few years. Both Scottish Hydro and Yorkshire Water have had site specific studies carried out for all their dam sites. Site specific studies are appreciably more rigorous than a regional study such as Musson and Winter's because they give more weight to the local geology. The detailed results of these studies are not in the public domain but summaries of them will be in the Note. The summary for the Yorkshire Water sites is shown below, together with the values from the BRE zone map:

Table N2
Results of Site Specific Studies of Dam Sites Owned by Yorkshire Water

ZONE A (125 sites)	**Peak ground acceleration (g) for stated return periods**			
Dam category	**I**	**II**	**III**	**IV**
Return period	**1 000 years**	**3 000 years**	**10 000 years**	**30 000 years**
Range	0.075 - 0.09	0.12 - 0.14	0.19 - 0.21	0.27 - 0.29
Mean	0.076	0.13	0.20	0.28
Growth factors	1.0	1.7	2.6	3.63
BRE Guide	0.10	0.15	0.25	0.375
ZONE B (10 sites)				
Range	0.058 - 0.067	0.10 - 0.11	0.16 - 0.17	0.23 - 0.23
Mean	0.062	0.10	0.16	0.23
Growth factors	1.0	1.66	2.63	3.76
BRE Guide	0.075	0.125	0.20	0.30

Thus site specific studies tend to show that at the higher return periods the zone PGAs are in many cases conservative by up to 20% and in some cases more.

This conclusion at first sight appears to be in conflict with the previous one relating to Musson and Winter's regional study but the answer lies in the hot spots where there are localised peaks of PGA. I believe that the overall conclusion to be drawn from this is that Musson and Winter is an improvement on the zone map for initial screening (which is likely to be the end of the matter for many of the lower category dams) but if the seismic stability of the dam is in doubt, that is to say if sophisticated studies or remedial works are in prospect, then a site specific study to determine the appropriate PGA is justified.

The issue of the appropriate earthquake return periods to be associated with each hazard category of dam (Guide Table 4 reproduced below) caused the most critical review and debate in the Working Party.

Guide Table 4
Return period (years) and peak ground accelerations for SEE

Dam	Return		PGA	
Category	**Period**	**Zone A**	**Zone B**	**Zone C**
IV	30,000	0.375g	0.30g	0.25g
III	10,000	0.25g	0.20g	0.15g
II	3,000	0.15g	0.125g	0.10g
I	1,000	0.10g	0.075g	0.05g

We made a great many enquiries on this issue and the findings may be briefly summarised as follows:

- The Maximum Credible Earthquake (MCE), unlike the PMF, is not well defined and practice around the world varies enormously. In some cases a probabilistic approach is used with return periods as low as 1000 years.
- The practice elsewhere in Europe for the highest category of dams is to use much lower values of earthquake intensity than is suggested in the Guide (viz. a return period of less than 30,000 years).
- Practice in the nuclear industry is to use a 10,000 year return period for natural hazards against release of radioactivity but with the proviso that there is no disproportionate increase in risk at levels more severe than this.

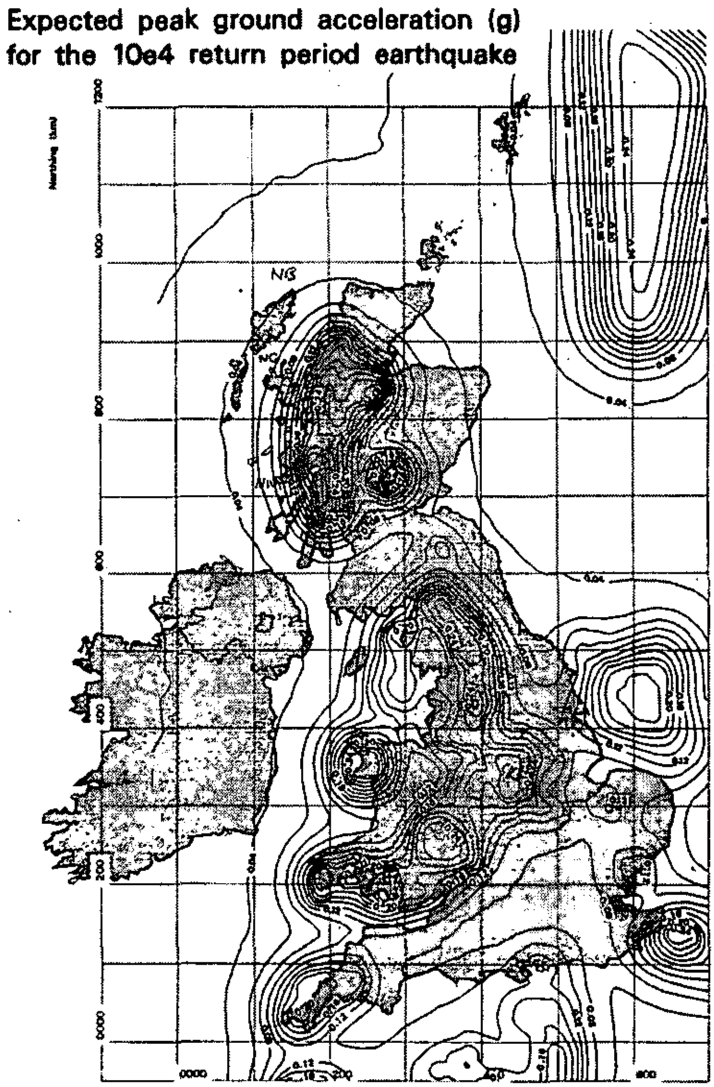

Figure 23. From Musson and Winter (1996)

Partly on the basis of these findings the Application Note will recommend that the Safety Evaluation Earthquake (SEE) for Category IV dams should be the MCE determined for existing dams on the basis of a 10,000 year return period event. However it also suggests that the assessment should ensure that there is no dramatic increase in risk for events which are more severe than this.

The Guide Table 4 will look like this:

Table N4
Criteria for selecting the Safety Evaluation Earthquake (SEE)

Dam Category	SEE
IV	MCE*
III	10 000 year event
II	3 000 year event
I	1 000 year event
	*or assessed equivalent

To conclude, I would like to stress, as did the foreword to the Guide, that this is not the last word on the subject and that the whole matter needs to be kept under review in the light of new research and developments in the field of risk generally.

Reference:

Musson, RMW: and Winter, PW (1996)	*'Seismic Hazard of the UK'* A report produced for the Department of Trade and Industry. AEA Technology

Colin Taylor (University of Bristol)
Seismic behaviour of ancillary structures
The paper in the proceedings bears no relation to what I'm going to talk about this morning. However, I will just make one comment about the paper, which is about the use of finite element analysis in dam safety assessment for earthquake loading. The main message of the paper is that you have to be very careful how you use the very powerful analytical programs that are available today. They can produce lots of very nice numbers but those numbers are very unreliable.

As Neil Sandilands said earlier, Scottish Hydro have commissioned a study with Bristol University to look at their appurtenant works and to develop some procedures or a process for commencing a rational seismic safety assessment of them. The study is starting off by making sure that the engineering science that underpins the safety assessment is well understood. Our studies are focusing on two case studies, the intake tower at Glascarnoch and the radial gate at Kilmorack. I'll start by making some general comments and then talk a little bit about some of the details of the things we've learned so far. The project is due to finish about the middle of next year.

The first thing I would say is that failures are usually unexpected. The reason a failure is unexpected may be because the engineer, who doesn't expect it, is poorly educated and lacks the knowledge that he should have or it could indicate a gap in the fundamental science - we just don't know collectively. It's therefore important that engineering science is well researched. This is where my role as an academic comes in as it's my job to acquire that basic knowledge and disseminate it. One thing I note from the participants' list at this Conference is that there are only two or three academics here, as far as I can tell. I just wonder whether that indicates a long-term weakness in the dam engineering profession in this country if there isn't much academic research going on to underpin the practical engineering.

The assessment methodology we're trying to develop will be staged, starting off with simple assessments and gradually introducing complexity when it is justified by the need and also, very importantly, by the quality of the input data that we can put into that assessment process. Analysis can take a wide range of paths, ranging from simple back-of-the-envelope calculations to detailed finite element analyses and detailed probabilistic risk assessments. All these methods of analysis have one thing in common: they produce numbers and that's all they produce - numbers. Engineers interpret those numbers and form a judgement and make a decision. It's very, very important to recognise that. Having lots of numbers does not imply greater reliability or confidence than if you've just got one or two well chosen numbers.

A safety assessment of a component within an appurtenant structure, be it a gate, pump or machine, needs to consider that component not only in its own right but also as part of the system. Many speakers in Session 1 used the word 'system' regularly. In an earthquake context that system is not just the dam. You have also got to look at the wider community - the local community, the regional community and, in some cases, the national community. After an earthquake you need the essentials to be available; you need water supply, you need power, you need telecommunications, and you need transport communication links as well. It's important to assess the safety of all dams and appurtenant works in that wider context. If I can give you an example: we did a seismic probability study of a hospital in Buenaventura in Columbia. This hospital is built on an island and there's a single bridge connecting the island to the mainland. The island is very poor, but the mainland is quite wealthy, so all the doctors and nurses live on the mainland. The hospital itself had many problems but we identified the really key vulnerable part as being the bridge because if the bridge collapsed, the medical staff would not be able to get into the hospital. We can see a similar scenario if you've got manually-operated fallback services: if you can't get the staff into the facility to operate them, then you've got a problem.

Nick Reilly talked about the work by Musson and Winter on looking at peak ground accelerations. One comment I would make is that peak ground acceleration on its own is not the best indicator of the damage capability of an earthquake. It's very much to do with how long the earthquake lasts - a 0.3g earthquake lasting two seconds may cause a lot less damage than a 0.2g earthquake lasting twenty seconds because we're dealing with a dynamic response. The response of the structure gradually builds up during the earthquake. In the UK we still have a very sparse seismic database and I suspect that when we have the next significant UK seismic event, about a magnitude of 4.5 or so, the Musson and Winter seismic map will change. We'll have a new hot spot.

A lot of seismic assessments have been done on dams in the UK. They have tended to concentrate on the dam structures and I think the message coming through from most of those assessments is that the body of the dam is unlikely to suffer any significant distress in the British-type of earthquake. That is a magnitude 6 happening once every 10,000 years with ground motion shaking for about 5 or 6 seconds - a short sharp shock. A real direct hit on the site. You might get an odd bit of cracking in the structure which might lead to slight leakage but it's unlikely you would get a catastrophic collapse. What we haven't done is to look at the appurtenant works such as parapets on the top of the structure, which will respond due to the amplification of the dam, the low level outlets and so on. These, I think, are more vulnerable than the main structures and this is what is really stimulating our study. It's essential that we learn the lessons of real earthquakes. This slide illustrates the damage from a real earthquake, the North Ridge earthquake in California which occurred a couple of years ago. I strongly recommend that if there is a big earthquake anywhere in the world that some of you try and get out there to look at the damage. There is an organisation in the UK called the Earthquake Engineering Field Investigation Team (EEFIT), which organises teams of industrialists and academics to go out and study the earthquake damage. It is the best method of learning.

This slide shows an access bridge to an intake tower at the Castaic dam near

San Francisco, which has moved on its piers. A number of things can be drawn from this. Firstly, there are services running across the bridge which have been disrupted. Although these cables did have a loop to allow for movement, obviously the loop was big enough on one side but not on the other as the cables are broken there. The dam had a 48-inch water main leading out from its base and the control for that water main was in the bottom of the intake tower. The water main fractured about 100m downstream of the dam, leading to a 30m high water spout. Engineers were very reluctant to go over the bridge to get into the intake tower to do anything about it! There was a tremendous lot of water flooding downstream because it was unsafe to go over the bridge. These logistical problems come out time and time again in real earthquakes.

A few years ago we carried out a research project where we looked at Wimbleball reservoir intake tower. Wimbleball dam is a diamond-headed buttress dam. Access to the concrete tower is via a connecting bridge. This plot illustrates what happens when an earthquake strikes a structure - it vibrates, like a tuning fork vibrating. If you hit the right note, the structure will vibrate violently. This structure exhibits a particular higher order mode, about 23 Hz or so, which wouldn't be excited strongly in an earthquake, but it illustrates the complex behaviour that can go on. This is a system which we must take account of. In this particular mode the tower can be seen to be vibrating strongly at the middle, and in the fundamental mode it will vibrate strongly at the top. This means that the equipment inside the tower will get a stronger shake at particular frequencies than just the ground motion. You typically get an amplification factor of about 3 from the base of the structure to the top. That means the equipment will get a stronger shake in some circumstances.

The accompanying table compares our computer predictions with some full-scale measurements that we did on the tower. This is where the engineering science is coming in. In some of these cases there's a very good comparison between the measured and actual frequencies and the predicted ones, which gives us some confidence that the elastic analysis we did is reasonably reliable - though it's only elastic.

To illustrate the effect of the amplification of a structure, let us look at another intake tower, that at the Los Angeles dam, again shaken in the North Ridge earthquake. Here the crane on top of the dam, which is lifting the gates, failed and toppled over because of the higher accelerations which it experienced. Whatever that crane was used for, it was no longer operable.

So if it served a safety function, that dam had a problem. It's these sorts of things that we need to look at.

Turning to the present study which is really only half-way through, we are looking at Glascarnoch intake tower. One thing I would draw your attention to is that the main power cable going into the tower runs along the bridge and goes across an expansion joint. If there is differential movement between the bridge and the tower there is a chance that that the power could be disrupted. We're also looking at the gates at Kilmorack and doing dynamic tests to understand the natural frequencies and mode shapes which will help us validate the computer analysis and the simple analysis that we're doing. The method used is to hit the gate with a 1500 lb sledge hammer. We record the dynamic response of the gate and from that we can measure its natural frequencies. The main frequencies of the radial arms are 19 Hz, 25 Hz and 29 Hz. These are well outside the seismic range and this is giving us some confidence that the simplified pseudostatic methods of analysis which are available would be appropriate for this sort of gate. You don't necessarily need to go into greater detail by carrying out the analysis for a full seismic assessment.

I'll finish off by showing some details just to draw your attention to the scope of the problems. The problems are often very minor and can be corrected relatively cheaply, frequently by just changing the routine maintenance procedures or changing future specifications. Standby batteries, which are very common in many structures, are also one of the most common failures in an earthquake. If the batteries aren't strapped down, they can jump out of their racks. Differential movement between the batteries can disrupt the connections. Very simple things such as these can actually cause a lot of problems.

In an earthquake, cabinets housing electronic control equipment will respond dynamically, so the equipment inside the cabinet will get a strong shake from the mounting. Cabinets which are very long will amplify the motions considerably, and need very careful attention. Circuit boards which are just plugged in, relying on friction grip of the rear connectors on the backplane to hold them in, could come out if subjected to a little bit of shaking. Connections would be disrupted, knocking out all your control. A good seismic design would have positive clips to keep the boards in place. In an earthquake loose objects, such as an N-strip left lying around, could bounce around and damage some of the components on the board. Free-standing cable reels could be knocked over, roll around and do some damage.

None of these things require sophisticated analysis to work them out, just common sense. For example, making sure electrical cables are appropriately anchored. When attaching a cable tray to the ceiling, are the small screws strong enough to take the loads, or would it be better to use a bigger screw? It might cost an extra 10p but that can make the difference between the whole thing collapsing or staying put. In the case of manual operation, where is the key stored? Is it in the key cabinet, just hooked on a hook? Could the key fall off the hook in an earthquake and fall down behind the cabinet where you can't get to it? Little things like that can have profound consequences.

Finally, differential movement is an important thing to consider. Take the case of some machinery operating a gate which is spanning across expansion joints. Differential movement between the pier and the bridge beam going across the gate might cause distortion to the mechanical equipment. Mechanical equipment can generally only accommodate very small distortions, so would the movement cause a problem?

So, in summary, our research is trying to categorise these problems and develop a simple stage process which will enable us to do a more rational seismic safety assessment than can be done at the moment.

Jack Lewin (Independent Consultant) and **Geoff Ballard** (Consultant)
Video - Matahina Dam after the Edgecumbe earthquake, New Zealand 1987
The Matahina Dam is situated on the North Island of New Zealand close to the Bay of Plenty. It was affected by New Zealand's worst earthquake that had its epicentre about 40km north of Matahina. The earthquake, measuring 6.6 on the Richter scale, showed a peak ground acceleration of $2361 mm/sec^2$.

The dam is 85 metres high. It has a core composed of weathered greywacke and rockfill shoulders. It is founded on alluvium with ignimbrite rock abutments. The reservoir capacity is about 25 million cubic metres and the spillway capacity 1900 cumecs.

After the earthquake, measurements indicated that there had been a vertical settlement of 90mm with crest deflections of up to 220mm. Drainage flows immediately after the event increased and were very cloudy, indicating possible damage to the grout curtain. Although these flows decreased later, significant damage at the left abutment was suspected and Arthur Penman will describe the remedial work that was undertaken.

There was no distortion of the spillway structure but there was indication that the left abutment had moved relative to the dam. The spillway gates remained operational and were operated by auxiliary power to lower the reservoir level.

Within the facility, a number of control cabinets were overturned after total failure of their hold-down arrangements. The switchgear yard of the associated power station suffered significant damage, although not always where it might have been expected. A number of large transformers were overturned. One transformer was right next to one that stood up perfectly adequately. Some that survived had actually shifted by several inches.

Discussion

Arthur Penman (Geotechnical Engineering Consultant)
Professor Jack Lewin has asked me to say a few words about Matahina Dam in view of my involvement in its extensive repair following the 6.6 magnitude Edgecumbe earthquake of 1987. This 85m high embankment dam, with slightly sloping rolled clay core supported by rockfill shoulders, was built during the mid-1960s and has been described by Galloway (1967 & 1970). The weathered greywacke core showed typical $w_L = 30$ and $I_P = 10$. In common with rolled clay cores of that time, it was placed fairly dry, as confirmed by negligible measured construction pore pressures and high value $c_u = 230\text{-}300\ kN/m^2$.

The columnar and regularly-jointed hard ignimbrite rock forming the abutments was not amenable to being trimmed to a smooth contact surface for the core. Benches formed naturally as the rock was excavated to remove weathered material and some were purposely made wider to be used as construction upstream to downstream roadways. During first filling in 1967, when the reservoir level was still 5m below TWL, there was a sudden increase of flow from the main drains that reached a peak of 570 l/s and became discoloured. Two weeks later, the weight of a bulldozer punched through the roof of a cavity that had formed above the right abutment. This was found to be a chimney of loose material extending down to a bench 12m below the dam crest. Sherard (1973) used this case history as one of his examples of dam cracking. It was argued that it was caused by a concentrated leak through a differential settlement crack in the core associated with a bench on the abutment. Differential settlement would rotate the direction of principal stresses adjacent to the irregular abutment and reduce the minor principal stress sufficiently to allow the occurrence of hydraulic fracture. The zone was excavated and repaired.

Following the earthquake the flow from the drains increased, with dirty water caused by silt in the drains being stirred up by the ground vibrations, but this soon cleared. The displaced transformers were put back on their bases, and generation was restarted only 14 hours after the earthquake. With the reservoir back up to full level, the leakage increased and an investigation was begun. At the left abutment the ignimbrite had settled 5mm. This value increased to 8mm during the following 12 weeks. The crest settlement was 73mm, increasing to 80mm. Inclined boreholes were put down in the core at both abutments and some caving zones were found. Additional piezometers were installed and these showed a head loss of only 4m through the core. Leakage measurements made at a weir on the main drainage outlet were found to under-register because of losses into the ground and the weir flows ceased when the reservoir was lowered to the minimum operational level.

Ten months later a sink hole appeared in the crest over the left abutment. Subsequently the reservoir was lowered and power generation had to stop. Detailed investigation showed core damage to a depth of 20m below crest level. At that stage, my advice was to excavate the core at both abutments, make a smooth contact surface between the core and the ignimbrite by trimming the rock and casting a thick covering of concrete, then to rebuild the ends of the core with a softer and more flexible material. There was no bottom outlet, but provision had been made for reservoir lowering by a 3.6m diameter tunnel with an invert 44m below the crest. At times of low river flow, this could lower the reservoir to about 37m below crest. Fortunately, the weight of overburden at this depth prevented hydraulic fracture, so there was no need to make the core softer below this depth. A search was made for a more clayey fill, and the limited amount that was found was mixed with excavated core material and wetted. This better material was carefully compacted into position so as to make good junctions between the new smooth concrete abutments and the excavated slopes at the ends of the core. There are some further details and drawings showing the shapes of the abutments given in my paper to the 8th BDS Conference in Exeter (Penman 1994). This extensive repair was carried out in 1988 and proved completely successful.

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Penman, ADM: (1994)	*'Examples of problems involving clay cores affecting dam safety'* Proc. 8th BDS Conference, Exeter, 'Reservoir Safety and Environment', Thomas Telford, London, pp. 42-51.
Sherard, JL: (1973)	*'Embankment dam cracking'* Chapter in Embankment Dam Engineering: Casagrande Volume, pp. 271-353.

Iain Moffat (University of Newcastle)
The video of Matahina Dam after the Edgecumbe, New Zealand, earthquake of 1987 provokes some sobering thoughts regarding the possible vulnerability of certain UK dams, and more generally, of associated critical components and ancillary structures, to crippling damage or failure in the event of a serious seismic event. There is urgent need for an improved appreciation of component response to seismic excitation and of what constitutes good detail design, whether of vulnerable structures, including gates and cast iron valves etc on the one hand, or of plant and equipment outfits, including their electrical or hydraulic operating systems, on the other. As dam engineers we have much to learn about good detail design against transient 'shock' loading protection from our naval engineering colleagues, and in particularly from the warship and submarine specialists.

A strong case can be presented for the preparation of project-specific 'Standard Response Procedures' (SRPs) to guide operating and headquarters staff in the event of specified occurrences including seismic, extreme flood and other incidents.

Turning to a little-recognised but nevertheless real hazard, and in no way to diminish the very important ongoing work in relation to the seismic hazard to UK dams, I would suggest that the aircraft collision hazard is of at least equal significance in some parts of the country. This is particularly so in relation to the possibility of an aircraft striking the crest of a dam or some critical auxiliary structure at high velocity.

Factors to be considered here include:

- location of the dam and its orientation relative to the prevailing flight-path (e.g. on the approach threshold to a civil airport or, more particularly, on flight paths intensively used by military aircraft on training missions);
- the type of aircraft regularly overflying the dam, i.e. civil or military (the former are relatively 'soft'; the latter are 'hard' with a much higher cross-sectional 'density' and a higher impact velocity);
- the seasonal weather envelope in proximity to the dam/reservoir complex.

Heavily-loaded military aircraft flying fast, 'zero altitude', training missions represent the most serious threat. In this context it should be appreciated that:

- in the case of military aircraft a number of identifiable high-profile dams are regularly used as navigational waypoints and/or as targets for aircraft flying attack profiles. Such aircraft may also be practising evasion tactics at high speed;
- sortie frequency may be intense, particularly within major exercise periods, and may also involve foreign aircrew unfamiliar with the terrain;
- in the event of an aircraft strike, the blast effect following ignition of vapourised fuel would represent an additional collateral threat;
- aircraft may, in some circumstances, be carrying a weapon load;
- navigational safety systems on a military aircraft are likely to be less comprehensive than those on its civil counterpart;
- laden mass at impact could be of the order of 25t and more;
- impact velocity may exceed 250 m/s (comparable to the velocity of a handgun bullet).

To conclude, I believe that an initial assessment of civil aircraft hazard has been carried out in a few instances. I suggest that the more critical aircraft hazard probably lies elsewhere and relates to military activity. Our interest in the seismic hazard developed from nuclear industry practice. It is also practice in the nuclear industry to assess the hazard and consequence associated with aircraft; logic decrees that we should give similar weight to the latter hazard on a number of UK dams.

Andrew Charles (Building Research Establishment)
In the discussion about seismic risk, the hazard posed by failure of small

components of ancillary works has been emphasised. However, the hazard posed by failure of the embankment dam should not be entirely dismissed. Bolton Seed suggested that any well built embankment dam can generally survive quite large earthquakes, but are old British embankment dams well built in the sense that Bolton Seed meant? Important considerations would include fill placed in relatively thin layers and heavily compacted and adequate factors of safety against slope instability. Many of our old embankments do not fit into this category. Thus the question of the stability of many of these old dams in seismic events cannot be altogether dismissed, despite the important matters concerning the behaviour of structures to which Dr Ballard and Dr Colin Taylor have drawn our attention.

James Martin (Scottish Hydro-Electric)
I'll tie together the point that Iain Moffat was making with the one Andrew's just made. I was one of the first civilians on the Falkland Islands after the war and I had the privilege of standing in a hole in which a Phantom with full bombload crashed. The volume of the hole, which was in solid rock, was about half the volume of this lecture theatre. The rock was tillite, which is a strong metamorphic rock. So that's a measure of the direct impact and I make the distinction between a training sortie and a serious wartime sortie. There is also no doubt that a dam close to that hole would have sustained very large ground accelerations indeed but of short duration, so that brings in the spectral response that Colin Taylor was talking about.

Chris Binnie (Independent Consultant)
I was interested in the discussion on seismic design of dams, particularly Colin Taylor's recommendation to visit earthquake sites. Lawrence Attewill and I were designing Gargar Dam in Algeria when an appreciable earthquake occurred nearby, from memory 60km away. We visited the site shortly afterwards and found a throw of about 1 to 1½m extending for several kilometres. Many buildings had been damaged and some flattened. Professor Ambraseys talked to a railway engine driver who had hit his head on the roof of his cab, indicating a vertical acceleration in excess of 1g. I wish to make the point that some civil engineers seem more concerned about horizontal acceleration and sometimes ignore vertical components. For the dam we finished up by choosing a design acceleration of 0.6g in any direction.

Colin Taylor (Bristol University)
Just to pick up a point from Geoff Ballard's very interesting presentation which I agree almost entirely with. I didn't mean to imply that there was a

large reliability database available. I don't believe there is. There is a database that's produced by Electrical Power Research Institute in the United States (EPRI) called the SQUG database, which draws together information from seismic qualification testing - that's actually shaking equipment on shaking tables - and observations of performance of that equipment in real earthquakes. That's a very useful resource as a starting point. At Bristol we've got a very large shaking table which we use for seismic qualification tests. We've done about 200 tests over the past 10 years - we must have tested thousands of components. Very few of those components failed. You can tell the ones which are going to fail because they are just clearly badly engineered. Those cabinets (shown in the slide) which had fallen over were not a surprise when I saw the gauge metal that the bolt was going through. In general, you don't need to use complicated remedial measures to significantly improve the reliability of those sorts of systems in earthquakes. You use a bit thicker metal and a bit bigger bolt. The calculations are very simple - it doesn't require a sophisticated analysis.

Geoff Ballard (Consultant)
I understood our initial speaker (in Session 1), Dr Andrew Charles, to say that other industries have a lot of accurate data on the reliability of their equipment and therefore risk assessment can be quite appropriate for those industries. By contrast he suggested that there was very little data available for dams and their ancillary equipment. First it should be noted that accurate reliability data is almost a contradiction in terms. Data you may want to use is always an historical record of other equipment, in some other environment, and with somewhat different design. Therefore, any data is uncertain and has to be used with a lot of engineering judgement. In using risk assessment the water industry is thus not that worse off than other industries; quantification of risk analysis will always require a great deal of engineering judgement, even though historical data can provide a useful calibration.

Andrew Charles (Building Research Establishment)
In reply to Dr Ballard's remarks on my presentation of the paper on risk analysis and safety management, the contrast I was drawing was 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 logic trees. An embankment dam and its foundation are quite different and the stages in any malfunctioning are not amenable to this type of analysis.

Geoff Ballard (Consultant)
Dr Charles makes the very interesting point that, in his view, judgements about the whole process of, say dam failure, are just as good as judgements about any of the individual contributors. This certainly is unusual and gives the industry some particular problems. However, the industry is not unique in having structural components as a major issue to deal with in their risk assessments. For example, a major chemical plant has to reach judgements about the integrity of large pressure vessels and they are also trying to achieve similar failure frequencies as the water industry.

Rod Bridle (Independent Civil Engineer)
My comment relates to the significance of "ultimate events" in reservoir safety practice. If we study F-N plots in the Charles, Tedd and Skinner paper (in Session 1), they bring home the uncomfortable fact that whatever the magnitude of event a dam is designed to withstand, there remains the possibility of some fatalities should a bigger event occur and cause failure of the dam. For example, from Figure 1 in that paper, even if we decided to take a negligible risk and took a 1 in 10 million event as our design criterion, we would be faced with the unpalatable fact that 1,000 lives would remain at risk. This is not a comfortable situation for the dam designer, less so for those at risk. However, some of the phenomena that might trigger failures are "ultimate event" phenomena. In other words there is a physical limit on their magnitude. We are all familiar with the Probable Maximum Flood and "science" can give us an estimate of Probable Maximum Precipitation adequate for an engineering estimate of the Probable Maximum Flood. Science also recognises, in principle, that there is a Maximum Credible Earthquake that could be experienced at any site, although estimating it is difficult and no methodology developed to date has been sufficiently accurate for application in reservoir safety situations.

Internal erosion, another major trigger of dam failure, is not an ultimate event phenomenon, although if we could retrofit the 'ultimate filter' to our embankment dams we might be able to prevent failure by internal erosion. I cannot think of other triggers of dam failure that are ultimate event phenomena. However, if we can establish the Probable Maximum Flood and the Maximum Credible Earthquake and design our dams to resist the effects of these, we would, in principle, completely eliminate risk of failure being caused by these hazards. This provides a good deal of comfort for reservoir engineers, and indeed for the general public, as we must be almost unique amongst the safety community in being able to build out the risk of the occurrence of certain types of failure. The inclusion of the term

Maximum Credible Earthquake in the new table in the Application Note to the Seismic Guide described to us by Nick Reilly, is a very significant step in this issue. More research is, of course, needed to assist us in arriving at estimates of the Maximum Credible Earthquake sufficient for our purposes.

Richard Vincent (Department of the Environment, Transport and the Regions)
With regard to the Review of the Reservoirs Act 1975, Ministers have decided that two of the original four deregulation proposals ((b) and (c) in the paper by Sims and Parr) should not go forward, wishing to see that safety of reservoir assets is maintained in the present manner. A further consultation on the two remaining proposals (transfer of panel appointments to the Institution of Civil Engineers, and establishment of the Environment Agency as the single enforcing authority in England and Wales) finished in July 1998. The deregulation process is now advancing with a view to these changes taking effect in April 1999, subject to the final views of Parliament.

Alan Burdekin (Agriculture, Environment and Fisheries Department, Scottish Office)
With the reference to the possibility of future legislative changes, I will take this opportunity to mention to the Conference what the expected position will be in Scotland following the planned confirmation of a Scottish Parliament next year.

Under the Scotland Bill, currently passing through the Westminster Parliament, reservoir safety in Scotland will become a "devolved matter" - a matter for the Scottish Parliament. After the Scottish Parliament has been established, the existing Reservoirs Act 1975 - and related statutory instruments - will still apply in Scotland but the provisions within it, in so far as they relate to Scotland, will be the responsibility of the new Scottish Ministers of the Scottish Executive. Any future reservoir safety legislation in Scotland will be separate - made by the Parliament in Edinburgh.

As far as Panel Engineers are concerned, until such time as the Scottish Parliament decides to make new legislation in this area, there is likely to be no visible change regarding reservoir safety.

It is intended that the present panel structure, and its arrangements, will remain but from then on Parliamentary arrangements will be different.

Written Contributions

Robert Freer (Consultant)

I have the following questions regarding Dr Sims and Mr Parr's paper:

1. In their revision of the guidance document for Panel Engineers, does the Working Party intend to put forward proposals for the continuing training of new panel engineers, especially of those who wish to become Panel AR Engineers but have had only limited personal experience of the design of dams? Does the Working Party consider that the critical part of the training could be undertaken as part of a formalised instruction in an academic institution?
2. To help clients and dam owners to match their safety needs with the safety resources (money and technical skills), would it be better to assess the safety requirements of each dam individually rather than on arbitrary characteristics of size?
3. Does the Working Party consider that this document may have potential value in helping those overseas clients who may wish to copy our system of dam safety procedures? In which case, would they recommend early completion and publication of this document?

Geoff Sims (Brown & Root)

In reply (on behalf of Noel Parr and the Working Party) to Mr Freer:

1. The guidance document will be in the form of an engineering guide to the implementation of the Reservoirs Act 1975. As such it will have no more statutory authority than the other engineering guides. It will seek to distil the wisdom and experience we now have through the operation of the Act since 1985. We have not envisaged covering the training of Panel AR Engineers in the guide. Accepting that this is an important issue, we consider that it merits more detailed and separate consideration than would be appropriate in this document.
2. Mr Freer will no doubt be aware of the research in progress on hazard assessment and rating for UK dams. This will consider the relevance of factors in addition to the reservoir volume in assessing the safety of a dam. It is in the interests of the owner for the individual properties of his dam to be considered in as much detail as is appropriate. The Act makes provision for this through the involvement in the process of qualified civil engineers who are expected to make judgements on all relevant factors. The guidance document will no doubt make this clear and will incorporate the available data from the research in hand.
3. We have no doubt that this guidance document will not only be of benefit to those engineers involved with the operation and maintenance of dams in this country, but will also assist those overseas who wish to follow the system we have developed in the UK.

SESSION 3
ASSESSMENT OF DAM PERFORMANCE

Chairman	Alan Johnston
Technical Reporter	John Sammons

Papers presented

1. Investigating internal erosion at Brent Dam.
   P Tedd, D P M Dutton and I R Holton
2. Investigation of a possible sinkhole at Walshaw Dean Upper Dam.
   A C Robertshaw, M S Atkinson and P Tedd
3. Bewl Water spillway investigation.
   I Davison

Presentations

1. Problems at March Haigh Dam.
   J Beaver
2. Distributed temperature sensing in dams - an overview.
   J Dornstädter
3. An evaluation of earth resistivity survey as a means of monitoring dam performance.
   V G Over

Papers not presented

1. Observation analysis at Llyn Brenig.
   A Thomas
2. The Lower Lias Clay at Barrow No. 3 Reservoir.
   N G Swannell
3. Kentmere - past, present and future mining subsidence.
   A K Hughes
4. Distributed temperature sensing in dams.
   J Dornstädter and M Aufleger
5. Enguri Dam : horizontal and vertical displacement, retrospective analysis.
   D Mirtskhulava, I Noniev, J Ebanoidze and T Kobasnidze

SESSION 3

Chair

The subject of this session, Assessment of Dam Performance, has attracted considerable interest and rightly so. Knowledge of how dams behave in service is crucial to how we perform our tasks as custodians of existing dams and the builders of new dams.

Eight papers have been published in the 'Assessment of Dam Performance' section of the Proceedings including papers from Georgia and Germany. Unfortunately we have time only for full presentations on three papers. All eight papers contain valuable information. I recommend them all to you and I would like to highlight key points from five papers which are not being presented fully.

The paper on Enguri dam, a double curvature arch dam, describes the difficulties of analysing horizontal and vertical movements of the dam. It also shows the value of doing so eg. the monitoring has revealed that the movement of the foot of the dam has caused tension in the rock foundation which has resulted in water penetration to closed fissures with subsequent damage to the grout curtain.

Andrew Thomas's paper on Llyn Brenig rockfill dam in Clwyd also tackles the topic of analysing performance and the delegates who visited the dam in the pre-meeting visit heard an excellent explanation from him on the value of appropriate computer software for devising proposals to compare actual with predicted readings.

The paper on Lower Lias Clay at Barrow No.3 reservoir, Bristol is a useful addition to the literature on the behaviour of this soil and in particular its residual strength.

The paper on Kentmere reservoir, Kendal, demonstrates a private owner taking a responsible attitude to undertaking work to preserve an environmental asset to the Lake District even though the reservoir is not used commercially.

The fifth paper looks to the future and an innovative method of measuring leakage using fibre optic temperature laser radar. One of the authors, Mr Dornstädter, is present and he will speak briefly at the end of the session.

Discussion

Robert Mann (Cuthbertson)
The photograph of the hole beneath the upstream face pitching at Walshaw Dean Upper Dam appeared similar to voiding which was observed beneath

pitching at and above top water level at Glenquey Reservoir, Perthshire. In that instance the cause was considered to be the removal of fines by wave action. The pitching had bridged over several other voids along the top part of the dam.

Andrew Robertshaw (Yorkshire Water)
The first Inspecting Engineer to inspect the void at Walshaw Dean Upper was Nick Reilly of Mott Macdonald who did raise the possibility of it having been caused by wave action. After further consideration, however, this was thought to be unlikely due to its location under the wavewall at the top of the upstream slope where waves would be unlikely to reach. Subsequent investigations concentrated on a search for some evidence of internal erosion but to date this has proved to be inconclusive.

John Massey (Mott MacDonald)
In response to a question from the floor regarding the possibility of wave action being the cause of the sink hole at Walshaw Dean Upper, in my capacity as AR Panel Engineer for the reservoir, I would make the point that this was an isolated hole on the face of the embankment. Close examination of the full length of the embankment had shown no signs of similar holes beneath the pitching, which one could expect if wave action was the cause.

Amongst other causes considered was the drying out and cracking of the clay core following the prolonged draw-down during the recent spillway reconstruction, plus the drought conditions prevailing at the time. The trial hole excavated over the site of the sink hole exposed the side of the core, and this was found to be in good condition. There was no abnormal crest settlement, no sign of slips in the embankment slopes and the sink hole was not near the valve tower or draw-off tunnel. The conclusion that we had not found the cause of the hole, and we had not found any signs of hazard to the safety of the dam, led to the recommendations that the hole should be refilled, and instruments left in place should be carefully monitored.

A review of the instrument readings and of the general situation should be made after a period of two years, ie. in 1999, by an AR Panel Engineer. Unfortunately this will not be me as I shall be retiring from the Panel in November this year. The investigations are my legacy to my successor.

Geoff Sims (Brown & Root)
Can I ask Ian Davison how much swelling strain was measured at the Bewl Water spillway? Have you measured strains on cores and have you estimated

the maximum potential strain of the concrete? The first observation of ASR from your account took place about 20 years after construction and it would be interesting to know how far the reaction has progressed.

My second observation is that there is a consensus that there is life after ASR. The first instinctive reaction to replace the affected concrete seems often to be changed as calm counsel prevails. The residual strength of ASR affected concrete is high particularly in compression compared with the insitu undamaged material. For instance, concrete that has swelled 5mm/m has a residual strength of 75% of the compressive strength of unaffected concrete at 28 days. Have you an estimate of what the stress level is within the structure and whether the concrete in its damaged state is still strong enough to resist it?

Have you considered whether it is possible to effectively reduce the seepage of water through the structure by effective waterproofing on the upstream or outer side?

Ian Davison (Montgomery Watson)
Strains have not been measured in the concrete. We did try and assess how much expansion had taken place but in the time that we had we didn't do any tests to quantify the expansion. As we were trying to establish whether the concrete was still expanding, we used a very simple alkaline immersion test, rather than the other test which takes longer. What we did was to examine the rest of the structure and carry out a structural assessment to see whether the concrete had been affected already. There is a possibility of raising the shaft in the future and we're quite happy it's not been adversely affected as yet. There were two options. One was just to release the stress and not replace all the blocks, but some of them have been so badly cracked they could well fall apart. The other option was just to replace the blocks and, at the same time, coat the outside of the insitu concrete to stop water getting into the cracks and affecting the reinforcement. There are, however, other factors to consider so we haven't finalised the remedial options yet.

Andrew Charles (Building Research Establishment)
The papers on Brent and Walshaw Dean Upper describe investigations which might be considered to be somewhat inconclusive. Case histories of this type are valuable and provide a corrective to the natural tendency to write papers describing problems which have been identified and correctly diagnosed, and where successful remedial works have been completed. The extent of the investigations at Brent and Walshaw Dean Upper may have

been greater than in many cases and it might encourage the preparation of papers describing problems which have not been fully resolved if, at future conferences, short papers and technical notes were encouraged.

Nick Reilly (Consulting Engineer)
With regard to the slide shown by Dr Tedd of the seepage into the culvert at Brent Dam, it looks remarkably similar to the ochreous deposits which arise from seepage flows into the adits in many Pennine dams. Assuming that it was actually clay, it is doubtful that the settlement facilities provided would have trapped all the material unless a flocculating agent had been used. In that case, the quantity of material removed per year could be greatly underestimated.

Referring to Andrew Robertshaw's comments that I was of the opinion that wave action could have caused the sinkhole at Walshaw Dam Upper, I did put this forward as a possible explanation at the time. As to the question of whether I have changed my mind since, the answer is yes - several times!

Paul Tedd (Building Research Establishment)
I agree that it is likely that we are not measuring all the material that is being eroded through the brickwork into the culvert at Brent dam. We did contemplate putting a flocculent into the sediment tanks but decided that with the very low flow rates, and using two settlement tanks, that a significant proportion of the eroded material could be collected.

I was reasonably convinced that the eroded material was clay and not ochre, however a recent elemental analysis of the eroded material indicated that it contained approximately equal amounts of iron, calcium and phosphorus in contrast to the in-situ clay which contained predominantly aluminium, silicon and a little potassium and iron. The eroded material is not ochre but it is very different to the clay surrounding the culvert. Investigations into the origin of the eroded material are continuing.

Derek Knight (Independent Consultant)
The paper by Tedd, Dutton and Holton on investigating internal erosion at Brent Dam adds usefully to the record of "concerns" with old dams. This particular dam, although of a very modest 9m height, is already 163 years old, and a probably unintended application of the Conference title is that the prospect for such aged reservoirs next century depends very much on keeping them healthy. Any suspicion of internal erosion, therefore, is quite properly taken very seriously. The discussion and conclusions on page 77

refer to draw-off tunnels and culverts acting "as drains to the surrounding fill" and potentially providing "a means of allowing eroded material to exit from the dam".
This contributor is reminded of just such an occurrence in 1986, when the 1300 year old Kantalai Tank (reservoir) failed suddenly, dramatically and with great loss of life, at the left bank sluice. A summary of the event and further references were given by this contributor in Dams & Reservoirs, November 1991, Vol. 7, No. 3, pp.30-31, but the point to be emphasised is that the disaster was triggered by a nearby, quite recent activity involving piling through the ancient embankment. This disturbed the mortar within the joints of the masonry culvert constituting the draw-off culvert, leading eventually to piping erosion and full-scale failure of the dam and reservoir.

The general lesson, therefore, for old UK embankment dams is the need for great care to be taken in any activity whatsoever on, through or near such dams. With the increasing pressures for cost-cutting, and the creeping de-centralisation of much human activity these days, it is becoming more difficult for those responsible for dam safety to be made fully aware by their staff and from their records of the possible effects of seemingly harmless actions on the dams in their care. Those actions include even the very investigations intended to enhance safety. At Brent Dam the use of ground probing radar was clearly an appropriate response to the need to minimise invasive surgical exploration.

Paul Tedd (Building Research Establishment)
Ground probing radar was considered as a first step in the investigation of Brent Reservoir. However, with inconclusive results it was necessary to undertake some very careful 'invasive surgical exploration' to try and understand where the eroded material is coming from. It is important to emphasise that geophysical methods such as ground probing radar often require expert interpretation. They should be considered as one of the many tools available for investigating old dams and should never be used in isolation.

Chris Owens (Bullen Consultants)
I was struck by the similarity between Bewl spillway and that at Ardingly, both presumably built using aggregates from a similar source. I note in the paper that the mix for the Bewl crest blocks was different. Were these cast off site with different materials? Is there any similar distress on the Ardingly and Bough Beech spillways?

Presentations

John Beaver (Halcrow Group)
Problems at March Haigh Dam
March Haigh Dam is situated in a remote moorland area near Marsden in West Yorkshire. There is no permanent vehicular access to the dam, and in the past when works were required, cross-country buggy-type vehicles were used to convey personnel and materials across the rough moorland.

The embankment forming the dam, which was constructed in the 1830's, is about 20m high at the maximum section and is believed to contain a relatively thin puddle clay core with earth fill shoulders. The upstream face is steep at about 1:1.5 and is protected with stone pitching, and the downstream face is grass covered. The whole structure is founded on millstone grits.

There is a single 12in (300mm) dia cast iron outlet pipe at the valley centre leading from a masonry-lined culvert in the upstream shoulder. This culvert is believed to extend from the upstream heel of the dam to about midway between the heel and the clay core. The drawoff pipe passes through the clay core with its outlet at the upstream end of a horseshoe-shaped masonry lined culvert under most of the width of the downstream shoulder. Access is available at the downstream toe for manual operation of the gate valves.

The crest of the dam had suffered significant settlement over the years, and in 1974 the clay core and the masonry wave wall were brought back up to level so that the design top water level (TWL) of the reservoir could be maintained with sufficient freeboard to guard against overtopping during the design flood.

Two years prior to the dam crest works, a small flow of water from a masonry joint about half way up the culvert on the left hand side was noticed by the owner's staff and was immediately reported to the Panel Engineer retained at the time, the late Mr Noel Cochrane. He asked for the issue to be examined for the possible erosion of fine material, but none was reported. He also asked for a pipe to be inserted at the point of leakage so that flow monitoring could be undertaken at fortnightly intervals. During the first 2-3 months of monitoring with the reservoir remaining at or near TWL, the leakage flow reduced. At the time there had been little rainfall and Mr Cochrane's interim conclusion was that the leakage was originating from the foundation rock through which the lower part of the culvert was thought to have been constructed.

Monitoring continued and a small amount of brown coloured sediment was collected which on analysis was found to be ferric hydroxide. Leakage flows, reservoir water levels and antecedent rainfall were monitored and plotted graphically over the next 3 years. The results were rather inconclusive, but with a slight tendency for correlation with reservoir water level. Mr Cochrane's 1976 Inspection Report reflected this but did point out that at some quite high water levels, leakage did increase significantly.

No action was taken, but monitoring continued in accordance with Mr Cochrane's recommendations, and by 1980, the leakage flow rate reduced to such a low level that it became unmeasurable. However, in 1982 there was a sudden substantial increase in flow at a time when rainfall was low and the reservoir water level relatively constant. The temperature of the leakage issue was found to be lower than that of the reservoir drawoff water in the summer and higher in the winter, which Mr Cochrane concluded would be the pattern expected if the source was groundwater.

During my inspection in July 1996, the leakage flow was slight and there was no evidence of fine material deposits. Just prior to the planned issue of my Inspection Report in September 1997, there was a substantial increase in flow to more than 20 litres/min with deposits of sandy material on the culvert floor below the leakage outlet pipe. I was concerned about the continuing deposits of fine material conveyed by the leakage water, and recommended that the water level in the reservoir be lowered as quickly as possible through the drawoff pipe.

The valves were opened, but after an initial flush of water, the flow reduced abruptly. Rods were passed up the drawoff pipe and a solid blockage was encountered at the point where the drawing of the dam cross-section shows the drawoff pipe entering the upstream masonry-lined culvert. Thus is it possible that there may have been a partial collapse of the culvert with a piece of masonry causing the blockage.

The conditions at the dam were now potentially serious, with the reservoir nearly full, a high level of leakage containing fine material and a blocked drawoff pipe. I recommended that the reservoir should be drawn down using pumps and siphons and this was done. A scheme is now under consideration for reinstating the drawoff. Sinking a shaft in the dam's upstream shoulder may be the only safe method since access to the lower parts of the upstream face may be difficult and dangerous. This is due to the considerable depth of silt known to be present within the deep incised valley where the drawoff inlet is thought to be located.

Piezometers are to be installed at two sections in the dam and on both sides of the drawoff pipe to monitor phreatic levels within the embankment and in the foundations, once the drawoff outlet has been re-established and the reservoir can be refilled under controlled conditions. The origin of the leak into the downstream culvert has yet to be established, but the possibilities are:

- groundwater within the rock foundations
- reservoir water arising from hydraulic fracture of the clay core
- leakage of reservoir water through the core at the point where the drawoff pipe passes through it
- leakage of reservoir water via open joints in the drawoff pipe. (This is now thought to be unlikely since leakage continues with the drawoff pipe blocked at its upstream end.)

Dam crest settlements over the last 8 years since the remedial works to the crest were completed have not been excessive, but as might be expected, settlement has been more in the valley centre where the dam height is greatest.

The problems which continue to be experienced at March Haigh Dam are serious, but worsening conditions which could have led to eventual failure of the dam have been kept under control by vigilant monitoring and surveillance.

Internal erosion of the dam and a blocked drawoff pipe are both serious matters for any dam, but in this case these have occurred concurrently which is somewhat unusual. Restoring the dam to a safe condition will be both expensive and in this case, technically demanding. The problem is further exacerbated if, as is often the case, the internal components of the dam section are not known with any certainty.

Chair
Can I now turn to two items which take us from problems left by previous generations to trying to make use of new technology to help with our methods of investigation. Firstly, one of the papers in our proceedings by Mr Dornstädter and Mr Aufleger of Germany is "Distributed Temperature Sensing in Dams". You may have seen Jürgen Dornstädter at the exhibition but he's going to take a couple of minutes to show us a few slides.

Jürgen Dornstädter (GTC Kappelmeyer GmbH)
Distributed temperature sensing in dams
Thank you, Mr Chairman, for giving me the opportunity to say some words about this new measuring system for dams in the 21st Century. Temperature measurement inside dams is a helpful tool to detect leaks and therefore possible erosion. How can we do this? Firstly, there is a temperature difference between the reservoir and the soil embankment due to the fact that the soil and other construction materials are very weak heat conductors. In this example, you can see a temperature anomaly inside the dam and in the foundation. This is due to the fact that seepage is occurring. Here's a slurry trench wall and a longitudinal section of the dam. The dam is only 7 metres high. Temperature soundings have been taken down to 20m at 10m intervals along the dam. You can see from the anomaly that water is passing through and underneath the dam.

To make life easier when taking measurements you can use a new technique called fibre optic temperature measurement. A laser pulse is sent into an ordinary fibre optic telecommunication cable. You send a laser pulse into the fibre, and all of the time the light is coming back because the photons are reacting with the glass molecules of the fibre, and so you get a signal coming back which is carrying information from the places where this back-scattering is happening. In the spectrum analysis there are two lines, which are temperature-dependent. This analysis is very complicated but it is all done by this machine for us so we don't have to think about what it's really doing! In the end you will get a temperature distribution all along the fibre and this you can do over lengths of up to 40km with an accuracy of 0.2ºC and a spatial resolution of 0.5m, so it's efficient for all construction engineering.

How can you use it? You can use it to monitor seepage behind a core with several lines of cables. You can bring the cables underneath asphaltic or concrete facing or you can use the cable to monitor the curing temperature of mass concrete. I will show you a few examples.

In this example the asphaltic facing of a canal has been renewed and underneath this facing we installed this fibre in a small trench. During the first filling we made measurements and you can see there's a very small anomaly in this area. This is due to a small fissure inside the asphaltic concrete that water entered. Over time the quantity diminished because the fine materials which were transported by the river closed this small fracture. But at the end of the rehabilitated section you can see that cold water (it's winter) is running underneath the facing because the area here was not renewed. So you have seepage underneath the new sealing.

The technique might also help you to find out the reason for your seepage. For example at this place small springs occurred. The owner was afraid that his new seal was leaking. We carried out measurements and what you could see was not seepage from his canal. The temperature in the canal was about 3ºC in winter and from the measurement profile along the section you can see very clearly that in the area where the springs occurred the temperature has increased. This is due to the fact that the groundwater levels were very high at that time of year and the groundwater passed underneath the re-sealed dam and came out here.

Vic Over (Bolton Institute)

An evaluation of earth resistivity survey as a means of monitoring dam performance

Many of the 150 plus earth dams in the North West of England are of Victorian construction. North West Water has a statutory requirement to monitor an ageing stock of dams and reservoirs by means of regular inspections in the form of 'walk over' surveys to visually check for signs of deterioration in performance. Water issuing from an embankment or surrounding ground is of particular concern as it could herald an imminent major failure. The source of a seepage and its flow path are initially a matter of conjecture. Thorough investigation is needed to establish how and why a flow has appeared and any necessary remedial action.

Identifying water flows by visual observation is less than ideal, because at this stage the root cause is well advanced. Ground resistivity measurements present the possibility of an 'early warning' system by monitoring the performance of an embankment and surrounding ground.

<u>Resistivity Survey</u>

Soil and rock are composed of mineral grains surrounded by void space, pore volume, which may be fully filled with water or an air/water mixture. Most minerals are insulators, as is the air content in partly saturated soil. Thus electrical current generally flows through pore water.

The fundamental properties of void volume, degree of saturation (or air content) and moisture content values all contribute to the measured value of resistivity. Thus any alteration of one or more of these properties due to changes within an embankment will alter the measured value of resistivity.

A critical question mark rests over whether field techniques can detect and locate these resistivity changes.

The technique of plotting electrical resistance sections in a vertical plane through the ground, electrical imaging, has become feasible in recent years though technological and software developments by geophysical equipment manufacturers.

The Project

The aim is to test the application of modern resistivity equipment and analysis to earth dams in the North West of England which have a characteristic assemblage of construction methods and materials. The plan is to monitor three or four embankments over a period of 12 to 18 months to obtain a general picture of their resistivity profiles and how these change with time.

Site selection has, however, been overtaken by need. Seepages have recently occurred at Gorton (East Manchester), Jumbles (North Bolton) and Guide (South East Blackburn), but their root causes are not clear. It is of considerable advantage to investigate these instances and hopefully, correlate the results with ongoing site investigation and remedial works. Soils samples from Jumbles will also enable comparative laboratory testing.

The display and discussion present some preliminary results from Guide reservoir. Resistivity results do not point to the expected source of a seepage, namely, flow through the interface of a raised level in the embankment. However, a central area of low resistivity within the core is identified. Guide is unusual in that water levels can be raised or lowered at will. Measurements taken nearly one month after raising the water level from 400mm to 200mm below top level caused an increase in extent of low resistivity regime. The next stage is to raise the reservoir to top level, the point at which seepage had previously occurred.

Summary

A recurring theme at this conference has been a concern about the future welfare of our elderly and somewhat incontinent stock of earth dams. Resistivity survey is a non-destructive indirect technique which has the potential to be well suited to monitoring and assessment. A significant amount of research investigation is required to establish characteristic resistivity profiles for earth dams; whether anomalies are due to either constructional and/or hydraulic processes; how profiles change with time and how anomalies are to be interpreted. In this respect a valuable exercise would be to monitor an earth dam during its construction and commissioning phases.

Finally, I wish to reinforce the principle that geophysics results do not constitute a 'test' and any interpretation should be confirmed by direct investigation.

Written Contributions

Colin Makinson (GIBB Ltd)
Residual strength of Lower Lias Clay
With reference to Mr Swannell's paper on 'The Lower Lias Clay at Barrow No.3 Reservoir' and the statement that this 'stratum appears to have been subject to relatively little investigation compared to other clays in the United Kingdom', I would draw attention to the work done on Lower Lias and Fuller's Earth Clays for the Batheaston Bypass. Both strata were extensively investigated by Exploration Associates and GIBB Ltd on behalf of the Highways Agency. Most of the A46/A4/A36 bypass site overlies Lower Lias Clay and periglacial derivatives such as Head and Landslipped material.

The area is 22km due east of the Barrow site and even though spatial variations in the Lias depositional regime are well documented (BGS, 1984 - reference below), the soil descriptions tally admirably apart from the use of the word 'laminated' rather than 'fissured'. In effect it was the weathered, overconsolidated, Lower Lias Clay soil blanketing the hillsides that was investigated rather than the unsoftened mudstone stratum.

GIBB Ltd's 1987 Interpretative Report, Volume 3, from the investigation of the preferred route corridor (following a 1983 regional investigation), devotes 24 pages and 43 figures to the Intact and Landslip Lower Lias Clay. The dates of the reservoir studies (1982 and 1992) bracket the Batheaston investigations which seem to have been overlooked in the desk study of the Lower Lias Clay, although presumably available through SWRO of the Department of Transport (later Highways Agency).

Fortunately there is a fair measure of agreement on the residual shear strength of the Lower Lias when back analysis is invoked in lieu of a straight dependence on laboratory testing of small samples. GIBB derived the following effective stress friction angles, assuming a nominal cohesion $c' = 2kN/m^2$ throughout, for use in sensitivity analyses of slope stability. An adequate stability factor of safety was demonstrated for a range of assumed groundwater conditions, from the extreme water at surface condition to the normal measured water table in the landslip.

ANGLE OF INTERNAL FRICTION (DEGREES)	**PEAK ϕ'**		**RESIDUAL ϕ'_r**
WATER TABLE CONDITION	EXTREME	NORMAL	RANGE *
Intact Lias Clay	27	24	17
Landslip Lias Clay	19	17	12.5

* In practice the $\phi'_r = 17^\circ$ value was applied to longer slip planes and the $\phi'_r = 12.5^\circ$ value to short lengths of slip plane lying only in clay and adversely orientated. The figures of 17º to 19º were obtained by back analysis of a number of regional landslips, including one with recent (mid 1960's) movement into the Kennet and Avon canal. The 12.5º figure comes from laboratory testing and Chandler *et al*, 1976 (Reference below and Reference 12 on Fig. 3-36).

Of course residual strength applies to all 'fossil' slip planes within the landslip envelope and indeed the mechanism of movement in modern temperate climate conditions is often by progressive remobilisation along pseudo-circular arc paths of small sections of the large sheet landslip. The curved graph correlating residual friction angle to overburden load, for the range of normal effective stress up to 250kN/m^2,and the scatter of these results is indicated on Fig. 3-36 (reproduced from the 1987 Interpretative Report). In the plot c'_r is incorporated as an increment in ϕ'_r. Further plots, Figs. 3-37 and 3-38, show the correlation of residual friction angle with plasticity index and clay fraction, in comparison with those given in Lupini et al, 1981 (Reference below and Reference 37 on Figs. 3-37 and 3-38).

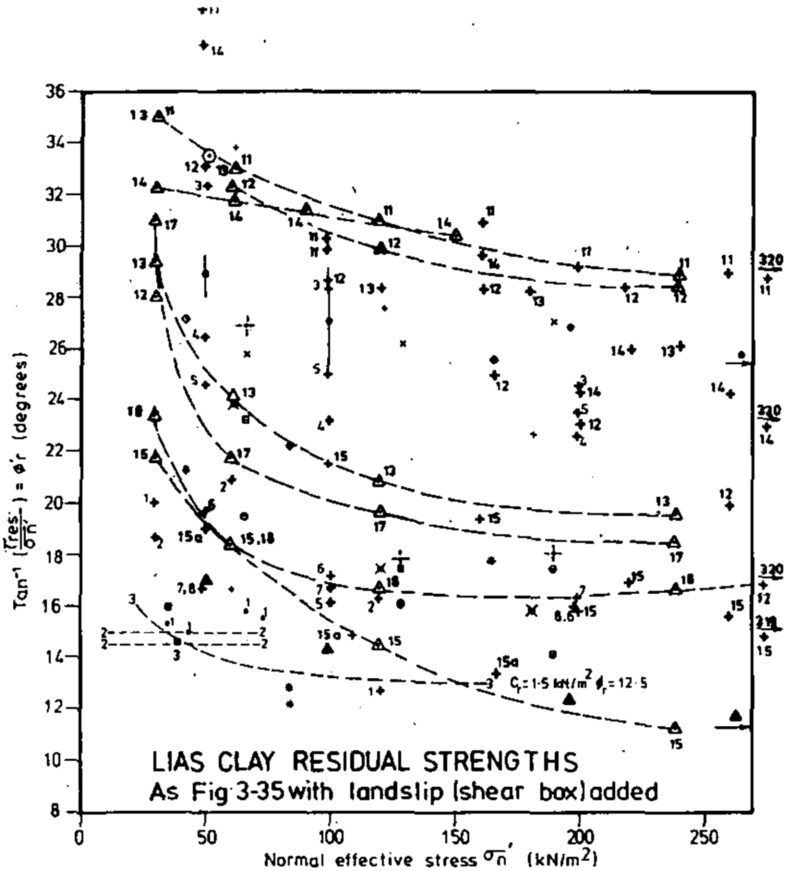
In 1993 a final investigation to further define the A46 landslips was carried out. One interesting feature was a 20m deep by 3m diameter inspection shaft driven through the 11m deep landslip mass, with precast concrete support rings like a sewage manhole. This picked up the cambering of the hillside, which resulted in irregular tilting of the thin blocks of limestone (not always continuous layers) and clearly defined the basal shear plane as a continuous feature. However there were numerous slickensided shear planes with variable inclinations, both above and below the landslip envelope as defined by the basal shear plane. These are clearly just the result of stress relief movements within the cambered and oversteepened hillside; remobilisation of landslips resulted from river erosion due to low sea levels late in the glacial epoch. If encountered in a small diameter, continuous cored, rotary borehole the full diameter slickensided discontinuities would have probably been interpreted as further slip planes. The envelope of any fossil landslide is therefore difficult to define but of primary importance for back analysis.

For the road construction, we were merely interested in upgrading the natural hillslope factor of safety of 1.0 nominal to 1.3 for road earthworks. It is therefore a reasonable risk to ignore the presence of short sections of slip plane entirely within clay at a lower residual strength than derived from back analysis. Such occurrences, if adversely orientated, merely cause localised collapse during construction. However for the case in question of an embankment dam, one would be more inclined to apply minimal residual strength parameters along the well defined slip plane and still expect the traditional factors of safety. The option of adopting the lower bound of possible shear strength parameters usually adopted for dam design is frankly not possible for a landslide situation, since the man-made loading will automatically force the natural Factor of Safety below unity.

Dr Charles in his Binnie Lecture appears to allow some discretion in accepting lower factors of safety in respect of the stability of an old embankment, but this would perhaps not be appropriate if residual strengths directly derived from back-analysis alone are utilised. Although the ring shear test, applying torsional shear to slurried samples, is a good indication of how low these residual parameters can be (ϕ'_r = 10 to 12º), we were chastised by external reviewers proposing 6º based on laboratory testing of leached (Fuller's Earth) clay samples, and dependent on pore pressure suctions to hold the slope up. If the Lower Lias is not well understood, it is not for want of in-depth study; but in the end judgement must prevail assisted by sensitivity studies for a range of possible slip envelopes and parameters.

References:

British Geological Survey: (1984)	*'Geology of the Bristol District: The Lower Jurassic Rocks'* HMSO
Chandler, RJ et al: (1976)	*'Valley slope sections in Jurassic strata near Bath, Somerset'* Phil. Trans. R.Soc. Lond. A238, pp.527-556
Lupini, JF et al: (1981)	*'The drained residual strength of cohesive soils'* Geotechnique, vol. 31, No. 2, pp.181-213



KEY

Published data

1 Chandler et al (1973)-Ref 31
2 Hutchinson et al (1974)-Ref 32
3 Chandler et al (1976)-Ref 12
■ Denotes back-analysed value
● Measured in lab tests
-- Envelope from lab tests

1983 AND PREVIOUS INVESTIGATIONS

Reversal shear box tests:
× LGR 6/7
• LGR 6/16
o LGR 17/17
⊡ LGR 17/19
┼ LGR 37/8
✱ LGR 37/12
◇ FEA 3/8
• FEA 17/16
⊙ BB 17/11

Ring shear tests:
▲ BB 17/5 5·6-6·0
● BB 17/11 11·1 11·4

1987 RING SHEAR TESTS

Symbol	Hole Pit / Depth	Grade	LL	PI	Cr′	φr′	<2μ	<75μ	Location
△ 11	PB 150 / 3·0	Sg 1	41	27	7	28	0	84·8	Tunnel N Approach
△ 12	PB 145 / 3·4	Sg 1	56	27	3	29	15·9	96·6	Tunnel N Approach
△ 13	PB 147 / 2·8	Sg 1	54	30	9	18	14·3	96·8	Tunnel N Approach
△ 14	TB 8 / 4·0	M.S	NP	NP	0	31	1·0	29·8	Old House Slip
△ 15	PB 122 / 1·5	Sg 1	71	35	9	10	23·4	99·1	A4/A46 Roundabout
△ 17	PB 79 / 1·0	g1 III	45	29	7	17	5·3	96·6	Upper A 36
△ 18	BB 33 / 4·4	Sg 1	52	25	4	17	25·0	100 0	Old House Slip

1987 INVESTIGATION DATA

Results for intact Lias samples from shear box tests (+1 to +8 as on Figure 3-16)
Results for landslip Lias samples from shear box tests (+11 to +16 as on Figure 3-33)

Figure 3.36 from Gibb (1987).

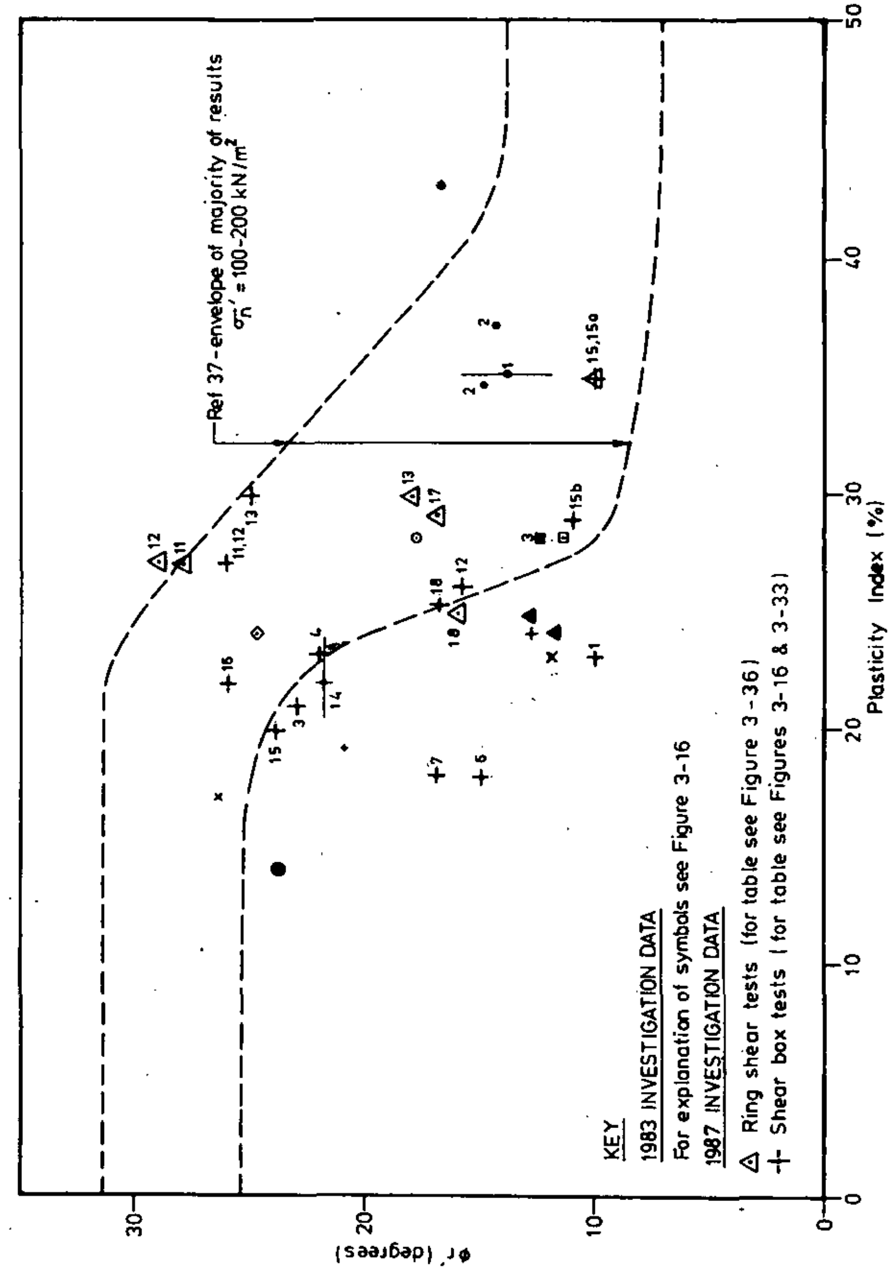


Figure 3.37 from Gibb (1987).

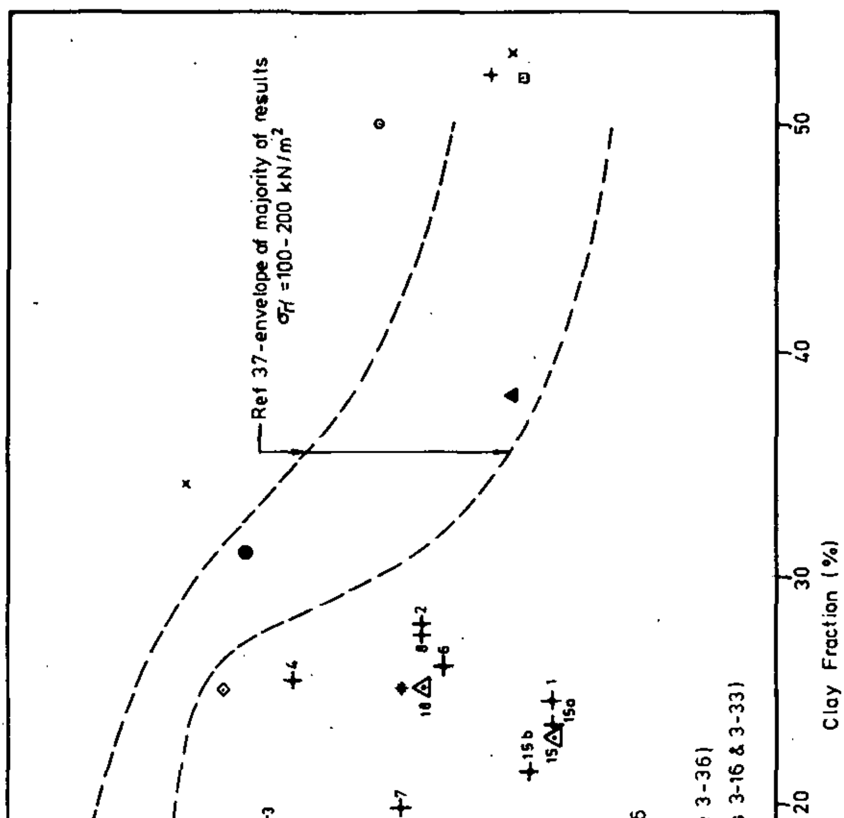


Figure 3.38 from Gibb (1987).

Nick Swannell (Halcrow Group)
I have read Colin Makinson's notes on Lower Lias Clay with great interest and am pleased to note a general agreement between our findings. I am also pleased that my paper has had the effect of bringing Mr Makinson's work into a more accessible domain. We had no knowledge of the Batheaston investigations during the study at Barrow No. 3 Reservoir and I would prefer to regard the data as being 'not discovered' rather than 'overlooked'.

SESSION 4
RESERVOIR OPERATION AND MANAGEMENT

Chairman	Andy Hughes
Technical Reporter	Henry Hewlett

Papers presented

1. Reservoirs and flood control: A Northern perspective.
   A MacDonald and G A McInally
2. Essential engineering criteria for the abandonment of tailings lagoons as environmental wetland features.
   D R Lamont, J R Leeming and M Brumby
3. Sediment management studies of Tarbela Dam, Pakistan.
   L J S Attewill, W R White, S M Tariq and A Bilgi

Papers not presented

1. Water storage for the Middle Level Cambridgeshire Fens.
   A Rowland
2. Challenges and opportunities for flood storage reservoirs.
   A T Pepper, D Pettifer and J Fitzsimons
3. Reservoir operation to control sedimentation: techniques for assessment.
   E Atkinson

SESSION 4

Chair

We have three very different papers in this session. The first one, prepared by Graham McInally of Babties, is on flood alleviation schemes; the second, by Donald Lamont, Bob Leeming and Martin Brumby, deals with 'Essential engineering criteria for the abandonment of tailings lagoons as environmental wetland features'; and the third paper, by Lawrence Attewill, Rodney White, Sadar Tariq and Attila Bilgi, is on the 'Sediment management studies of Tarbela Dam, Pakistan'.

Discussion

Andy Rowland (Binnie Black & Veatch)
With reference to the paper on Tarbela Dam, have the authors considered the effects of releasing large quantities of silt on the river system downstream? Will there be deposition resulting in a reduction of carrying capacity of the river and consequential increase in risk of flooding?

Rodney White (HR Wallingford)
The paper was written at the completion of the original study which did not encompass downstream effects. We were subsequently commissioned to look at the effects downstream of Tarbela. The short answer is that there is very little effect, the fine sediments travel downstream to the Attock Gorge without significant deposition. Beyond the Attock Gorge there is a proposed new dam and the effects of fine sediments from Tarbela will have to be considered in the design of this project.

John Beaver (Halcrow Group)
Referring to the paper by Attewill et al, what, if they are going to proceed with the regular flushing regime, will be the effect on the waterways of the coarser fractions coming down?

Rodney White (HR Wallingford)
The model shows that as far as the Attock Gorge, which is 30-40 miles downstream of Tarbela, the fine sediments simply pass over the coarse gravel river bed which is, and was prior to the construction of Tarbela, present in that reach. So, from that point of view there is no problem and, in a way, we would simply be re-establishing conditions prior to the construction of Tarbela. There are two differences, however. First, Tarbela attenuates flood flows so peak discharges are somewhat lower than they were prior to construction. Second, Tarbela will store the larger bed material sizes almost indefinitely and, because of this, the size distribution of sediments passing Tarbela during flushing will differ marginally from the distribution prior to construction. I do not subscribe to the view that the sluicing of sediment through Tarbela will induce dramatic changes in conditions further downstream.

Ian Gowans (Cuthbertson)
Lamont et al are to be congratulated on their excellent paper and the quality of their projects. Cuthbertson were appointed to register about 45 washland reservoirs in Yorkshire that fell within the ambit of the Act. The format of reports required by the Act (Schedule 2. Regulation 4 of the Statutory Instrument 1986 No.468) refers to the following:

"(v) adequacy and condition of the waste weir or overflow and connecting channels

(vi) alterations in levels of overflow cills

(vii) adequacy of margin between the top of bank level and overflow level."

These items are not appropriate to washland reservoirs which are non-impounding, off-stream flood attenuation reservoirs. The format of the Reservoir Inspection Reports, whilst containing sections covering the points set out in the Statutory Instrument, was adjusted to deal with the particular circumstances of the washlands.

Mike Atkinson (Ç L Associates)
In the written paper on the abandonment of tailings lagoons, under the heading of public safety, the authors state that the minimum shear strength for the crust on the soft deposits should be about 6 kN/m^2 before public access is contemplated. I question the authors on the basis for choosing this value; in my opinion, a significantly higher strength would be appropriate, based on a detailed study of the intertidal mudflats at Immingham on the Humber Estuary. Here, the measured strength is 10 kN/m^2 at the surface, increasing gradually with depth. However, even with this higher strength it is extremely difficult for a person to walk on the mudflats without sinking in to the point of risking being trapped in the mud.

Martin Brumby (R J B Mining)
The way it was derived was in two ways: first of all, there's been some scientific study done on the effect of overtipping lagoons using small bulldozers with wide tracks and different thicknesses of layers. The conclusion reached was that unless the top layer of deposits was at a strength around about 5 or 6 kN/m^2 you weren't going to achieve anything. Basically, the material would just punch in.

The other way that I looked at it was to use a vane test. I've worked basically on the principle that if I can walk on the deposits with my sort of general profile (that I'm sure you can all see), and the deposits will withstand my weight and I can take a measurement and get round about 6 kN/m^2 then I think most people will probably be reasonably safe because I've got fairly small feet for my size! I have also done similar exercises in places such as the Dee Estuary and that's the kind of strength you get in the sort of deposits on the side of the Dee Estuary, for example, so there's nothing terribly high-powered about it.

As far as public safety is concerned I think the idea is to try to discourage people from going into the lagoon in the first place. Partly that will be done by encouraging reeds and rushes and what have you around the edge, partly, I guess, by appropriate signs, and partly by making sure that the gradients around the perimeter of the lagoon in the abandoned state are such that if people get in, get wet and dirty, they're unlikely to drown.

At the end of the day it has to be recognised, I'm sure it will be, that any water body anywhere - even a large puddle - is perfectly capable of killing a toddler or a drunk or somebody having a fit or something similar. There's no such thing as a safe water body anywhere. Our concerns, both from the Health and Safety Executive and mine from RJB, were basically to ensure that there weren't any nasty geotechnical implications, if you want to put it that way, or long-term stability problems and also to make sure that the risks associated with people wandering into lagoons when they were drunk etc were minimised to the lowest level practical.

Jonathan Hinks (Halcrow Group)
As has been explained, there are two large irrigation outlets at Tarbela. Tunnel No 4 at the right abutment has a capacity of 2600m^3/sec. Tunnel No 5 at the left abutment has a discharge capacity of 2300m^3/sec. The cill of Tunnel No 4 is 390 ft below TWL. However the existing tunnels will not be effective for flushing sediments except perhaps in the immediate vicinity of the tunnels. A similar situation was anticipated at the Mrica Project in Indonesia where the bottom outlet was positioned vertically beneath the power intake to try to keep it free of sediments. However, it was always recognised that it would not be effective for more general flushing of sediments. That was some years ago and reading Mr Atkinson's paper on p.226 it is clear that knowledge has advanced in respect to the art of sluicing and flushing. The paper by Mr Attewill and colleagues describes a more radical and ambitious scheme for dealing with sediments at Tarbela and will be of great interest in relation to many schemes around the world as reservoirs silt up.

I would however like to pick up on a couple of points in relation to the possibility of liquefaction of the foreset slope causing blocking of the intakes. From Figure 3 in the paper we see that the foreset slope is presently 7 or 8 miles upstream of the dam. It is advancing about $^1/_2$ mile per year so the risk is getting greater with time.

There have been a number of cases of submarine flow slides on ocean beds but I have not heard of earthquake induced liquefaction of sediments in reservoirs. It is true that the intakes were blocked at the Ambuklao scheme in the Philippines after an earthquake in 1990 but detailed enquiries suggest that this was not a case of liquefaction but rather of soils on the reservoir banks being loosened by an earthquake and then being washed into the reservoir by monsoon rains.

The paper mentions a peak ground acceleration of 0.13g to cause liquefaction but I think this figure needs to be treated with caution. A peak ground acceleration of 0.27g was measured at Tarbela dam in February 1996 for an earthquake with a surface magnitude of 5.2 and epicentre close to the dam. Accelerations at the foreset slope would have been slightly attenuated. No liquefaction was observed. It is very tempting to characterise earthquakes in terms of PGA but as Dr Taylor pointed out this morning other parameters such as frequency and duration of shaking are likely to be just as important. The other parameter which is of interest at Tarbela is the time taken for the liquefied material to reach the intakes. If there is sufficient time and adequate warning it might be possible to close the intake gates before the liquefied material arrives. However it is not easy to predict the speed of travel of the liquefied material and I should be very interested to know whether anybody can shed light on this problem. Our estimate, based on data for submarine events, was that it might take about an hour which would not really give time to close the gates even if a warning system were to be set up. So my question is whether anybody can point to any data which would be relevant to the question of speed of travel of the liquefied material?

Lawrence Attewill (Tams Consultants)
I would endorse Jonathan Hinks' comments entirely: we recognise that the 0.13g is not anything much more than an indicative figure based on what's done in the '80s and we have recommended in our report that a good deal more work is done on the whole question of the behaviour of the liquefied delta. However, such a study would confirm or change the slope of the limit line which might eventually only alter the timing of the schemes involved. The level of the studies so far isn't up to that point.

Chris Binnie (Independent Consultant)
I am concerned that unless the profession designs, builds and publicises the environmental benefits of dams and reservoirs, society will not allow them to build more.

This slide of the area of Abberton Reservoir prior to construction shows poor grade agricultural land with large fields and sparse hedges. The reservoir is now a great attraction for waders, geese, and tree-nesting cormorants and has become a SSSI and an internationally recognised RAMSAR site.
The following slides of Singapore reservoirs show how, in a country with a

shortage of land, reservoirs have been developed for birds, monkeys, fish and turtles, and for recreation such as golf, landscaped garden, concerts, bicycling, etc.

This is a slide of Marchlyn, the dam to be seen tomorrow. I was the Project Engineer for the 70m high dam and I would ask that delegates judge not only its engineering but also its landscaped effects.

SESSION 5 : PART 1
CONSTRUCTION AND REHABILITATION CASE HISTORIES

Chairman — Geoff Sims
Technical Reporter — Ian Gowans

Papers presented

1. Rehabilitation of Holmestyes Reservoir.
   T Dyke and P J Williams (presented by J Hinks)
2. Grouting the puddle clay core at Barrow No 3 Reservoir, Bristol.
   T St John, R A Nicholls and K W Senior (presented by C Hunt)
3. The rehabilitation of Luxhay Dam, Somerset.
   J P Millmore, J F Stables and F E Shannon
4. The use of a composite HDPE membrane/bentonite-cement slurry trench cut-off at Broadwood Loch, Cumbernauld.
   K M H Barr, C W Berry and P J Barker
5. Ireland Colliery Reservoir: A reservoir created by deep mining subsidence.
   A K Hughes and C J Beech

SESSION 5 : PART 1

Chair

I wondered whether many people would be here on Saturday morning and I'm delighted to be faced with considerably more than fifty percent of delegates. By any measure they must be the better half of the Conference!

May I remind you of the yellow comment forms. That attractive box there is the one that contains the completed forms which tells posterity of what you think of Ian Carter's efforts. I'm happy to take the opportunity of saying that I have been particularly impressed with the way that the Conference has gone so far and I hope that the comments will reflect that.

The timetable on Saturday morning is always critical and I will be seeking to maximise the period for discussion. The programme of this first part of Session 5 is on the overhead in front.

Discussion

The rehabilitation of Holmestyes Reservoir [presented by Jonathan Hinks (Halcrow Group)]

Alan Johnston (Babtie Group)
The paper on Holmestyes Reservoir describes a ring of grout holes circling the valve shaft and these are shown in Fig 2 as including holes drilled down through the puddle clay core of the dam. Since the core appeared to be satisfactory, was there any danger of damaging the core in this operation?

Jonathan Hinks (Halcrow Group)
The core is effectively extended upstream around the valve shaft. It is not intact as evidenced by the leakage into the valve shaft and the need for an upstream clay blanket soon after the original construction. Drilling into this material was necessary to reduce the leakage into the shaft. The main waterproof membrane at Holmestyes is the upstream clay blanket as evidenced by low piezometric pressures in the upstream shoulder.

Grouting the puddle clay core at Barrow No 3 Reservoir, Bristol [presented by Colin Hunt (Bristol Water)]

Ian Hay (RKL-Arup)
In his presentation of the paper "Grouting of the puddle clay core at Barrow No 3 Reservoir, Bristol", Colin Hunt mentioned that there were eighteen "major" failures and seven "minor" failures whilst the reservoir was being constructed. Could he please explain what determined whether a failure was "major" or "minor"?

Chair
What form of records were kept to enable this question to be answered?

Rod Bridle (Independent Civil Engineer)
Following on from Colin Hunt's presentation on Barrow No 3 Reservoir, I have a few observations:

The history of the reservoir, referenced as Watson Hawksley (1982), was prepared, in his usual meticulous fashion, by Peter Horswill, Chief Engineering Geologist, Montgomery Watson. The source materials were the Bristol Waterworks Board Minute books, old drawings and the T&C Hawksley archive. It is pleasing that the history Peter prepared is being put to good use in maintaining the safety of the reservoir.

The Engineer on commencement of the Works was James Simpson. Following the numerous landslips Thomas Hawksley was called in. The slopes were flattened by construction of "poultices" (nowadays we would probably say berms) to about 1 in 12 in places, probably unprecedented at that time.

The fundamental cause of the instability was that much of the embankment was built on the foundered masses and toes of landslips in the Lower Lias Clay on the slopes above and across the reservoir. To this day, the area below this reservoir is named "The Wild Country" on maps, reflecting the hummocky ground and scrubby vegetation there, typical of landslip debris. Nick Swannell describes the investigations into the geotechnical parameters of the Lower Lias Clay in his conference paper (Page 92).

Delegates may be interested to know a little of a more recent historic event in relation to Barrow No. 3. Soon after I joined T&C Hawksley in 1970, I was summoned to Mr Hawksley's office. This was "young" Thomas, the great grandson of Thomas the First. He was poring over a site investigation report from Soil Mechanics Ltd. I noticed that the moisture content plots showed a number of horizons where the moisture content rocketed upwards and thought that this was probably indicative of a failure surface, or at least a pause in construction. He also had a copy of Terzaghi and Peck open. As a recent graduate in soil mechanics from Imperial College I was hopeful that I would get a chance to make a name for myself by carrying out some investigation and analysis of these interesting results. "Now" he said, as I perked up expectantly, "I've studied this report, it talks about undrained strength which it calls 'c_u'. I've looked in Terzaghi and Peck and found undrained strength, but they call it 's_u' ". I explained that the undrained shear strength was known as c_u in English and s_u in American, thinking that my familiarity with such things would lead me into the assignment I was hoping for. However, all I got was "Hmm, you soil mechanics types should get your terminology straight" and my splutterings thereafter didn't get me anything to do on the job. That investigation led to the 1970 grouting.

I learned afterwards that "young" Thomas was a very conscientious engineer who liked to do most of the engineering on his projects himself.

In relation to an earlier appeal for innovation in carrying out repairs to cores, I am not offering any, but just wanted to say that many cores become cracked at the top because of desiccation. Replacement of the top part of the core with a slurry, a plastic concrete wall or something similar overcomes the desiccation problem and maintains the core at lower levels where desiccation is less likely to be a problem. Although we think of cores as being extremely flexible, the amount of movement of an old core is relatively small as most supervising engineers will know from the settlement records of their own reservoirs. Grouting can be used to seal leaks but, more importantly, it also raises the minimum stress in the core thereby reducing the potential for hydraulic fracture, which after desiccation, is a common cause of cracking in cores.

Colin Hunt (Bristol Water)
All embankment slips were important. They were all failures. The classification of "major" slips were those at the embankment core. They often sheared through the clay core of the dam. There was a study carried out by Montgomery Watson in 1982. That report gives a complete history of the Barrow No 3 Reservoir and the information is provided by Mr Bridle's contribution above.

The rehabilitation of Luxhay Dam, Somerset [presented by Jim Millmore (Babtie Group)]

John Sammons (Independent Consultant)
Jim Millmore indicated that the site investigation using boreholes found nothing conclusive about the cause of leakage. I wondered firstly what he had hoped to discover and, secondly, whether other forms of investigation such as geophysics or temperature monitoring might have been of some use.

Paul Tedd (Building Research Establishment)
I think that the approach that Mr Millmore adopted in terms of investigation was probably what I would do because you can go on investigating for only so long. Bearing in mind that they had established that the leak was in the top metre, to go ahead and do what they did seemed like a sensible thing to do. I am not sure they would have found out any more had they put in temperature probes or some other techniques because all these other techniques don't always, or rarely, give you conclusive information. Mr Millmore said that he was concerned about the brittle behaviour of the cement-bentonite and the material under confined triaxial conditions is brittle under low effective confining pressures which is essentially what they've got there. If there is a possibility of reasonably large drawdown settlements

at the dam, is he at all concerned that you may get cracking occurring on the underside of the actual wall, the new wall in the middle of the dam?

Ian Hay (RKL-Arup)
I endorse the approach to the design of the remedial works at Luxhay Dam described by Messrs Millmore, Stables and Shannon in their paper "Rehabilitation of Luxhay Dam, Somerset". The initial site investigation had revealed no conclusive evidence as to the cause of the leakage that was observed when the reservoir was within 1 metre of top water level. The designers believed that further investigations were unlikely to provide conclusive evidence of the exact cause of leakage nor would they lead to a reduction in the cost of the permanent remedial works. They therefore took the sensible decision to curtail any further investigation, interpret the information which they already had and rely on their engineering judgement in designing the remedial works.

In an earlier session at this Conference, we were urged that if we did not have enough information on which to base our decisions, we should do everything possible to obtain additional information, if necessary by further monitoring or site investigation. However, an equally strong message has emerged at the Conference: don't rely too much on the numbers in an analysis - *use engineering judgement also.* All designers face this dilemma - deciding when they have sufficient information to enable them to reliably exercise their engineering judgement.

Jim Millmore (Babtie Group)
We expected to find a clay core lower than top water level with possibly weak material in the core with hydraulic fracturing. Gravels could also be present in the core. Geophysical and temperature monitoring were considered but in this instance it was considered most unlikely to find conclusive results. Paul Tedd supported the form of investigations undertaken. In this instance, we felt it was unlikely that we would find more information using the other techniques. The leaks would most likely occur through the top 1 metre of core, the core material being in a brittle condition. Large draw down settlement had occurred and the proposal for a new slurry wall in the middle of the core was appropriate.

The slurry wall built into the core material of the dam was more flexible than initial tests suggested. Tests on cubes taken of the slurry indicated brittle conditions which were not reflected in samples in the slurry wall which were found to be more flexible. A finite element analysis by Keller showed that the diaphragm wall and the existing embankment core material were very close in properties.

In connection with what were we expecting to find, we were thinking there may have been a whole series of possibilities. For example, the top of the clay core may have been lower than the top water level; there may have been weak material within the body of the clay core which could have indicated hydraulic fracturing or collapse in some way of the puddle clay at some time in its history; we were looking for the possibility of gravels. These were some of the options we considered. In terms of geophysical methods, we did consider geophysical methods to try to identify any anomalies but the advice we were constantly receiving was that we were unlikely to find the sort of differences in material likely to be encountered because the type of material or the variations in material were not sufficiently distinct compared to the differences or anomalies you need to have to give meaningful results with geophysics. With regard to temperature measurements, yes we did consider all sorts of novel things associated with temperatures and we did try to measure the temperature of the water on the surface of the dam embankment. We also contemplated model aircraft with infrared photography, but we realised that none of this was going to be sufficiently conclusive, again on the basis of the advice we were receiving.

Referring to Dr Tedd's contribution, in connection with the possibility of something untoward occurring at the base of the diaphragm wall, this in fact was a real point of concern. The material, based on the initial testing, was perceived to be far more rigid than we were looking for. The subsequent testing found that the material was a little more flexible than had earlier been indicated and more in line with the material we had specified. The finite element analysis which was undertaken by Keller to try to establish the interaction between the diaphragm wall and the existing embankment took into account the raising and lowering of the reservoir and we looked very carefully at the likely movement of the embankment and the diaphragm wall. We did not find that there would be sufficient deformation to cause us concern.

Vic Over (Bolton Institute)
I agree with Mr Millmore's reservations about applying geophysical investigations as an emergency measure with the aim of producing immediate results. In this context any anomalies encountered, such as zones of low resistivity, may well be small and difficult to interpret. I believe this reinforces the research I am leading on geophysical monitoring. An important feature

is not just the presence of an anomaly, which may be either due to a constructional feature or development of seepage, but the development of slight changes over a period of time. Regular monitoring enables a dynamic profile to be built up. An increased rate of change in the resistivity profile could herald the appearance of a seepage and indicate its flowpath.

Andy Hughes (RKL-Arup)
The paper on Luxhay Dam draws attention to the importance of reservoir supervision and regular routine visits by the Undertaker's staff. I have found as a Panel AR Engineer that most owners in the public sector accept two visits by the Supervising Engineer. It is important for Panel Engineers to assess what level of supervision is provided by the owner, whether it be given by the Supervising Engineer, "waterman", reservoir keeper etc. The DETR (DOE) report format requires this. My concerns are that as time goes on, pressures are brought to bear to reduce staffing levels which are likely to affect the level of supervision provided.

I request that all Inspecting Engineers assist our colleagues responsible for reservoir supervision in stressing the importance of supervision to their owners.

The use of composite HDPE membrane/bentonite-cement slurry trench cut-off at Broadwood Loch, Cumbernauld [presented by Kenneth Barr (Fairhurst)]

Alan Johnston (Babtie Group)
The paper on Broadwood Loch demonstrates the value of utilising the experience of specialist contractors before finalising a design. It would be interesting to know how much was saved by changing from a concrete cut-off to the composite bentonite/HDPE trench.

Kenneth Barr (Fairhurst)
There was a saving in tender costs of about £90,000, or about 30% of the cost of the original concrete cut-off wall design.

Andrew Thomas (Independent Supervising Engineer)
The dam at Dolwen Reservoir in North East Wales was constructed in 1905 with a clay core supported by earth shoulders. In the mid 1970s the core was raised with a concrete diaphragm wall. Recently a leakage has developed. When the reservoir level is below the level of the bottom of the concrete, there is no discernible leakage but above that level leakage starts.

It is not surprising that leakage is occurring at this level since concrete and clay are not compatible and with regards to movement the two cannot be expected to act as one homogenous mass.

This observation also leads to a reconsideration of using cement grout in an earth structure. The grout will form a structure like coral which may be quite fragile and unlikely to have the characteristics to resist settlement or heave in the earth or movement due to seismic forces.

The ideal solution is to repair with like materials such as raising a clay core with similar clay worked in a similar fashion to the old material. Some progress has been made in better compatibility with the use of bentonite but, alternatively, engineers should be looking more widely at other materials rather than using the traditional cement based compounds. There is not usually a need for structural strength in the earth and the main need is to provide an impermeable barrier so maybe the answer is to consider materials such as plastic polymers, possibly polyurethane foam. This may provide the impermeability with the flexibility and compatibility that is needed. The question to the authors of the papers is what consideration was given to the compatibility between the new materials introduced and the existing materials and the second question to the audience at large is what consideration is being given to the use of materials other than the traditional ones.

Kenneth Barr (Fairhurst)
The benefit of flexibility was recognised by the designers, and was one of the reasons why the alternative design was accepted on technical grounds. Hardened bentonite/cement slurries can accommodate some movement, and the required minimum strain at failure of the mix adopted at Broadwood was 5%, as is commonly specified. The actual strain at failure of test samples generally exceeded 20%. If strains exceeding the capacity of the set slurry are experienced in-situ, the geomembrane will ensure that the cut-off continues to perform within its specification.

Andy Hughes (RKL-Arup)
It appears that the designers considered that the HDPE membrane is vitally important to the performance of the dam. What is known about the longevity of this sort of membrane in a bentonite slurry trench?. What systems are in place to measure performance, and how was the depth of cut-off designed/ checked?

Kenneth Barr (Fairhurst)
HDPE has a recognised track record in other applications, and its use as a geomembrane in slurry trenches is now well established in the landfill industry. There has been a considerable amount of research about its performance in landfill applications both by the specialist contractors, and by agencies such as the BRE. The research includes its performance in much more aggressive environments than are normally encountered in dam engineering. BRE Digest 395, referenced in the text of the paper, gives information on applicable test methods and includes references to other sources of information about durability.

Performance of the cut-off is monitored by hydraulic piezometers located in the dam foundation upstream and downstream of the wall.

The cut-off was designed to fully penetrate the alluvial and glacial overburden, and to be keyed a minimum of 600mm into rock. The depth was checked by manually dipping the slurry trench at rockhead and on completion of excavation.

Derek Knight (Independent Consultant)
The paper by Barr, Berry and Barker describes the use of a composite HDPE membrane/bentonite-cement slurry cut-off, and comments (page 281) that "A more recent innovation however is the inclusion, within these walls, of an HDPE membrane". Four prior examples of such use are quoted between 1988 and 1992.

In a brief description of a major HDPE membrane cut-off constructed in 1980 in the Dead Sea, Jordan, this contributor referred to it as "the only such example known to the Author" (Dams & Reservoirs, November 1997, Vol 7, No 3, Page 30). That particular cut-off was 14.5km long and 7m deep, and is fully described by Brice and Woodward in their paper "Arab Potash solar evaporation system : design and development of a novel membrane cut-off wall", Proc ICE, February 1984, Pages 185-205. The further examples given by Barr, Berry and Barker are a useful addition to what can be expected to become an increasing list.

It is pertinent to add that some old UK dams have, in recent years, had their cast iron draw-off pipes through the embankment fill lined internally with HDPE linings, as a safeguard against cracking and consequential internal erosion of the surrounding dam fill. The authors rightly state (page 290) that the use of an HDPE membrane in dam engineering is advantageous in the "provision of increased reliability where loss of water is a significant issue". This principle applies equally to pipework as to cut-offs.

Jim Stables (Consultant, formerly Wessex Water)
Could Mr Barr give us an indication as to the cost of the Broadwood Loch project? He stated that there was a saving of £90,000 which he said was about 10% of the project cost. That suggests a total cost of close to £1M or £900,000 which seems a different order of magnitude from the costs at Luxhay.

Kenneth Barr (Fairhurst)
The saving of £90,000 was about 10% of the project tender cost, which included construction of the embankment, rock grouting, the reinforced concrete overflow and outlet structure as well as the cut-off. The cost of the cut-off was about £190,000, or £120/m^2. This appears to be a little higher than at Luxhay, but reflects the greater depth, requiring excavation by crane-mounted grab rather than backhoe, and the cost of the geomembrane.

Ireland Colliery Reservoir: A reservoir created by deep mining subsidence [presented by Chris Beech (Derbyshire County Council)]

Martin Airey (Mott Macdonald)
In view of the fact that the reservoir has been formed by mining subsidence, has the possibility of future mining subsidence been considered when specifying the scope and nature of the geotechnical investigations? Has it been necessary to include special measures in the design of the works, particularly for the modified railway embankment, to accommodate any future mining subsidence and thus ensure that the freeboard is maintained?

Chris Beech (Derbyshire County Council)
The 2m subsidence fromBritish Coal mining had substantially occurred by 1993. Settlement has now substantially ceased. The camber was allowed in the embankment to allow for further minor settlement.

Andy Hughes (RKL-Arup)
Settlement on the embankment has now ceased. In the design of the embankment, the incorporation of the old railway embankments has caused me some concern and I wrote to the Client indicating that further work may be required to this part of the design. However, the modified embankment has performed extremely well.

Written Contribution

Kenneth Barr (Fairhurst)

The authors anticipated that there would be other examples of similar cut-offs constructed elsewhere and are grateful to Mr Knight for drawing their attention to the Arab Potash project. They have read the paper by Brice and Woodward with considerable interest. It is noted that different methods were used for trench excavation and different types of slurry adopted at the Arab Potash project compared to Broadwood Loch. These are obviously site-specific. The major technical advance since 1980 is the jointing of the HDPE membrane. This was done at the Arab Potash project by 1m overlaps of the membrane. There are now a number of methods available by forming inter-panel joints including the Geolock system used successfully at Broadwood.

SESSION 5 : PART 2
CONSTRUCTION AND REHABILITATION CASE HISTORIES

Chairman	Geoff Sims
Technical Reporter	John Scriven

Papers presented

1. Winscar Dam: Investigations and repairs to asphaltic concrete membrane.
   A C Wilson and A C Robertshaw
2. Raising Llysyfran and Brianne dams.
   R A N Hughes
3. The restoration of Rufford Lake.
   P G De Lande Long and C W Scott
4. Remedial works to upstream face protection, Megget Reservoir.
   D Gallacher, R M Doake and D Hay-Smith (presented by I Gowans)
5. Stabilisation of Tai Tam Tuk Dam, Hong Kong.
   D Gallacher and R J Mann
6. A drained synthetic geomembrane system for rehabilitation and construction of dams.
   A M Scuero and G L Vaschetti

Papers not presented

1. The ICOLD Committee on Rehabilitation of Dams.
   G P Sims and P Tedd
2. Recent developments in the seismic analysis of concrete gravity dams.
   C A Taylor and W E Daniell

Presentation

1. Chambon Dam
   G L Vaschetti

SESSION 5 : PART 2

<u>Discussion</u>

John Sammons (Independent Consulting Engineer)
A geomembrane similar to that described by G L Vaschetti was used as an upstream impervious barrier in construction by Impresit Bakalori of a 3.5km long sandfill embankment dam at Jibiya in northern Nigeria. The Italian designer particularly selected a geomembrane because of its ability to cope

with potential displacements, the dam being built over some 15m to 20m depth of collapsible sand. Geodam, the geomembrane used, was multi-layered, the layers being formed by being sprayed and bonded together during manufacture. It had a felt base and a fibre glass core and layers with special UV protection and good welding properties. A sample was available for view.

The geomembrane was tied along the upstream toe of the dam into a plastic bentonite concrete wall, which fully penetrated the sand foundation; a grout curtain continued the cut-off into the rock beneath. The geomembrane was weighted down and protected from wave action by 4m by 2m concrete slabs cast-in-situ over a felt cushion layer.

Did Gabriella Vaschetti have any comments on protection of such geomembranes from wave action?

Gabriella Vaschetti (Carpi Italia)
A comment on the difference between the membrane you mentioned and the CARPI membrane: the membrane used by CARPI is a geocomposite, consisting of a PVC geomembrane, typically 2 to 2.5mm thick, plasticised and stabilised to UV rays, which is the element providing impermeability, coupled during extrusion to a geotextile which is a transition element for drainage capability, additional anti-puncture protection, and additional dimensional stability.

The CARPI waterproofing geomembrane is one single material, homogeneous in its whole mass; that is to say the same highly performing material is present in the whole thickness of the waterproofing liner, to guarantee imperviousness and resistance. Elongation at break is superior to 240%.

The waterproofing membrane that Mr Sammons mentioned is made with different layers. The UV resistance on the outer surface is of limited thickness. The inner layers are not UV resistant and provide mechanical resistance.

Concerning wave action, ballasting can provide protection. CARPI installed a ballast layer at Bovilla dam, where actually the main reason for ballasting was to avoid uplift by the suction exerted by the very strong winds of the region. Bovilla, a rockfill dam in Albania, is the highest embankment dam in the world where a waterproofing geomembrane is the only watertight element. The geocomposite membrane was installed on the upstream face over stabilised gravel. A geotextile was positioned over the geocomposite as a transition and protection layer, and then concrete slabs were cast in place to ballast the membrane. The system was installed in 1996.

John Ackers (Binnie Black & Veatch)
I have a number of questions concerning the remedial works at Megget Reservoir.

Q1 I understand from the paper that it was accepted that the presence of the bitumen grout would impair the dissipation of wave energy. I would expect there to be a complementary effect on wave runup, which could be increased somewhat. Were any physical or other studies made into that effect?

Q2 What conclusions were reached regarding the likely effect of the grouting on the wave runup?

Q3 What was the effect of the new wind-wave assessments on the design wave height adopted for the assessment of the freeboard requirements for the dam?

Q4 Was the original dam freeboard sufficient to accommodate the runup resulting from the new assessments of wave height and any increase in runup ratio?

Ian Gowans (Cuthbertson)
In response to John Ackers:

Q1 No physical studies were made into the effect of the bitumen on dissipation of wave energy. The effect on wave run-up was studied initially by reference to the current ICE Guide assuming permeable rip-rap and subsequently a more refined assessment was made by the Owen overtopping method recommended in Hydraulics Research HRW Report SR 459.

Q2 The likely effect of grouting on the wave run-up was assessed by selecting a Roughness Value r in the Owen method appropriate for stones set in cement rather than for rubble layers, which is very conservative bearing in mind that the grouted rip-rap remains permeable.

Q3 The new wind-wave assessment increased the design wave height for assessment of freeboard.

Q4 The assessed overtopping discharge from grouted rip-rap was very low in relation to the upper limit for damage to a protected crest, and therefore the original dam freeboard was sufficient to accommodate the runup.

Arthur Penman (Geotechnical Engineering Consultant)
I would like to say a few words about Winscar and ask Mr Robertshaw a question. It was clear to us at BRE that it was better to place the impervious element on the upstream face of a rockfill embankment than within it, so that the whole of the rockfill would be resisting thrust from the reservoir water and a minimum volume of rockfill would be required. But following the bad experiences with concrete membranes on dumped rockfill in other parts of the world, British engineers were reluctant to embark on this type of design. Our work on rockfill showed that with a correct grading, ie with sufficient fines to fully bed every large lump of rock, and with placement in layers, even up to 2m thick, compacted with smooth vibrating rollers, amounts of deformation, as compared with the old no-fines dumped rockfill, were negligible. John Humphreys became quite enthusiastic about the idea of using upstream membranes and was looking into the possibility of Balfour Beatty joining with a German firm for the rights of placing asphaltic concrete membranes in Britain. This all led to the design of Winscar being as a compacted sandstone rockfill dam of quite steep slopes and the first in England to have an upstream bituminous membrane.

As so often happens, the site investigation proved a little optimistic about the depth to bedrock and quality of overburden, so the amount of stripping to formation level turned out to be greater than catered for in the design. The result was that the vertical toe wall that formed the plinth for the membrane had to be a little higher than intended. As we all know from compaction behind such things as bridge abutments, it is not easy to place rockfill in as dense a condition just behind the wall as it is out on the open dam fill. This means that there is a pocket of rockfill just behind the toe wall that is not as stiff as the rest of the fill. So when the reservoir water pressure comes to act on the membrane, it tends to get pushed down just behind the wall, producing a local hollow that puts tensile strains into the membrane that can cause tensile cracking.

When the reservoir was filled, springs developed in the rose gardens of the left valley side downstream of the dam, opposite the houses of Dunford Bridge that bordered the stream. It was assumed that water was passing behind the right abutment and the grout curtain was extended to cut off this flow. There was some reduction, but it was not cured. Later the reservoir was emptied, enabling the area of membrane adjacent to the toe wall to be examined in detail, after the silt had been hosed off. Small cracks were found of such an area that under the head of the full reservoir the flow through them could account for most of the observed leakage. The lesson from this is to avoid where possible a vertical plinth wall, using instead a horizontal plinth structure anchored down to bedrock, or backfill behind a wall with a material that can more readily be compacted to a stiffness compatible with the rest of the rockfill.

To endorse Alan Johnston's views on safety when working on bituminous membranes at the upstream slope of 1 on 1.7, it should be remembered that a man was killed at Winscar. Wearing a black plastic coat and leggings because of the rain, he lost his footing on the upper part of the membrane and falling prone he stretched out his arms to increase his area in contact with the wet membrane, but with nothing to catch hold of, he accelerated down the slope, slowly turning as he went, and had the awful misfortune to go headfirst into concrete at the bottom.

My question to the Authors is about the position and direction of the cracks found during the 1996 survey. Was there any indication that some were caused by continued settlement of the membrane over the area of the 1980 cracks? It would be expected that once reservoir pressure had compressed the slightly softer rockfill adjacent to the toe wall, little further compression would develop.

Andrew Robertshaw (Yorkshire Water)
The initial problems on first filling at Winscar that Professor Penman refers to were reported on in some detail by Chris Routh at the ICOLD Conference in San Francisco and this is referred to in our paper. Although the reservoir was not completely drained down in 1996 the investigations and repairs that are described in the paper did not indicate any concentration of defects near to the covering to the toe wall and they were, in fact, quite uniformly distributed over the face of the dam. Unlike Marchlyn Mawr which was visited yesterday there is no toe gallery at Winscar which made the problems during first filling extremely difficult to detect although after they had been found they were resolved very quickly. If there had been a toe gallery there is no doubt that the detection time would have been reduced significantly. The leakage from the whole foundation is collected in an extensive underdrainage system which is closely monitored against reservoir level and rainfall.

Andy Hughes (RKL-Arup)
A question for Chris Scott. Monoslab blocks do not appear to be the most

appropriate for use in the spillway at Rufford Lake. It appears that no damage was sustained before remedial works so why did damage occur after remedial works? Has the dam been overtopped?

A question for Gabriella Vaschetti. Mention was made of allowing for seismic movement. How is this done and who decides the degree of movement required between adjacent blocks?

Chris Scott (Binnie Black & Veatch)
The monoslab protection downstream of the spillway crest was installed at Rufford during earlier restoration works undertaken in 1974. The spillway was sized for floods up to the 1 in 100 year event with the auxiliary spillway intended to provide additional capacity for larger floods. It is understood that some minor repair work was required to the monoslabs between 1974 and 1988, mostly along the toe of the spillway where it discharges into the outlet channel. No damage of the sort described in the paper was reported and the spillway was understood to perform satisfactorily during overtopping. The freeboard of this spillway crest above normal lake level was 400mm.

As described in the paper, the restoration of the lake following the 1988 subsidence involved lowering the spillway crest level by 250mm, hence reducing the freeboard between the overflow gates and the spillway crest to 150mm. This reduced freeboard had two consequences: firstly, the spillway would be overtopped more frequently and secondly, waves on the lake could slop over the crest. The effect of the wave slop was some local washing out of material within the monoslabs immediately downstream of the crest, preventing grass becoming established in the monoslabs close to the crest. This lack of grass cover allowed flood water to penetrate the monoslabs when a flood occurred. The difference between the pre- and post-restoration situation was the effect of the wave slop in preventing grass becoming established in the monoslabs close to the spillway crest. The wave slop was a result of the reduced freeboard to the spillway crest.

In answer to the follow up query, it is understood that the auxiliary spillway has not been overtopped either before or after the 1990 restoration works.

Gabriella Vaschetti (Carpi Italia)
In answer to Dr Hughes. In dams located in seismic areas on which CARPI installed the geomembrane system, it was usually the owner who gave the indication and the specifications concerning the accommodation of movements due to seismic activity. To assess the behaviour of the

geomembrane system under these conditions, CARPI performs large scale testing in its laboratory. The laboratory includes a "seismic equipment" in which sudden opening of a crack can be simulated. The geomembrane system is installed over a platform consisting of a fixed part and a part that can tilt simultaneously. The sudden opening of a crack is thus performed, according to specific requirements, and observations are made on the behaviour of the system. This type of test was made for example before installation of the system on Chambon dam, and according to the requirements of the owner, Electricité de France.

Jim Claydon (Yorkshire Water)
There are several interesting comparisons between Winscar and Marchlyn Mawr. I was on site at Winscar during construction.

The toe wall at Winscar was designed to be short, with a vertical downstream face below a rounded corner detail. In practice it was built taller than designed because of the depth of unsuitable material being more than expected. The construction sequence dictated that the position was fixed before the foundation was stripped because of the rapid weathering of the shale when it was exposed. Over part of the length of the toe wall a fillet of mass concrete at a slope of 1 on 1 was placed against the downstream face of the toe to reduce the depth of fill and guard against differential settlement. At Marchlyn Mawr this detail was built into the design.

The culvert at Winscar passes through the upstream membrane at the toe wall. It is taller than the rest of the toe wall. The fill behind it was given extra compaction but the fillet of mass concrete was not placed there. The crack described by Chris Routh in his paper was at this location. There is no equivalent culvert at Marchlyn Mawr.

I agree with other speakers that a longitudinal gallery in the toe wall would have been desirable to monitor the performance of the membrane.

Jonathan Hinks (Halcrow Group)
My question is for Mr Mann concerning the Tai Tam Tuk Dam in Hong Kong. The dam is not straight and points upstream. Maybe temperature variations are less in Hong Kong than in Scotland, but nevertheless I wonder whether consideration has been given to the possibility of longitudinal compression in the dam causing horizontal cracking on the downstream face near the change of direction. Something similar happened at the Mullardoch Dam in Scotland, although in that case the apex pointed

downstream and the cracks were in the upstream face.

Robert Mann (Cuthbertson)
During the last formal inspections by Mr McLeish and Mr Gallacher the upstream and downstream faces were thoroughly examined and the dam was remarkably free of cracks. The dam is monolithic, and it seems to have performed very well indeed throughout its life.

Rod Bridle (Independent Civil Engineer)
Much progress is being made in the application of underwater engineering developed in the offshore oil industry for carrying out rehabilitation work on dams while water remains without draining the reservoir. The draw-off pipes and valve replacement enabling contract currently in progress at Ladybower Reservoir, where DSND Oceantech are the diving contractors, is a case in point. In view of the need for repairs at Winscar, would it be possible to carry out these repairs underwater? This is a question directed to Niek Leguit of Hesselberg Hydro. Also could Dr Vaschetti tell us if the CARPI/Oceaneering techniques for fixing new CARPI membranes underwater could also be applied to repair damaged membranes?

Gabriella Vaschetti (Carpi Italia)
The CARPI/Oceaneering techniques can be applied also to repair damaged membranes underwater, avoiding the need to dewater the reservoir. As a matter of fact, the Lost Creek project was the outcome of research, as well as the field experience acquired in repair installations. Research was performed by CARPI and then jointly by CARPI and Oceaneering during a two phase project granted and financed by the US Army Corps of Engineers, which resulted in the design and underwater construction of a geomembrane system suitable for underwater installations. Field experience has been accumulated in repairs accomplished underwater, in Portugal and US.

Niek Leguit (Hesselberg Hydro)
After making a hole in the asphaltic lining to remove the defective material, you have to fill it again and there, on a steep slope, the problem starts. You are filling a hole on a steep slope with a hot material which tends to flow down. After compaction and cooling a small crack will appear at the top joint, between fresh and existing material. Step two is to repair the just executed location again by filling the top joint with mastic, which together with the repair in dense asphaltic concrete will guarantee a waterproof lining.

Under water you can repair with asphaltic materials eg. mastic, but not with asphaltic concrete like that used to repair Winscar. This composition needs compaction to perform. Under water with the quick cooling you do not have the time to do so. Mastic is an overfilled material and needs cooling only.

John Sammons (Independent Consulting Engineer)
Figures 1 and 4 in the paper on raising Llysyfran and Brianne dams each show a step where air can be entrained directly downstream of the raised spillway crest blocks. Being a novice in hydraulics, I wonder why, if beneficial, this feature is not used in original designs? Is it just a way of fitting the new geometry to the old?

Aled Hughes (Binnie, Black & Veatch)
Mr Sammons is right in his observation that the step downstream of the overflow raising at Llysyfran and Brianne dams was adopted as a convenient way to fit new geometry to the old. The alternative of cutting into the existing weir concrete to avoid a featheredge tail to the new concrete would have been costly and difficult. The step facilitates a clean break away with incidental aeration of the flow. At spillways where aeration slots are deliberately introduced for the avoidance of cavitation attack associated with high velocity flow they will normally be placed much further down the glacis.

Ian Gowans (Cuthbertson)
The original crest of Llysyfran dam had the 45º upstream slope which designers at that time incorporated to deal with ice pressures on the crests of spillway (the theory was that expanding ice on a 45º slope expansion slipped up the slope reducing the horizontal pressures at the weir crest). The new shape of the crest profile involves a vertical face. Was the effect of ice and wind blown ice taken into consideration in the new design?

Aled Hughes (Binnie Black & Veatch)
As far as the designers of the raising are aware, the 45º ramp upstream of the original overflow weir crest arose from the geometry of the overflow section monoliths, which have the same downstream and upstream slope as the non-overflow monoliths forming the rest of the dam.

Llysyfran lies in the south west of the country at low altitude, close to the sea, and ice formation has never been observed in the reservoir. It has not been foreseen that ice loadings could ever exceed the hydrostatic loadings resulting from floods at this location.

Alan Johnston (Babtie Group)
The wave surcharge calculated for Llyn Brianne dam is stated in Mr Hughes' paper to be 0.85m. Has that value been checked against the recommendations in the 3rd Edition of Floods and Reservoir Safety to use the total length of the reservoir in calculating the fetch? From Mr Hughes' slides the reservoir appears to be comparatively long and narrow and thus be one where the change in approach to the topic of fetch, wave surcharge and wave run-up might well affect the raised crest.

Aled Hughes (Binnie, Black & Veatch)
The wave surcharge of 0.85m calculated for Llyn Brianne dam was arrived at following the recommendations in the 3rd Edition of Floods and Reservoir Safety. It is calculated for the longest straight line fetch (F_{50}) measured from the right hand end of the dam, the remainder of the dam being more sheltered. The reservoir is not exposed to prevailing winds and the upstream limbs of the reservoir lie in narrow V-shaped valleys distinctly angled to the main limb of the reservoir. Therefore it has not been considered necessary to consider a zig-zag fetch to the furthest limit of this reservoir in arriving at the appropriate estimate of wave surcharge to apply.

John Ackers (Binnie Black & Veatch)
I can say that the position of the dam is rather sheltered. The lake is quite long but if my recollection is correct, it turns sharp left just as it approaches the dam face.

Ian Hay (RKL-Arup)
In relation to the work at Winscar, what are the latest recommendations regarding a single-stage or two-stage application of an asphaltic membrane?

Niek Leguit (Hesselberg Hydro)
State of the art bituminous facing consists of:

- Granular drainage layer stabilised with bituminous emulsion or cutback bitumen
- Asphaltic binder layer of 70mm minimum. This layer is still permeable after compaction.
- Dense asphaltic concrete of 80mm minimum. This layer is impermeable after compaction.
- Sealcoat: a hot applied mastic or nowadays, most of the time, a bituminous emulsion project which is applied cold.

Alan Johnston (Babtie Group)
A table in the paper by Messrs Robertshaw & Wilson indicates that double layer construction has been superseded in the UK by single layer construction. This has been largely on the grounds of cost because there are many examples of satisfactory performance of double layer construction from around the world. An example is Dungonnell Dam in Northern Ireland which I inspected recently. Completed in 1970 the only work required so far has been the re-sealing of the top joint i.e. at the junction of the membrane and the crest.

The problem at Winscar would probably not have occurred if single layer construction had been used because there would not have been a lower impermeable layer to trap the water. However, as far as is known, the lower layer has remained water-tight and it has been a valuable second line of defence.

Supplementary Presentation on Chambon Dam

Gabriella Vaschetti (Carpi Italia)
Chambon dam is well known in the ICOLD community. It is a 136m high gravity dam owned by Electricité de France, whom we thank for their courtesy in providing the drawing. Chambon is subject to alkali-aggregate reaction and holds the swelling record in France. As a remedial measure, EDF has been performing a series of slot cuts to relieve tension. Eight slots, height varying from 30 to 40 metres, have been made. In the case of Chambon, installation of the CARPI geomembrane system had the additional advantage of providing a waterproofing system that could efficiently accommodate the slot cutting and easily restore impermeability.

The geomembrane system was installed over the entire upstream face of the dam in the upper 40 metres. Before each cut was performed, CARPI removed a strip of waterproofing liner from the area of the slot. After the cut had been completed, a strip of the same PVC membrane material was positioned to cover the area where the liner had been removed. The strip was welded over the main PVC liner, thus restoring total impermeability of the upstream face.

The details of the phases of the procedures before and after slot-cutting are as follows:

- After the membrane has been cut to the width necessary to easily perform the slot-cutting, the membrane is anchored along the

perimeter of the removed area. Some sealing material along the anchorage avoids intrusion behind the liner of debris due to slot cutting, which could affect the drainage layer behind the membrane. After the slot has been completed, the PVC strip is welded over the main liner. The final integrity of the liner relies on the reliability of the welds: PVC can be welded easily and welds are totally reliable, provided they are all controlled. Accurate Quality Control is paramount for the efficiency of the system.
The PVC cover strip bridges the entire opening of the slot immediately after it has been installed, and there is no water head on it. Then the slot starts closing. The first slots close quickly, so in principle there should be no gap left when the water head is applied. With the last slots, closing is slower, and the PVC membrane may have to bridge a gap with the water head applied. A very thin stainless steel plate was installed over the area of all slots to support the membrane. No damage has been reported. The performance of the PVC membrane in the event it has to bridge an opening under different water heads is investigated by CARPI in large-scale tests in the laboratory.

Discussion (continued)

Rod Bridle (Independent Civil Engineer)
Would Dr Vaschetti like to comment on the point that sealing concrete dams using membranes, or other means, prevents the entry of water into the concrete and as water is a vital ingredient in the AAR, ASR reaction, the process may be slowed and eventually cease?

Gabriella Vaschetti (Carpi Italia)
This is an important and delicate point, because it is disputed what is the amount of relative humidity that is needed to feed the reaction. 80% is a figure agreed upon by many, including ICOLD and Bureau of Reclamation. Data on how much the membrane has succeeded in slowing the reaction are not yet available. It is known that the drained membrane system does deprive the dam of a potentially harmful element, water. Drainage not only removes water infiltrating from the outside crest or foundations, but also deprives the dam body of saturation water. This is due to the water migration inside the dam body: saturation water tends to migrate towards the warmer surface of the dam, the waterproof membrane stops the migration of water, and the condensed water is drained in the drainage gap between the membrane and the dam surface. The drained waterproofing system therefore is continuously extracting water that is already inside the dam. This is in line with the need

to decrease an element that is feeding the reaction. We know the reaction is fed by the aggregate, but also by the water: theoretically, if the water content in the dam is brought below a certain figure, and the drained membrane system follows the line, the reaction can be stopped.

Robert Mann (Cuthbertson)
In reply to questions by Geoff Sims on the date of construction of Tai Tam Tuk Dam and on measures taken or planned to ensure efficiency of the drains. The dam was completed in 1918. We have observed an apparent reduction in efficiency of the trial drains over time since drilling in 1991. A recommendation has been made for routine monitoring of the piezometers and drainage discharges, to determine whether or not clearance of the drains is necessary to maintain their efficiency. All the drains are straight in bore and reasonably accessible so re-drilling should be a fairly straightforward matter if it is required.

Vic Over (Bolton Institute)
Referring to the paper on Winscar Dam, Figure 2 on page 298, it would be useful to have an indication of the dimensions of the test holes, the size of copper plates and the voltage of the battery (which appears to be a 12V vehicle battery).

An applied D.C. current would cause gassing at and near the electrodes as a result of water disassociating into oxygen and hydrogen. I believe this test method could be flawed. BS 1377: Part 9: 1990 Section 5 states in Note 2 that "The use of direct current can cause polarization effects and lead to an uncharacteristically high resistance being measured". A high resistance around holes 1 and 2 could reduce the voltage measured in hole 3. An erroneous interpretation would then be that a water layer does not exist between the asphalt layers. A side effect may be that the pressure of released gas could cause the top asphaltic layer to lift. Further, I am sure that generating hydrogen gas in a test hole presents a safety issue.

A more appropriate technique, in my opinion, would be the application of an earth resistivity meter passing alternating current through two current electrodes (no gassing) with voltage measurement made via two further electrodes. High voltage sources would enable quite small diameter test holes to be drilled down to the asphalt layer boundary. A further advantage would be that a number of voltage measurements could be spread around the test area.

Written Contribution

Alan Wilson (Babtie Group) and **Andrew Robertshaw** (Yorkshire Water)
In reply to Mr Over, the tests were generally undertaken in holes within about 10m radius which were previously cut to repair faulty areas of the top dense asphalt layer. These holes, typically had an area of $0.25m^2$ and a depth of 0.1m. The copper plate electrodes were approximately $0.01m^2$ and the circuit was connected to a 6V battery.

It is appreciated that theoretically oxygen and hydrogen will be produced at the electrodes when undertaking electrical testing using the circuit described in the paper. In practice, it is found that during the short period that the circuit is energised, the volume of gas produced is not significant, appearing as fine bubbles which are retained on the electrodes and does not present a safety hazard in the well ventilated locations where the tests were conducted.

The equipment used in the electrical continuity test has the merit of being simple and constructed from readily available components. A generally similar system has been used successfully by the Contractor for almost 20 years to help locate leaks in lagoons lined with polymer or bitumen based membranes. The more sophisticated technique suggested by Mr Over using alternating current and an earth resistivity meter is of interest and may be considered for future applications.

Mr Over's comments concerning polarisation effects leading to high electrical resistance are noted. The procedure used in this instance proved to be highly successful in indicating the general extent of debonding. However, it is accepted that the method is not infallible.

VISIT TO DINORWIC PUMPED STORAGE SCHEME

Visitors to Marchlyn Mawr were welcomed at the Electric Mountain Visitors Centre, Llanberis, by Owen Williams, Company Civil Engineer, First Hydro and Jim Graham, Geotechnical Engineer of Mott MacDonald. They gave introductory presentations on the main features of the Dinorwic pumped storage scheme, with particular reference to the operation and monitoring of the Marchlyn Mawr, Surge Pond and Peris reservoirs. Dinorwic's rapid response to surges in demand was graphically illustrated; the station can be operating at full power within a matter of seconds. Detailed analysis of TV and sporting schedules ensures that surges in demand are anticipated and catered for.

Chris Binnie, who was Project Engineer for the upper dam at Marchlyn, set out the background behind the development of the scheme in the following presentation.

Need for Dinorwic Scheme and Design of Marchlyn

Conventional pumped storage schemes pump during times of low demand and cheap electricity and generate at times of high demand and hence high revenue. In the 1960's many people bought television sets. At the end of a programme many millions of people would switch on a kettle to brew tea and this increased the electrical demand very rapidly. This caused serious problems for the CEGB as almost all the generation was nuclear and coal and it could take 20 minutes to run these up to full power, and they could not cope with a power surge occurring in a couple of minutes. In addition the Board was installing sets of 660 MW and was concerned about the effects if one or two of them tripped out.

The way to cope with this was to have hydro sets spinning in air synchronised to the grid. By opening the guide vanes power could be brought on in a few seconds. Originally the target for full power response had been 10 seconds but it had only been possible to achieve about double this. However this was still very fast and sufficient to minimise system breakdown.

The CEGB had built Ffestiniog in the early 1960's. This had proved the system but at about 350 MW it had been insufficient. Three alternative sites were investigated, and, in 1972, it had been decided to proceed with Dinorwic. This was to be 1320 MW power out (2 x 660 MW sets) and 1800 MW in, the capacity of the existing North Wales line which the Board considered it would not get permission to increase. This resulted in a requirement for 6 Mm^3 of storage at the upper reservoir at Marchlyn. The need for submergence of the intake and low pressure tunnel meant a minimum water level of 600m OD, resulting in a top water level of 633m OD, subsequently raised to 634m OD.

The upper reservoir was a high level glacial cwm with a 30m deep lake retained by a terminal moraine with very large boulders up to 7m across. Rock for the dam could easily be obtained from an abandoned quarry nearby along with an abandoned waste tip. Initially it had been expected that because there was a 30m deep lake there would be sufficient fine soil on the site to form a "clay" core dam. In the event no material of sufficient impermeability could be found on site, the nearest appropriate site being 8 miles away. Because of this, and the concern about the effect of daily variations in reservoir level of 30m in 6 hours on a conventional upstream shoulder, it was decided to go for a deck type dam. This could be concrete or asphaltic. The asphaltic deck could be more susceptible to the high rainfall at Marchlyn but a concrete deck would require much more material to be hauled up to the site. The deck was to be laid over the insitu moraine over its lower half and there was doubt as to the compressibility of the moraine. In the event it was decided to build an asphaltic deck at a slope of 1:2.

During the drilling investigation the right abutment was found to be tight and a grout cap was deemed sufficient. The central section was thought groutable but a grout gallery was provided in case of problems so future grouting could be done without taking the scheme out of commission. The left abutment was a deeply weathered spur. Removing and replacing this would have been very expensive. In the event it was decided to form a grout curtain but to expect appreciable leakage. Because of the possible instability of the steep hillside downstream it was decided to have a drainage gallery in the spur to ensure water pressures in it were low.

The dam is in the Snowdonia National Park. Every effort was made to reduce the environmental impact. The dam was curved in plan; the edges were filletted; there were no straight lines of the dam itself. As much as possible, the existing moorland vegetation was re-established. The downstream slope was given an uneven shape and a straight stone boundary wall was continued up over the dam. The result was that from a distance few delegates could actually identify the dam.

GEOFFREY BINNIE LECTURE 1998
LIVES OF EMBANKMENT DAMS: CONSTRUCTION TO OLD AGE

by Dr. Andrew Charles (Building Research Establishment)

SUMMARY

This account of British embankment dams has been presented in the form of an analogy with human life, no period of which, from birth to death, is exempt from troubles. However, at the different stages of our existence we are vulnerable to different maladies. We may suffer from diseases associated with our genetic inheritance or with our life style or we may suffer some unforeseeable accident. The seriousness of an illness can be very dependent on the stage of our life at which it strikes, the rapidity with which it progresses, the accuracy and speed of its diagnosis and the availability of medical treatment. Our evaluation of our own state of health may be faulty and our response to symptoms may be characterised by excessive anxiety or, at the other extreme, a fatal complacency.

There are close parallels in the life of an embankment dam. The dam can be afflicted by problems during construction of the embankment, during first filling of the reservoir or during the long term operation of the reservoir, a period stretching from first filling into old age. As with human life, there may be radical changes in circumstances during the life of the dam which require major adaptations. Problems may be closely associated with the design and construction of the dam, with the mode of operation of the reservoir or with some extreme event not anticipated in the design. Advancing years will be accompanied by deterioration, whether such deterioration is a threat to reservoir safety depends not only on the nature of the deterioration but also on the rate and predictability with which the process of decay develops. The seriousness of these problems will depend on whether or not the reservoir is full when the problem occurs, how quickly it is detected, the nature of the malady, the accuracy with which it is diagnosed, and the rapidity with which emergency and remedial action is taken.

This lecture is concerned with some of the geotechnical aspects of the behaviour of embankment dams and, in particular, the hazards encountered at different stages of their life. There has had to be a high degree of selectivity in the topics which are considered. Three phenomena are examined; the behaviour of wet clay cores during embankment construction, collapse compression on inundation during first filling of the reservoir and the long term threat posed by internal erosion which is now likely to be the major hazard for old embankment dams. Detection of this hidden process is not easy.

A requirement for a major programme of works to upgrade old embankment dams to modern standards needs to be assessed in the light of the risks posed by internal erosion and slope instability. There has been major expenditure on overflow works to meet improved flood standards, but there does not appear to have been any comparable effort to upgrade old embankment dams. The need for ongoing research should be considered. The major threat to old embankment dams comes from internal erosion and further work on this phenomenon should be undertaken.

1. INTRODUCTION

On 12th March 1872, a paper describing some recently built embankment dams was presented at the Institution of Civil Engineers. Thomas Hawksley was in the chair. In the discussion, Mr Bateman remarked that he was "somewhat surprised at the sections of the embankments". Another eminent Victorian dam engineer, Sir Robert Rawlinson, who had been the Government Inspector at the Dale Dyke failure in 1864 and who was to be President of the Institution of Civil Engineers in 1894-95, commented that "he could not tell whether the diagrams of reservoir embankments exhibited were for instruction or for warning. If for instruction, he thought they were calculated to teach a very bad lesson: if for warning he would point out a few of their malarrangements". Even in those robust Victorian times, the author of the paper under discussion cannot have failed to detect that this observation fell well short of an unqualified endorsement of his engineering skills! This year is the centenary of the death of Rawlinson and it is appropriate that we make use of his pungent comment to identify those two vital elements of technical communication; instruction and warning.

Before looking at some incidents in the lives of embankment dams which have been selected for instruction and for warning, there are some introductory issues which merit consideration.

1.1 Vicissitudes of life in this world

No period of human life, from birth to death, is exempt from troubles. However, at the different stages of our existence we are vulnerable to different maladies. We may suffer from diseases associated with our genetic inheritance or with our life style, or we may suffer some unforeseeable accident. The seriousness of an illness can be very dependent on the stage of our life at which it strikes, the rapidity with which it progresses, the accuracy and speed of its diagnosis and the availability of medical treatment. Our evaluation of our own state of health may be faulty and our response to symptoms may be characterised by excessive anxiety or, at the other extreme, a fatal complacency.

There are close parallels in the life of an embankment dam. The dam can be afflicted by problems during construction of the embankment, during first filling of the reservoir or during the long term operation of the reservoir, a period stretching from first filling into old age. As with human life, there may be radical changes in circumstances during the life of the dam which require major adaptations. Problems may be closely associated with the design and construction of the dam, with the mode of operation of the reservoir or with some extreme event not anticipated in the design.

Advancing years will be accompanied by deterioration, whether such deterioration is a threat to safety depends not only on the nature of the deterioration but also on the rate and predictability with which the process of decay develops. The seriousness of these problems will depend on whether or not the reservoir is full when the problem occurs, how quickly it is detected, the nature of the malady, the accuracy with which it is diagnosed, and the rapidity with which emergency and remedial action is taken.

1.2 Function of an embankment dam

A meaningful examination and assessment of a particular human life needs to be made against the background of the objectives of that life and the extent to which those objectives were achieved. Some individuals may have varied, peculiar or, indeed, dangerous objectives. But, it is also pertinent to evaluate a particular human life in the light of the principal objective of all human life, or, as the Shorter Catechism phrases it, "the chief end of man". The catechism tells us that "Man's chief end is to glorify God and to enjoy him for ever" (Assembly of Divines at Westminster, 1643). In an analogous manner, reservoirs may have varying degrees of success in serving particular purposes such as water supply, hydroelectric power generation, or environmental improvement, but the basic purpose of the dam is a much simpler matter. What is the chief end of an embankment dam? The function of an embankment dam is to safely retain a reservoir of water; clearly a more mundane purpose than man's chief end and there is no "for ever" in this definition.

To safely retain a reservoir of water, a dam must satisfy both structural and hydraulic criteria.

- The embankment must be able to safely transmit to the foundation both forces due to the selfweight of the embankment fill and also hydrostatic and seepage forces associated with the reservoir water. The slopes of the embankment must be stable.

- The embankment and its foundation must contain effective barriers or seals of sufficiently low permeability to ensure that losses of water due to seepage and leakage are kept within acceptable limits. It must also contain materials with suitable filter and drainage properties to ensure that pore pressures are not transmitted downstream and that seepage and leakage do not result in unacceptable loss of material from the embankment and its foundation.

1.3 Significance of the subject

It could be questioned why the life of an embankment dam should be of interest. A personal response might be that the subject is technically interesting. Furthermore, it may be the job of many of the delegates at this 10th Conference of the British Dam Society to be interested. At the wider level of society, it needs to be emphasised that embankment dams represent a valuable and, indeed, a vital element of the infrastructure. Reservoirs can enhance the environment. In some cases, failure could endanger public safety on a massive scale and have a very negative effect on the environment. Lafitte (1996) calculated that a major failure in Switzerland could cost CHF 20×10^9.

1.4 Scope and organisation of the subject

This lecture is concerned with some of the geotechnical aspects of the lives of embankment dams. It does not deal with floods, the hydraulics of spillway flow or the structural performance of ancillary structures. It relates primarily, although not exclusively, to British dams and it is largely based on research carried out at the Building Research Establishment (BRE). Even with these limitations, the subject is a large one and there has had to be a high degree of selectivity in the topics which are considered.

The stages in the life of an embankment dam can be listed as follows:

- design
- construction
- first filling of the reservoir
- normal operation of the reservoir
  - minor fluctuations in reservoir level
  - major operational drawdown
- abnormal operation of the reservoir
  - serious incidents
  - emergency drawdown
  - remedial works
- demolition

1.5 Dams and the Building Research Establishment

Work on dams has taken place over many years at the Building Research Establishment. The failure of the Chingford reservoir embankment at the end of July 1937 while under construction was investigated by what was then known as the Building Research Station (Cooling and Golder, 1942). During the 31 years the author has worked at BRE, research on dams has been principally concerned with the safety of embankment dams, although at an earlier date, during the Second World War, there was a connection between BRE and the destruction of concrete and masonry dams! Figure 1 shows a recent view of the 1/50 scale model of the Mohne dam built in the grounds of BRE in 1940/41. The Journal of the Institution of Water Engineers (1955) recorded that allotment holders at Garston, near Watford, were annoyed early in 1941 when a mysterious and sudden onrush of water swept down a nearby hill and inundated their plots.

Figure 1. Model of Mohne dam at Building Research Establishment.

2. EMBANKMENT CONSTRUCTION

Embankment construction might be considered to be the first stage in the life of an embankment dam, but much will have happened before construction commences on the site. In a world which appears to be increasingly full of environmental objectors, the planning and design stage may be a lengthy one and, indeed, the project may be aborted.

2.1 Fill placement

Embankment construction involves the placement and compaction of unsaturated fill materials. The initial degree of saturation of the earthfill will depend on the moisture content of the fill as placed and the amount of compaction. Modern procedures for the specification and control of the placement and compaction of fill were introduced by Proctor (1933). He recognised that it was possible "to compact a soil so firm and hard as to appear entirely suitable for a dam and for this same soil to become very soft and unstable when percolating water saturates it". Thus Proctor was principally concerned with collapse compression on wetting or softening as he described it.

Some method was needed for determining and limiting the softening of an earthfill dam which could occur when the soils contained in it became completely saturated with water. A basic issue relates to the amount of energy which is needed to reduce the air voids in the fill to a minimum. Proctor (1948) demonstrated that this was related to the undrained shear strength of the soil and that the subsequent compression behaviour was also related to strength after compaction. With a particular compactive effort, a cohesive fill can be brought to various degrees of compactness depending on the moisture content of the fill. For a given compactive effort there is a maximum dry density which is achieved at the optimum moisture content. Compacted dry of optimum the fill may have large air voids, whereas compacted significantly wet of optimum, a minimum air voids of the order of 3% can be achieved.

The relationship between dry density and moisture content obtained by Proctor (1948) for a clay fill using different compactive efforts is shown in, Figure 2. Fill compacted significantly wet of optimum, using the standard compactive effort of about 600 kJ/m^3, has a minimum air voids of about 3%. The relationship between the penetration resistance measured with the Proctor needle and the moisture content of the fully compacted fill is also shown in Figure 2. The undrained shear strength would be of the order of $^1/_{10}$ of the penetration resistance. The compaction energy required to compact a clay soil is approximately linearly related to the undrained shear strength. The following expression gives an indication of the required energy, but can only be very approximate without precise definitions of full compaction and the method of measuring c_u:

$$E/V = 3c_u$$

where E is the energy required to compact a volume V of clay soil with undrained shear strength c_u. Thus, the compactive effort required to reduce the air voids to less than 5% in a puddle clay, which could have been placed with c_u as low as 10 kPa, would only be about $^1/_{10}$ of that required to fully compact a rolled clay where c_u was of the order of 100 kPa.

Two moisture content limits are required for cohesive fills as a moisture content that is too high can lead to placement difficulties and large construction pore pressures and a moisture content that is too low can lead to excessive settlement of the fill when the fill becomes saturated. Reducing the air voids of a clay fill to 5% or less should ensure that a very low permeability is achieved and that the fill is not susceptible to collapse compression. The drier the fill the greater the compactive effort which will be required to meet this air voids criterion.

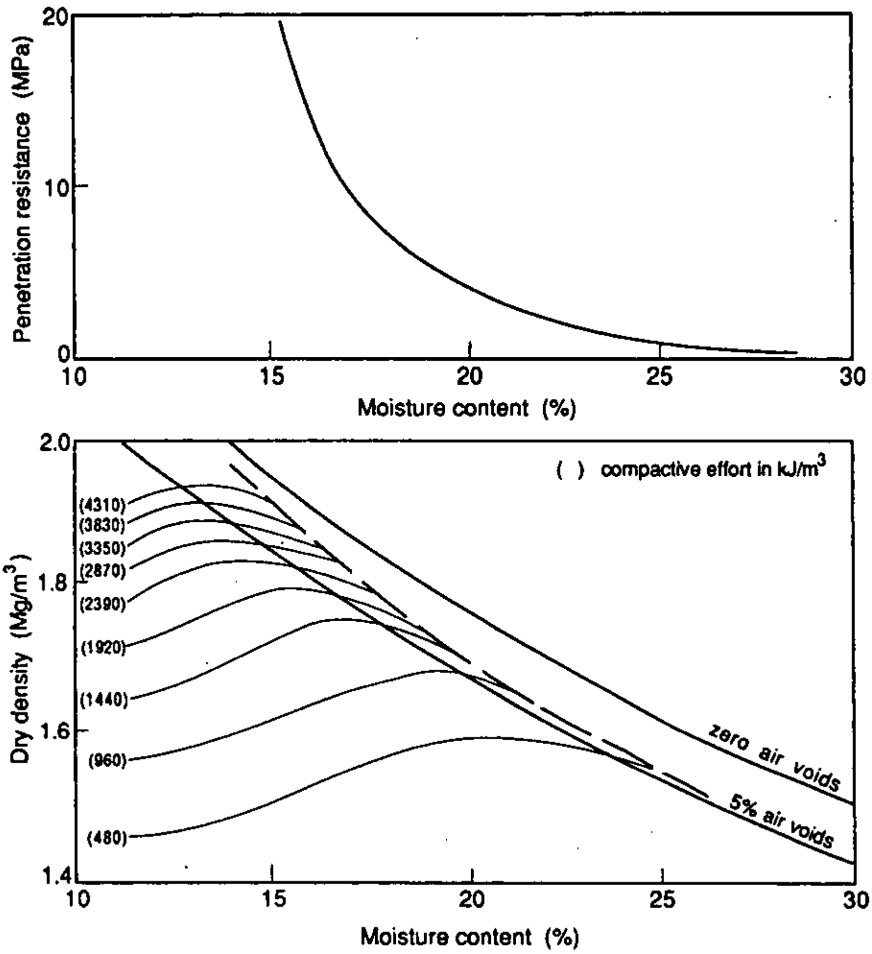


Figure 2. Compaction of clay fill (after Proctor, 1948).

2.2 Slope stability

Figure 3. Failure of Carsington dam.

Assuming that reservoir impounding does not commence until the embankment has been completed, problems during construction relate to the stability of the embankment. The use of large modern machinery and also financial considerations have encouraged short construction periods and high construction pore pressures may have insufficient time to dissipate during embankment construction. In a central clay core this can be an advantage, but it is undesirable in the shoulder fill. For many embankments the factor of safety is at, or close to, a minimum at the end of construction and therefore many slope failures have occurred during embankment construction with the reservoir empty. From the perspective of public safety, it is clearly advantageous that the factor of safety aganst slope instability should be a minimum during embankment construction where instability cannot lead to an uncontrolled release of reservoir water.

The fact that many embankments which have high construction pore pressures may be at their least stable during construction could be considered a positive factor as it provides a type of proof loading, with the assurance that, if there is instability, it will occur before impounding commences. However, although failure of an embankment during construction and before reservoir impounding has commenced may not pose any threat to public safety, the client may incur substantial costs in rebuilding the dam and losses due to the delay in commissioning the project. The accepted tender sum for the reconstruction of Carsington dam, which had suffered a major upstream slip in June 1984 when the embankment was almost at full height, was £17.8 million (Banyard et al, 1992). Figure 3 shows the slope failure at Carsington.

2.3 Behaviour of a wet clay core

Movements may be large where embankments are built on soft clay foundations or where embankments have a soft, compressible clay core. It may be questioned whether these movements are a cause for concern.

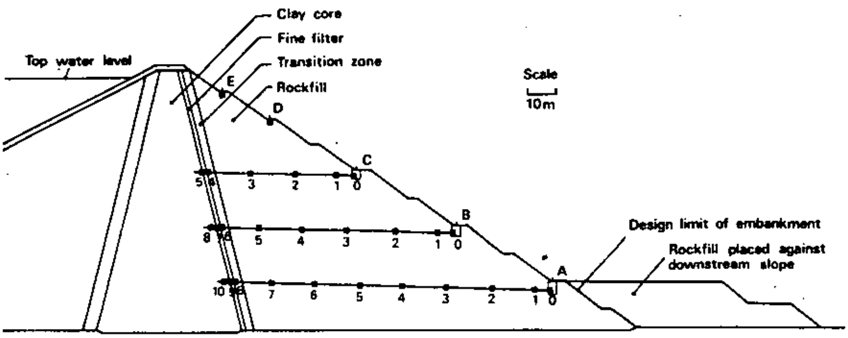


Figure 4. Cross-section of Llyn Brianne dam showing location of BRE instrumentation.

BRE made extensive measurements of the deformations that occurred during the construction of Llyn Brianne dam in the period between 1969 and 1971 (Penman and Charles, 1973a). The location of instrumentation is shown in Figure 4. The central clay core was placed wet of Proctor optimum moisture content at a relatively low undrained shear strength. The rockfill shoulders were heavily compacted and had a high angle of shearing resistance. The effective stresses in the core were very low during embankment construction and pressure from the wet clay core produced a major lateral thrust on the rockfill shoulders. The downstream rockfill was subjected to a greater lateral thrust during construction than it would subsequently suffer from the reservoir water and the effect of reservoir impounding on the downstream rockfill was therefore small. No additional downstream movement occurred at the clay core/ downstream rockfill interface during impounding. Figure 5 shows that large lateral movements occurred during embankment construction. During the final stages of embankment construction lateral movements as large as 20mm per metre height of fill placed were measured. Clearly these large constructional movements were not a sign of incipient failure as the factor of safety against slope instability was quite satisfactory.

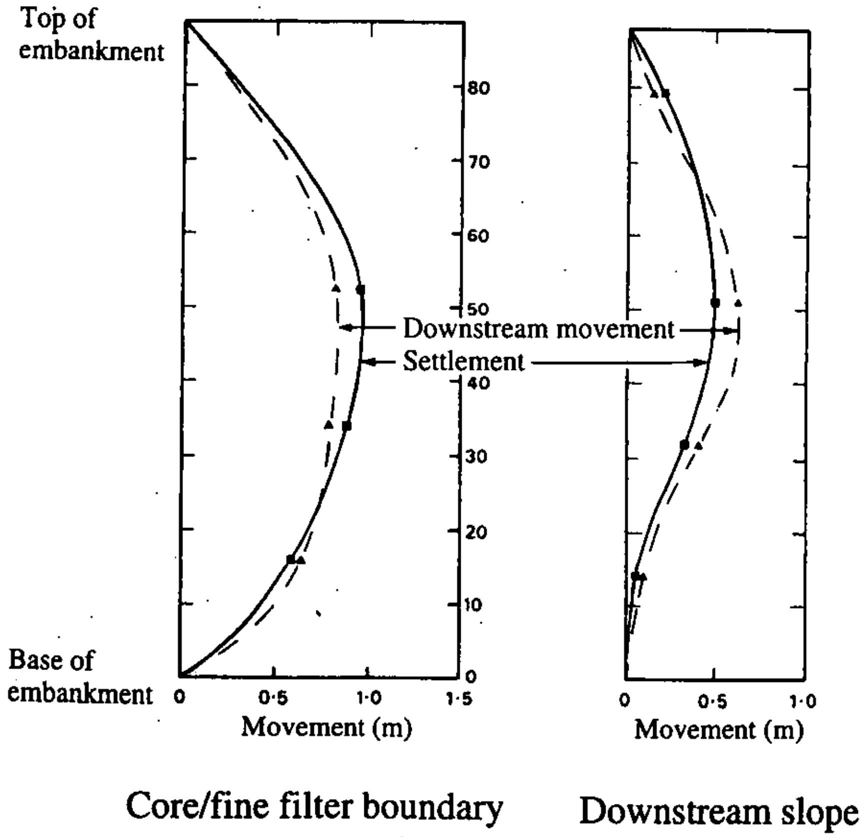


Figure 5. Construction movements at Llyn Brianne dam (after Penman and Charles, 1973a).

The movements were not a problem, merely a sign of the healthy behaviour of a wet plastic clay core.

Marulanda (1997) has described a vertical movement of 7m during construction of the 248m high Guavia dam in Colombia. About 90% of the instrumentation passing through the core was broken and it is believed that the tubing was severed by the large deformations.

There is good reason to place a clay core at a high moisture content, to ensure that it is soft and flexible (Penman and Charles, 1973b). High construction pore pressures are likely and these should ensure that hydraulic fracture does not occur when the reservoir is filled. High pore pressures will reduce the factor of safety against slope instability during embankment construction and help to ensure that this is the critical period for stability. In these circumstances large deformations do not indicate unsatisfactory behaviour. Although it could be claimed that it was necessary to use puddle clay in cores because of the limited compactive effort available in earlier years, technically it was a good approach from a number of points of view.

3. FIRST FILLING OF RESERVOIR

First filling of the reservoir is the critical stage in the life of a dam during which about 50% of failures of embankment dams occur (ICOLD, 1995). Close surveillance and monitoring are essential. The Reservoirs Act 1975 recognises the importance of this period and strictly regulates impounding.

3.1 Impounding process

Reservoir impounding has usually commenced after the completion of embankment construction at British dams. As first filling is so critical, ideally the rate of rise of the reservoir level should be closely controlled. However, this is not possible except for fully bunded reservoirs where all the water is pumped in. Where a river is impounded, the degree of control will be dependent on the outlet capacity in relation to the inflow.

A slow and gradual filling of the reservoir is generally considered to be desirable. Hoeg et al (1993) observed that experience has shown that the rate of rise of the reservoir water level may affect the development of cracks in the core. On the basis of detailed analyses, Dounias et al (1996) concluded that rapid undrained final impounding is much more likely to induce hydraulic fracture. The reservoir level at Teton dam had been rising at a rate of 0.9m per day and was close to top water level when failure occurred (Arthur, 1976).

Some problems such as slope stability may have been at their most critical during embankment construction. Other hazards such as internal erosion are only tested as the reservoir is filled. Unsatisfactory performance associated with the rising reservoir level may be related to several facets of the geotechnical behaviour of the embankment:

- inadequate shear strength, due to, for example, increased pore pressures, can cause slope instability,
- internal erosion may be associated with hydraulic fracture, piping or suffosion,
- inadequate compaction may lead to collapse compression of submerged fill.

3.2 Collapse compression on inundation

Poorly compacted dry fills almost invariably undergo a reduction in volume when their moisture content is increased. The phenomenon, which can occur without any change in applied total stress, has been termed "collapse settlement" because it was considered to be associated with a collapse of the soil structure.

Collapse compression of earthfill and rockfill materials has been observed in many embankment dams. It can have an adverse effect on the performance of embankment dams and effects such as longitudinal cracking, localised settlement and erosion have been attributed to it. There can be confusion between the effects of collapse compression and the effects of internal erosion; collapse around instrumentation has been mistaken for internal erosion.

Collapse settlement is not necessarily harmful and it has been argued that in certain circumstances collapse can be beneficial. Significant collapse compression occurred during reservoir impounding at Beliche dam in Portugal. It was claimed that this was a good thing as it would increase the stresses in the clay core (Naylor et al, 1997).

Although collapse compression is not confined to first filling of the reservoir, it often occurs at this stage and it is appropriate to examine the phenomenon at this point. Two situations need to be considered; a general collapse compression of the embankment fill and collapse compression within a limited zone of fill.

(a) General collapse compression of embankment fill

During first filling of the reservoir, the upstream shoulder fill of a central core dam will be submerged. The shoulder fill of old puddle clay core dams was not heavily compacted in comparison with modern practice and the fill would have had significant collapse potential. The current profile of the upstream slope usually confirms that such movements have taken place.

Settlement was measured at the restored opencast mining site at Horsley in Northumberland as the water table rose some 34m through the fill as shown in Figure 6 (Charles et al, 1993). The 70m deep mudstone and sandstone backfill had not been systematically compacted and the settlement caused by the rising water table at Horsley may give an indication of the settlement which would have occurred on first filling of the reservoir in the upstream shoulder of old puddle clay core dams built in the Pennines with similar embankment fill.

If the upstream fill has been placed dry or has not been well compacted, collapse compression can produce longitudinal cracks along the contact between the upstream shoulder and the core. At the 155m high Canales rockfill dam in Spain a deep step developed due to collapse compression of the upstream shoulder during first filling of the reservoir (Giron Caro, 1997). The final stage of impounding from December 1995 to January 1997 is shown in Figure 7 and it is seen that the depth of the step increased from 0.6m to 1.0m as the reservoir level rose 28m.

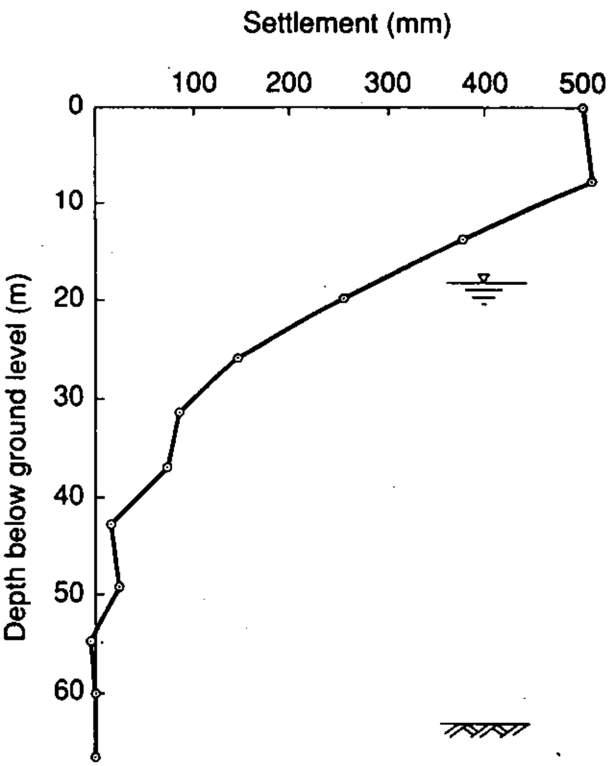


Figure 6. Settlement of opencast mining backfill due to rising ground water level (after Charles et al, 1993).

Heavy compaction of fill placed at an appropriate moisture content can eliminate collapse potential. Llyn Brianne dam has shoulders of heavily compacted rockfill and the deformations in the upstream and downstream shoulders monitored during first filling of the reservoir indicated that collapse potential had almost been eliminated.

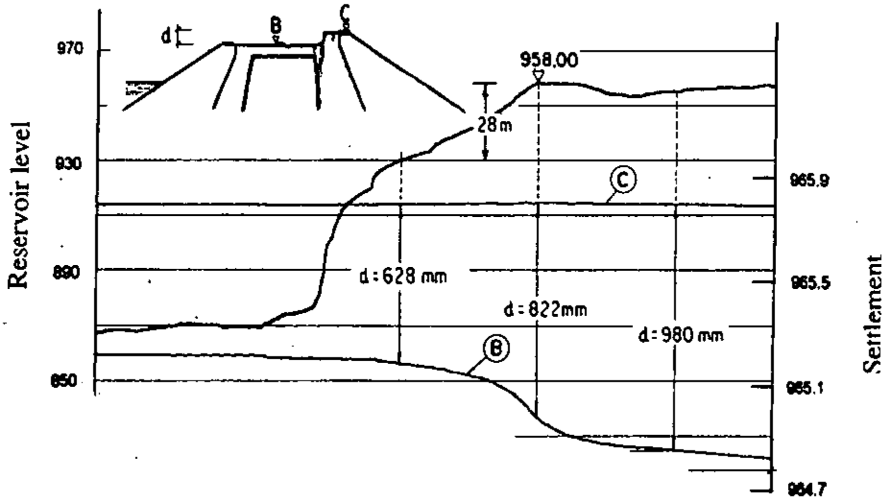


Figure 7. Collapse compression at Canales dam during the first filling (after Giron Caro, 1997).

(b) Localised collapse compression of zones of fill

Localised collapse can be triggered by the wetting of limited zones of fill. Wherever collapse occurs in a limited zone of soil, there will be a redistribution of stress. A test carried out in the 4m deep test pit at BRE has demonstrated this reduction in stress on wetting. Zones of dry fill (1m x 1m x 1m) with significant collapse potential were built into a colliery spoil fill. The fill surrounding the zones was sufficiently wet to have little or no collapse potential. Figure 8 shows the major reduction in vertical stress in a zone of dry fill when the fill was submerged (Skinner et al, 1997).

This type of localised phenomenon can occur in embankment dams due to wetting of the downstream fill by reservoir water seeping through or around a central core, by downward percolation of surface water from the crest or downstream slope or by leakage through an upstream membrane. The collapsing zone of soil will be effectively unloaded and the stress on the surrounding soil will increase. The soil in the collapsed zone will then be vulnerable to internal erosion. Where clay cores have relative dry zones with collapse potential, such stress transfers will occur on saturation. This can be linked with the hypothesis of Bravo-Guillen and Perez-Romero (1997) that preferential seepage paths rather than hydraulic fracture can often form the initiating mechanism for internal erosion.

Peterson and Iverson (1953) attributed the failures of some low earth dams in western Canada on first reservoir filling to this type of mechanism. The glacial fills had been placed at very low moisture contents with poor

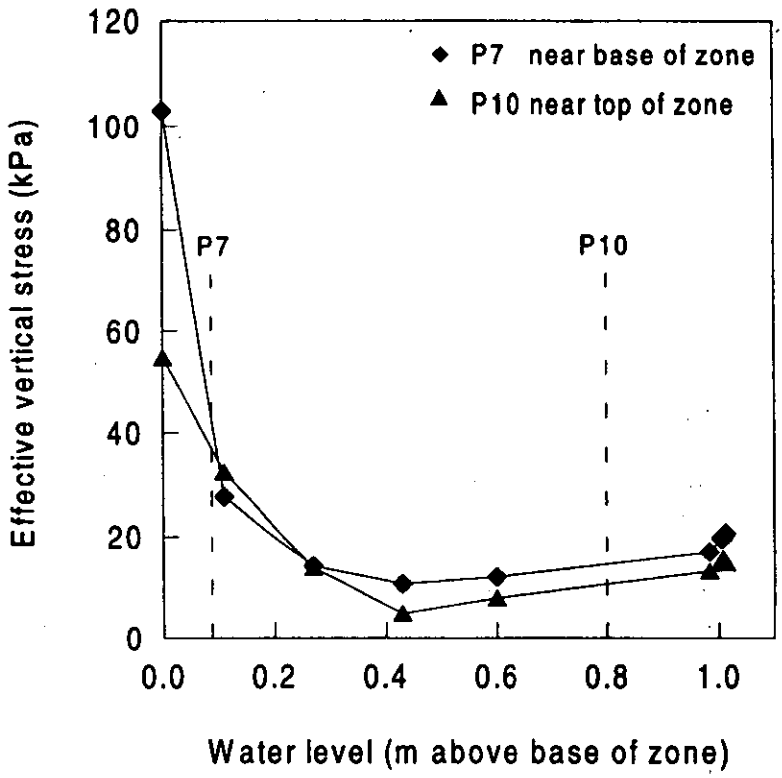


Figure 8. Reduction in vertical stress due to localised collapse compression (after Skinner et al, 1997).

compaction and it was believed that when the reservoir water wetted the fill and caused collapse, layers of better compacted material arched over the collapsing fill. In this way preferential seepage paths were created and erosion ensued.

3.3 Failure of Dale Dyke dam

In human experience, the early death of an individual just approaching maturity is particularly poignant. The failure of a dam during first filling of the reservoir can be disastrous and the collapse of Dale Dyke dam on 11th March 1864 with first filling of the reservoir vitually complete was a subject that Geoffrey Binnie examined in a paper to the Quarterly Journal of Engineering Geology (Binnie, 1978) and then subsequently re-examined in his book (Binnie, 1981). It seems appropriate in this Geoffrey Binnie lecture to briefly revisit this most catastrophic of British dam failures.

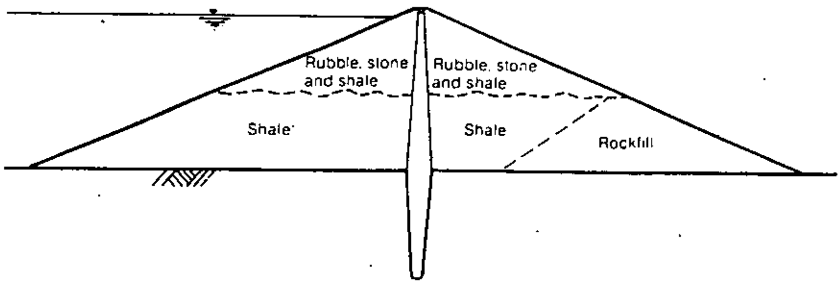


Figure 9. Cross-section of Dale Dyke dam (after Binnie, 1978).

The 29m high Dale Dyke dam had an excessively thin core, only 1.2m wide at the top and 4.8m wide at original ground level, and the shoulder fill was particularly poorly compacted, being placed in excessively thick layers probably 2m deep. Figure 9 shows a cross-section of the dam as deduced by Binnie (1978) from evidence given at the inquest. An explanation of the cause of failure must account for two significant features; the rapidity with which the embankment failed and a horizontal longitudinal crack which was observed shortly before failure on the downstream slope just below the crest.

In the discussion of the paper by Dounias et al (1996) which analysed the failure and cracking of old British dams, Naylor (1998) raised the issue of collapse settlement on first filling of the reservoir at Dale Dyke dam. The reply by the authors was somewhat dismissive. However, in the discussion on the paper on the Dale Dyke failure by Binnie (1978), both Mr Carlyle and Professor Bishop identified collapse compression as a likely major contributor to the failure.

The deep layers of loose fill in the shoulders of the embankment would have had a large collapse potential and rainfall during construction would only have affected the surface of each layer. During first filling of the reservoir substantial collapse compression would have occurred in the upstream fill. However, immediately downstream of the core, the fill would have held up the core and adjacent upstream fill. At the time of failure there had been a period of heavy rainfall and on 10th March there was a gale warning. There was no wave wall and the storm spray and rainfall would begin to inundate the downstream slope. Collapse compression would have begun in the shoulder fill immediately downstream of the core and settlement of the crest would have ensued. The reduction in volume of the wetted fill could have caused the longitudinal crack.

In the reply to the discussion on his paper, Binnie rejected the views of Carlyle and Bishop on the grounds that the collapse compression mechanism could not explain the rapid settlement of the embankment. If collapse compression was solely due to spray and rainfall acting on the surface of the downstream fill, it is likely that this was correct. However, if hydraulic fracture of the core had already resulted in zones of downstream fill at relatively low levels adjacent to the core becoming inundated, then it is much more feasible that the sudden settlement of the crest of the embankment was due to collapse compression as the zones of inundated fill became larger and arching of the fill above them broke down.

4. RESERVOIR OPERATION

The working life of the dam/reservoir system must be the principal concern, particularly in countries where few new dams are likely to be built. Failure of the dam in operation with the reservoir full can have serious economic and operational consequences. However, where failure would cause loss of life, public safety should be the primary issue.

In our society many seek to make their offspring as independent as possible at a very early age. While the lack of wisdom in this should be self evident, it is true that at some point our children inevitably become independent of parental control. In contrast, the embankment dam will always require care and attention.

4.1 Behaviour during normal operation

The normal operation of the reservoir will depend on its function. For example a landscape reservoir built in the time of Capability Brown may always be full; water supply reservoirs will usually undergo relatively small fluctuations in level with occasional much larger changes in water level during periods of drought; major changes in water level may occur on a daily basis at a pumped storage reservoir.

With the reservoir full, the embankment dam may appear to be just sitting there in equilibrium with its environment. In reality it is ageing in a world in which the Second Law of Thermodynamics expresses the reality that in any closed system there must always be a decrease of order and organisation; unless there is an external intervention, entropy increases. It must be expected that this universal law of deterioration applies even to structures such as embankment dams which appear to be inert, durable and in harmony with their environment. Rawlinson (1859) commented "The Hindoos had a saying that 'an arch never sleeps'; the same might be said, with equal truth, of water".

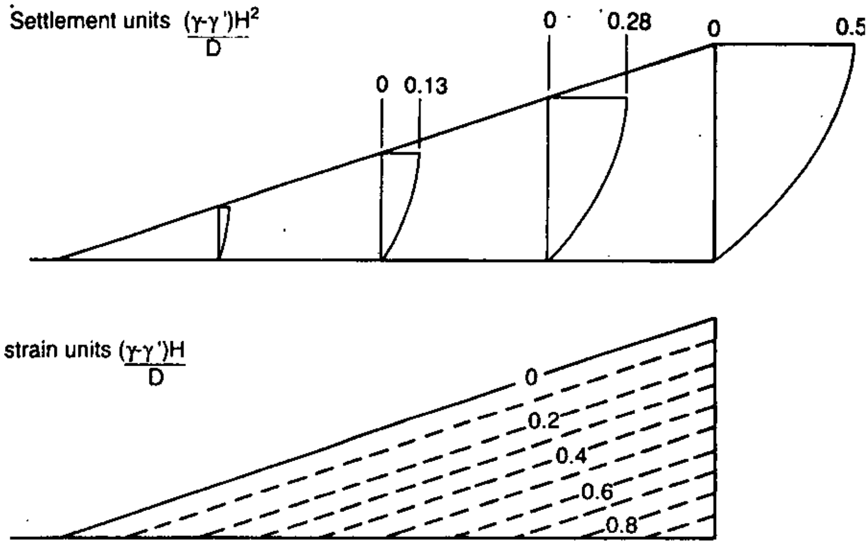


Figure 10a. Response to reservoir drawdown: central watertight element (after Tedd et al, 1994).

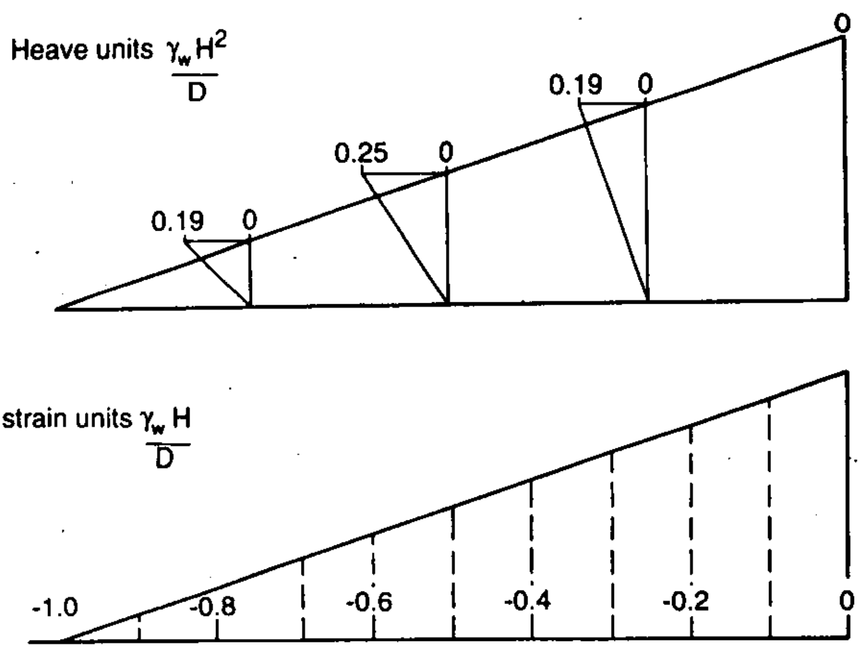


Figure 10b Response to reservoir drawdown: watertight element on upstream slope (after Tedd et al, 1994).

It is important to distinguish between normal and abnormal behaviour of the dam during normal reservoir operation. For example, what deformations are normal during fluctuations in reservoir level? To answer such an enquiry, it is necessary to understand the mechanisms of behaviour of the embankment. The pattern and magnitude of deformations during normal operation depend on the position of the watertight element and, in particular, whether it is located on the upstream face of the embankment or at a central location within the embankment. Figure 10 shows the different response to reservoir drawdown in the two cases.

Most of the failures which have caused loss of life can be attributed to two basic causes; overtopping during floods and internal erosion. The former hazard is largely the province of hydrology and the estimation of the design flood. Overflows designed to modern flood standards should prevent this type of failure. The latter hazard may be associated with piping or hydraulic fracture and, in new dams, should be prevented by appropriately designed filters and careful design of the watertight element. Relatively few catastrophic failures have been due to slope instability associated with inadequate shear strength or high pore pressures; internal erosion must be considered the major remaining threat to old embankment dams which do not have filters designed to modern standards.

4.2 Internal erosion processes

Internal erosion involves the removal of solid material, usually in suspension, from within an embankment or its foundation by the flow of water. Loss of water due to seepage and leakage will rarely be of economic significance, but the flow of water through the embankment and its foundation may cause internal erosion which could threaten the security of the dam. Where surveys of failures and serious incidents have been carried out, invariably internal erosion is found to be one of the main threats to the safety of embankment dams. In a recent statistical analysis, internal erosion was reckoned to be the primary cause of some 27% of failures and a secondary cause of 18% of failures (ICOLD, 1995). The importance of the subject has been recognised with the formation of a European Working Group operating under the auspices of the European Club of ICOLD (Charles, 1998).

Two types of internal erosion have been distinguished; a general erosion sometimes termed "suffosion" and localised erosion eg piping which often is manifested by sinkholes. Fry et al (1997) considered that piping is the more dangerous and the more rapid. Internal erosion is essentially a hidden, weak link, phenomenon in which the internal fabric of the embankment is destroyed. Until some feature such as a sinkhole appears at the surface of the soil, it is difficult to identify and investigate. The various mechanisms of internal erosion may be associated with construction defects and weaknesses or with deformations and stress conditions within the embankment. It can be dependent on relatively minor details of design and construction. Although certain types of fill may be peculiarly vulnerable to internal erosion, it should not be thought that the phenomenon is restricted to particular soils. For example, since 1970 in France some 70 cases of internal erosion have been noted in various types of fill (Fry et al, 1997). The provision of adequate filter protection is the best defence against internal erosion, but many old dams do not possess such protection. Cohesionless soils such as fine sands and silts are particularly vulnerable to internal erosion. Although cohesive soils are expected to be more resistant to erosion, this expectation requires qualification. Some clays are dispersive and highly erodible. Although non-dispersive clays may be resistant to erosion, once a fracture is formed it is likely to remain open due to cohesion and concentrated leakage will occur. While some fills appear to be self-healing, in other materials internal erosion can lead to a rapid failure of the dam.

Hydraulic fracture of the core by the reservoir water pressure may initiate internal erosion. Lofquist (1992) suggested that two conditions were necessary for hydraulic fracture to occur; the first relates to the stress conditions, the second to the presence of an initiating zone, such as an existing crack or more permeable layer, which is in hydraulic connection with the reservoir. Hydraulic fracture may occur where a thin clay core is supported by stiffer granular shoulders and internal stress transfer is caused by differential settlement. Significant stress transfer can also occur along the axis of the dam in steep sided valleys. Measurements by Charles and Watts (1987) showed that the stress conditions found in narrow clay filled cutoff trenches rendered some old British dams particularly vulnerable to hydraulic fracture. The ratio $\sigma_{ha}/\gamma_w h_w$ can be used as an indicator of susceptibility to hydraulic fracture, where σ_{ha} is the horizontal total stress acting in the direction along the axis of the dam and $\gamma_w h_w$ is the full reservoir pressure at the depth of measurement. Where $\sigma_{ha}/\gamma_w h_w > 1$, hydraulic fracture is unlikely. Figure 11 shows $\sigma_{ha}/\gamma_w h_w$ plotted against z/B where z is the depth below the crest at which the stress was measured and 2B is the width of the core at that depth.

Quite frequently unexpectedly high pore pressures have developed within a core close to its downstream face and many explanations of this phenomenon have been advanced (Charles, 1997). It seems probable that in many cases the situation does not represent a problem.

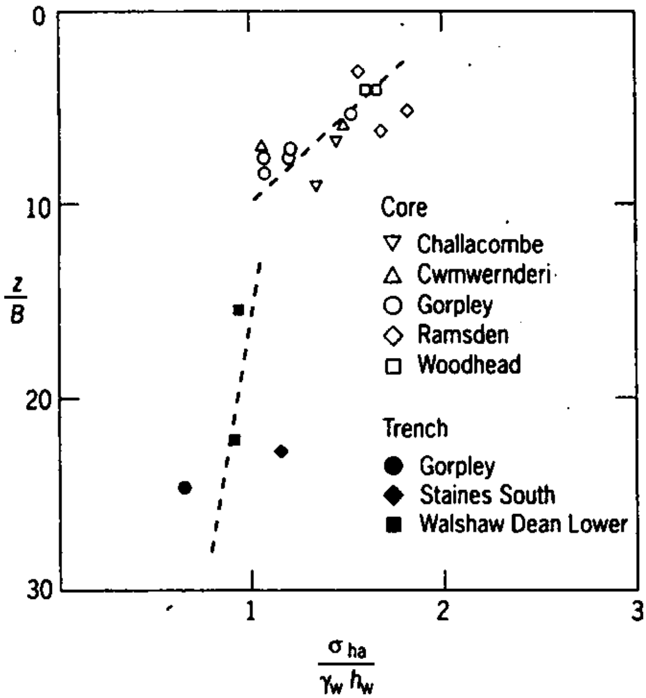


Figure 11. Stresses measured in puddle clay cores and cut-off trenches (after Tedd and Charles, 1991).

4.3 Internal erosion case histories

From the standpoint of dam safety two key questions concern, firstly, whether or not internal erosion will lead to a hazardous situation which threatens the security of the dam, and, secondly, if it does, the speed with which the hazard will develop. Could an old embankment dam, with a long record of apparently satisfactory behaviour, fail suddenly and catastropically due to internal erosion? It is reassuring that many internal erosion problems have occurred during first filling of the reservoir and, where problems have occurred at a later stage, their development has generally been sufficiently slow that routine surveillance and monitoring have detected the situation in time for remedial action to be taken. The Warmwithens failure presented an exception (Wickham, 1992). If it is concluded that there is some risk of a rapid internal erosion failure, albeit slight, it is important to identify features in an embankment dam which are critical in rendering it vulnerable to such a development and to determine what warning signs could be expected and how much warning these would give prior to failure.

Leakage and associated internal erosion have developed in very different ways at different dams. Figure 12 shows the development of the leakage flow at the 10m high Warmwithens clay fill dam in the period immediately prior to failure on 24th November 1970 as deduced from the data presented by Wickham (1992). The flow increased in a few hours from a few l/s to over 1000 l/s.

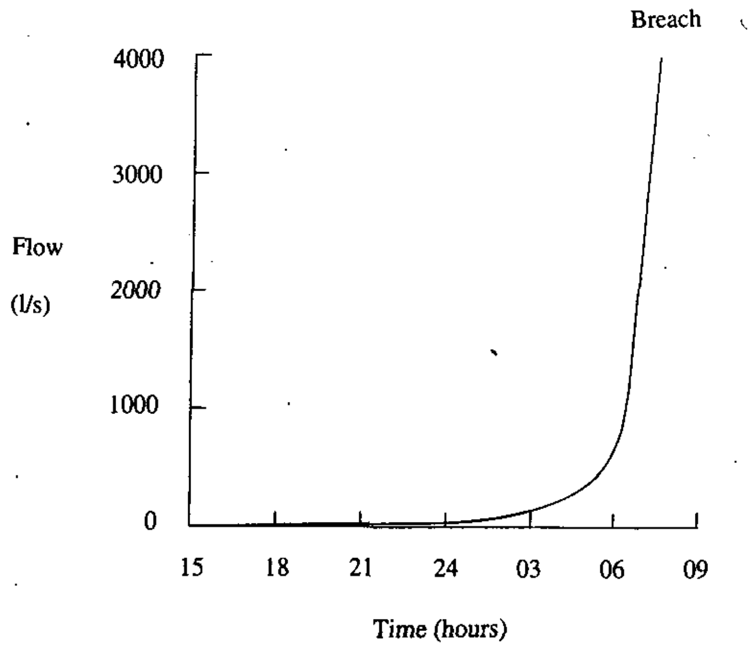


Figure 12. Flow rate at Warmwithens dam as failure developed.

In contrast to the rapid failure at Warmwithens, the Songa dam in Norway demonstrated self-healing when, on 11th August 1994, the leakage suddenly increased. The leakage increased from the normal 1.25 l/s to 107 l/s in 20 minutes, as shown in Figure 13, and then returned to the normal flow within six hours (Torblaa and Rikartsen, 1997). The 42m high rockfill dam has a central moraine core.

The rapid lowering of the reservoir level by almost 1m per day which was carried out at the Peruca dam in Croatia following the explosions on 28th January 1993 was almost certainly a major factor in saving the dam from collapse. Leakage flow and reservoir level are plotted on Figure 14. The maximum flow was 570 l/s (Rupcic, 1997). The lowering of the reservoir level by 5m by Captain Mark Gray of the Royal Marines prior to the attack on the dam was also an important factor. The 63m high rockfill dam has a narrow vertical clay core.

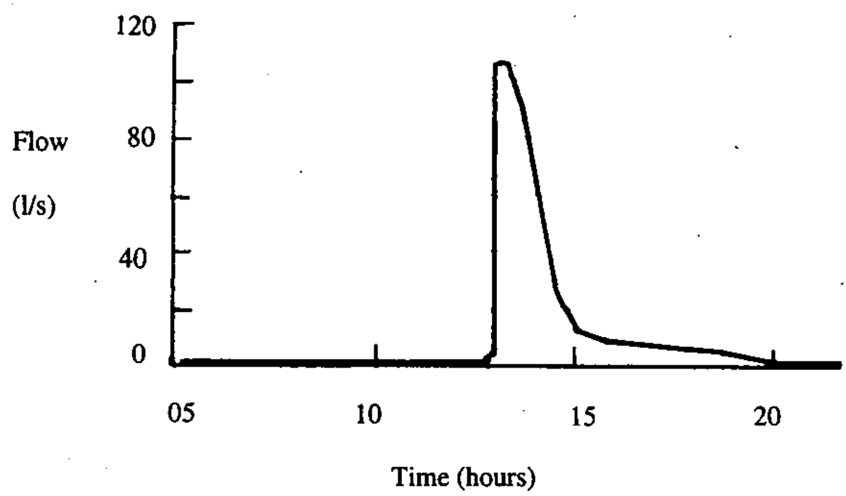


Figure 13. Sudden change in leakage rate at Songa dam, Norway (after Torblaa and Rikartsen, 1997).

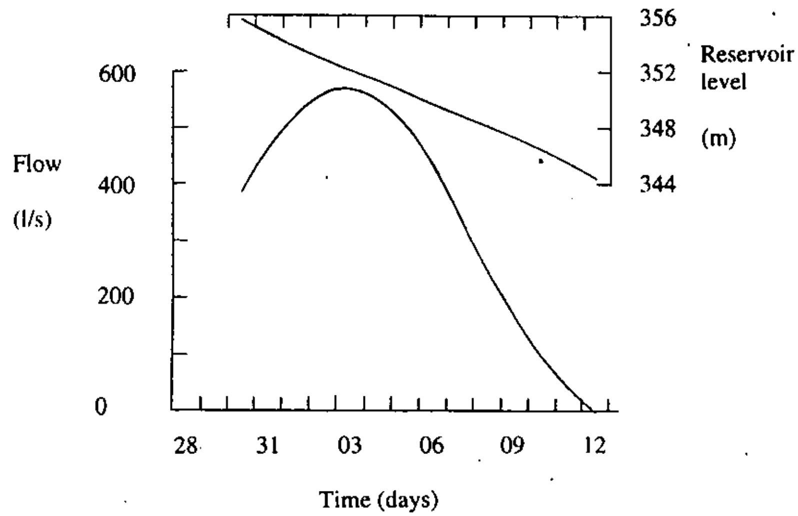


Figure 14. Flow rate following explosions at Peruca dam.

The significance of the rate at which the internal erosion incident develops is further illustrated by the experiences at the 93m high Teton dam and the 39m high Fontenelle dam in the USA. Both dams had cores of erodible silt and granular shoulders. Internal erosion occurred at Fontenelle a few months after first filling of the reservoir (Bellport, 1967) and at Teton with the reservoir almost full for the first time (Arthur, 1976). At Fontenelle leakage increased to 600 l/s, some 8000m^3 of material was lost and an area of the dam crest about 6m in diameter collapsed with a drop of 10m. Rockfill was dumped into the hole, emergency measures were taken to lower the reservoir and a breach was averted. In contrast, Teton failed with a catastrophic release of the reservoir water. Although the symptoms of distress were similar at Fontenelle and Teton, and similar actions were taken, the outcome was vastly different due, it would appear, to the different speeds at which erosion occurred and to the presence at Fontenelle of low level outlets which made it possible to lower the reservoir level by 1.2m per day. There are implications for frequency of dam inspection and emergency reservoir drawdown rates.

4.4 Significance of interfaces

While internal erosion can occur within the body of the fill, frequently problems have occurred at the interface between fill and concrete structures (Charles, 1997). Factors which are of particular importance include the control of the placement of fill and the stress conditions in the fill. There can also be weaknesses associated with interfaces between successive layers of fill, but appropriate fill placement procedures should prevent this.

An example where stress conditions are favourable is provided by Queen's Valley dam on Jersey. The dam was completed in 1991. The central vertical bituminous core is founded on the roof of the inspection gallery, as seen in Figure 15, and, as bituminous cores are very thin, the interface of core and concrete is clearly a critical location. However, providing that there is not excessive hang-up of the core on the stiff rockfill shoulders, there will always be vertical compressive stress at the interface. Therefore, there should be no possibility of hydraulic separation along the interface due to the pressure of the reservoir water.

Any structure which passes through an embankment dam from upstream to downstream presents a potential hazard to dam safety and internal erosion failures have often been associated with the presence of this type of structure. Dolcetta (1997) explained that Italian regulations prohibit such structures in embankment dams. The current modification of Audenshaw reservoir in Manchester has unavoidably involved the construction of two culverts passing through the new clay fill embankment. The stress conditions in the fill at the side of a culvert are much less favourable than for the fill on the roof of the culvert. Great care has been taken in ensuring appropriate fill placement against the culvert walls as shown in Figure 16.

Figure 15. Base of bituminous core on roof of inspection gallery at Queen's Valley dam, Jersey.

Figure 16.
Trial compaction of clay fill against concrete structure in modification of Audenshaw reservoir.

5. RISK AND SAFETY

The work of actuaries in life assurance offices is concerned with attempts to calculate probabilities of life expectancy. It has been said that the profession attracts those who have found that practising as an accountant is too exciting! It is easy to mock but many people do insure their lives and doubtless the work of the actuary is necessary. This leads to the subject of risk and safety as applied to embankment dams.

5.1 Risk analysis and safety management

It is now being recognised that there is a need to develop a form of quantitative risk analysis which is suitable for the analysis of reservoir safety. Two systems must be examined to evaluate public safety; the dam-reservoir system and the downstream valley system. In the past there was reluctance in the United Kingdom to examine the downstream valley system as it implied that a dam could fail. This line of thought has a close analogy with the belief that it is unwise to have lifeboats on a cruise liner as it might alarm the passengers.

The use of risk analysis means that it is necessary to address the issue of acceptable risk (Charles et al, 1998) and, where the risk is assessed as unacceptable, it will be necessary to mitigate the risk. Appropriate safety management is a fundamental component of a risk mitigation strategy which will have managerial, legal and technical aspects. It is expected that quality management of reservoir safety will be much more explicit in future.

5.2 Observational method

The uncertainties with old embankment dams are such that the analysis of behaviour can easily lead to diagnosis of problems and consequent remedial solutions which are either overconservative or dangerous. The antidote is for the safety evaluation of an embankment dam under normal operating conditions to be linked to field observations. However, observations as such do not enhance, still less ensure, the safety of a dam. The observations must be carried out within an appropriate risk management framework. A form of observational method which is appropriate for the safety evaluation and risk management of old embankment dams has been described by Charles (1993) and Charles et al (1996).

In reservoir inspection and supervision, signs of change in the condition or the behaviour of the embankment should be detected by visual surveillance or by instrumentation readings. The engineer responsible for reservoir safety then must attempt to relate the observations to a likely mechanism of behaviour and determine whether or not there is an incipient problem. In the light of the diagnosis of the cause of the problem and of previous experience of similar situations, the need for further investigation or other actions will be determined (Charles et al, 1996). The responsibilities of personnel should be clearly defined and appropriate procedures must be in place to ensure that there is a sufficiently rapid response to developments. There should be contingency plans for remedial measures and emergency reservoir drawdown.

5.3 Safety standards

With an ageing population of dams the relationship between safety standards of existing dams and safety standards for new dams has to be addressed. As there is no justification for accepting a higher level of risk to public safety for old dams, the safety evaluation flood and earthquake criteria should be identical. However, it can be argued that a slightly lower calculated factor of safety against slope instability is acceptable for an old dam as this safety factor includes a factor of ignorance and the existing dam has successfully survived the critical periods of construction and reservoir filling.

6. CONCLUSIONS

In conclusion it is appropriate to quote Geoffrey Binnie who, in presenting the 12th Dickinson Memorial Lecture on "The Evolution of British Dams" to the Newcomen Society at the Science Museum on 12th May 1976, affirmed that "British dams, although modest in size, deserve a place in the history of civil engineering". It might be added that there is much for instruction and for warning in the subject.

This account of British embankment dams has been presented in the form of an analogy with human life and it has been pointed out that, as with mankind, the problems that affect earthfill dams differ in the successive stages of life. The potential consequences of problems occurring during construction and problems occurring during reservoir operation are very different; during embankment construction, failure may be limited to economic loss, whereas once a substantial volume of water has been impounded, public safety may be threatened.

A soft, flexible core will effectively prestress the embankment shoulders during construction and reduce deformations in the core and downstream shoulder during reservoir impounding. The vulnerability to hydraulic fracture during first filling of the reservoir will be minimised. The embankment design should make allowance for the adverse effect of such pore pressures on slope stability.

First filling of the reservoir is a critical time for dam safety and close surveillance and monitoring is essential. Problems during first filling may be associated with slope instability or with internal erosion. The dam may be most vulnerable to the former hazard with a relatively low reservoir level whereas it will be most vulnerable to the latter hazard with the reservoir at top water level. A relatively slow, controlled rate of filling should reduce the susceptibility to hydraulic fracture.

Collapse compression on inundation is likely to occur when a fill that was not well compacted or was placed relatively dry is inundated for the first time. However, collapse compression may occur in situations where it does not cause a problem.

When consulted by a patient, the skilled physician will want to see the records of the patient's medical history, to carry out a careful examination noting symptoms and, possibly, to undertake tests in order to assess whether there is a problem. When there is a problem, an accurate diagnosis is required if a cure is to be found. The diagnosis will depend heavily on the knowledge and experience of the physician. It can be dangerous to treat the symptom rather than the disease; for a patient suffering from recurring severe headaches merely to take aspirin could prove a fatal mistake. It is vital that reservoir safety continues to be in the hands of experienced and knowledgeable civil engineers. Internal erosion is now likely to be the major hazard for old embankment dams as problems such as overtopping in extreme floods have been largely dealt with. Detection of this hidden process is not easy.

With many types of earthfill dam, slope instability is most likely to occur during embankment construction. Thus, in some cases, if the dam does not become unstable during construction, subsequent instability may be unlikely; in effect, construction is a form of proof loading. During normal reservoir operation, movements will occur which may not be an indication of a problem. However, as these old dams have not been designed to modern standards, the embankment may have a factor of safety close to unity and any small perturbation could result in instability. Where failure would pose a major risk to life and property, it is surprising if the slope stability of a large embankment dam has not been analysed; it is more than 60 years since Fellenius (1936) described the calculation of the stability of earth dams.

A major problem facing Western society is an ageing population; the ratio of the retired to the working is steadily increasing. How can this state of affairs be sustained? There is also an ageing infrastructure and, in particular, an ageing population of embankment dams. Reservoir safety in the United Kingdom is largely concerned with the performance of old embankment dams many of which are very old. While there is a good safety record, the consequences of failure could be enormous. An observational approach is required for the safety evaluation of embankment dams during normal reservoir operation. Surveillance and monitoring need to be integrated into an appropriate scheme of safety management. Risk analysis provides a suitable framework and the techniques of probabilistic risk assessment may help in decision making, but the crucial importance of correct diagnosis remains. Quality management concepts such as long term planning and prevention being preferable to detection are highly relevant.

A requirement for a major programme of works to upgrade old embankment dams to modern standards needs to be assessed in the light of the risks posed by internal erosion and slope instability. There has been major expenditure on overflow works to meet improved flood standards, but there does not appear to have been any comparable effort to upgrade old embankments. Are the embankments less significant for safety than the overflow arrangements?

Finally, the need for ongoing research should be considered. This was recognised in the report of Dr Coats (1993) on the DETR programme of reservoir safety research in which it is recommended that "... the funding of reservoir safety research be increased." This has not taken place. However, there are other bodies such as EPSRC and CIRIA which fund some dam safety research and some work is sponsored by dam owners, although this is usually site specific and may not be published. The major threat to old embankment dams comes from internal erosion and further work on this phenomenon should be undertaken.

ACKNOWLEDGEMENTS

The work on reservoir safety described in this lecture has been carried out with the collaboration of many past and present BRE colleagues, particularly Dr Arthur Penman until his retirement from BRE in 1982 and, over the last 15 years, Dr Paul Tedd. From 1983 onwards, dam safety research at BRE has been undertaken as part of the DETR Reservoir Safety Research Programme; this programme has been under the direction of, successively, John Phillips, Colin Wright and, currently, Richard Vincent, all of whom have provided encouragement and support. As would be expected in an invited lecture of this type, the opinions expressed are those of the author and not necessarily those of DETR or BRE.

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VOTE OF THANKS
by Rodney Bridle

Andrew, Chairman, Ladies and Gentlemen, it is my privilege and a pleasure to propose a vote of thanks to Dr. Andrew Charles for presenting the 1998 Geoffrey Binnie lecture to us. As we might have expected, knowing Andrew, the lecture has been thorough, accurate and interesting, and presented with occasional touches of Charles-style understated humour. The scope of the lecture has been magnificent and we must all thank Andrew for the devoted care he has taken in preparing it.

Being asked to present the Geffrey Binnie lecture, the most prestigious in the British Dam Society's calendar, not long after having prepared the General Report on Question 73, "Special Problems Associated with Earthfill Dams" for the International Commission on Large Dams Congress in Florence last year, at the same time as coping with being newly privatised at BRE and with all his many research lines, clearly was a daunting task. However, I for one and I suspect many others, knew from past experience that Andrew would take up the challenge, almost with relish, and deliver the goods in his usual concise and excellent fashion.

Looking back, I realise that Andrew has always been the man for the big job. When I was at Imperial College in 1969 and 1970, in the hallowed halls of the Soils Mechanics Research Laboratory, all kinds of experiments were in progress. Some involved 38mm triaxial cells, others 100mm triaxial cells, but dominating the scene from one corner was a tall steel drum about 1m in diameter. I noticed that it had to be fed from time to time from a wheelbarrow of what looked like 40mm crushed stone. The drum contained a large diameter triaxial cell and the stone was the model rockfill that Andrew was testing as part of his PhD research which ultimately led to the famous curved failure envelope for rockfills, published by Andrew and Marcio Soares in 1984.

Now his Binnie lecture has brought together the soil mechanics and geotechnical engineering aspects of the various phases of the life of our embankment dams. Some are already known to us, though it is nice to hear new presentations in a modern geotechnical context on compactive effort, slope stability and wet core behaviour.

Others are newly presented here. The insights into collapse mechanisms and how these may have occurred, or perhaps remain to occur locally, on some of our old embankment dams, placed with very little compaction, have been most illuminating. The new insight into the Dale Dyke failure is timely and gives us food for thought in the context of our new responsibilities in relation to the stability of dams, stimulated by the new seismic stability requirements.

Internal erosion remains the most unpredictable of the hazards faced by our dams. Andrew's report on the work in progress to better understand it, and how to take steps to reduce the risk from it, will provide some comfort to us all but we'd all like to know more, and support Andrew's plea for more research.

Andrew's further wise words, adding to those in the Conference paper and other papers in "Dams & Resevoirs" and elsewhere, on risk and safety and the application of the observational method in this context, are most welcome.

I believe Andrew's Binnie lecture will become a vitally important reference for all engineers responsible for our embankment dams. I think it will mark the beginning of a new phase in how we carry out surveillance and monitoring of embankment dams and lead us into the better application of modern geotechnical principles and investigations to maintain the safety of our dams.