British Dam Society

Dams 2000

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Preface

The 11th conference of the British Dam Society, *Dams 2000*, was held at the University of Bath in 2000. The proceedings of the conference constitute a valuable collection of 38 papers covering a wide variety of topics related to reservoir safety and the rehabilitation of existing dams, largely in the UK. Dam break analyses and design floods are discussed including reference to some surprising results found by using the recently published *Flood Estimation Handbook*. Experience with hydraulic structures includes siphons, gate performance and valve repairs. Risk and reservoir safety feature in a number of papers including a summary of the recent guide to the *Reservoirs Act 1975* and work undertaken in the CIRIA research project on risk management.

A number of papers examine the environmental and economic benefits and disbenefits of developing new dams and reservoirs in the UK and overseas. Planning of new reservoirs and refurbishment of existing ones requires greater attention to be paid to analysis and mitigation of environmental and social impacts than was customary in the past. The World Commission on Dams issued guidelines in 2000 on dam development that set new standards in these areas.

Many of the papers are cases histories describing investigations and rehabilitation of existing dams to extend asset life. They include geophysics and temperature measurements to detect leakage paths in dams, remedial works such as grouting, slurry walls and membranes.

The Geoffrey Binnie Lecture *Taken for granted* was given by Alan Johnston of the Babtie Group and is published in the Society's journal *Dams & Reservoirs*.

Acknowledgement

The organising committee are grateful to members of the British Dam Society for their time spent in reviewing papers submitted to the conference.

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Lessons learned from overseas experience

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Design floods for UK reservoirs – a personal view of current issues

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SYNPOSIS. The paper discusses aspects of the design flood estimation methods that are routinely applied to reservoired catchments in the UK, often as part of statutory flood safety appraisals. The publication of the Flood Estimation Handbook provides an opportune moment to review issues surrounding the application of the rainfall-runoff method to reservoir flood estimation. The paper provides a personal view of the important issues facing hydrologists and reservoir engineers in estimating floods for reservoirs. The paper also contains suggestions on how reservoir design flood estimation in the UK might be improved.

INTRODUCTION

The unit hydrograph rainfall-runoff and losses model was first described in the Flood Studies Report (FSR) (NERC, 1975), and updated in Flood Studies Supplementary Report No.16 (Boorman, 1985). It has been the basis for the estimation of design floods to determine the adequacy of the freeboard and spillway arrangements for reservoirs in the UK for the last 25 years.

The application of FSR rainfall–runoff model to derive the design flood events, required to assess whether the existing freeboard and outflow arrangements of a particular dam meets the flood and wave surcharge standards recommended in the ICE guides on Floods and Reservoir Safety (ICE, 1978, 1989 and 1996), has in many cases significantly increased the magnitude of design flood. It is a moot point as to whether the recommended safety standards clearly set out in Table 1 of the relevant ICE guides or the improvement in design flood methodology following the publication of the Flood Studies Report, has been the more responsible for the numerous improvement works undertaken.

The Flood Estimation Handbook (FEH) published by the Institute of Hydrology consists of the five volumes as follows:

- Volume 1 - Overview;
- Volume 2 - Rainfall frequency estimation;
- Volume 3 - Statistical procedures for flood frequency estimation;
- Volume 4 - Restatement and application of the FSR rainfall-runoff method;
- Volume 5 - Catchment descriptors.

Dams 2000, Thomas Telford, London, 2000.

These volumes contain the results of a 5 year program of research supported by the Ministry of Agriculture, Fisheries and Food, the Environment Agency, the Scottish Office and the Northern Ireland Office.

Most reservoir engineers will no doubt be pleased to learn that the FEH does not present a new method of reservoir flood estimation or indeed recommend one. Volume 4 of the FEH does, however, provide a comprehensive technical rewrite of the FSR unit hydrograph rainfall-runoff method incorporating the numerous enhancements contained in eighteen Flood Studies Supplementary Reports and other relevant research published in the Institute of Hydrology (IH) Report series, various technical journals and conference proceedings. Volume 4 currently provides the most up to date guidance on the use of the FSR unit hydrograph method for reservoir flood estimation.

Reservoir engineers and other interested parties should not be lulled into a feeling of complacency that all is well with flood estimation and that no further improvements are required. We are unsure how widely it is recognised that the FSR unit hydrograph rainfall-runoff model often provides only coarse flood estimates, and particularly so when key model parameters are derived from catchment characteristics rather than local data. For example Boorman et al (1990) showed that with estimates of unit hydrograph time-to-peak (Tp) and catchment standard percentage runoff (SPR) based on catchment characteristics, the mean annual floods of a sample of 74 gauged catchments were overestimated by 22% on average, and the 25 year floods by 41%. That the method provides realistic or acceptable reservoir design flood estimates is, as Reed & Field (1992) observed, "an act of faith".

Outlined below is a brief review of elements of the FSR rainfall-runoff model approach plus our personal views of some steps that could be taken to help provide improved design flood estimates for UK reservoirs.

THE FSR RAINFALL-RUNOFF METHOD - GENERAL

Figure 1, copied from Volume 4 of the FEH, illustrates the main steps in the FSR unit hydrograph rainfall-runoff method. In summary a design storm rainfall profile is converted to a flood hydrograph using a deterministic unit hydrograph and losses model. The main steps in the approach are:

- To construct a total rainfall hyetograph for the design event;
- To assess the proportion of rainfall which contributes directly to the flow in the river (constant percentage runoff);
- To determine the catchment response to effective rainfall (NB the unit hydrograph shape is dictated by the time-to-peak); and
- To calculate the quantity of flow in the river prior to the event (baseflow).

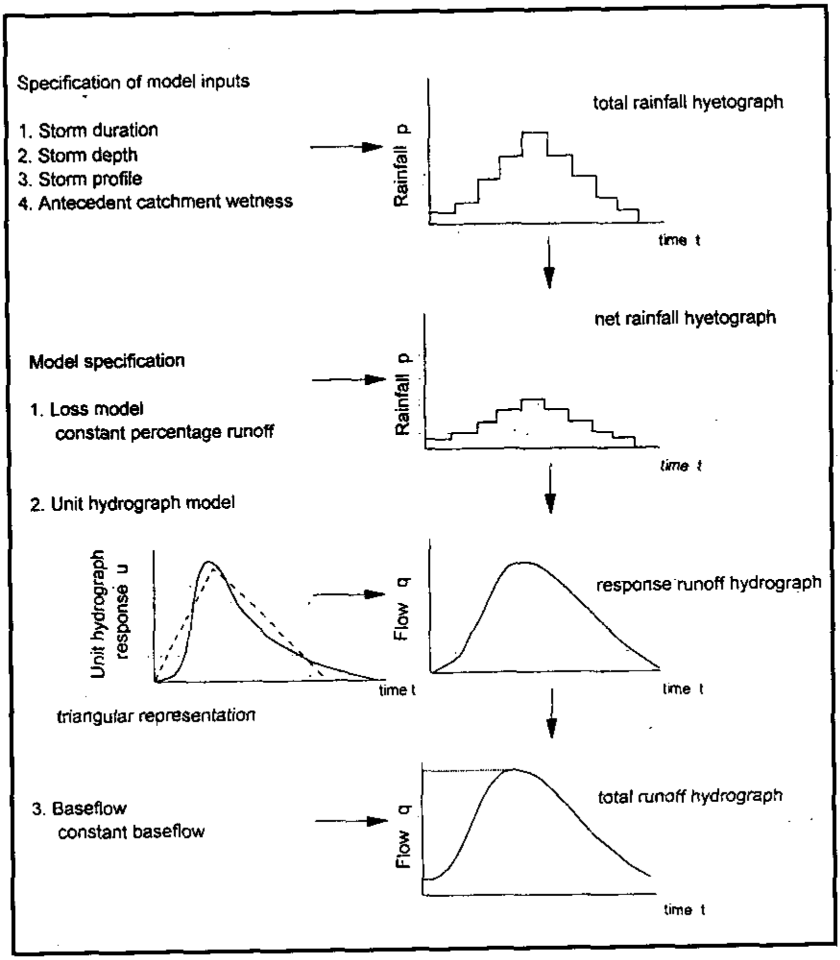

Fig. 1. Flood estimation using the FSR rainfall-runoff method (copied from Volume 4 of Flood Estimation Handbook)

Whilst the basic unit hydrograph rainfall-runoff methodology is not greatly altered from earlier publications, some of the model parameter estimation equations have been updated in the FEH. Tables B.1 to B.3 of Volume 4 of the FEH summarise the various changes to the model parameter estimation equations.

CATCHMENT CHARACTERISTICS

The FEH makes use of new catchment descriptors based on digital data sets rather than manually derived descriptors from maps. The catchment descriptors for the areas draining to any given reservoir will henceforth be digitally-derived from a CD-ROM which accompanies the FEH. This CD-ROM, which makes use of the Institute of Hydrology digital terrain model, can provide the relevant catchment descriptors for any point in the United Kingdom which drains an area greater than 0.5 km^2.

Use of the CD-ROM should help to minimise errors in defining catchment characteristics, which were all to common with the manually derived values obtained from Ordnance Survey maps and the maps in Volume V of the FSR. It should be noted, however, that the Institute of Hydrology digital terrain model (IHDTM) is based on 1: 50,000 scale mapping and that for some areas the generation of IHDTM-drainage paths is flawed. For some drainage areas, the IHDTM may provide a value for catchment area that differs significantly from the area enclosed by the topographic boundary drawn manually from the pattern of contours on a 1: 25,000 scale Ordnance Survey maps.

Floods and Reservoir Safety (ICE, 1996) recommends that catchment charateristics should not be defined solely from an examination of Ordnance Survey maps, and that site inspections should be carried out. Given the imperfections of the digital terrain model in some areas, and the fact that the median size of a reservoir catchment in the UK is only about 4 km^2, it is essential to check that all areas draining to the reservoir are defined accurately. It should be noted that Section 7.2 of Volume 5 of the FEH provides a method for correcting the digitally derived catchment characteristics where necessary.

DESIGN STORM PRECIPITATION

Volume 2 of the FEH contains a comprehensive reassessment of the UK rainfall depth-duration-frequency statistics. At the time of writing we have not had the opportunity to investigate personally the extent of the changes in the rainfall frequency statistics. We understand, however, from presentations made by members of the FEH development team that the over-generalisation inherent in the rainfall statistics contained in the FSR (see Dales & Reed, 1989) has been removed.

Neither the all year probable maximum precipitation (PMP) estimates contained in Volume II of the FSR, nor the seasonal PMP values first published in the 1978 edition of Floods and Reservoir Safety, have been reviewed or revised in the FEH. Similarly there have not been any changes to previous recommendations in the FSR and the 1978 guide to the FSR (Sutcliffe, 1978) its supplements about design storm profiles. The lack of attention to these topics no doubt stems from the terms of reference of the FEH team, but the omission is disappointing from the standpoint of reservoir design flood estimation in the UK.

The point PMP estimates for the UK shown on the maps contained in Volume V the 1975 FSR range from 110 to 190 mm for a duration of 2 hours, and from 250 to 400 mm for 24 hours. The appropriateness of these published values for some areas has questioned, amongst others, by Acreman (1989), Clark (1995) and Austin et al (1995).

Acreman (1989) concluded that the Halifax storm on 19 May 1989 produced 193 mm of rain in about 2 hours, which exceeds the 2 hour point PMP value of 160 mm for the region provided by the maps contained in the FSR. Similarly Clark (1995) has produced significantly higher PMP estimates for south-west England than the FSR, for all durations between 2 and 24 hours. The findings of these two studies are, however, by no means universally accepted.

The study by Austin et al (1995) entitled "Radar-based estimation of Probable Maximum Precipitation and Flood" is considered to be more significant in that it was published by the Meteorological Office, and was subject to scientific and technical review by an outside Steering Group. This study used radar data for convective storms together with numerical models of storm systems to estimate PMP for Ladybower, Valehouse and Stocks reservoirs which were considered as test catchments. Based on the results obtained by applying the preferred storm model to the test catchments the report concluded that:

- PMP is:
  - less than FSR values for storm durations less than 2 hours;
  - close to FSR values for storm durations from 2 to 11 hours; and
  - greater than FSR values for storm durations greater than 11 hours;
- PMP events for durations greater than about11 hours are likely to result from a class of meteorological systems known as Mesoscale Convective Systems whereas shorter duration PMP events are likely to result from super cell or multi-cell thunderstorms.
- Storm hyetographs for 12 hour duration storms are double peaked and quite different from the FSR profiles.

In our judgement the generally adopted PMP estimates for the UK, which originate from the 1975 FSR, need to be reviewed in the light of current best practise as it appears possible that PMP values have been underestimated for some storm durations.

The critical storm duration used to compute the design flood inflow to a single reservoir is usually obtained for the equation:

$$D = Tp\ (1 + SAAR/1000) * (RESLAG)\ \text{hours}$$

Where

D is the critical storm duration
Tp is the unit hydrograph time-to-peak;
SAAR is the standard average annual rainfall; and
RESLAG is the time between the peak inflow and maximum reservoir level.

For a typical Pennine reservoir with a simple uncontrolled shute spillway the critical storm duration is often within the range 4.25 to 6.5 hours. For such durations it is possible to envisage an intense, and almost stationary, thunderstorm producing an hyetograph which approximates to the stacked bell-shaped that is recommended for use in PMF calculations. For larger reservoirs such as Woodhead, Celyn and Pontsticill, which have throttled or "morning glory" type spillways, the critical storm duration may be closer to 15 hours. In our view the use of the stacked bell-shaped storm rainfall profile becomes increasingly conservative and unrealistic for durations greater than about 9 hours.

For PMF events the 1975 FSR considered only a summer storm profile, which is consistent with an isolated super cell or multi-celled thunderstorm. In the light of recent research into Mesoscale Convective Systems, which have only been identified in the UK over the last 10 to 15 years, we would like to see the issue of design storm profiles for extreme events re-examined. More attention might also usefully be given to winter events, which were not considered in the studies by Austin et al (1995).

PERCENTAGE RUNOFF

Percentage runoff is the proportion of the design storm rainfall hyetograph that becomes direct runoff to a river. The FSR rainfall-runoff model assumes that percentage runoff is constant throughout a design storm event, and is applied to each increment of the total design storm hyetograph.

The current percentage runoff model used in the FSR rainfall-runoff method is as presented by Boorman (1985). Percentage runoff is made up of the catchment parameter, Standard Percentage Runoff (SPR), which represents the normal runoff potential of the catchment, and dynamic terms that reflect the variation in runoff depending on the wetness of the catchment prior to the storm and the total storm precipitation.

The preferred method of deriving estimates of the parameter SPR is by the analyses of observed flood events using the procedure described in Appendices A.5 and A.6 of the FEH. Unfortunately the coincident rainfall and runoff data required for the application of the procedure are rarely available for small reservoired catchments. Where some flood data are available, the SPR values obtained from individual events are often very different; as can be opinions as to the most representative.

In recent years, following the publication of Institute of Hydrology Report 126 (Boorman et al.1995), catchment SPR values have often been derived from Soil Survey maps using the HOST soil class fractions. Now that the drudgery of the calculation of SPR from HOST has been removed by use of the FEH CD-ROM referred to above, there is likely to be an even greater temptation to adopt a catchment SPR value based on HOST rather than local data.

A more reliable approach is to utilise the close relationship that exists between SPR and the baseflow index (BFI). Although promoted originally as a low flow index, BFI is also a valuable index for flood estimation as it correlates well with SPR, which can be estimated from the equation:

$$SPR = 72.0 - 66.5\ BFI$$

A catchment BFI can be calculated from as little a one years' record of mean daily flows, by the procedure described in Institute of Hydrology Report 108 (Gustard et al, 1992). Also the Hydrometric Register and Statistics 1991-95 provides a BFI value for each of the 1400 gauging stations listed, thereby providing a large pool of potential donor catchments from which SPR values may be transferred to an ungauged catchment with similar characteristics.

THE FSR UNIT HYDROGRAPH AND ESTIMATION OF Tp(0)

In the FSR rainfall-runoff method, the unit hydrograph is a simple triangle whose shape is controlled by the parameter time-to-peak (Tp). The unit hydrograph peak and the time base (TB) are calculated as functions of time-to-peak as illustrated in Figure 2.

It is essential that the time-to-peak is estimated as accurately as possible because the shape of the unit hydrograph effectively determines the shape of the flood hydrograph for a given catchment. If the estimate of the time-to-peak is too short the unit hydrograph will have also have too short a time base and too high a peak, and the design flood hydrograph will be overly short with a conservatively high peak. The reverse is true if the estimated time to peak is too long.

The preferred method of deriving estimates of time-to-peak is by the analysis of observed flood events using the procedures set out in Appendices A.5 and A.6 of the FEH. Although the rainfall and runoff data required for these procedures are rarely available for reservoired catchments, there is sometimes sufficient data available to estimate the unit hydrograph time-to-peak. It can be obtained from the close relationship that exists between exists between Tp(0) and catchment lag (LAG) via the equation:

$$Tp(0) = 0.879\ LAG^{0.951}$$

Where Tp(0) is the time-to-peak of the instantaneous unit hydrograph (hours) and LAG is the time in hours from the centroid of the storm rainfall to the flood peak.

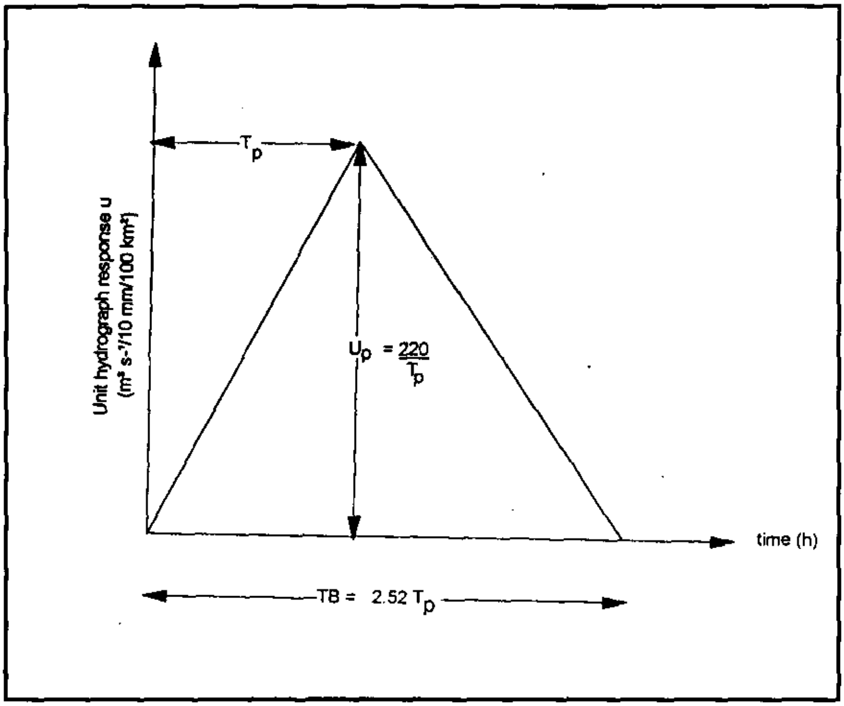

Fig. 2. FSR triangular unit hydrograph (copied from Volume 4 of Flood Estimation Handbook)

Unfortunately all too frequently, the time-to-peak of the unit hydrograph for reservoired catchments is estimated from catchment descriptors using a generalised relationship derived by regression analysis of the values provided by gauged catchments throughout the UK.

The original time-to-peak parameter estimation equation contained in the 1975 FSR was based on the regression analysis of data from 130 gauged catchments. This equation was superseded by the revised equation produced by Institute of Hydrology Report 94 (Boorman, 1985) based on data from 175 gauged catchments. The most up-to-date equation given in the FEH, based on data from 204 gauged catchments, is:

$$Tp(0) = 4.270\ DPSBAR^{-0.35}\ PROPWET^{-0.80}\ DPLBAR^{0.54}\ (1+URBEXT)^{-5.77}$$

Where

Tp(0) is the time-to-peak of the instantaneous unit hydrograph (hours);
DPSBAR is the mean drainage path slope (m/km);
PROPWET is the proportion of time when the soil moisture deficit was below 6mm during the period 1961-90;

DPLBAR is the mean drainage path length (km); and
URBEXT is the extent of urban and suburban land cover.

The above equation is in line with expectation that the steeper, wetter and more urbanised the catchment, the faster should be its response. It should be noted however that the factorial standard error of the new equation given in the FEH is 1.85. This relatively large value means, for example, that if the value of Tp(0) obtained from catchment descriptors is say 1.5 hours, then it is only 68% certain that the true value of Tp(0) lies between 0.8 and 2.8 hours.

For a hypothetical Pennine reservoir with a catchment area of 4 km^2 and a Tp(0) of 1.5 hours, the peak inflow in a 10,000 year flood from a 4.75 hour design storm would be about 39 m^3/s. However, with exactly the same design storm the peak inflow would be about 50 m^3/s if the value of Tp(0) is 0.8 hours but only 26 m^3/s if the value of Tp(0) is 2.8 hours. The maximum stillwater level for the hypothetical reservoir varies by +/-0.3m from the maximum level computed using a Tp(0) of 1.5 hours

An alternative to using the above equation is to transpose information from a gauged catchment that is hydrologically similar to areas draining to the reservoir for which a design flood hydrograph is required. Table A.3 of Volume 4 of the FEH lists the results of flood event analyses from the UK Flood Event Archive (Houghton-Carr and Boorman, 1991) is a useful source of potential donor catchments. This table contains time-to-peak data for about 270 gauged catchments,. Further parts of the software WINFAP-FEH, developed by the Centre for Ecology and Hydrology to facilitate the application of the statistical method of flood estimation, can be used to help select a suitable donor catchment. The routines used in WINFAP-FEH to compile the pooling-group of gauged catchments required to construct a flood growth curve for an individual site can equally well be used to provide a pooling group of hydrologically similar catchments which might serve as donor catchments.

For PMF events it is standard practise to reduce the estimated value of Tp(0) by one-third, to reflect the more rapid catchment response that is believed to occur in extreme conditions. The impact of this adjustment is to reduce the time base of the unit hydrograph by one-third and to increase all ordinates, including the unit hydrograph peak by one-half.

On several occasions we have been challenged as to whether this very significant adjustment to the unit hydrograph shape is warranted. Our standard response is to refer to the supporting evidence for the adjustment set out in Section 6.6.3 of Volume I of the 1975 FSR. Nevertheless we consider that it would be worthwhile to revisit this topic in the light of the larger database of severe events now available. Consideration might also be given as to whether the adjustment is equally valid for both largely rural and highly urbanised catchments.

CONCLUSIONS

To improve the quality of reservoir flood estimates there is a need to move away from the reliance on flood estimates based on parameters derived from catchment characteristics. Increasing the use of local data can do this. Data can be either from the actual catchment or from a hydrologically similar catchment selected using the approach suggested in the FEH. The possibility of using the results of the flood event analyses published in Table A.3 of Volume 4 of the FEH should not be overlooked.

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Revised design storm rainfall estimates obtained from the Flood Estimation Handbook (FEH)

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In the second half of January 2000 the Centre for Ecology and Hydrology dispatched copies of the FEH-CD rom to those who had placed orders for the FEH software. This software provides, together with several other important functions, a very convenient means of deriving rainfall depth-area-duration-frequency estimates based on the new procedures contained in the FEH, for any catchment in the United Kingdom (above a minimum threshold of 0.5 km^2).

Our current understanding of the new procedures, based on presentations made by members of the FEH team and a review of Volume 2 of the FEH, is that they represent an advance on the corresponding rainfall estimation methods provided by the 1975 Flood Studies Report. We attribute this advance partly to the improved data analysis and mapping techniques employed by the FEH team and partly to the larger rainfall database now available. We anticipate, therefore, that the new procedures will overall provide more reliable rainfall estimates throughout the UK.

We also understand that FEH research was primarily tailored to improve rainfall depth-duration-area-frequency estimates in the 2 to 1000 year return period range. We were pleasantly surprised, therefore, when we found that the FEH-CD rom (Version1.0) could be used to provide storm rainfall estimates for events with a return period of 10000 years. We were somewhat taken aback, however, when we compared the 10000 year catchment rainfalls provided by the FEH-CD rom for some reservoirs we have studied recently, with the 10000 year and PMP design values computed using FSR methodology. Not only were the 10000 year rainfalls obtained from the FEH-CD rom significantly larger than the corresponding estimates computed from FSR methodology, but in some cases the new 10000 year rainfall estimates were also higher than the PMP values.

The following table compares 10000 year rainfall and PMP estimates for a selection of ten reservoir catchments in England and Wales. For most reservoirs the storm durations listed are relevant to a particular design case examined. On the basis of the examples shown it would appear that either the method of extrapolation adopted in the FEH is providing overly high rainfall estimates for events with a return period of 10000 years or the 10000 year and

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PMP estimates computed using FSR methodology are too low. A further possibility is that both the FEH and FSR methods are valid approaches and the range of design rainfall estimates are simply a true reflection of the uncertainty attached to them. Needless to say this topic warrants further consideration.

Table 1. Comparison of FSR and FEH Catchment Rainfall Estimates

Reservoir	Storm Duration (hrs)	FSR Catchment Rainfall (mm)			FEH Catchment Rainfall (mm)	
		5 yr	10,000 yr	Summer PMP	5 yr	10,000 yr
NW London						
Brent	7.25	32.0	164	208	31.7	263
East Anglia						
Ravensthorpe	11.5	36.8	182	214	41.5	263
Solihull						
Olton	5.75	32.1	168	205	28.0	201
Nidderdale						
Angram	5.25	40.3	191	201	36.3	259
North West						
Stocks	6.0	39.2	184	205	38.9	266
	12.0	52.7	223	244	52.9	316
Adlington	6.5	30.3	161	206	31.9	221
	18.5	42.1	197	256	44.8	256
Mid Wales						
Egnant	5.25	36.7	185	221	39.3	262
South Wales						
L.Neuadd	5.75	51.4	221	224	46.5	295
West Wales						
Llysyfran	10.5	45.9	204	264	50.3	263
South West						
L.Slade	3.75	28.6	156	188	31.1	214

Reservoir inundation studies: a concrete dam owner's perspective

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L A COWAN, Babtie Group, UK.

SYNOPSIS. Scottish and Southern Energy plc are completing reservoir inundation studies on all seventy six reservoirs operated under the Reservoirs Act, 1975. Concrete dams, principally gravity and to a lesser extent buttress, tend to predominate. At sixteen of the reservoirs, large floodgates form a dominant feature of the dam structure. The six main generation schemes are generally based on reservoir cascade systems.

An initial pilot study was carried out on the Glenmoriston catchment to investigate the specific needs of inundation studies in relation to cascade systems, concrete dam structures and large gated structures. The study was approached from the perspectives of risk assessment, asset management, operational control and contingency planning. Consideration was given to the flows which could be generated by dam failure, gate failure or malfunction, bottom outlet failure and by various extreme flood events. The pilot study was used to develop a policy document for wider application.

INTRODUCTION

Scottish and Southern Energy plc, formed by the merger in 1998 of Scottish Hydro-Electric and Southern Electric, is the largest generator of conventional hydro power in the UK. The Company owns and operates 66 hydro power stations with a total installed capacity of 1100 MW as well as the 300 MW Foyers pumped storage scheme.

The 66 power stations are fed from 76 main reservoirs registered under the Reservoirs Act, 1975 (1) which are impounded by 93 dams.

Scottish and Southern Energy's stock of dams is markedly different from any other UK dam owner. The dams are predominantly concrete structures with 45 gravity dams, 7 buttress, 5 arch, 1 pre-stressed, 14 earth embankment, 3 concrete faced rockfill, 9 composite concrete gravity and embankment and 8 small cut-off sections. One reservoir registered under the Act is a 5 km long concrete lined open trapezoidal aqueduct. Fifty-six of the dams are included

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in the ICOLD world register, including one arch dam where the reservoir is marginally below the capacity for inclusion in the Reservoirs Act, 1975.

The North of Scotland is well served with natural reservoir sites, which are generally in U-shaped glaciated valleys. There are therefore no very high dams by international standards, but many have significant crest lengths. The buttress dam at Sloy is the highest at 56 m and Mullardoch Dam with a crest length of 727 m is the longest. Loch Quoich, impounded by a 38 m high concrete faced rock fill dam is the largest reservoir in the UK with a capacity of $382 \times 10^6 m^3$.

Other unique structures within the diverse range of concrete dams include Monar, a 35 m high arch dam and the only double curvature arch dam in the UK, and Allt-na-Lairage which is a pre-stressed concrete dam.

The majority of the reservoirs are in multi-reservoir cascade systems, which maximise the water utilisation and storage potential of the catchments. As a consequence, the downstream dams in a number of instances are virtually run of river systems with relatively little storage and a requirement to pass considerable floods. These structures generally have floodgates to assist with flood routing. A variety of different types of gates are used including radial, drum, tilting and vertical direct lift roller gates. In total sixteen dams have gates for the release of floodwater and four have syphons. Under extreme flood conditions many of the dams will be subject to significant levels of overtopping and in such instances the Inspecting Engineer considers the effect of this.

Nine of the dams are more than fifty years old, most significantly those constructed by the Grampian Electricity Company as part of the Tummel Scheme in 1932. The oldest structure is Loch Mhor dam, which is of concrete gravity construction with, associated masonry and earthfill elements, all constructed in 1896. The remainder were generally constructed during the programme of intensive hydro-electric development between 1947 and 1963. The large majority of dams can therefore be considered as modern, incorporating advances in technology since 1945.

THE NEED FOR INUNDATION STUDIES

Scottish and Southern Energy's approach to the development of risk based dam safety and asset management systems are well documented, (2), (3) (4) and (5). Reservoir inundation studies form a significant element in this overall process.

One of the key reasons for undertaking dam risk assessment is the hazard posed to third parties by uncontrolled release of water from the reservoir, principally the risk to life and property downstream. The scale of this loss can only be judged if the consequences of a dam failure are accurately assessed. Large capacity reservoirs sitting high in steep valleys above major towns are easy to identify as high hazard, but small communities tens of kilometres downstream of moderately sized dams in less steep valleys are not so easy to judge. The geographical disposition of much of Scottish and Southern Energy's reservoir stock is such that the latter case is normally the more relevant.

As decisions on applicable flood and seismic standards are related to the category of reservoir hazard the consequences of applying inappropriate criteria to the dams is a potential source of third party risk or unnecessary expenditure for the owner. Dam breach analysis and inundation modelling provide a consistent means to assess the hazard of a reservoir. The company therefore undertook a pilot study to examine the value of inundation studies to their particular circumstances.

PILOT STUDY

The pilot study was carried out on the Glenmoriston catchment within the Garry/Moriston Scheme, covering Dundreggan, Cluanie and Loyne Dams. The catchment had been recently reconsidered in relation to flood studies for the 1997 Inspecting Engineer's inspection, Failure Mode Effect and Criticality Assessments had also been carried out as part of ongoing risk assessment studies.

The scheme is a small cascade system with the two upper dams of Loyne and Cluanie feeding water direct or via Ceannacroc Power Station into the River Moriston and through to Dundreggan Reservoir. The location of the dams is shown on Figure 1 with their main characteristics indicated within Table 1 below. Cluanie and Dundreggan dams are indicated in cross section respectively on Figures 2 and 3.

Table 1. Characteristics of dams within pilot study

	Loyne	Cluanie	Dundreggan
Type of dam	Concrete gravity, free overflow spillweir	Concrete gravity, free overflow spillweir	Two radial gates, tilting gate and concrete gravity
Year completed	1956	1955	1957
Flood category (Note 1)	A(Min)	A(Min)	A(Min)
Capacity (x 10^6 m3)	45.5	203	1.64
Crest length (m)	549	675	155
Height (m)	18.11	35.28	15.85

Note 1 - From Table 1 of "Floods and Reservoir Safety" (6)

The catchment area comprises typical upland catchment with areas of steep and forested valley sides on the lower slopes. The River Moriston is typical of many rivers in the Scottish Highlands with a steep riverbed in the upper reaches, comprising of large boulders, down to granular material in the flatter lower reaches.

The pilot study reservoirs and catchment area comprised many of the attributes common to Scottish and Southern Energy's reservoir asset base and were chosen to be representative of the typical problems that will be encountered during the wider scale studies on all other schemes.

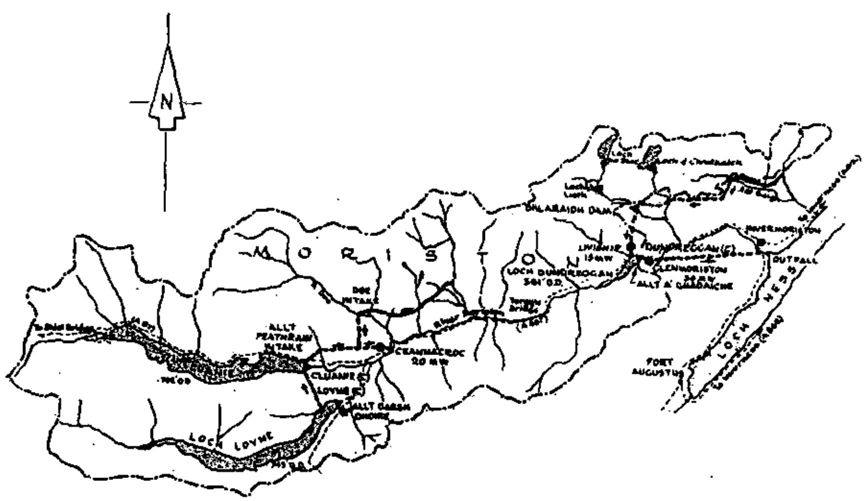

Fig. 1. Pilot study area : Glenmoriston catchment

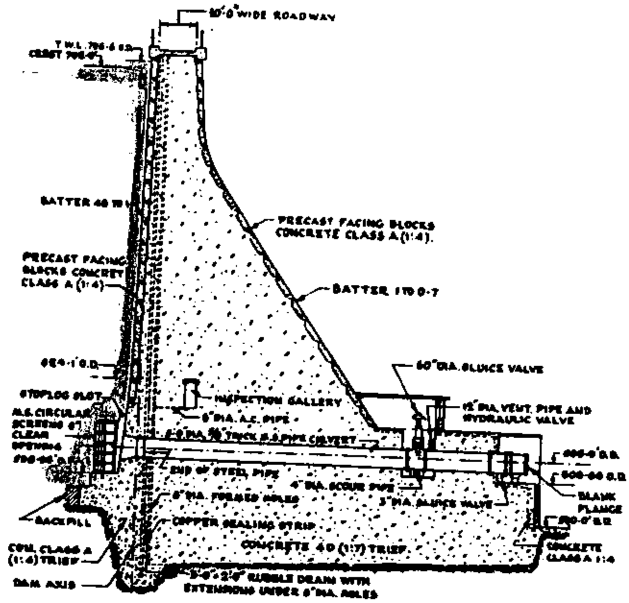

Fig. 2. Cluanie Dam : Cross Section

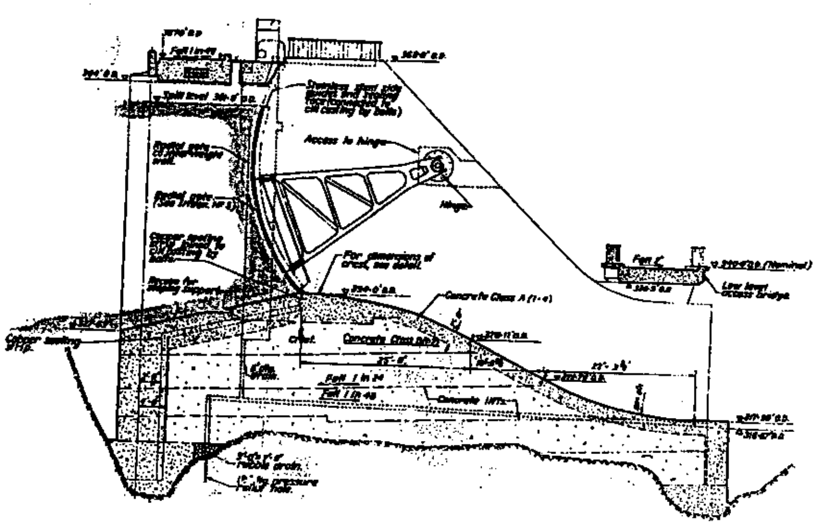

Fig. 3. Dundreggan Dam : Cross Section through radial gate

PILOT STUDY OBJECTIVES

Each of the reservoirs were reviewed to :

- Identify potential failure scenarios.
- Gain a better understanding of potential flood flows from dam breach, gate failure, PMF flood flows and bottom outlet failures and the effects these flows may have on the downstream valley.
- Agree a format for inundation maps to suit Scottish and Southern Energy's operational and other requirements.
- Prepare inundation maps for each of the three reservoirs or combination as appropriate.
- Produce sufficient information to allow the preparation of a brief applicable to future inundation studies, which may be carried out on any Scottish and Southern Energy reservoir.

APPROACH TO STUDY

The objectives were achieved by carrying out the following modelling and associated work.

- ◆ Potential failure scenarios were considered.
- ◆ Initial "standard" dambreak computer simulations were carried out for all reservoirs to provide a working computer model of the system encompassing PMF plus dam breach and routed PMF events.
- ◆ Hydraulic studies were completed to determine potential peak flows and/or flood peak hydrographs for gate operation, encompassing failure/malfunction or operational error and for bottom outlet failure.
- ◆ Assessment of the flow from the above hydraulic studies with other natural flood event peak flows to determine similarity. Selected flows were then routed down the valley to assess potential inundation areas.
- ◆ Additional, non-standard dambreak runs were undertaken to assess the sensitivity of dam failure duration, breach dimensions, spillway capacity, initial reservoir level and inflow conditions.
- ◆ Sample inundation maps were then prepared.

DAM BREACH

The calculation method for the dambreak analysis was based on the methodology recommended in "Reservoir Contingency Planning - Methodology for Preparation of Flood Inundation Maps", (7) and the use of the computer modelling package DAMBRKUK. The initial "standard" dambreak runs were assessed to determine whether the standard input conditions for DAMBRKUK should be varied for Scottish and Southern Energy's asset base. The results are discussed in the following sections.

Inflow Conditions
Inflow conditions assumed a PMF or other return period event entering the reservoir prior to dam failure.

Cascade Conditions
For the reservoir cascade system failure of the lower dam (Dundreggan) was assumed to result due to the flood wave from failure of one of the upper reservoirs (either Loyne or Cluanie). This generated a combined floodwave below the lower reservoir. For this cascade situation assuming a failure of Dundreggan, rather than no failure, did not significantly affect the results downstream of Dundreggan.

For other cases, however, particularly where relatively small storage volumes supply large downstream reservoirs, the likelihood of the downstream dam failing due to failure of the upstream dam will be reviewed on an individual basis and may require more detailed analysis.

Valley Roughness
Manning's coefficient 'n' for valley and river roughness was initially set at 0.04 for all models, which is consistent with an upland rural valley and within the recommended range of 0.03 to 0.05. In order to stabilise the model this had to be amended to 0.044 for the standard model runs. This minor alteration to the 'n' value to achieve model stability was considered unlikely to affect predicted flood levels significantly.

As part of the sensitivity analysis an 'n' value of 0.08 was used to simulate the effect of densely forested reaches. This demonstrated that the higher 'n' value resulted in significantly lower velocities (30-40%) and considerably greater flood depths (30-50%). For future studies it may be necessary to carry out two simulations, using both a high and low value of Manning's 'n'.

At lower magnitude flow runs, such as for some gate failure scenarios, the roughness coefficient for the natural river channels is likely to have a greater influence on the depth than the sections used for dambreak flows. Should detailed study be required for lower magnitude flow events then consideration will be given to more accurate modelling, for example incorporating information on channel dimensions gained from a valley walkover or survey.

Time of Breach Formation
The dam failure duration for concrete gravity structures was considered to be instantaneous and set at 0.2 hours, as recommended in the DAMBRKUK user manual. Once model stability was reached this was extended further to examine the sensitivity of this parameter. The peak discharge reduces dramatically with increased time to failure and therefore reduces levels of

inundation along the valley. Reducing the time of failure below 0.2 hours did not have a significant effect on the peak discharge.

Spillway Capacity\Reservoir Water levels at the Point of Failure

Reservoir water levels were set at top water level for commencement of modelling and in general the dams set to fail when the water level under flood conditions reached dam crest level or peak water level.

Where the stillwater flood level did not reach the crest of the dam the spillway capacity was effectively reduced by 15 - 50 % to artificially raise the water level to simulate increased loading conditions which could conceivably induce a dam failure. Otherwise the dams were set to fail at the peak overtopping water level reached during a PMF event.

Sensitivity analysis demonstrated that for Loyne and Cluanie the water level in the reservoir at the point of failure has a relatively small effect on the peak dambreak discharge. Reservoir water levels at the point of failure were seen to have a significant effect at Dundreggan.

Breach Dimensions

The final dimensions of a breach and the time of formation are likely to have the most significant effect on the peak outflow following a dam failure. The DAMBRKUK user manual recommends a final breach width of less than or equal to 0.5 x dam length and a breach height equal to the height of the dam, for a gravity dam. It is credible, however, that under some circumstances the breach width could be the complete length of the dam. Similarly, it is possible that a much smaller breach than that recommended could form.

Several breach configurations were considered, ranging from failure of a single monolith through multiple blocks to entire dam failure. This assumed loss of the entire dam was recognised as being extreme but would present the worst case scenario for the preparation of inundation maps. The results for Cluanie are summarised in Table 2.

Table 2. Dambreak flow and velocity for Cluanie breach scenarios.

Breach	0.15 km d\s of Cluanie dam		At Dundreggan reservoir	
	Flow (m^3/s).	Velocity (m/s).	Flow (m^3/s).	Velocity (m/s).
Full breach	103,900	20.1	40,150	5.8
10 blocks	41,600	15.6	14,950	2.55
1 block	4,225	7.9	2,500	1.4

Further studies into failure modes specific to concrete dams are considered to be required to define failure modes and to validate the above assumptions.

Secondary Damming
Secondary damming at bridges and restrictions in the downstream valley caused by debris brought down by the flood wave, was examined. For this study the magnitude and velocity of the initial dambreak flood wave from the reservoirs was considered to be such that the typical bridges crossing the river would present an insignificant barrier to the flow or would be washed out during the initial flood. For lower flow modelling with associated slower water velocities, the restrictions presented by the bridges and the damming behind bridges may be of more significance. The validity of including the effects of secondary damming within any particular analysis needs to be assessed on a case by case basis and in light of the predicted flood flows and velocities from the initial dambreak models.

GATE OPERATION
Various flood study runs were undertaken to simulate floods of various magnitudes passing through Dundreggan reservoir with one or all gates operational and all gates closed.

Complete structural failure of the gates could conceivably occur at any time with the worst case scenario being when the dam crest is being overtopped at times of maximum flood flow. In the case of Dundreggan this generally equated to peak inflow. Gate malfunction or operational error leading to failure to open followed by sudden opening of the gates during a flood event would represent a more conceivable operational scenario.

The model runs shown in Table 3 were completed to determine outflow hydrographs from the reservoir allowing the main parameters to be developed in relation to gate operation.

Table 3. Model runs for gate operation

Case	Scenario	Flood Event	Initial water level
1	Gates fail to open, sudden structural failure of all gates	PMF	Top water level
2	Gates fail to open, sudden structural failure of all gates	PMF	Invert of gates
3	Gates fail to open, sudden structural failure of all gates	1 in 150 year	Top water level
4	Gates fail to open, sudden opening of all gates	PMF	Varied to worst case
5	Gates fail to open, sudden opening of all gates	PMF	Invert of gates
6	Gates closed followed by sudden opening or structural failure of all gates.	No flood	Top water level

The gates were set to fail at a water level that generated the maximum peak outflow, which was governed by three main effects. Namely, size of flood event entering the reservoir, the water level within the reservoir at the time of gate failure/opening and the coincidence of peaks produced by gate failure/opening and the flood event inflow.

The PMF event outflow was found to be greater than the peak outflow resulting from a gate failure with no flood. The effect of coincidence of peaks for cases 4 and 5, where the gate opening was set to occur before overtopping commenced, demonstrated that for Dundreggan it was not possible to combine the peak flows produced from the gate opening and flood event (the reservoir was filled before the PMF peak arrived). The initial water level was the only parameter varied in cases 1 and 2, however the peak outflow and resultant levels were found to be the same, highlighting that the initial water level has little effect on the peak outflow for the floods modelled. The results at Dundreggan may be fairly typical of gated structures where there is a relatively small storage capacity, compared to the volume of a PMF event.

BOTTOM OUTLET FAILURE

As part of the study, bottom outlet failure was also considered where failure of part of the hydraulic system at the dam could discharge stored water from the reservoir into the downstream river channel. The elements of the hydraulic system which could be considered included, ground sluices, scour culverts, dewatering culverts, compensation flow outlets and diversion tunnels built during dam construction, excluding high level spillway gates and tunnel intakes to power stations. In general power station discharges are remote from the dam and this flow would not contribute to the immediate vicinity of a bottom outlet failure. Each case will require to be considered and where appropriate the full load flow added to a potential bottom outlet failure flow.

Two scenarios for failure of a bottom outlet were considered. Firstly by failure of a gate or valve to open leading to a higher than normal reservoir level followed by a complete structural failure or a sudden opening. Secondly, the complete removal of a downstream valve by other external factors giving rise to flows greater than would be the case with the valve open at normal top water level.

The maximum rate of flow for each outlet device was checked for the peak flow, which could ensue following a credible failure or failures. The magnitude of these flows was then considered in relation to the scale of potential effects if they were routed down the valley. For the pilot study all bottom outlet failure flows were found to be significantly less than the respective 1 in 150 year flood event flow.

NATURAL FLOOD FLOWS

Flood flows for the return period events associated with the dam category were routed down the valley to allow comparison between potential dam failure and natural events. These events are considerably lower than dambreak flows as can be seen from Table 4. The DAMBRKUK software was used for this routing, but it was recognised that the accuracy of cross section data, valley/river roughness values and the effect of incoming flood flows downstream of the dam could lead to errors.

Table 4. Comparison of Dambreak and Design Flood Events.

Reservoir	Dambreak flow (m^3/s) (Complete failure)	PMF flow (m^3/s).	1 in 10,000 yr. flow (m^3/s).
Loyne	28,990	288	223
Cluanie	103,900	424	347
Dundreggan	8,526	2,010	1,390

The information obtained, therefore, can only be taken as indicative. However it does provide useful data for comparison between different flood events.

Data from return period flood study outflow hydrographs was presented on key cross sections within the inundation maps. It is considered very suitable for use by Scottish and Southern Energy's Control Engineers in rapid assessment of potential flood depth and general inundation in the valley downstream of a dam passing an extreme flood discharge over the spillway or through flood gates.

Sensitive areas where lower magnitude events need to be considered may require a more detailed prediction of the flood effects, particularly where it can be seen that a change in predicted levels could result in significantly more or less inundation. As Scottish and Southern Energy's inundation mapping policy develops such areas may be subject to more traditional river modelling techniques.

INUNDATION MAPPING

Inundation maps were prepared for each dam for the two cases of PMF plus dambreak and PMF without dambreak. Each identified the following items; estimated maximum extent of flooding, areas liable to inundation only, areas liable to inundation and structural damage, flood hydrograph at key cross sections, depth profile at key cross sections, time of arrival at key sections for 1 m over bankfull and maximum depth, available transport routes and potential helicopter landing sites.

The other return period events and gate failure\maloperation events considered have been presented as supporting information within the inundation mapping documentation. Water levels with supporting hydrographs to allow approximation of downstream levels during such events, are given at key cross sections on the maps.

POLICY DOCUMENT

A Policy Document has been prepared based on the results of the pilot study, covering Scottish and Southern Energy's aims for inundation studies and sets out the framework within which the studies will be carried out. This includes a summary of the technical issues raised by the pilot study and the conclusions reached to ensure common application throughout.

The principal situations to be modelled and for which information will be provided are shown on Table 5. Supporting information consists of hydrographs, water level and velocity data at key cross sections.

Table 5. Inundation Mapping Format

Event	*Level of Information to be produced*
PMF and full dam breach	Inundation Mapping
PMF and partial dam breach	Supporting Information
PMF event (no associated dam breach)	Inundation Mapping
1 in 10,000 year natural flood event (no associated dam breach)	Supporting Information
1 in 150 year natural flood event (no associated dam breach)	Supporting Information
Gate Failure with PMF flood event (worst case)	Supporting Information
Gate Opening/Failure with no flood event	Supporting Information

Bottom outlet failure flows will be assessed on a case by case basis and modelling together with the production of supporting information may be undertaken. Other gate failure/opening scenarios may be considered for individual dams.

CONCLUSIONS

The pilot study set out to achieve several objectives. Dam failure times, breach size and breach development can all have a considerable influence on the discharge flow and were studied for sensitivity of outcome. The passage of a probable maximum flood also represents a potential inundation case that requires to be considered by an owner. In addition to these Scottish and Southern Energy is concerned with the passage of lesser floods, and in particular with the implications of floodgate operation or malfunction.

The contingency planning aspects of this information are now being considered by Scottish and Southern Energy and it is the intention to disseminate this to appropriate emergency services as part of the integrated response to events on hydro generation schemes.

The output from the pilot study convinced Scottish and Southern Energy of the value of having this decision making information available on all reservoirs within the cascade systems and a programme of studies is now in hand to cover all the major, multiple reservoir schemes by Spring of 2001.

The use of the DAMBRK UK software package is well established within the larger reservoir owning companies in the UK. Commentary on the use and presentation of the output from this package is well documented in the industry, (7), with further commentary to be included within the CIRIA Guide on "Risk and Reservoirs" (8). There is, however, no legislative requirement to use such a means of assessing hazard nor is there published dam safety guidance on the application or interpretation of such modelling in deciding hazard. As a result, those owners who have carried out this type of study have applied differing criteria to the presentation and dissemination of the results.

The output from the studies will be fed into Scottish and Southern Energy's overall risk management programme and may be expected to initiate some changes in currently held views on hazard and the consequent acceptability of risk. In particular, it will give additional support to Inspecting Engineers in determining flood hazard category. Scottish and Southern Energy regard all of this work as consistent with the emergency plans required for major industrial plants under EEC Directive, 1984 " Control of Major Accident Hazards" (CIMAH) although these regulations do not apply to dams and reservoirs. The studies also meet Scottish and Southern Energy's objectives of knowing and understanding the hazards associated with dams and reservoirs as part of the dam safety programme.

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Bohernabreena Reservoirs, Dublin: the impact of Hurricane Charlie

A ROWLAND, Binnie Black & Veatch, UK

SYNOPSIS. Upper and Lower Bohernabreena are two reservoirs in cascade on the River Dodder in the Wicklow Hills near Dublin. An inspection by WJ Carlyle in 1974 and associated flood studies identified that the capacities of both spillways fell far short of the required PMF standard.

On 25 August 1986 an offshoot depression from Hurricane Charlie passed over Ireland and nearly 200 mm of rainfall, the highest since records began, fell over a 24 hour period. The stillwater level in Lower Bohernabreena rose to 940 mm above spillweir level, equivalent to a flow of 90 m^3/s. The level recorder at the upper reservoir did not function correctly but the outflow would have been of similar magnitude. Damage occurred to both spillways.

The consequences at Lower Bohernabreena could have been worse had the meteorological office not informed the reservoir superintendent of the exceptional rainfall, giving time for the outlets to be opened to lower the level of the reservoir in advance of the flood passing through the reservoir.

INTRODUCTION

The Upper and Lower Bohernabreena Reservoirs were constructed between 1885 and 1887 on the River Dodder in the Wicklow Hills some 8 km upstream of the outskirts of Dublin. The upper reservoir is used for water supply whilst the lower is operated for river regulation. The layout is shown on Figure 1.

Both dams are of typical clay construction with puddle clay core. The upstream slope is 1 to 3 and the downstream slope 1 to 2.

The dams are in sound condition but the spillways are of inadequate capacity. Failure of the upper dam would cause the lower dam to be overtopped and to fail. Breach of the lower dam would certainly lead to extensive damage and loss of life. Both reservoirs have therefore been placed in Category A of flood risk and the overflow provisions should be capable of passing the Probable Maximum Flood.

Dams 2000, Thomas Telford, London, 2000.

The overflow weir lengths are in fact nearly adequate to pass the PMF but the geometry of the spillways restricts the discharge capacity. The upper reservoir can pass the 10 000 year flood provided the bywash channel is not blocked. The lower reservoir cannot pass the 10 000 year flood.

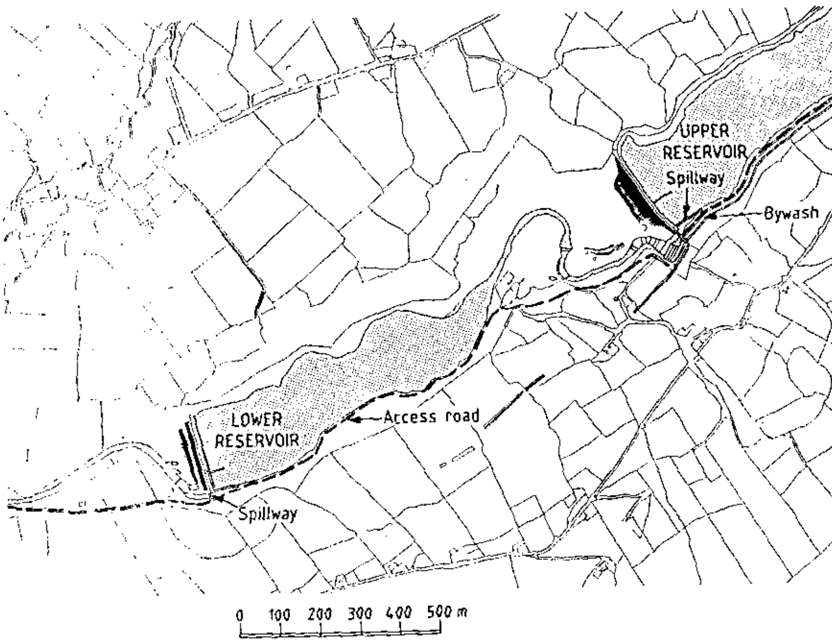

Figure 1 Layout of Upper and Lower Bohernabreena Reservoirs

DESCRIPTION OF SPILLWAYS

Upper reservoir

The spillway is at the left abutment of the dam (Figure 2). The spillweir crest is 60.89 m long and discharges into a side channel spillway. This channel passes under the access bridge on the axis of the dam and thence into a small basin from where it discharges down the main spillway over a control weir. The bywash channel, skirting the left side of the reservoir, also discharges into this basin.

The spillway below the lower control weir consists of a series of near horizontal steps connected by steeply sloping ramps. In plan this spillway turns through a sharp left hand bend before discharging into the natural channel below the dam and thence into the lower reservoir.

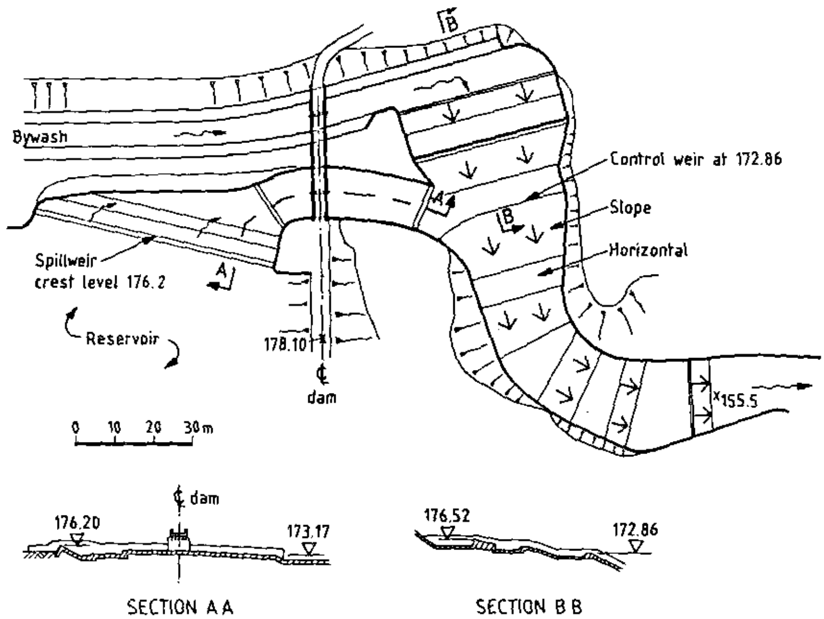

Figure 2 Spillway at Upper Bohernabreena

Although the side channel spillweir crest is long enough to pass safely the PMF discharge, the spillway chokes at much lower flows at the restricted bridge section. This causes the flow to back up drowning the main weir crest and causing unacceptable water levels in the reservoir. The maximum capacity is about 150 m^3/s. Compare this with the predicted outflows shown in Table 1.

Table 1. Summary of flood routings for upper reservoir – freeboard 1.83 m

Event	Peak inflow m^3/s	Peak outflow m^3/s	Flood rise m
PMF (bywash blocked)	329	318	3.1
10 000 year (bywash blocked	183	164	2.4
10 000 year (bywash clear)	129	111	1.4

The other main problem with the existing arrangement is that the sharp bend on the lower part of the chute causes the flow to pile up on the outside of the bend and at fairly moderate discharges the outer wall is overtopped. The location of this bend is close to the foot of the dam and any serious overtopping could lead to scouring at the toe of the dam.

Lower reservoir

The spillway is at the left abutment of the dam (Figure 3). The spillweir crest is at right angles to the axis of the dam and discharges as a side-channel spillway into a channel leading directly into a steep curved spillway chute.

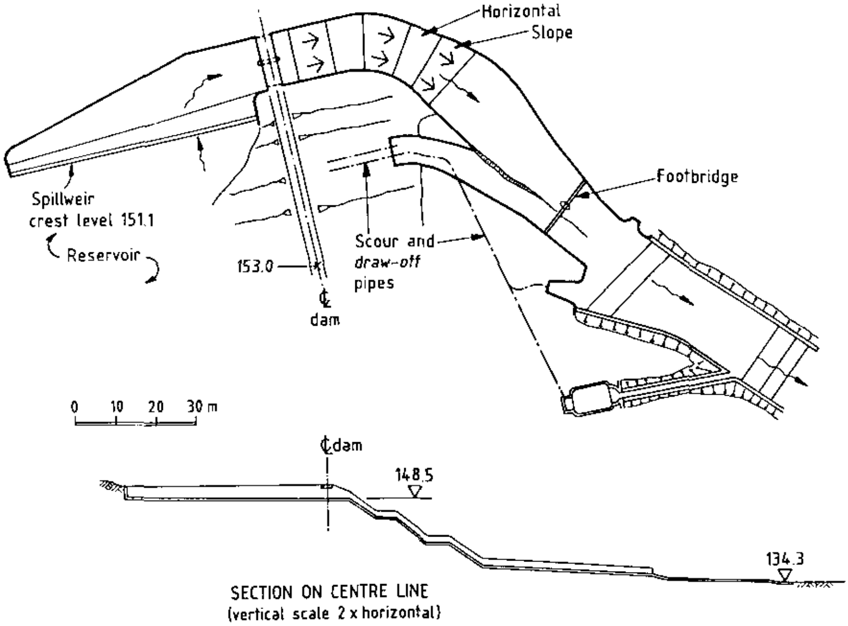

Figure 3 Spillway at Lower Bohernabreena

Provided modular discharge could be maintained over the 61 m long weir, the PMF could be passed with the water level only partly up the masonry wave wall. The channel from the spillweir to the head of the chute is inadequate to take this flow. In particular the control section under the bridge on the axis of the dam causes the flow to back up and drown the main spillweir at flows in excess of about 65 m^3/s. The maximum flow is about 125 m^3/s, somewhat less than the predicted outflows for the PMF and 10,000 year flood given in Table 2.

Table 2. Summary of flood routings for lower reservoir – freeboard 1.87 m

Event	Peak inflow m^3/s	Peak outflow m^3/s	Flood rise m
PMF	379	377	3.1
10 000 year	165	164	2.5
1 000 year	111	106	1.2

The main spillway consists of a series of horizontal steps connected by steep ramps. The flow will be supercritical throughout. In plan, the spillway takes a fairly sharp right hand bend midway down and there will be overtopping of the outside wall at moderate discharges. Although this does not endanger the dam directly, undercutting of the slope could cause slips to occur that may block the spillway, pushing the water over the right wall of the spillway onto the downstream shoulder of the dam.

HURRICANE CHARLIE

A weather system associated with Hurricane Charlie passed over Ireland on Monday 25 August 1986. Over a 24 hour period, the highest rainfall since records began fell on the Dodder catchment. Three rain gauges in the catchment recorded rainfalls of 182.5 mm, 190.2 mm and 165.3 mm (Corby, 1986). At the last of these, the intensity of rainfall was not fully recorded as the gauge suffered water damage during the storm but the maximum intensity was of the order of 20 mm per hour. The average rainfall during the 24 hour period over the whole catchment upstream of Dublin has been calculated as 134 mm (McDaid and O'Riordan)

Although there was a very similar storm in August 1905 and a slightly smaller one in September 1931, the return period of the August 1986 event has been estimated to be between 100 and 400 years, and even as high as 460 years by some analysts. A return period of around 250 years is probably realistic. It is notable that all three major events occurred in late summer.

The water level records at the upper reservoir were truncated at 1.23 ft (0.37 m), when the gauge failed. The peak discharge over the weir was probably about 40 to 50 m^3/s. The peak flow recorded in the bywash was 1590 cusecs (45 m^3/s) which, combined with the discharge over the weir, would give a total discharge down the chute approaching 90 m^3/s. A flow of about this magnitude was sustained from 8 pm on 25 August to 1.30 am the following morning.

Even at a flow significantly lower than the existing maximum capacity coping stones to the crest of low control weir at the top of the chute were dislodged. As could be expected there was also overtopping of the wall on the outside of the bend at the bottom of the chute resulting in some erosion behind the wall. There was also erosion of the river bed immediately downstream of the spillway.

At the lower reservoir, the peak level recorded at 11 pm on 25 August was 3'-1" (0.94 m) equivalent to a flow of about 90 m^3/s. Water submerged the arches of the bridge and overtopped the spillway walls causing damage to coping stones and erosion of the fill behind.

The reservoir keeper reports standing on the bridge over the spillway and feeling the bridge vibrating, stating that resting his hands on the parapet was like holding a jackhammer. He also saw five coping stones from the spillway walls being projected by the water 10 to 12 feet into the air accompanied by a loud noise like an explosion.

The Meteorological Office advised Dublin Corporation that there was exceptional rainfall in the catchment area and the decision was made to open all outlets at the reservoirs. The reduction in water level in the lower reservoir prior to the peak of the flood certainly mitigated the consequences of the storm. Although a total of 315 residential and 25 commercial properties were flooded within the city boundary, it was concluded that the reservoirs helped to alleviate downstream flooding.

PROPOSED MODIFICATIONS

Modifications to the spillways have been designed and model testing was carried out at University College in Dublin to confirm that the proposed complex geometry was satisfactory.

Upper reservoir

At the upper reservoir, the side-channel will be widened by extending it to incorporate the existing bywash channel and the floor will be lowered by around 2 m The outside wall of the upper part of the spillway will be realigned. Flow patterns in the lower spillway will be improved by the construction of splitter walls. The arrangement is shown on Figure 4.

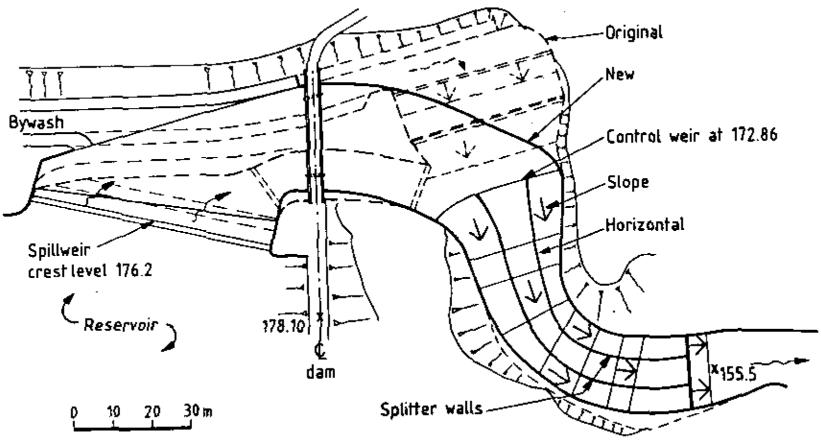

Figure 4 Proposed modifications to spillway at Upper Bohernabreena

Lower reservoir
Significant widening of the side channel at the lower reservoir is restricted by the steeply rising hillside which forms the left abutment. Some widening is proposed but lowering of the floor of the upper part of the spillway by 3 m is necessary to achieve the required discharge capacity.

The outside wall of the steep chute will be realigned to reduce the problem of overtopping. The inside wall remains relatively unchanged but will have to be heightened. The layout is shown on Figure 5.

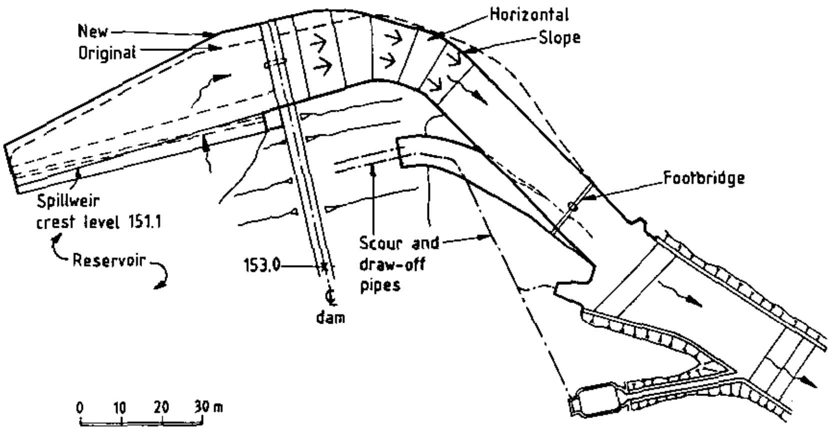

Figure 5 Proposed modifications to spillway at Lower Bohernabreena

Implementation
Detailed design is currently being undertaken and a contract for the construction is expected to be awarded during the last quarter of 2000.

CONCLUSIONS
Although the weir length of both spillways is theoretically adequate for the design flood, throttling of the flow within the discharge channel severely restricts the discharge capacity.

Even at the throttled discharges the geometry of the spillways resulted in the risk of potentially serious damage to the dams.

The value of a reservoir keeper in being able to respond quickly to abnormal situations has been established. In addition, his observations of conditions and the performance of the spillways highlight the potentially destructive power of water and have been valuable in developing proposals for the modifications.

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Prediction of downstream destruction following dam failure: no quick solution

F R TARRANT, Binnie Black & Veatch, UK
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SYNOPSIS. Based on the results of over 150 dam break analyses carried out for reservoirs in all parts of the United Kingdom, an attempt has been made to relate extent of damage to three easily definable parameters - dam height, reservoir volume and valley slope. Three examples described in the text demonstrate that valley shape and conditions have a very significant effect on the extent of damage and highlight the difficulty in establishing such a relationship. The best relationship is between extent of damage and dam height, as perhaps would be expected, but the scatter remains to great for this to be used for any purpose other than assisting engineers in defining the downstream limit of a dam break model. The results confirm that the use of simple formulae for extent of damage of a set distance downstream for assessing the hazard ranking of a major or high risk reservoir is not appropriate. Such decisions must remain the judgement of the engineers unless a full dam break study is carried out.

INTRODUCTION

The safety of dams is of paramount importance. Modern design methods and construction control have resulted in a very low risk of failure. However, the risk of failure can never economically be reduced to zero. Whilst any risk remains, it is the responsibility of reservoir owners to ensure that the extent of downstream hazard is known and is managed. The most effective tool in determining the hazard is dam break analysis.

Methods of risk analysis have been and continue to be developed for determining the level of risk of failure of a dam. Combining knowledge of the population at risk with the probability of failure of a dam gives a reservoir owner a guide to whether the safety of a particular dam is acceptable. In this paper we are concerned with prediction of the extent and severity of damage downstream of a dam following failure.

It is recognised that a full and rigorous analysis of every dam cannot necessarily be justified. The problem is how to determine which reservoirs have a high number of people at risk. Since adapting the dam break program DAMBRK from the United States for use in the United Kingdom in 1990 (DoE 1990), Binnie Black & Veatch have carried out analyses of 150 reservoirs. The results of these studies have been used to compare

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extent of damage to various parameters so that the dominant parameters could be identified and some form of rough relationships established.

SUMMARY OF RESULTS

Results from some of the many studies carried out by Binnie Black & Veatch are summarised in Table 1. The symbol > indicates that the length of destruction or damage exceeded the modelled distance downstream of the dam. The results indicate the extent of destruction that can be caused by the catastrophic failure of a dam.

Table 1. Length of Total Destruction and Partial Structural Damage for reservoirs of capacity greater than 5 million m^3

Dam	Capacity at TWL	Dam height (max.)	Dam type 1=Embankment 2=Gravity	Length of total destruction	Length of partial structural damage
	(Mm3)	(m)		(km)	(km)
1	124.00	37	1	>24	>24
2	84.84	33	1	>52	>52
3	73.97	58	1	>85	>85
4	59.70	26	2	82	88
5	57.76	23	1	8	14
6	40.71	20	1	>15	>15
7	27.72	18	1	6	7
8	25.72	15	1	>5	>5
9	20.30	17	1	2	3
10	17.55	19	1	9	10
11	13.40	39	1	17	38
12	12.01	31	1	>31	>31
13	9.47	36	2	55	67
14	8.99	37	2	55	95
15	6.79	19	1	>11	>11
16	6.65	29	1	10	42
17	6.05	20	1	22	25
18	5.10	25	1	>16	>16

CASE STUDIES

To illustrate the extent of destruction and how the area of inundation can be affected by valley conditions, the damage predictions for three reservoirs in the United Kingdom are described. For reasons of client confidentiality we are unable to identify the reservoirs. We must also stress that there is no concern over the condition and safety of the dams. Two of the reservoirs are impounded by gravity dams and the third is impounded by an embankment dam.

Reservoir 1

The reservoir has a capacity of 41 million m^3 (No.6 in Table 1) and is impounded by a 10 m high concrete gravity dam faced with coursed masonry. The overflow provisions are designed for the Probable Maximum Flood and the 'least unlikely' mode of failure would be sliding of the dam as a result of build-up of excessive uplift pressures within the foundation. Using the DAMBRK program for the predicted outflow hydrograph through the breach, areas of inundation damage, partial structural damage and total destruction in the valley downstream were estimated.

The peak flow 1 km downstream of the reservoir could be 12 500 m^3/s. The peak flow could be halved after 16 km and reduced to under 1 000 m^3/s about 24 km downstream of the dam. The time for the peak of the flood to travel 6 km could be about 2 hours. Maximum flood depths could be 14 m just downstream of the dam and 10 m at a distance of 11 km downstream. Maximum velocities could be up to 8 m/s.

The zone of total destruction could extend for 13 km and seriously affect a small town. The extent would be greatly truncated as, at the confluence with another river, the valley broadens very significantly and contains a large natural lake. Downstream of the lake, the valley narrows again and there is another short zone of total destruction.

A total of 159 houses, 9 larger buildings, 2 farms and a hospital are within the zone of total destruction. Research carried out by the US Bureau of Reclamation (USBR,1986) suggests that 1.5 hours is a critical minimum time for effective evacuation. Assuming that immediate warning could be given of the breach and using standard USBR procedures, the average loss of life could be 32 people. If there was no warning the death toll could rise to in excess of 300.

Reservoir 2

This reservoir (No.4 in Table 1) is impounded by a 26 m high gravity dam of cyclopean masonry structure, comprising large stones bedded on and infilled with cement mortar. The reservoir has a capacity of 60 million m^3. It is 7.5 km long and only 0.8 km wide. This shape affects the discharge hydrograph that must be applied to the breach. The 'least unlikely' mode of failure is as a result of a build-up of pressure within the dam.

The peak flow from the breach is estimated to be 4500 m^3/s. This reduces slowly and the peak flow could still be in excess of 3000 m^3/s some 44 km downstream of the dam and could occur 5 hours after the breach. Maximum velocities could vary between 5 m/s and 7.5 m/s depending on the topography. The peak velocity is estimated to be 9 m/s. Maximum flood depths also vary with the topography and could be in the range of 4 to 7 m.

The valley broadens and narrows along its length and the zones of total destruction would be fragmented. The initial zone of total destruction extends for 37 km and the end of the final zone is 82 km downstream of the dam.

Reservoir 3

The first two examples described above are located in highland Britain. The third is in lowland Britain. Here the valleys are generally broader and flatter than the narrow steep valleys of highland areas. This has a significant effect on the extent of damage. In narrow steeply sided valleys the extent and amount of partial structural damage would be little greater than that of total destruction. In lowland valleys the converse is often true.

The third reservoir (No.5 in Table 1) has a storage volume of 58 million m^3 and is impounded by a 23 m high embankment dam with clay core and clay shoulders. It is a pumped storage reservoir and the overflow provisions are designed for the Probable Maximum Flood. The 'least unlikely' mode of failure is internal erosion adjacent to the draw-off tunnel leading to a breach.

The peak outflow from the breach is estimated to be about 9500 m^3/s. After 7 km the peak flow could have reduced to 6500 m^3/s. At this point the valley in which the reservoir is constructed joins a larger valley and as the water slows the peak discharge could reduce to 4000 m^3/s. Flood depths could be in the range 4 m to 6 m, although for the first kilometre below the dam the depth could be up to 11 m. Velocities are lower than for highland valleys and could be expected not to exceed 5 m/s.

The zone of total destruction extends for just 8 km downstream of the dam and the zone of partial structural damage for a further 6 km. Although there are 720 buildings at risk of partial structural damage, only 46 are within the zone of total destruction. The estimate of the population at risk is about 450, giving a potential loss of life of 40 people.

ASSESSMENT OF RESULTS

The results of hazard analyses carried out for reservoirs with a storage volume of over 1 million m^3 capacity have been analysed to establish any relationships between the extent of total destruction or partial structural damage and reservoir characteristics. The reservoir characteristics investigated were dam height, storage capacity, slope and shape of the valley downstream.

The results of this investigation revealed that the best correlation that can be established was between dam height and length of partial structural damage downstream of the dam. Figure 1 shows an estimated best fit line through a wide scatter of points of dam height related to distance downstream of partial structural damage, but also two envelope lines providing a band within which the likely resulting length of partial structural damage could lie. For example, if a dam of height 30m should fail, the length of partial destruction is likely to be between 10km and 45km. However, the scatter of points and the outliers on the graph show that other factors need to be considered when assessing the potential extent of damage, including the volume of the reservoir and the valley shape and gradient.

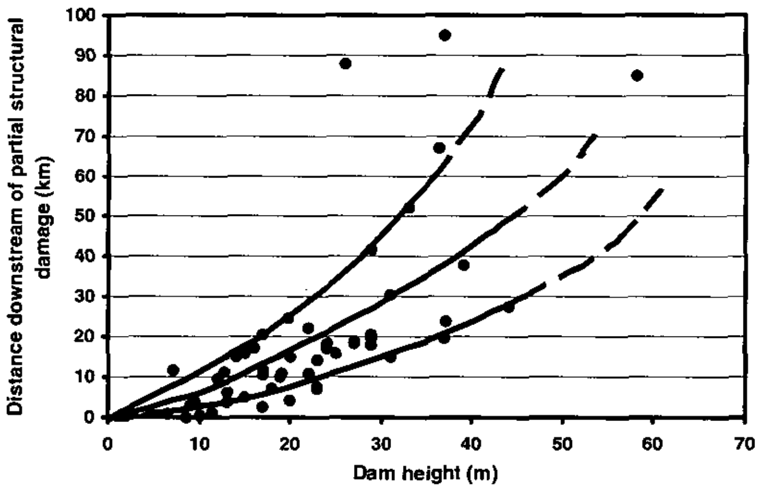

Fig. 1. Dam height plotted against distance downstream of partial structural damage

Dam height and reservoir capacity are generally related, but plotting reservoir capacity against length of partial structural damage revealed an even wider scatter of points. Although Figure 2 shows two lines enveloping the majority of points, these are tenuous and show a wide range of possible resulting lengths of partial structural damage. This further confirms that other factors are involved in predicting the extent of damage downstream of a dam failure.

An analysis of valley slope with extent of damage downstream again showed little correlation. Figure 3 shows valley slope plotted against length of partial structural damage. If outliers in the data set are ignored, a tenuous relationship of increasing length of damage with increasing valley slope can be established.

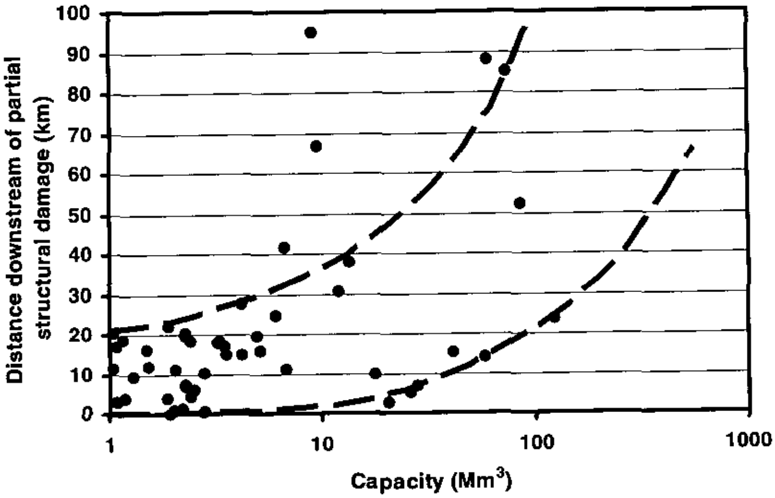

Fig. 2. Capacity plotted against distance downstream of partial structural damage

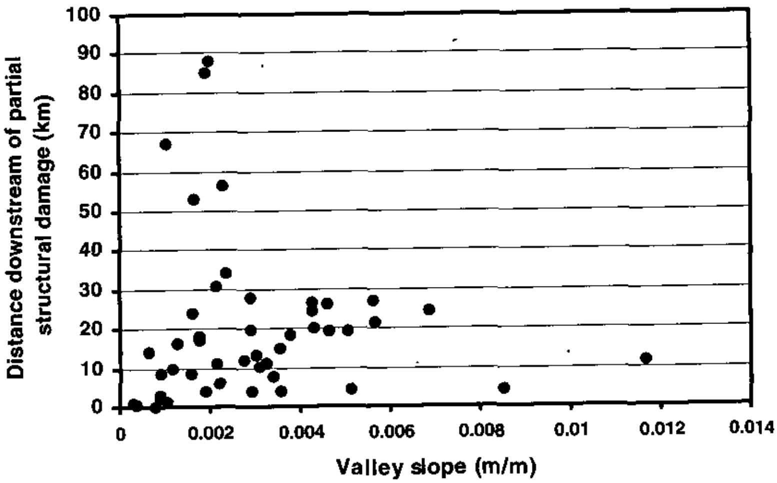

Fig. 3. Valley slope plotted against distance downstream of partial structural damage

Valley shape was investigated together with valley slope, but was difficult to assess because shape usually varies along the downstream length. General statements can be made such as that the extent of damage

CADAM: A European Concerted Action Project on Dambreak Modelling

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SYNOPSIS. The *Concerted Action project on Dambreak Modelling* involved participants from 10 different countries across Europe and ran between Feb 98 and. Jan 2000. The aim of the project was to review dambreak modelling codes and practice from first principles through to application, to try and identify modelling best practice, effectiveness of codes and research needs. Topics covered included the analysis and modelling of flood wave propagation, breaching of embankments and dambreak sediment effects. The programme of study was such that the performance of modelling codes were compared against progressively more complex conditions from simple flume tests through physical models of real valleys and finally to a real dam-break test case (the Malpasset failure). The study conclusions are presented in a final project report, published by both the EC and the IAHR. This paper provides a brief summary of the key issues identified.

INTRODUCTION

The first legislation in Europe for dam-break risk analysis was presented in France in 1968, following the 1959 Malpasset dam-break that was responsible for more than 400 injuries. Since then, and especially more recently, many European countries have established legal requirements. However the techniques applied when undertaking the specified work can vary greatly. The perception of risks related to natural or industrial disasters has also evolved, leading to public demand for higher standards of safety and risk assessment studies. Considering the relatively high mean population density within Europe, a dam-break incident could result in considerable injury and damage; efficient emergency planning is therefore essential to avoid or minimise potential impacts.

Dam-break analyses therefore play an essential role when considering reservoir safety, both for developing emergency plans for existing structures and in focussing planning issues for new ones. The rapid and continuing development of computing power and techniques during the last 15 years has allowed significant advances in the numerical modelling techniques that may be applied to dam-break analysis.

CADAM was funded by the European Commission as a Concerted Action Programme that ran for a period of two years from February 1998. Under

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these terms, funding was provided only to pay for travel and subsistence costs for meetings, and for project co-ordination. All work undertaken during the study was therefore achieved through the integration of existing university and national research projects. HR Wallingford co-ordinated the project, with additional financial support from the DETR.

The project continued work started by the IAHR Working Group (established by Alain Petitjean following the IAHR Congress in 1995) and had the following aims:

- The exchange of dam-break modelling information between participants, with a special emphasis on the links between Universities, Research Organisations and Industry.
- To promote the comparison of numerical dam-break models and modelling procedures with analytical, experimental and field data.
- To promote the comparison and validation of software packages developed or used by the participants.
- To define and promote co-operative research.

These aims were pursued through a number of objectives:

- To establish needs of industry, considering a means of identifying dam owners, operators, inspectors etc. throughout Europe.
- To link research with industry needs - encourage participation; distribute newsletters to dam owners and other interested parties.
- To create a database of test cases (analytical, experimental, real life) available for reference.
- To establish the state-of-the-art guidelines and current best practices for dam-break modelling within the technical scope of the Concerted Action. This leads towards establishing recommended European standard methods, procedures and practices for dam-break assessments.
- To determine future RTD requirements.

CONCERTED ACTION PROGRAMME

The project involved participants from over 10 different countries across Europe. All member states were encouraged to participate, with attendance at the programme workshops open to all and to expert meetings by invitation. Also, links with other experts around the world were welcomed to ensure that state-of-the-art techniques and practices were considered. The programme of meetings planned for the presentation, discussion and dissemination of results and information were as follows:

Meeting 1 Wallingford, UK. 2/3rd March 98(Expert Meeting)

A review of test cases and modelling work undertaken by the group up to the start of CADAM, followed by a review of test cases considered during the previous 6 months. Typical test cases included flood wave propagation around bends, over obstructions and spreading on a flat surface (physical modelling undertaken in laboratory flumes).

Meeting 2 Munich, Germany 8/9th October 98(Open Workshop)
Presentations and discussion on the current state of the art in breach formation modelling and sediment transport during dam-break events.

Meeting 3 Milan, Italy May6/7th 99(Expert Meeting)
Comparison and analysis of numerical model performance against a physical model of a real valley (Toce River, Italy) plus an update on breach modelling research.

Meeting 4 Zaragoza, Spain Nov 18/19th 99(Symposium)
Comparison and analysis of numerical model performance against a real failure test case (Malpasset failure) plus a presentation of the results and conclusions drawn from the work of the *Concerted Action* over the two-year study period.

MODELLING COMPARISONS

The programme of tests progressed from simple conditions to test the basic numerical stability of modelling codes, through to a real dambreak test case – the Malpasset failure. The aim of the programme was to progressively increase the complexity of the modelling, and in doing so to try and identify which models performed best under which conditions. Both breach models and flood routing models were considered during the project.

Flood Routing Analysis

Numerical Models The models applied in CADAM ranged from commercially available software to codes developed 'in-house' by the various participants. Participants ranged from 'End User' organisations such as ENEL (Italy), EDF (France) and Vattenfall (Sweden) to consultancy companies and universities undertaking research in this field. Many of the European participants codes were 2D codes based on depth averaged Saint Venant shallow water equations, but applying different numerical schemes utilising different orders of accuracy and source term implementations. Codes more familiar to the UK market included DAMBRK and ISIS (1D model - implicit finite difference Preissmann Scheme).

Analytical Tests Initial test cases were relatively simple, with analytical solutions against which the numerical modelling results could be compared. These tests included:

- Flume with vertical sides, varying bed level and width. No flow – water at rest.
- Flume with (submerged) rectangular shaped bump. Steady flow conditions.
- Dam-break flow along horizontal, rectangular flume with a dry bed. No friction used.
- Dam-break flow along horizontal, rectangular flume with a wet bed. No friction used.

- Dam-break flow along horizontal, rectangular flume with a dry bed. Friction used.

These tests were designed to create and expose numerical 'difficulties' including shock waves, dry fronts, source terms, numerical diffusion and *sonic points*. Results were presented and discussed at the 2nd IAHR Working Group meeting held in Lisbon, Nov. 96 (EDF, 1997).

Flume Tests Following the analytical tests, a series of more complex tests were devised for which physical models provided data (Fig 1). The aim was to check the ability of the numerical codes to handle firstly, specific 2D features, and then important source terms. These tests were:

- Dam-break wave along a rectangular flume with 90° bend to the left.
- Dam-break wave along a rectangular flume with a symmetrical *channel constriction.*
- Dam-break wave along a rectangular flume expanding onto a wider channel (asymmetrical).
- Dam-break wave along a rectangular flume with 45° bend to the left.
- Dam-break wave along a rectangular flume with a triangular (weir type) obstruction to flow.

The first three test cases were presented and discussed at the 3rd IAHR working Group meeting in Brussels (UCL, June 97) and the remaining two at the 1st CADAM meeting in Wallingford (CADAM, March 98).

Photos courtesy of Sandra Soares, Université Catholique de Louvain, Belgium

Fig. 1 Shock waves generated from 'dambreak' flow around a 45° bend.

'Real Valley' Physical Model A model of the Toce River in Italy was used for the analysis of model performance against 'real valley' conditions (Fig 2). The advantage of comparing the numerical models against a physical model, at this stage in the project, was that the model data would not include any effects from sediment or debris that might mask features of numerical model performance.

The model was provided by ENEL and, at a scale of 1:100, represented a 5km stretch of the Toce River, downstream of a large reservoir. An automated valve controlled flow in the model such that a flood hydrograph simulating partial or total dam failure could be simulated. Features within the downstream valley included a storage reservoir, barrage, bridges and villages (Fig 3).

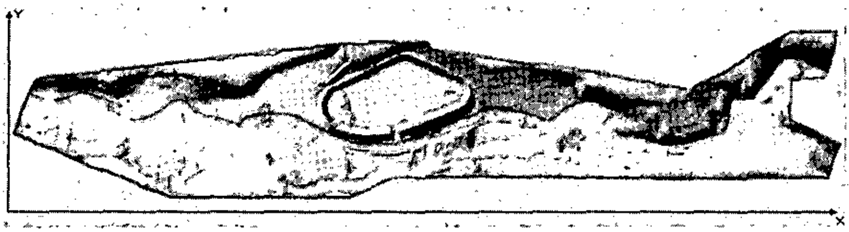

Fig. 2 Digital plan model showing the Toce River model

Photos courtesy of Prof JM Hiver, Université Libre de Bruxelles, Belgium

Fig. 3 Bridge structure on the Toce model and a dambreak flow simulation

Real Failure Test Case – The Malpasset Failure The Malpasset Dam failure was selected as the real case study for the project since:

- The data was readily available through EDF (France)
- It offered a different data set to the commonly used Teton failure
- In addition to field observations for peak flood levels there were also timings for the failure of three power supply centres
- Data from a physical model study undertaken by EDF in 1964 (Scale 1:400) was also available

Modelling focused on the first 15km of valley downstream of the dam for which there was field data to compare against model predictions. This stretch of valley included features such as steep sided valleys, side valleys / tributaries and bridge / road crossings.

Breach Analysis

One of the four CADAM meetings was devoted to breach formation and sediment and debris effects. A comparison of the performance of breach models was undertaken using two test cases. The first test case was based on physical modelling work performed at the Federal Armed Forces University in Munich. The simulation tested was for a homogeneous embankment represented by a physical model approximately 30cm high. The second test case was based on data from the Finnish Environment Institute, derived from past collaborative research work undertaken with the Chinese. This work analysed the failure of an embankment dam some 5.6m high (Loukola et al, 1993).

SELECTED RESEARCH FINDINGS

The following sections highlight some selected issues identified during the CADAM project:

Flood Routing

It was not possible to uniquely define a single best model or single best approach for dambreak modelling within the scope of the study since the various models and approaches performed differently under varying test conditions. Equally, a more in-depth analysis of the significant quantities of test data collected is now required to understand some of the performance features identified. It was possible, however, to identify some recurring features and issues that should be considered when defining best practice for dambreak modelling. These include (in no particular order):

Wave Arrival time The speed of propagation of the flood wave is an important component of dambreak modelling since it allows emergency planners to identify when inundation of a particular area may be expected. It was found that 1D and 2D models failed to reproduce this accurately and that 1D models consistently under predicted the time (i.e. flood wave propagated too quickly) and 2D models consistently over predicted this time (i.e. flood wave propagated too slowly).

Figure 4 shows wave travel times for one set of test data. The 1D models (left) show a scatter of results, probably due to the range of numerical methods applied. Results shown spread across the observed data. Later tests showed a tendency to under predict the wave speed. Many of the 2D models used similar numerical methods perhaps resulting in the tight clustering of data, however the results here (and repeated later) show a consistent over prediction of wave speed (right).

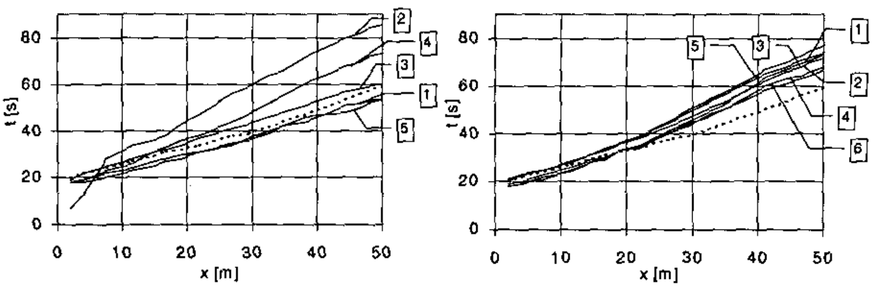

Fig. 4 Summary of flood wave travel times for 1D models (left) and 2D models (right)

Flood wave speed is poorly modelled – 1D models over predict wave speed, 2D models under predict wave speed.

Use of 1D or 2D Models It was found that the 1D models performed well in comparison with the 2D models for many of the test cases considered. It is clear, however, that there are instances where a 2D model predicts conditions more effectively than a 1D model. In these situations a 2D model should be used or the 1D model should be constructed to allow for 2D effects. These situations are where flow is predominantly 2 dimensional and include flow spreading across large flat areas (coastal plains, valley confluences etc), dead storage areas within valleys and highly meandering valleys. Simulation of these features using a 1D model will require experienced identification of flow features, reduction of flow cross section and addition of headloss along the channel.

A promising development that may offer a significant increase in model accuracy from a 1D model but without the heavy data processing requirements of a 2D model, is the use of a 'patched' model. This is where areas of 2D flow may be modelled using a 2D approach 'patched' within a 1D model (Fig 5). This technique requires further development and validation, but seems to offer significant potential.

In relation to the additional effort required for 2D modelling, 1D models perform well but cannot be relied upon to simulate truly 2D flow conditions. An experienced modeller is required to apply a 1D model correctly to simulate some 2D flow conditions.

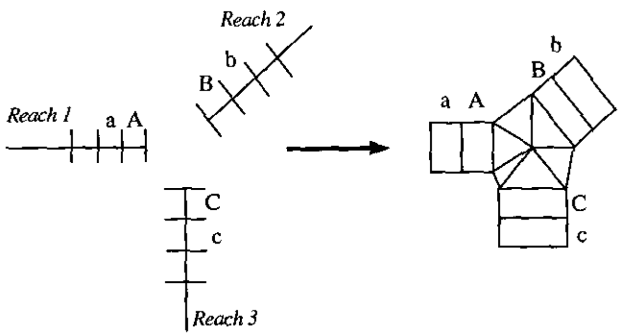

Fig. 5 2D patches within a 1D model to improve model accuracy whilst limiting processing requirements

Modeller Assumptions It was clear just from the test cases undertaken (and also supported by an independent study undertaken by the USBR (Graham, 1998)) that the assumptions made by modellers in setting up their models, can significantly affect the results produced. Graham (1998) deliberately gave identical topographic and structure data to two dambreak modellers and asked them to undertake independent dambreak studies for the same site. The results varied significantly, and particularly in terms of flood wave arrival times. Variations in breach formation, valley roughness and simulation of structures contributed to the differences.

Modelling assumptions can significantly affect the model results. Different modellers may produce different results for an identical study. Care should be taken to ensure only experienced modellers are used and that all aspects and assumptions made are considered.

Debris and Sediment Effects It is unusual to find debris and sediment effects considered in detail for dambreak studies but it is clear from case studies and ongoing research that the movement of sediment and debris under dambreak conditions can be extreme and will significantly affect topography, which in turn affects potential flood levels. Case studies in the US have shown bed level variations in the order of 5 to 10m.

Debris and sediment effects can have a significant impact on flood water levels and should be considered during a dambreak study. These effects offer a significant source of error in flood prediction.

Mesh Convergence The definition of a model grid in 2D models, or the spacing of cross sections in 1D models, can significantly affect the predicted results. Models should be checked as a matter of routine to ensure that the grid spacing is appropriate for the conditions modelled and that further refinement does not significantly change the modelling results.

Mesh or section spacing should be routinely checked when modelling

Breach Modelling

Existing models are very limited in their ability to reliably predict discharge and the time of formation of breaches. Figure 6 below shows a typical scatter of modelling results found for the CADAM test cases. Models comprised a range of university and commercial codes, including the NWS BREACH code.

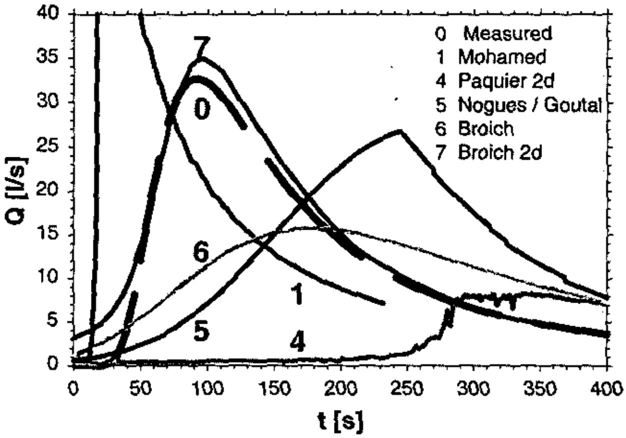

Fig. 6 Typical scatter of model results trying to predict breach formation

It is also clear that there is little guidance available on failure mechanisms of structures, which adds to the uncertainty of conditions assumed by modellers.

There are no existing breach models that can reliably predict breach formation through embankments. Discharge prediction may be within an order of magnitude, whilst the time of breach formation is even worse. Prediction of breach formation time due to a piping failure is not yet possible.

The NWS BREACH model is only calibrated against a very limited data set. The author (Danny Fread) confirmed that it is based on approximately 5 data sets.

Existing breach models should be used with caution and as an indicative tool only. A range of parameters and conditions should be modelled to assess model performance and results generated.

There is a clear need to develop more reliable predictive tools that are based on a combination of soil mechanics and hydraulic theory.

End User Needs

Throughout CADAM, the project focused on the practical needs of end users. Attempts were made to quantify a number of issues, both by end users and academic researchers alike. The initial response to the question of what accuracy models could offer and what was required from end users was limited. Without agreement on such issues it is impossible to determine whether existing modelling tools are sufficient or not! This perhaps reflects the current uncertainty of end users with regards to legislation and appropriate safety measures and of modeller's appreciation of processes and data accuracy. It was suggested that the level of modelling accuracy should be appropriate for the site in question (i.e. more detailed for urban areas). Water level prediction should be appropriate to the mapping required, and the mapping should be at a scale sufficient for emergency planning use (i.e. to identify flood levels in relation to individual properties). This suggests an inundation mapping scale of approximately 1:5000 for developed areas.

Inundation maps should be undertaken at a scale appropriate for use in emergency planning. For urban areas it is suggested that this should be at a scale of 1:5000 or greater. Modelling accuracy should be consistent with the detail of mapping required (i.e. for the end user of the data)

SOME ADDITIONAL POINTS ON DAMBRK_UK and BREACH

During the project, work undertaken by HR Wallingford identified a number of potential problems with the DAMBRK_UK and BREACH software packages.

Under certain conditions, it was found that the DAMBRK_UK package created artificial flow volume during the running of a simulation. For the limited conditions investigated this volume error was found to be as high as +13% (Mohamed (1998)). This error tended to be on the positive side, meaning that the flood levels predicted would be pessimistic. It may be assumed that similar errors exist in the original DAMBRK code. It was noted that model performance varied between DAMBRK, FLDWAV (released to replace DAMBRK) and BOSS DAMBRK. A detailed investigation into the magnitude and implications of these errors has not yet been undertaken.

Similarly, problems were also found with the BREACH software package. Under some conditions, predicted flood hydrographs were found to vary significantly with only minor modifications to input parameters. This erratic behaviour was discovered when considering the differences between piping and overtopping failure, by tending the piping location towards the crest of the dam. Erratic performance was also confirmed by a number of other CADAM members.

Figure 7 shows a plot of flood hydrographs generated by BREACH for an overtopping failure and a piping failure located just 3cm below the crest. Logic dictates that these hydrographs should be very similar however the results show a significant difference in both the volume of the hydrograph as well as the timing.

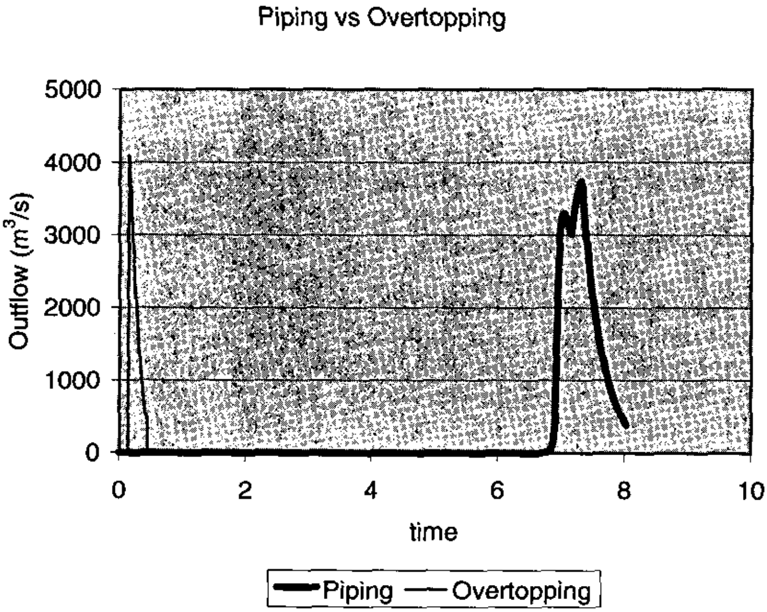

Fig. 7 Different outflow hydrographs produced by breach for an overtopping failure and a piping failure located 3cm below the crest.

CONCLUSIONS

The CADAM project has reviewed dambreak modelling codes and practice and identified a range of issues relating to model performance and accuracy. A number of these issues have been outlined above. When considering all aspects contributing to a dambreak study it was found that breach formation prediction, debris and sediment effects and modeller assumptions contribute greatly to potential prediction errors.

Full details of all findings and conclusions may be found in the project report which has been published by both the EC and the IAHR, and which may also be found on the project website.

ACKNOWLEDGEMENTS

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Early siphon spillways

J C ACKERS, Binnie Black & Veatch, UK

SYNOPSIS This paper gives an account of the early history of siphons, focusing particularly on their adoption and development by British engineers and their use for reservoir spillways.

ORIGINS

Siphons were invented over 2000 years ago. *Encyclopaedia Britannica* cites an example of a siphonic water clock documented by Ctesibius of Alexandria in 250BC. The use of siphons in reservoir spillways came somewhat later, but perhaps by the 1840s according to one reference. Dr W L Lowe-Brown (see Naylor, 1937) 'found an interesting early reference to them in an article dated 1843 in which the theory and advantages of siphon spillways were given clearly and in some detail'. The author of that article, Mr Robert Mallett, was reported to be enthusiastic, in spite of the siphons which he described being small. What is not certain in Lowe-Brown's account is whether Mallet was describing actual siphons or only the concept.

According to Prof A H Gibson *et al* (1931), reporting model tests of a number of siphon designs at Manchester University, siphon spillways were first used in Europe in about 1870. The early types were not automatic, requiring an ejector to prime them, but the development of self-priming siphons apparently occurred at about the same time in Italy and Prussia. Gibson reported these to be of 'comparatively small dimensions, and not very efficient in operation'. It appears that the technology was subsequently adopted in France, then in the USA, where a number of technical papers and reports appeared between 1910 and 1926..

In the discussions on the 1931 Gibson *et al* paper, Mr A B Buckley reported that he had visited several siphons in Italy and had obtained a list of 80, with operating heads of between 1m and 12m, which had been constructed there before about 1915. He understood that the largest such installation was in Norway, with a discharge capacity of 600 m^3/s, but unfortunately he provided no further information about it.

Mr G H Stickney, in his paper to the ASCE in 1922, reported that he had designed the first siphon spillway to be built in the USA, in 1909, and had obtained a patent in 1911. However, in discussions on the paper, Mr W P Creager, the author of a well known textbook on dam engineering, claimed the

Dams 2000, Thomas Telford, London, 2000.

credit for the first siphon spillway built in the USA for himself. Stickney listed at least 15 separate siphon spillways built in the USA up to 1922, mostly in association with canals and hydropower plants.

PRINCIPLES OF SIPHON OPERATION

Siphons, as used in spillways, generally have the following modes of flow, at progressively increasing heads in the upstream pond or forebay:

Weir flow: this occurs over the internal crest of the siphon at low heads and discharges, when the barrel is open to the atmosphere at the inlet and/or outlet.

Sub-atmospheric weir flow, which occurs after the inlet and outlet have been virtually sealed against air entry and some of the air has been expelled from the outlet, reducing the pressure and raising the head over the crest to above the upstream pond level. To sustain this mode, air expelled from the outlet with the water flow is balanced by air drawn into the inlet.

Partialised flow occurs when the discharge is sufficient to entrain large quantities of air. The siphon passes a mixture of air and water, drawing in either a continuous stream of air or frequent gulps through partially or intermittently submerged air vents, or under the upstream lip. This mode, in which the water discharge through the siphon can continuously match the discharge entering the forebay, is only stable in a properly designed air-regulated siphon. In other cases, this mode is passed through rapidly en route to blackwater flow.

Blackwater flow occurs at the highest forebay levels, when the upstream lip and air vents are submerged and no air can enter the barrel.

The transition between weir flow and partialised or blackwater flow is known as 'priming'. A properly designed air-regulated siphon has stable stage/discharge characteristics, which are virtually identical on rising and falling forebay levels. Siphons of the 'make-and-break' type, on the other hand, generally exhibit hysteresis, with the forebay water level for de-priming being rather lower than the priming level. This behaviour can cause operational problems and an unacceptable risk of sudden downstream flooding.

The key components of priming and depriming are:

- controlling the entry of air to the barrel;
- sealing the outlet; and
- entraining and expelling the air.

There are several methods of achieving these, a discussion of which is beyond the scope of the present paper, but can be found in a BHRA report (Charlton, 1962), which also includes illustrations of a wide variety of examples, many taken from the early references.

THE INDIAN CONTRIBUTION

The first siphon spillway constructed to the design of a British engineer appears to have been in India. The hydropower station bypass at Renala was designed in 1922 by Mr E S Crump, apparently with the guidance of Stickney's ASCE paper.

In the correspondence on the 1931 paper by Gibson *et al*, Mr C C Inglis, who was later the first director of the Hydraulics Research Station at Wallingford, claimed that Crump's design (Figure 1) was 'by far the most efficient'. This is a justifiable claim, as it was a simple and effective air-regulated siphon, capable of self-priming through the entrainment of air in the vertical downstream leg, and of operating without hysteresis between the rising and falling stage. This was achieved by the presence of a large air vent extending to one fifth of the barrel height above the crest level.

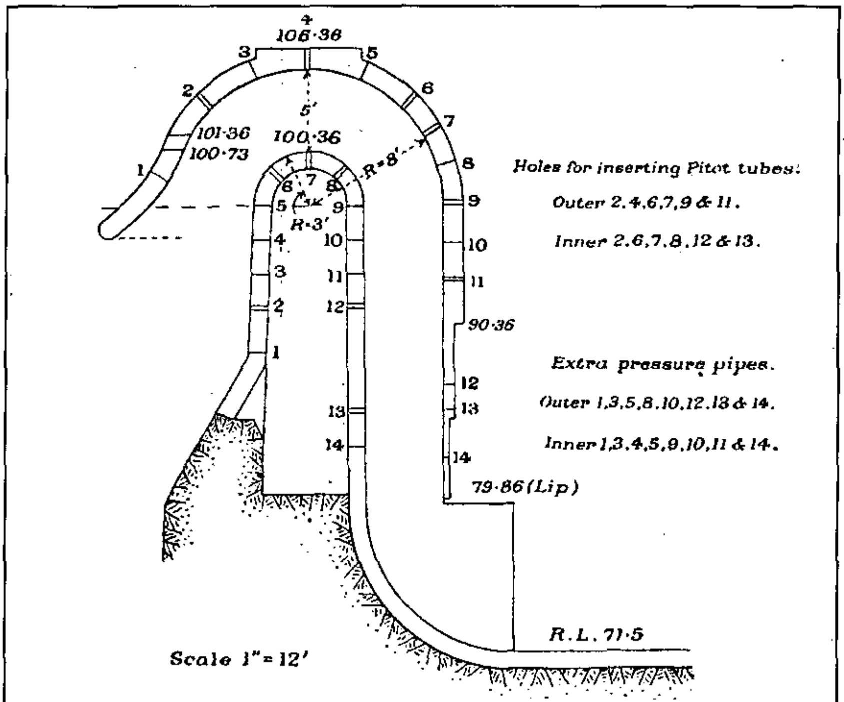

Fig. 1 Full-size model of Crump's Renala siphons (see Gibson, 1931)

Crump's achievement should not be underestimated. Although the ASCE paper (Stickney, 1922) showed how air regulation could be achieved, there was no mention of the risks of cavitation against the inside of the crest radius. Indeed, a couple of the American designs shown in the ASCE paper had very tight crest radii, so may have been vulnerable to cavitation. Crump correctly identified the occurrence of free vortex flow around the crest curve and designed the Renala siphons accordingly, to avoid the risk of cavitation.

Crump is now probably best known for originating the triangular profile of weir which is named after him, and was described as 'the most brilliantly original thinker of all the hydraulicians produced by the PWD in India in recent years' (Inglis, 1949).

In the period 1925–29 Crump and his colleagues carried out experiments on two models of the Renala siphons, one to a scale of 1:8 and the other at full size, with the objectives of determining pressures throughout the structure and the factors affecting the discharge characteristics, including the priming heads. By the use of two models, they could also investigate scaling effects, which showed that priming occurred at a lesser relative head in the prototype than in the model: 1.15ft in the full-scale model, compared with the equivalent of 1.8ft in the 1:8 model (Inglis, 1949).

The Renala siphons were perhaps even more remarkable for having been constructed using reinforced brickwork in a period of only ten days (see correspondence on Davies's paper, 1927). This was no mean feat, as there were 35 barrels, each 4ft wide by 5ft high.

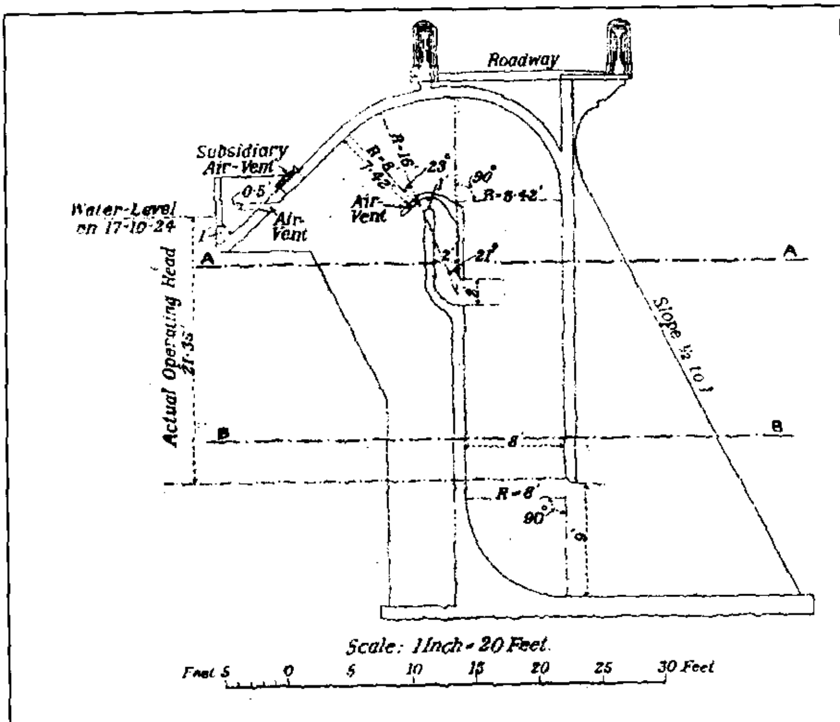

Fig. 2 Maramsilli reservoir siphon 'Type A' (Davies, 1927)

At about the same time as Crump's pioneering work, Mr Powys Davies was responsible for the design and construction of a siphon spillway for Maramsilli reservoir, completed in 1923. This is described in his paper presented to ICE in 1927 and comprised 34 barrels, to two different designs, both using integral 'baby siphons' to assist in priming. Figure 2 shows the 'Type A' design, which was used for seven batteries of four siphons. Davies reported having been 'deputed by Government to study the operation of spillways on the continent of Europe' prior to his return to India in early 1921, so had presumably come across the concept of the baby siphon there.

BRITAIN PONDERS

One of the first proposals for siphon spillways in Britain was in 1929, when parliamentary powers were sought for the addition of siphon spillways to some of the reservoirs in the Longdendale Valley. Mr W J E Binnie (in discussions on the 1931 Gibson *et al* paper) explained that these reservoirs 'had been constructed many years previously, before the intensities of floods had been much studied'. He reported that various bodies, including those 'who would have suffered very severely if the reservoirs burst, had opposed the introduction of the siphons on the ground that siphons were dangerous to construct. He did not know quite what was meant by that; but it had been maintained that they were liable to be blocked by ice; and unless there was a consensus of opinion of engineers in this country in favour of the introduction of siphons there would be great difficulty in getting the necessary powers to construct them.'

A number of well-known engineers were present at the 1927 and 1931 meetings of the Institution of Civil Engineers, and model tests had been undertaken at both Manchester University (Gibson *et al*, 1931) and Imperial College in that period. Not surprisingly, the first British siphon spillways appeared soon afterwards.

THE SCOTTISH RIVALS

The first two siphon spillways to be completed in the UK were at Dunalastair dam in November 1933 and Laggan dam in July 1934. There seems to have been intense rivalry between the proponents of these siphons. In his 1936 ICE paper on the Lochaber water-power scheme, Mr A H Naylor reported the inclusion of six siphons as part of the overall spillway provision at Laggan dam. He claimed this as 'the first instance in Great Britain of the embodiment of a large siphon spillway' in the design of a dam. This was challenged in the discussion on the paper by Mr H H Gibb, who said that the four siphons at Dunalastair, in the Tummel development of the Grampian scheme, had been in operation earlier and that two of them were larger than the Laggan siphons. Naylor acknowledged that the Dunalastair siphons had been brought into operation first, but said that the Laggan siphons were designed first, but completed later 'due to the magnitude of Laggan dam' and reasserted that they 'were the first large siphons to be used in conjunction with a large dam in Britain'.

At both Laggan and Dunalastair dams, the siphons provided about one quarter of the design flood discharge capacity assessed at the time. Figure 3 shows the design of the Laggan siphons, with a design discharge capacity of 600 ft^3/s per barrel. The key features are:

- the tapered outlet, to control the discharge and hence the minimum pressures, so as to avoid cavitation;

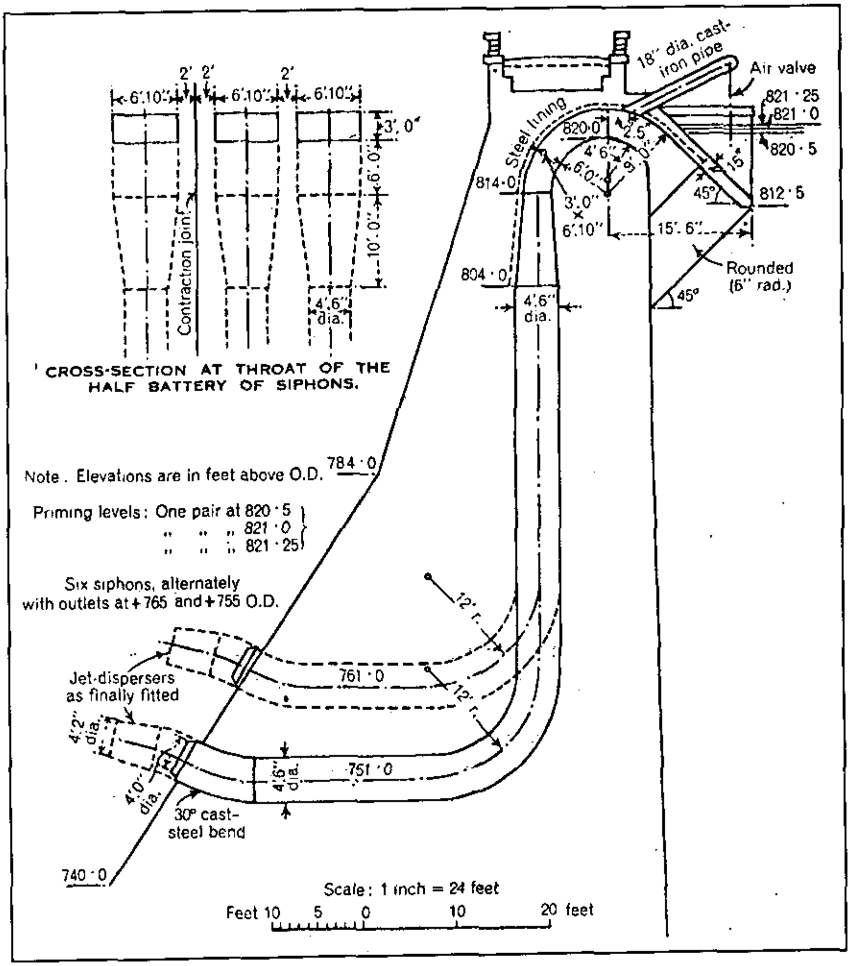

Fig. 3 Laggan dam siphons (Naylor, 1937)

- the inlet cowl, to prevent interruption of the flow by heavy wave action and to protect against vortices; and

- the use of float-operated valves to control the air venting and thus the priming and depriming levels.

BELLMOUTH SIPHONS

The Jubilee (Shing Mun) reservoir in the New Territories of Hong Kong was constructed between 1933 and 1937, including two spillways: one incorporating a battery of six siphons and the other a bellmouth. The bellmouth spillway was model tested in Hong Kong by Mr G M Binnie and described in his 1938 ICE paper, in which he also put forward the concept of combining the features, by installing siphons around the rim of a bellmouth spillway. This was included in his model testing programme (Figure 4) and shown to work successfully with a vertical shaft discharging to an almost horizontal tunnel, but was not successful with a sloping tunnel. As there was already a commitment to a sloping tunnel for Jubilee reservoir, the concept was not taken up on that occasion, but was subsequently adopted at several other reservoirs, including Batang Padang in Malaysia and Shek Pik and High Island in Hong Kong.

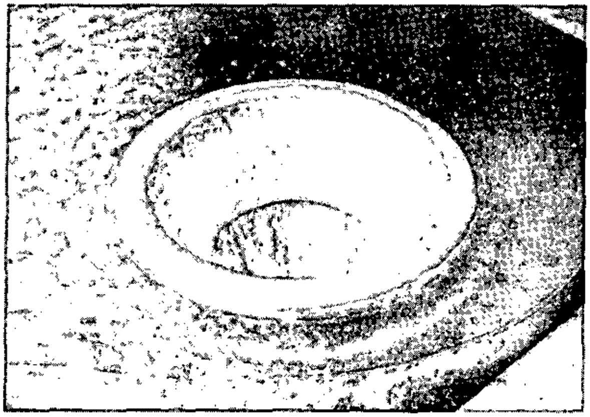

Fig. 4 Siphon bellmouth model (Binnie, 1938)

Binnie's experimental facilities in Hong Kong are of particular interest to dam engineers: 'A stream with a sufficient flow and a suitable site for an experimental hydraulic station was found in the neighbourhood, and three small dams...were constructed across it.' The upper dam was a 14ft high thin arch dam to form the storage reservoir for the tests; the middle dam a 35ft high gravity section to represent the prototype reservoir and contain the models; the lower dam a 7ft high gravity section forming a stilling basin and incorporating a measuring weir. The various models were built to scales of between 1/43.5 and 1/19, partly with the purpose of checking for scale effects in priming the siphons and air-water flow in the spillway tunnel.

RETRO-FITTING SIPHONS

Brent reservoir in north London contains what is thought to be the first case in Britain of retrofitting siphons at an earlier dam, following the first inspection under the 1930 Act by Mr W J E Binnie in February 1934. The reservoir was originally built in 1835 to supply water to the Grand Union Canal and enlarged in 1851. The spillway comprised an arched masonry weir with an effective length of 81ft, positioned near the centre of the embankment dam. Two low-level culverts were provided in the same structure, with hand-operated sluice gates.

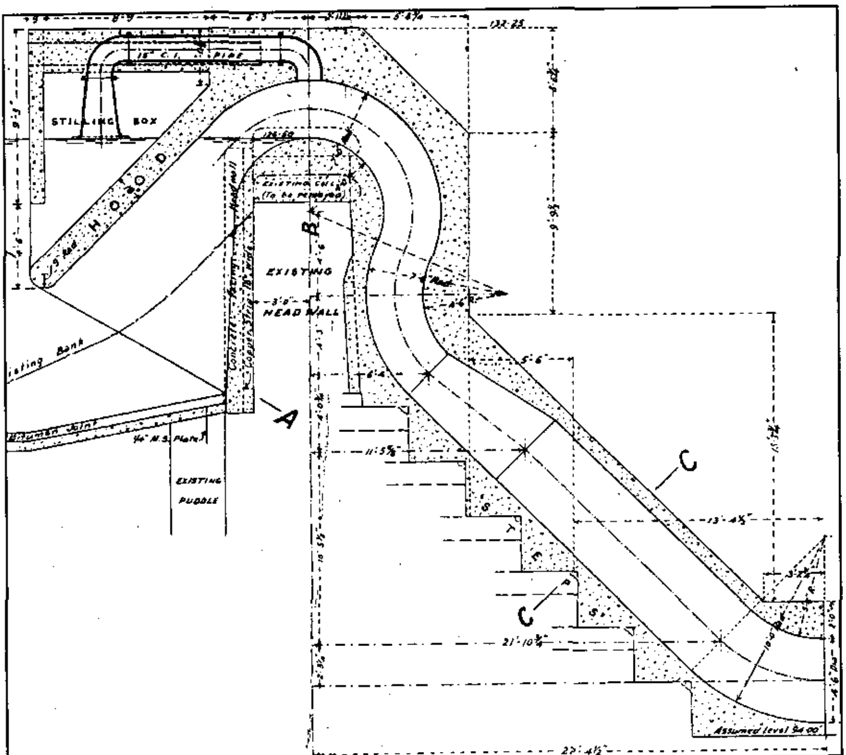

Fig. 5 Brent reservoir central siphon (Civil Engineering, 1937)

The five siphons – one central barrel and a pair of barrels on either side – were fitted into and over the old weir, positioned to avoid conflict with the low-level culverts (Civil Engineering, 1937). The design discharge capacity was 600 ft^3/s (17 m^3/s) per barrel, but model tests at Imperial College (Digby, 1972) and Newcastle University (Valentine, 1994) have shown a more realistic figure to be about 14 m^3/s.

The design (Figure 5) features an extended upstream hood to avoid problems with ice and a stilling box to reduce the effects of wave action. The central siphon has a lower crest than the other four, which have the same crest level but differing elevations of air inlet. It was probably the intention of the designer that the air inlet elevation would control both priming and depriming, maintaining the reservoir level within a narrow band when the siphons are operating. However, subsequent experience with siphons of this design, including the model tests on the Brent siphons at Imperial College and Newcastle University, have shown that they operate as 'make-and-break' siphons. They prime by the combined effect of:

- the rising water level closing the entrance to the air vent pipe which is connected to the crown; and
- a curtain of water forming across the S-bend in the downstream leg of the barrel, sealing the outlet against air entry, entraining air in the water flow and carrying it out of the barrel, thereby lowering internal pressures and increasing the water flow.

The air vent pipes are elevated between 0.04m and 0.33m above the crest, but still higher upstream heads are needed to initiate priming through air entrainment, with the result that the four higher siphons are likely to prime at the same water level, or nearly so. Upon priming, the siphons run straight up to blackwater, resulting in a rapid increase in downstream flood flows.

Depriming on falling headwater levels occurs when the air vent pipe is re-exposed. As this happens at a lower level than priming, the rating curve exhibits hysteresis. Consideration has been given since the 1960s to modifying these siphons so that they are properly air-regulated, in order to avoid sudden increases in downstream flows and overcome the hysteresis.

SIPHONS TODAY

Siphons remain an effective method of discharging floodwater from reservoirs with only a small rise in flood level, provided that proper attention is given to a number of design issues, including:

- effective and reliable priming and depriming;
- the use of air-regulation, to avoid hysteresis and flow instabilities associated with make-and-break designs; and
- effective control of operating head, to ensure that the discharge capacity and minimum pressure are controlled, to avoid cavitation.

The use of air regulation and careful design to achieve a progressive increase in stage with discharge should also eliminate potential problems from sudden increases in downstream discharge.

Probably the greatest discouragement to the use of siphons arises from the combined effects of standards of acceptable risk, flood severity, hydrological uncertainty and advances in hydrological science, which have generally conspired to increase the required design discharges for reservoir spillways over the years. The ultimate blackwater discharge capacity of a given siphon is proportional to the square root of the operating head (which is the difference between the reservoir water level and the elevation of either the siphon outlet or the tailwater, depending on the design) and is also governed by cavitation considerations. There is therefore little scope for increasing the discharge capacity of a siphon spillway by raising the available dam freeboard, or for upgrading a siphon spillway without major reconstruction.

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Dam innovation – a selection of innovative features and curiosities

J C ACKERS, Binnie Black & Veatch, UK
C W SCOTT, Binnie Black & Veatch, UK

SYNOPSIS This paper highlights a number of case histories of dams with unusual or even curious features:

- the cascade of reservoirs, in which the upstream reservoir has the lower water level;
- tipping flood gates;
- the dams with corrugated iron wavewalls;
- the dam with a highway crash barrier as wave protection; and
- the valve tower which doubles as a chimney.

UPHILL FLOWING WATER IN LANCASHIRE?
The Worthington group comprises three reservoirs, Adlington, Arley and Worthington, built between 1850 and 1870 to supply water to Wigan. The layout of the reservoirs is shown in Figure 1. The reservoirs impound runoff from Buckow Brook and are constructed within the valley of the River Douglas which, at the time of construction, was heavily polluted. The river was therefore diverted through a tunnel, rejoining its original course downstream of Worthington Reservoir.

Sir Robert Rawlinson engineered the reservoirs, which were constructed under powers obtained from Parliament under the Wigan Waterworks Act of 1853, the Local Government Supplemental Act of 1858 and the Wigan Waterworks Act of 1860. Under the 1853 Act, the Board of Health were empowered to divert the course of the Douglas River and construct two embankment dams in the valley forming a reservoir approximately covering the area currently occupied by Arley and Adlington reservoirs.

All went well until, in 1858, local colliery owners objected to the reservoir construction north of a fault approximately aligned with Adlington dam. A substantial redesign resulted which required a further Act of Parliament. Interim powers were incorporated into the 1858 Local Government Supplementary Act. The revised layout involved moving Arley dam downstream and the construction of Adlington dam. Flow from the Buckow Brook not required for the reservoirs was passed to the Douglas tunnel by an overflow culvert.

Dams 2000, Thomas Telford, London, 2000.

The 1860 Act granted powers for the construction of Worthington reservoir. The top water level of both reservoirs is 66.06m OD.

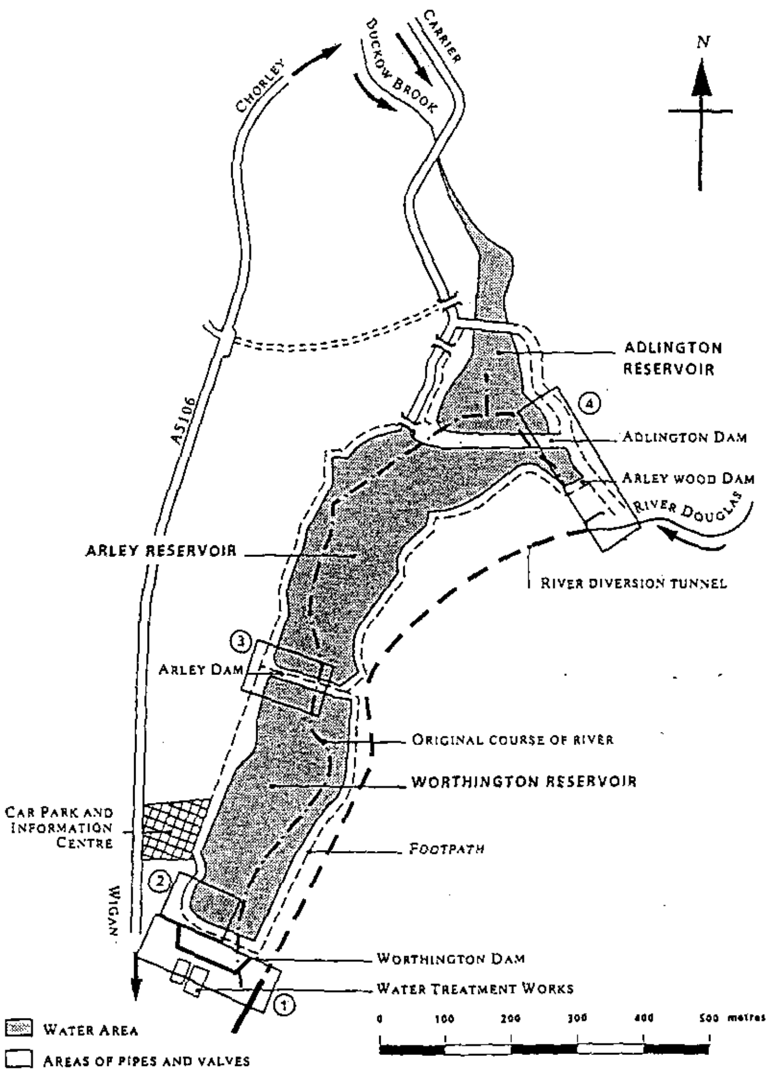

Fig. 1 Layout of Worthington reservoirs (North West Water)

Adlington reservoir, which has a top water level of 61.65m OD, was created at a later date by raising the entrance to the overflow culvert. The water from Adlington is used to supplement the required compensation flow. An iron safety railing with elaborate corner posts, which gave rise to the name 'bedstead overflow', was installed at the top of the raised overflow. This arrangement was later replaced with the metal cage shown on Figure 2, but the name has been retained.

Water is brought to Arley and Worthington reservoirs, which have a very small direct catchment, from Buckow Brook, via a trapezoidal channel called the Carrier, which bypasses Adlington reservoir at a higher elevation. Arley and Worthington are interconnected via a two-way spillway in the dividing embankment. In addition the Arley outlet tower allows transfers between Arley and Worthington. The spillway from Arley reservoir is at its upstream end and discharges to the River Douglas just upstream of the tunnel.

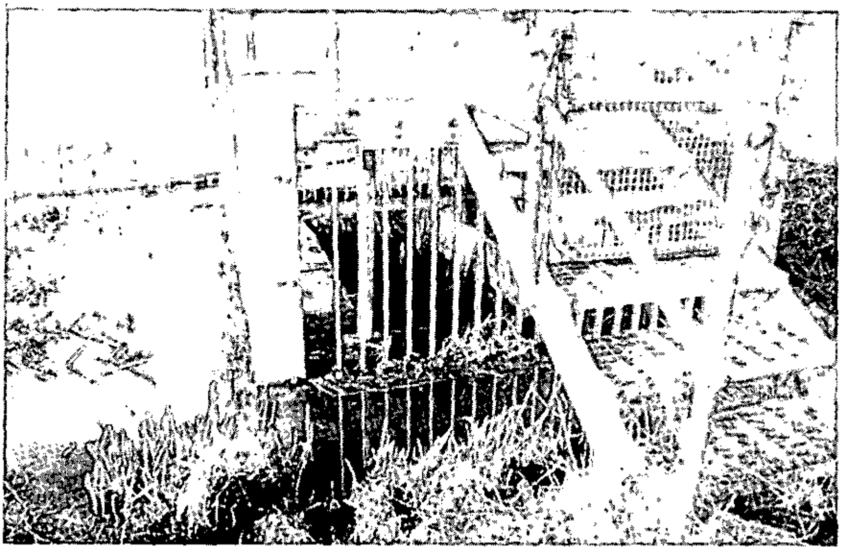

Fig. 2 'Bedstead' overflow, Adlington reservoir

Thomas Hawksley's Upper and Lower Rivington reservoirs form part of the eight-reservoir Rivington Group and were built between 1850 and 1857; as at Worthington, the two reservoirs are effectively one, being interconnected at the dividing embankment. The oddity is that the reservoirs now have spillways at different crest levels: the spillway crest for Upper Rivington reservoir is the lower of the two; 129.28m OD compared with 129.41m OD for the Lower Rivington spillway (King & Pearce, 1992).

TIPPING FLOOD GATES

Fontburn reservoir in Northumberland was built around the turn of the century for Tynemouth Corporation and is now owned and operated by Northumbrian Water. It was designed by James Mansergh & Sons and a contract was let to

Mr George Lawson in 1900. However the works were taken over in December 1903 and completed by the Corporation.

The embankment dam is about 25m high and over 300m along the crest. The main spillway is a bellmouth, discharging to a tunnel, which was used for river diversion during construction. An auxiliary spillway with concrete tipping gates was built following recommendations made by Mr R H Cuthbertson in 1958 and following model tests at Heriot-Watt College, Edinburgh.

The auxiliary spillway comprises 20 tipping gates (Figure 3), providing an overall net width of 270ft. The sill level is 2ft above the crest of the bellmouth, the ten central gates are designed to tip at a head of 1.75ft; the outer ten gates at a head of 2ft. The gates pivot about stainless steel hinge pins and rest against the downstream sloping faces of piers on their upstream side. A concrete step downstream provides a water cushion from leakage past the gates, so that the gates can operate without damaging themselves. A single gate has been tested by installing timber planks spanning across the upstream piers and pumping water into the space. All the gates are checked annually for free movement.

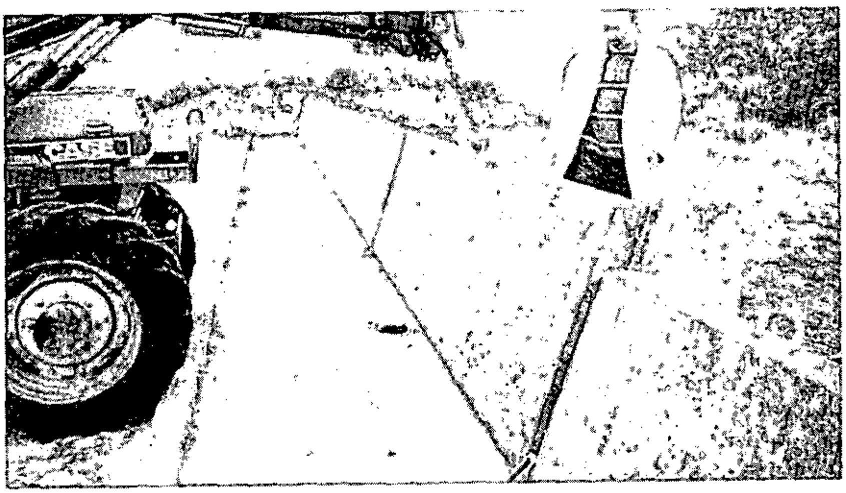

Fig. 3 Tipping gates at Fontburn reservoir, under annual test (photograph courtesy of Cuthbertson Maunsell)

Subsequently much larger tipping gates, with theoretical tipping heads of 1.54m and 1.95m, were installed in auxiliary spillways at Greenfield and Yeoman Hey reservoirs, near Manchester. These were described in a paper at the 1988 BNCOLD Conference (Ackers & Hughes). The idea has also been taken up and developed further in the prefabricated metal Hydroplus gates.

METAL WAVEWALLS AND WAVE PROTECTION

The use of metal wave protection or spray deflectors featured at three reservoirs owned by Dee Valley Water until the late 1980s. Sadly, two of these cases of innovation reached the ends of their useful lives and have since been replaced.

The contract for Cae Llwyd impounding reservoir, signed in 1875, was for the value of £6070. It is commendably brief and the BQ runs to a single page with only seven items. When the first author examined the original drawings in 1984, he found that one of them was drawn on the back of a print showing three alternative schemes for Mersey crossings – one a suspension bridge and the other two tunnels.

The earth dam, with a puddle-clay core, is about 15m high and 180m along the crest. The original drawings show a freeboard of 6ft and no evidence of a wavewall. An 1883 drawing suggests that the impounding level was raised by 2ft either during or soon after construction, without a corresponding rise in dam crest level. In his first inspection under the 1930 Act, Mr E W Dixon reported that 'The top of the embankment at its lowest point is about 4 feet above top water or spillway level and the water face of same is protected with squared stone pitching to full height. There is also a galvanised sheeting 3 feet in height as a protection against wave action.' Dixon was wrong about the height of the sheeting, which was only about 26 inches (Figure 4).

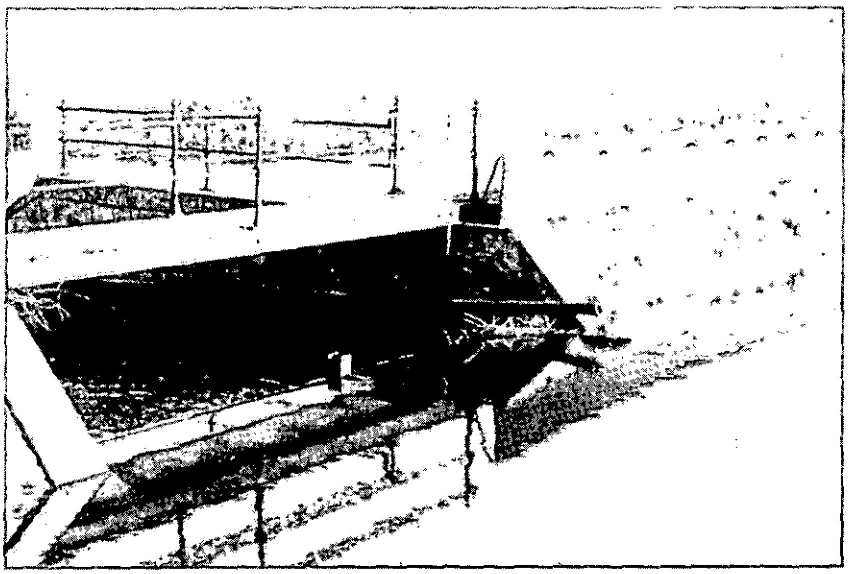

Fig. 4 Cae Llwyd metal wavewall

The 1953 inspecting engineer reported the galvanised sheeting as in good condition and considered the 4ft freeboard to the dam crest as sufficient. His two successors, in 1963 and 1973, made no comment on the condition of the metal wavewall and considered the freeboard sufficient, although by 1973 it had reduced to 3.3ft. When Mr R M Arah inspected the reservoir in 1983, the metal wavewall had reached the end of its useful life and the minimum freeboard to the dam crest was surveyed as 0.97m. This was found to be inadequate in relation to new flood and wave assessments, and there was strong doubt about the durability of the metal wavewall under more than minor attack. Accordingly, the metal wavewall was replaced with a 0.60m high concrete wall and an auxiliary spillway was also constructed.

Ty Mawr reservoir is a short distance downstream of Cae Llwyd and was completed in 1907. The owner has a set of well preserved ink and colourwash drawings detailing progress on the puddle trench between 1904 and 1906. A drawing dated 1914 entitled *'Raising of the puddle bar'* shows very serious settlements of up to 6ft 3in only seven or eight years after construction, but there are no records of what remedial works were undertaken, although they were evidently successful and there are no present-day signs of either stability problems or major reconstruction in the past.

Ty Mawr is an off-line reservoir, being linked via a short tunnel to the stream, just upstream of a weir which sets the reservoir's impoundment level. The 600m long embankment dam, with a maximum height of about 13m, encompasses just over half the perimeter of the reservoir, whilst the surface area of the reservoir occupies over half of the direct catchment.

In his 1932 report, Mr E W Dixon noted that the freeboard was 5ft and that most of the crest was equipped with 3ft high galvanised sheeting. (In this case, the height was reported correctly.) The metal wavewall was still in good condition in 1953, and the 1963 and 1973 inspecting engineers made no reference to it. By 1983, it was slightly corroded and some parts were loose and rattling in the wind. The minimum dam crest freeboard was surveyed as about 1.25m. A flood study showed this to be inadequate according to modern standards, especially if no reliance could be placed upon the metal wavewall, which was clearly near the end of its useful life.

The metal wavewall was replaced with a 0.60m high concrete wall and the connecting tunnel to the stream was modified by the incorporation of a large flap gate to throttle flood inflows, but have little effect on outflows. This also allowed the top water level to be raised by 150mm to increase the storage capacity, if required, by the use of stoplogs.

Pendinas reservoir was formed in about 1896 by a total of four earth dams with puddle-clay cores and is filled largely by a catchwater. The second inspection was carried out in 1943 by Mr R F Baker, who described the dams in detail, referring to a number of plans which are still in the possession of the Company. The drawings do not, however, show the ingenious wave protection at one of the dams, where steel highway crash-barriers on timber posts have been used to support the edge of additional riprap placed on the upper part of the inner slope (Figure 5). Unfortunately, there is no information about when this was installed or who should take the credit for the idea.

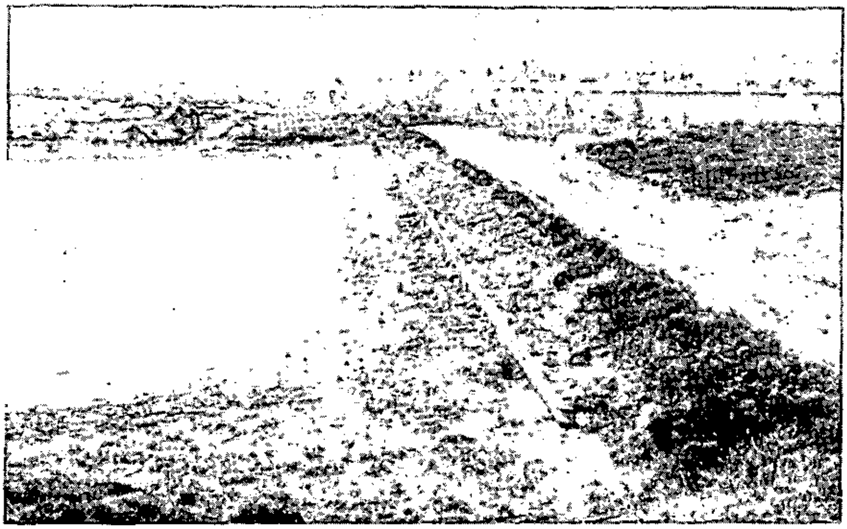

Fig. 5 Pendinas crash barrier wave protection

When inspected in 1983 the minimum freeboards at the four dams were surveyed at between 0.36m and 0.61m, which was insufficient in relation to new flood and wave assessments. Although the dam with the least freeboard had a masonry wavewall, the others did not. Subsequent remedial measures included adding a wavewall to a second dam and widening the spillway to reduce the flood rise. The dam with the crash barrier was placed into Category D of the 1978 *ICE Guide* and escaped the need for crest raising or the substitution of more conventional wave protection.

OUTLET TOWER OR CHIMNEY?

The valve tower at Barbrook reservoir near Chesterfield is of circular cast iron construction located at the mid-point of the main dam. The lower part of the tower and scour valve are caked in soot. This is because fires used to be lit at the foot of the tower to prevent the water in the draw-off pipes from freezing. A chimney in the valve house vented the smoke. The following quotation from an operative at the site describes how this was done.

'I was at Barbrook from 1974 until...Feb 1990, the ice would form when the wind was blowing in a north-easterly direction even if the temperature was only one degree below freezing point. Yes, we did use coal braziers to stop the inlet pipe in the tunnel from freezing up. However, the ice that was formed was not solid but like a frazzy ice, like slush, this ice used to stop the water from flowing through the inlet pipe. The coal was taken in bags up to the far end of the tunnel where the res inlet pipe came in, the only down side to this was the fact that the ashes had to be removed the same way. The fires had to be kept burning for 24 hours a day during the cold periods....Prior to my period at Barbrook, I often heard it said that in order to generate extra heat when the inlet pipe had actually frozen, a pile of sacks used to be placed near the pipe and then doused with paraffin, a match would then be thrown at the rags whilst taking very big and quick steps down the tunnel before the pipes went bang!'

ACKNOWLEDGEMENTS

North West Water for permission to include information from their files concerning the Worthington group of reservoirs, including the 1997 inspection reports by John Smith; supervising engineer Keith Swettenham for showing the first author over the works in 1998.

Northumbrian Water for permission to include information about Fontburn reservoir; Ian Gowans for providing information from Cuthbertson's archives and from his recent inspection report; and John Laing of RKL-Arup, the supervising engineer, for providing information about testing the tipping gates.

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Severn-Trent Water for permission to include information on Barbrook; Neil Williams and Paul Bingham for the operator's tale and photographs.

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Refurbishment of the scour facilities at Green Withens Reservoir

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SYNOPSIS. Reservoir owners are often troubled by leaking valves and pipework. little attention seems to have been given by the designers of the late 19^{th} century towards the need for, or the ease of, periodical replacement of these features. This is unfortunate as valves and pipes tend to age more rapidly than most other components of large raised reservoirs. A problem of this kind developed on the scour facility at Green Withens reservoir in Yorkshire. this paper describes the investigations, studies and the refurbishment carried out in an attempt to resolve the situation.

INTRODUCTION

Green Withens Reservoir is used by Yorkshire Water for water supply. It has a storage capacity of 1.36 Mm^3 and lies near to the head of the River Calder, close to the M62 cross-Pennine motorway. It is an impounding reservoir that is augmented by catchwaters that intercept runoff from the moorland around Blackstone Edge. Two embankments retain the reservoir: a 25 m high main dam and a 13 m high saddle dam, as shown on Figure 1. The reservoir outlet works lie on the right bank. Wakefield Corporation Waterworks commissioned Mr H Rofe to design the works and Thomas Oliver constructed them between 1892 and 1898.

Some one hundred years later in 1998, a report of a statutory inspection by Rodney Bridle, Inspecting Engineer, commented on the unsatisfactory condition of the scour valves. He noted that even though the valves could be opened successfully during tests, the operators were unable to seat the valves properly on closure, allowing water to flow by continuously. The valves had last been reconditioned in 1935-36 and the situation had steadily worsened over the years such that significant quantities of water were now escaping past the valves. The steady flow was contributing to the deterioration of the tunnel lining downstream and consequently he recommended investigation and resolution of the situation.

Dams 2000, Thomas Telford, London, 2000.

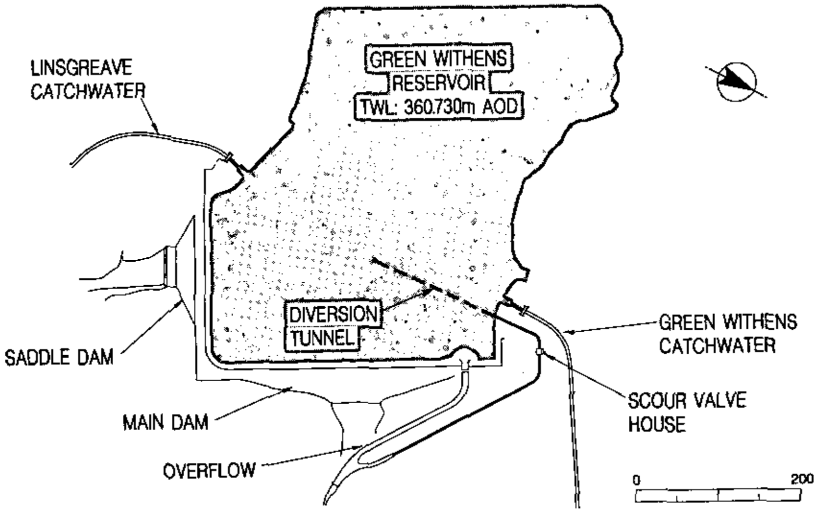

Fig. 1. General arrangement of reservoir

SCOUR ARRANGEMENT

The natural stream into the reservoir basin was diverted during construction via a tunnel on the left bank of the valley. Its alignment followed a curved path that led from the lowest point of the basin, around the footprint of the main dam and emerged beside the stilling basin, as illustrated on Figure 1. The tunnel was driven through *Namurian* sandstones and shales and was brick lined throughout. A twin-cell rectangular shaft was constructed about halfway along the line, close to the left abutment of the main dam.

The river diversion was closed after completion of the embankments with bulkhead walls being formed inside the shaft. A 10.5 m long section of 24-inch diameter pipe was installed beforehand to provide a scour outlet to the reservoir. Bellmouths were fitted at the entry and exit points and two valves, acting as an isolating–service pair provided control. The in-line valves were built into the brickwork and the base of the shaft was backfilled with concrete, as was fairly common practice at the time. The base of the shaft currently lies about 10 m above the level of the valves. They are manually operated by capstans at ground level, which are housed in a small stone building. The general arrangement is shown on Figure 2.

The scour valves are unusual, although the manufacturers, *Glenfield Ltd*, did supply a number of them elsewhere in the country. The valve is closed by a sliding gate, which is connected to its operating spindle by a yoke and pin fitting. The valve body is fitted with a gunmetal facing ring that mates with a similar one attached to the gate. A hinge at the base of the spindle allows rotation and the force of the water pushes the faces together to form a seal.

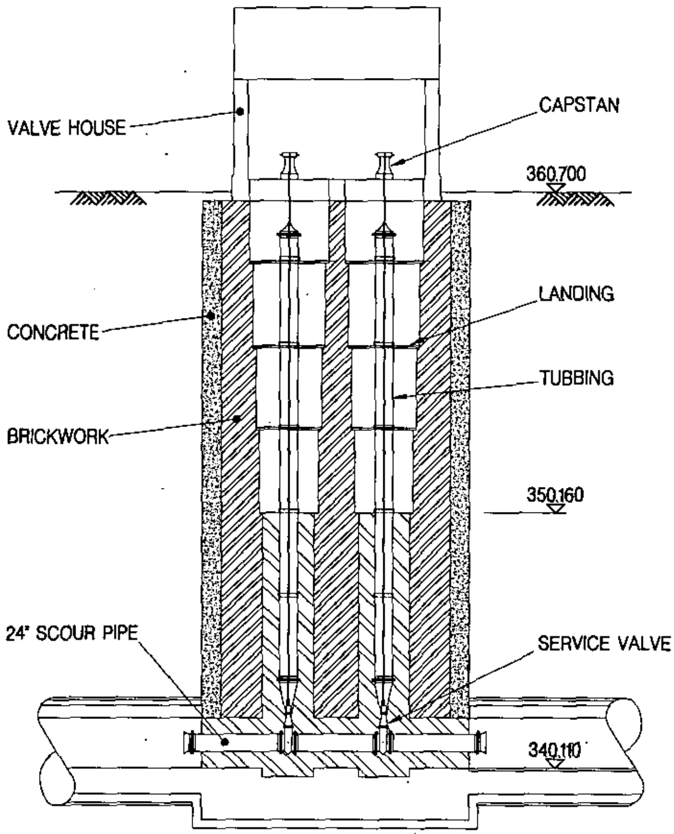

Fig. 2. General arrangement of the scour facility

Rather than a reducing bonnet ~~section~~, the valve body is fitted with an expansion piece which increases to accept the socket of a 28-inch diameter nominal bore standpipe ('tubbing'), which then extends above reservoir top water level in a series of socket and spigot pipes. The operating spindles are connected to geared capstans at ground level and the valves are operated manually by hand wheel. Each valve requires several hundred turns to fully open the valve. The capstans are supported on a cast iron platform with large dressed stone flagstone floor panels.

VALUE MANAGEMENT WORKSHOP

A value management (VM) workshop was held to identify the preferred way forward. Representatives from Operations, the Reservoir Safety Section and Capital Investment from YWS met with the engineering consultants and the Qualified Engineer. The objective of the exercise was to explore potential solutions to the problem of the ageing scour facility.

The exercise enabled a working strategy to be developed. This had two main strands. In the first instance, the approach would be to determine by CCTV survey if it might be practicable to withdraw parts of the valves. If this were to be possible, then one valve would be withdrawn and repaired. If the refurbishment proved to be unsuccessful, then the alternative would be to seek an engineering solution to overcome the problem.

A number of prospective solutions were identified. Each was discussed and ranked according to its perceived 'best whole life value'. Those solutions that did not meet the 'fitness for purpose' criteria were discarded and by the end of the VM workshop, four main options remained. The potential solutions were as follows.

1. 'Do nothing' – i.e. accept current condition, monitor deterioration and maintain scour and tunnel as necessary.
2. Add new actuated valve at the head of the scour tunnel.
3. Extend new pipeline through scour tunnel and fit manual control valve at the downstream end.
4. Construct a new shaft immediately downstream of the valve house for the purpose of installing a new manual control valve at the same position as in option 2.

INVESTIGATION OF THE VALVES AND PIPEWORK

The scour pipework is mostly inaccessible. The only section that can be visually inspected is that length downstream of the service valve. Inspection is made more difficult by the continuous flow of water but it was clear that the metalwork surfaces were pitted and corroded. Also, despite several minor adjustments of the valve position, it could be seen that the main direction of the water jets was from the top half of the gate on both sides, as shown on Plate 1.

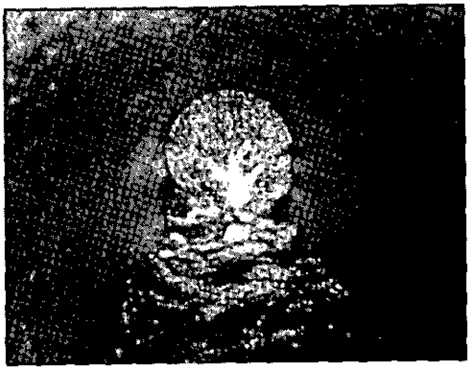

Plate 1 : Flow condition in the 24 inch pipe prior to gate removal

The condition of the section upstream of the service valve was unknown, although it was expected to be in a worse condition, due to the greater amount of water being let through the isolating valve. Further investigation by remotely operated vessel (ROV) from the upstream bellmouth might have been possible but this task would be very difficult since:

- the bellmouth lies some 250 m from the entrance to the tunnel,
- the scour tunnel is probably partly filled with fine sediment, and,
- manoeuvrability / visibility inside the tunnel is likely to be very poor.

The cause of the closure problem was unknown but it was thought that the valves might have been overdriven on one or more occasions, which could have damaged them enough to prevent perfect seating. This theory seemed plausible, given the mechanical advantage afforded by the gearing, the ease of operation and the absence of visual or audible confirmation of closure.

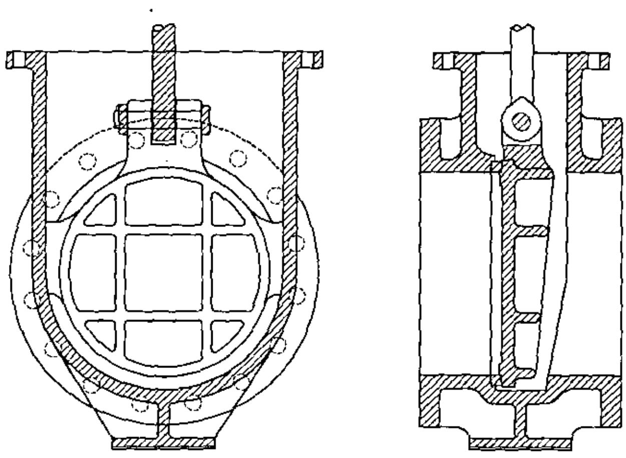

Fig. 3. 24-inch valve gate details

A CCTV survey was carried out to investigate the conditions inside the 28-inch diameter 'tubbing' with a view to removing the gate. The survey proved that the arrangement had not been constructed as shown on the record drawings. It confirmed that the operating spindle was assembled from five tubular rods 12 feet long. The diameter of the individual rods varied from 4-inch at the lowest and highest rods, 5-inch at the intermediate positions while the central rod was 6-inch.

The rods were joined using screwed solid internal stub shafts, which each added about 75 mm to the overall length of the operating rod. However, the survey could not determine whether it would be possible to split the rods using conventional methods.

Digging down inside the twin cell shaft to exhume the valve body was considered but the inadequacy of the isolation valve and the overall disturbance would have necessitated the reservoir to be complete empty beforehand. A controlled drawdown of the reservoir started in March 1999 and a contingency plan was devised for bypassing the catchwaters and over-pumping the natural stream. However, this ran into difficulties once the water level had dropped by about 15 m, as the water quality of the discharge decreased and the Environment Agency became concerned about potential environmental damage downstream.

VALVE REFURBISHMENT WORKS

The opposition to reservoir drawdown necessitated a change in approach. The services of a specialist contractor, George Green (Keighley) Ltd, were sought to assist with the investigation of valve repair *in situ*. Current health and safety regulations preclude any possibility of manual work inside the pipe hence the approach was aimed at partial removal. Therefore they devised a method statement, which involved the following works:

- Installation of a lifting gantry and a 2-tonne hoist over the headstock.
- Disconnection and removal of the capstan from the spindle.
- Extraction of gate/spindle by clamping, lifting and splitting the rods.
- Removal of the assembly to the workshop for assessment.
- Gate refurbishment, as needed, by casting and machining a new face.
- Refitting the assembly on site.

Careful attention was given to the health, safety and security aspects prior to the start of the works. YWS were responsible for the operation of the valves and the issue of permits to work. The shaft was a confined space and gas detectors were used to monitor the atmosphere inside the working area. Temporary lighting was provided and scaffolding was erected around the pipe column to provide good access. All works were undertaken with safety harnesses. Care was also taken to protect the opening in the valve house floor and to ensure that objects were not allowed to fall inside the 'tubbing'.

The works were carried out over a period of 5 days in August 1999. The extraction of the spindle did not give rise to any special problems and was recovered with minor damage. All the brass spindle rods were found to be in good order although one of the smaller diameter pieces had a slight bow.

The jointing pin of the top rod had only been screwed about four turns into the next rod and the lock nut had not been tightened. While it had survived in this state for over 60 years, the incomplete penetration of the stub into the shaft would have resulted in the total length of the spindle being greater than the design length. As such this could have caused the operators to over tighten the valve on closing, pushing the gate past its normal seating position and into the clearance void at the bottom of the valve body.

Plate 2 : Condition of gate upon withdrawal

All of the old stub shafts were removed and could not be re-used because of their condition. The female threads within the rods were manually chased out and the new stubs were specially cut in order to suit the non-standard thread (six per inch). The gate guides were corroded but cleaned up, while the leading edge was skimmed by the same amount as the gate seal ring. The gate-fixing pin was replaced, as were the top fixings to the gearbox.

The gate seal face was found to be a gunmetal ring fixed by copper rivets. Its face was uneven and clearly would not have sealed against a flat surface. A fine skim was therefore machined from the ring to remove the surface irregularities. Unfortunately it was not possible to refurbish the mating surface within the valve body (on health and safety grounds), which somewhat reduced the chances of effecting a good seal in the repaired valve.

The reservoir level was raised after the replacement of the gate and spindle into the valve body in order to test the effectiveness of the repairs. If the work had been successful then the intention was to follow up with similar treatment to the isolating valve. However, by September 1999 it became apparent that an effective seal could not be achieved as water, albeit in smaller quantities, was still passing by the gate. Almost certainly, the problem lay with the worn, pitted seating ring but repair to that element would have necessitated complete drawdown of the reservoir, which would have been undesirable on environmental grounds.

FEASIBILITY STUDIES

Following the failure to resolve the problem by valve repair, the study reverted to a detailed assessment of the four options identified by the VM workshop. Each scheme was examined in sufficient detail to enable reliable cost estimates to be drawn up. A summary of the main advantages and limitations is given in Table 1. Surprisingly, Options 2 and 3 were only slightly cheaper than the best technical solution (Option 4) and all three were much more expensive than Option 1. Clearly, considerable costs will accrue even from this approach. However, there is the possibility that a complete reservoir drawdown will happen at some time in the future, which would provide an opportunity to repair the body of the valve.

Table 1. Summary of the merits and limitations of the prospective solutions

Option		Advantages	Disadvantages
1	'Do nothing'	Large expenditure deferred for the time being.	The valves and pipework will continue to deteriorate. There will be extra maintenance requirements in the tunnel. Current scour valves are not designed to control flow rate.
2	'Actuated valve'	No loss of existing scour capacity. Provides control over rate of flow.	Access required to bring plant, equipment and materials to tunnel. Difficulties with valve transport in tunnel and future maintenance. Modifications needed at tunnel head to accommodate plug valve.
3	'Extended pipeline'	Ease of operation. Provides control over rate of flow.	Access to tunnel portal needed. Difficult to move pipework and valve inside tunnel. Some loss of scour capacity. Complicated series of bends needed at 70° bend inside tunnel. Pipe will restrict access for future monitoring and surveillance.
4	'New shaft'	No new access required to the tunnel portal. No loss of scour capacity. Ease of installation and maintenance. Improved arrangement for future maintenance. Improved ventilation.	Difficult ground excavation adjacent to existing structure.

LESSONS LEARNT

Leaking valves and pipework at reservoirs is a common problem but the solution is frequently very difficult to find, especially if the operators need to maintain supplies to the treatment and distribution system. The reason is usually related to access or safety constraints. At Green Withens Reservoir the problem is due to the encasement of the valves inside 10 m of concrete and brick infill. No serious thought seems to have been given by the late 19th Century Engineers toward the removal and replacement of the metalwork components, which are arguably the mostly rapidly ageing elements in a dam-reservoir system.

The situation at Green Withens fortunately did not compromise reservoir safety and ultimately it was possible to accept the so-called 'Do nothing' option. However, considerable effort was expended in the attempt to resolve the situation and valuable experience was gained in the process. We conclude that the cause of the problem was a combination of factors that include: ageing (corrosion and erosion of metal over time), selection of an inappropriate type of valve, and finally, faulty refurbishment or refitting.

Although the work at Green Withens Reservoir has not been an unqualified success, there has nevertheless been a considerable improvement in the situation. YWS have now established a benchmark and will be able in future to monitor the performance qualitatively and quantitatively, in line with current best practice (Reader *et al*, 1997). It is possible that future performance data will provide an insight into both the rate and the nature of valve deterioration.

The arrangement at Green Withens Reservoir is not particularly unusual amongst dams of the Victorian era. Clearly, problems of this kind will become more prevalent as the country's stock of reservoirs ages. Those responsible for reservoir safety will need to develop effective strategies to combat the problem in the future.

ACKNOWLEDGEMENTS

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Towards total acceptance of fully automated gates

P D TOWNSHEND, Flowgate Projects (Pty) Ltd., RSA

SYNOPSIS. Ungated spillways offer the safest form of spillway but are generally more costly than gated spillways for the volume of water stored. Gated spillways offer a more cost-effective use of water storage but must not jeopardize dam safety. Most commonly used spillway gates are mechanically driven reliant on external power supply. There is however a substantial record of these types of gates not operating when required, thereby placing the dam's safety in jeopardy. The ideal is to have automatic gates which do not suffer from these problems. A range of fully automatic water control equipment has been operating for more than 20 years in South Africa from which a new generation of spillway gates has been developed which meets nearly all the requirements of an ideal spillway gate. This paper introduces these gates.

INTRODUCTION

The big dam debate continues unabated. The environmental lobby, with some justification, opposes the construction of large new dams. With a few exceptions, the era of new dams being built in developed countries has waned. However the pressure for increased water supply remains a challenge to the dam engineer. In addition to steadily increasing demand for water for population and economic growth, a new environmental requirement has appeared to increase in-stream flow requirements, all of which require the dam engineer to find more storage! With the option of new dams no longer available, the dam engineer needs to consider raising existing dams to obtain more storage.

Further, with the re-evaluation of the hydrology for dams, the design floods and PMFs are generally increased significantly so that the existing spillways then become inadequate and dam safety is compromised.

One option to increase spillway capacity is to lower the spillway crest and to fit spillway gates to retain the original full supply level.

Dams 2000, Thomas Telford, London, 2000.

DAM RAISING – WHAT ARE THE CHOICES?
Fortunately, or unfortunately, the dam engineer has a number of options to consider and each option must be carefully considered against the requirements of the site and costs. The following logic diagram in fig. 1 offers some of the options available, which fall into two primary categories – gated or ungated spillways.

Whilst ungated spillways offer the safest form of spillway, they are not usually the most effective in terms of cost for the extra volume of water stored as well as suffering other disadvantages.

Gated spillways offer a more cost effective use of the additional storage but invariably leave doubt over the safety of the dam due to possible malfunctioning of the gates.

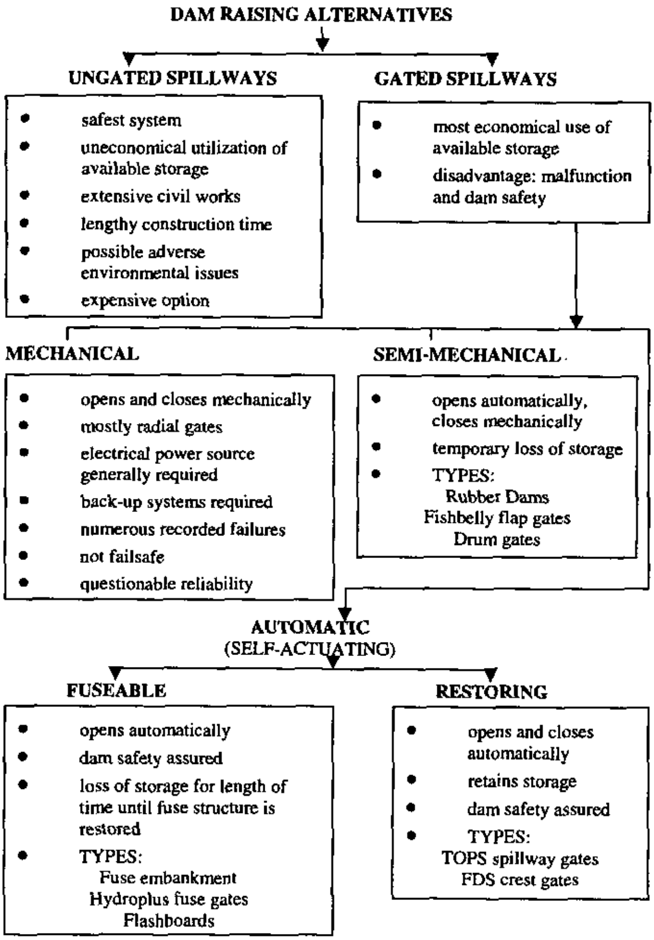


Fig. 1. Spillway gate alternatives

The gated spillways fall generally into four broad categories i.e. fully mechanical, semi-mechanical, fuseable, fully automatic and self-actuating. Each type of gate has its advantages but yet most also have one or more major disadvantages. This is illustrated in the following matrix of features.

DAM RAISING ALTERNATIVES

MATRIX OF FEATURES

Type of Spillway	Ungated	Gated			
Features	Conventional raising	Mechanical	Semi-mechanical	Fuseable	Fully automatic
Cost	X	X	T	T	T
Environmental	0	0	T	T	T
Maintenance	T	X	X	0	0
Retain storage after large floods	T	T	0	X	T
Time to construct	X	0	0	T	T
Reliability	T	X	0	T	T
No operator or external power required	T	X	X	T	T
Emergency draw-down /large releases	X	T	T	X	T

X – DISADVANTAGES

0 – POSSIBLE DISADVANTAGES

T - ADVANTAGE

Ideally one would require a gate to be as flexible in operation as a mechanically driven radial gate but which would not be reliant on external mechanical power to operate it, which poses the possibility of dam safety being compromised.

Such type of fully automatic self-actuating gates exist in Southern Africa and these are discussed more fully herein.

FULLY AUTOMATIC SPILLWAY GATES
The two types of automatic spillway gates discussed here are the

- Fluid Dynamics Systems (FDS) Crest Gates and
- TOPS Gates.

Both gates make use of variable buoyancy or ballast tanks. The natural force of the water is used to operate them and they are therefore totally self-actuating. Although fully automatic, they can be overridden manually if required for controlled releases.

They are also self restoring gates which will retain the increased storage after the passage of floods. Being fully automatic, the gates do not require operators and human error is therefore eliminated from the operation of the gates. Also, the minimum of maintenance is required.

These gates are therefore ideal for remote sites but are not necessarily confined to them. On some larger dams where infrastructure and operators exist, these gates can be activated by small power equipment and controlled by instrumentation or telemetry.

The automatic feature is therefore considered an advantage for dam safety in the event of electrical or control faults as the gates will open automatically as an important backup feature.

THE CREST GATE

The FDS Crest gate consists of a buoyancy tank connected by ducted radial arms to an upstream axle. The buoyancy tanks seals against the cill of the spillway and between vertically sided piers. An inlet weir situated upstream is connected to the hollow axle to allow water to flow into the buoyancy tank. A pipe drains the buoyancy tank to the downstream side of the weir.

The FDS Crest gate operation is described diagrammatically in the following figures.

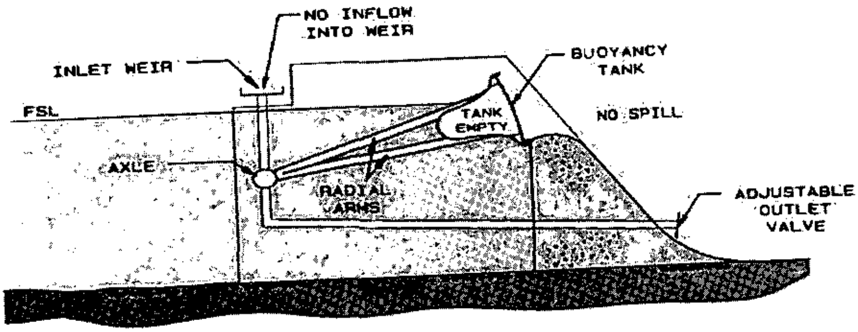


Fig. 2(a)

The empty buoyancy tank floats on the water and seals against the spillway cill and sides to increase the water-level above the spillway level.

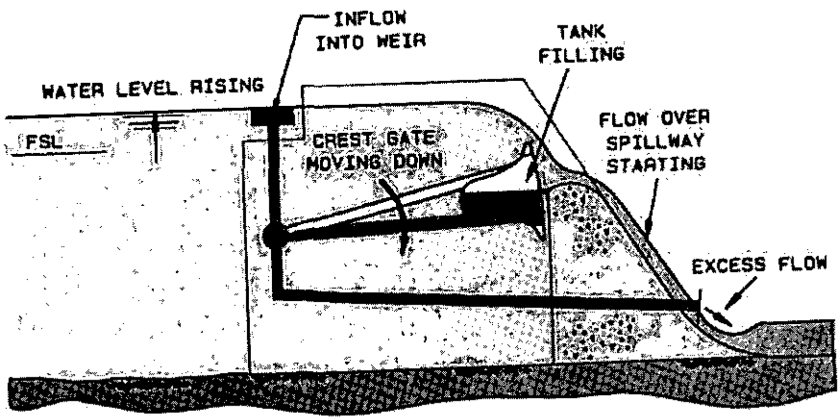


Fig.2(b)

As the upstream water level rises due to a flood, water flows into the inlet weir and into the buoyancy tank at a rate greater than the discharge rate from the tank. The buoyancy tank fills and submerges.

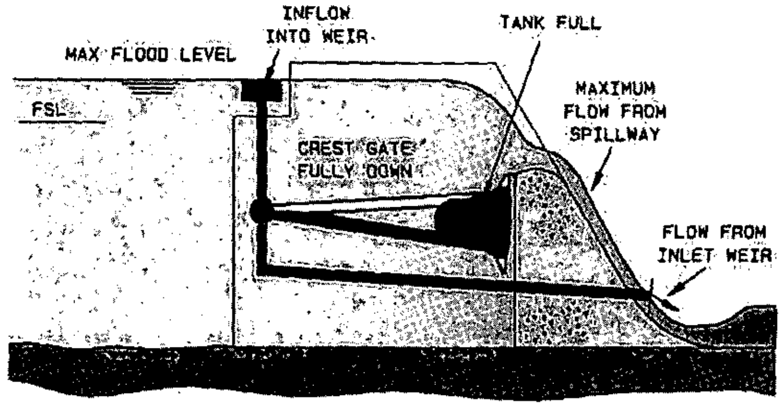


Fig. 2(c)

In the totally submerged position, the buoyancy tank is full and the gate fully open to offer an unobstructed spillway to pass floodwaters and debris.

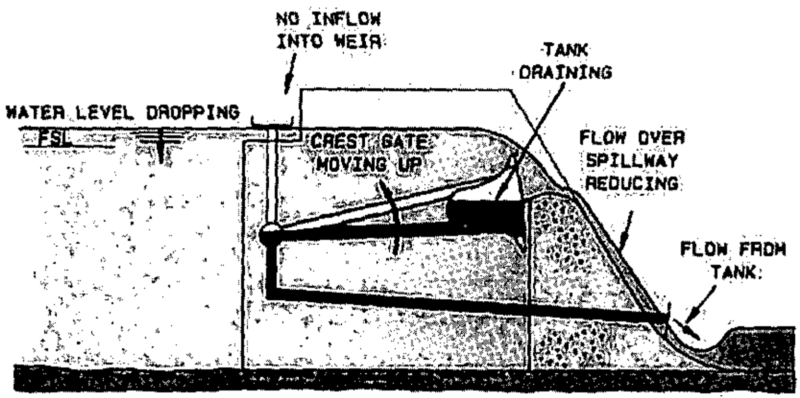


Fig. 2(d)

As the flood level recedes, water ceases to flow into the inlet weir. The buoyancy tank drains until empty and the gate floats into its fully closed position as shown in figure 2(a).

This gate is suitable for weirs and dams where the depth of water over the gate is considerable and the debris load high. For weirs it is usually used in conjunction with the FDS Scour gate which maintains a sediment free pool of water for the crest gate to open fully.

THE TOPS GATE

The TOPS gate has similar features to the crest gate but can operate on a wider selection of spillways including side channel spillways. It also has other operational features which some dam engineers appreciate. However, the selection of which type of gate to use is very much site-dependent.

The TOPS gate consists of a ballast tank attached to a closure plate which seals against the spillway cill and vertical sides of piers to retain the increased water level above the spillway. The gate is attached to two trunnions positioned above the water level.

The operation of the TOPS gate is described diagrammatically in the following figures.

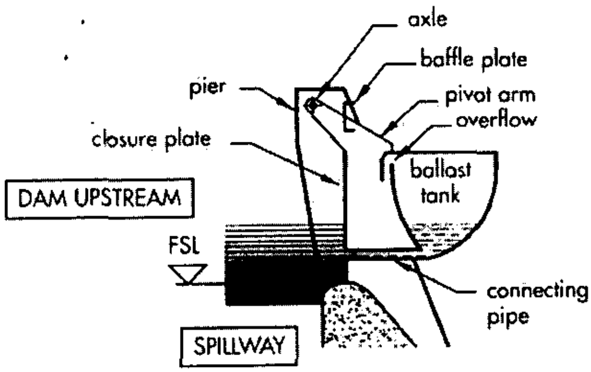


Fig. 3(a)

The ballast tank is connected by conduits to the closure plate so that the water level in the dam and the ballast tank are in equilibrium. The mass of water in the ballast tank, together with the gate's self-weight create a closing moment about the axle which is greater than the opening moment induced by the upstream water level, and hence the gate remains closed.

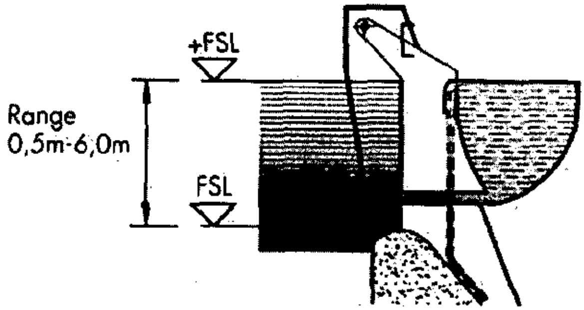


Fig 3(b)
The gate remains closed for all upstream water levels up to the increased supply level. The ballast tank is then full and excess water is spilled from outlets on the upstream side of the tank.

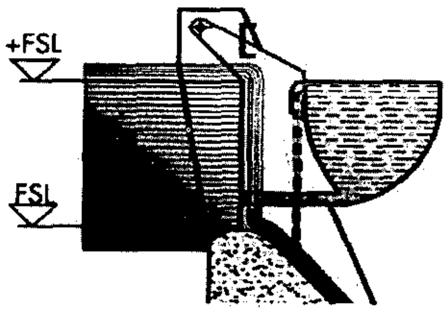


Fig. 3(c)
In order for the gate not to open for all small order floods, surplus flood waters pass over the closure plate and discharge between the closure plate and ballast tank.

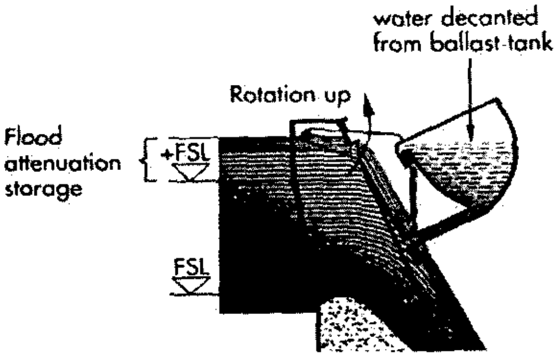


Fig. 3(d)

For larger order floods, the upstream water level will rise and the opening moment exceeds the closing moment. The gate then rotates upwards and outwards and by so doing, decants water from the ballast tank through the outlets thereby making the gate lighter. The gate then opens easily with increased upstream flood levels.

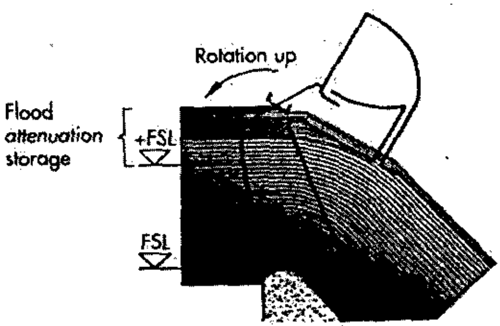


Fig. 3(e)

In its fully open position, the ballast tank is empty and the gate rides on the flood waters with very little backup of upstream water level. The offset in the pivot arm causes the gate to rise as it rotates, thereby creating sufficient waterway to pass the design flood.

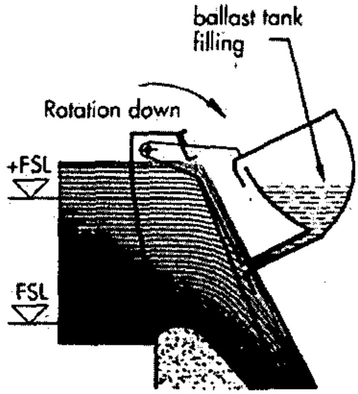


Fig. 3(f)

As the flood water level recedes, the gate will rotate downwards and the ballast tank fills partially in order to balance the opening and closing moments.

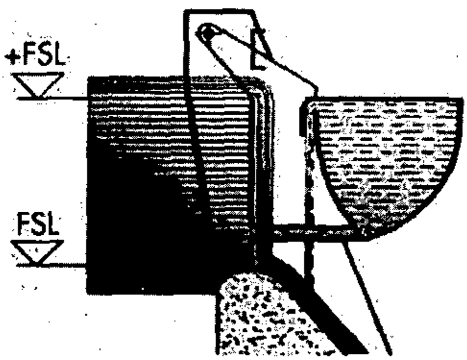


Fig. 3(g)

As the flood subsides, the gate will close completely and surplus flow will pass through the gate until in fig. 3(h), the gate is closed to retain the increased full supply level.

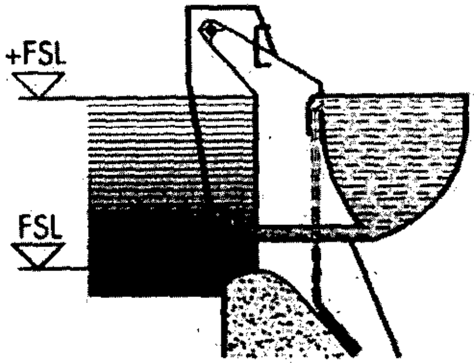


Fig. 3(h)

The TOPS gate is as flexible as a radial gate in operation and can be used as both an emergency and regulating gate. Some of the main features of the gate are:

- It is totally automatic and self-actuating in both opening to pass floods and closing after floods to retain the increased water level.
- It can be manually opened either by an operator or by an electrically actuated valve by instrumentation or telemetry. The TOPS gate can therefore be used to regulate controlled releases from the dam.
- By means of different opening mechanisms, certain gates of a multi-gated spillway can be opened fully at the start of an incoming flood. This assists greatly in flood routing and in containing the peak discharge for large order floods.
- It incorporates a number of safety and backup facilities to ensure that the gates operate as intended and that dam safety is not compromised.
- The gates are modular to form a multigated spillway. This assists in dam safety as well as reduces the possibility of losing the whole spillway system as a result of one failure
- The gates can fit most types of spillways, including curved spillways provided the radius of horizontal curvature is not too tight.
- The gate is constructed in metal and is therefore robust and easily repaired or modified by standard steelwork practices.

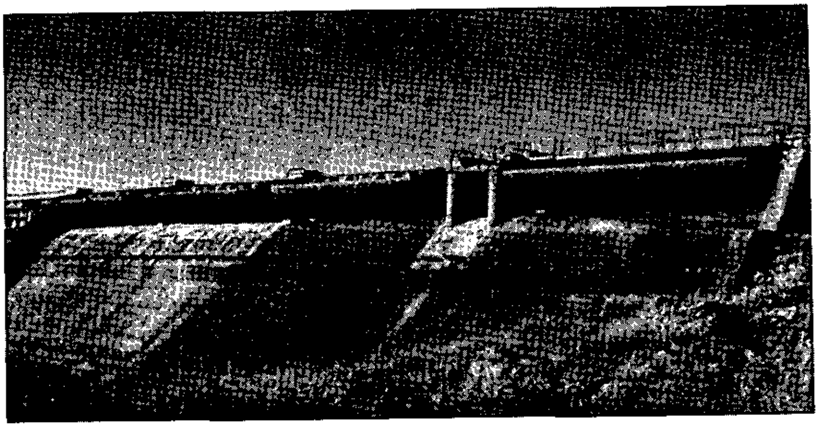
- The gate can be inspected and maintained in position against a full head of water without the need to draw down the water level.

  This is achieved by means of a floatable maintenance gate, which is attached to pivots on the piers and flooded to sink into a closed position. Once maintenance is completed, the intervening chamber is flooded and the maintenance gate floats upwards where it is detached from the piers and towed away, or to the following gate if a maintenance programme is instituted. The TOPS gate also has trunnions positioned above water level where they are accessible for inspection.

CURRENT STATUS OF THESE AUTOMATIC SPILLWAY GATES
The FDS Crest gates are installed at more than 10 sites around Southern Africa and have been working successfully for approximately ten years on average. Some of the earlier gates have been subjected to major floods and were observed to operate as designed during and after the floods.

Crest gates at Tswasa weir, Groot Marico River, South Africa.

The TOPS Gate is a more recent gate and has been installed on the Belfast dam in South Africa where the water level is increased by 2.0m.



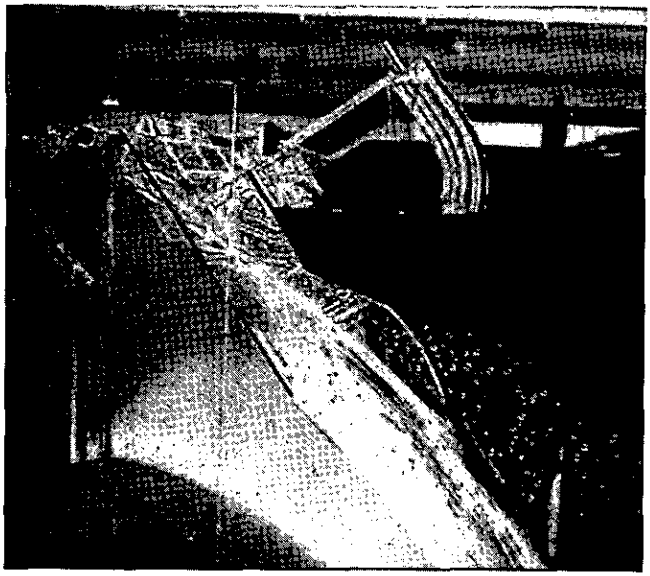
TOPS gates fitted to the Belfast Dam, South Africa.

The TOPS Gates are currently being considered for dams in Southern Africa and Australia.

Extensive model testing is being conducted by the University of Stellenbosch, South Africa, which will result in a comprehensive technical report by an independent hydraulics professor.

The results of the testing is very promising and confirms that the gate operates as intended. The results also indicate that the gate's fully open position when the ballast tank is empty, the TOPS gate has very little increase in the water level in the dam when compared to an ungated spillway.

The TOPS gate therefore has very little effect on the maximum discharge when compared to an ungated spillway



Model test of 3.5m high TOPS gate for Bougouriba Dam, Burkina Faso, Africa.

CONCLUSION

Automatic, self-actuating spillway gates are operating in Southern Africa, which offer the dam engineers an attractive alternative to consider in raising dams.

Attention has been given to backup and safety devices to ensure that they do not jeopardise dam safety. These gates give the flexibility in operation as mechanically driven gates but offer the more important feature in that they will operate automatically when required.

These automatic spillway gates can therefore be used with confidence by dam engineers.

Three cases of gate vibration

M. NOBLE, Scottish and Southern Energy plc
J. LEWIN, Consultant, UK

SYNOPSIS. At Dundreggan, vibration of one of the radial gates occurred at low openings. An unusual gate lip design resulted in fatigue cracks at stress concentrations in the web of vertical beams stiffening the gate skin plate assembly. Alterations and repairs to the gate lip and stiffener beams are described. On commissioning new radial gates of the overflow and undershoot type at Teddington Weir, severe vibration occurred during flow under the gates due to intermittent flow reattachment at the timber block sill seals. The modification to the sill seal is described. Vibration of the gates associated with a new weir at Torrumbarry on Australia's Murray River occurred over a limited range of partial gate opening, caused by periodic eddy shedding at the lip of the gates. The investigation and solution are described.

DUNDREGGAN RADIAL GATE

Introduction

Scottish Hydro-Electric owns and operates 76 reservoirs covered by the UK Reservoirs Act. At 14 dams, flood flow is controlled by 28 gates, seven of which are radial gates. Two radial gates manufactured by Glenfield and Kennedy and installed in the late 1950s are sited at Dundreggan on the Morrison River, near Loch Ness. A change in the operation of one of the gates caused gate vibration at low openings. An initial assessment of the cause of vibration pointed to the unusual design of the gate lip. A hydraulic model study carried out at Imperial College confirmed this.

The gate vibration, which occurred over some time, caused fatigue cracks at stress concentrations in the web of the vertical beams stiffening the gate skin plate assembly. The heaviest concentration of cracks occurred at or near the horizontal girders which tie the gate arms. These were the areas subjected to high shear stress and contraflexure during vibration. The structural integrity of the modified gate was subject to detailed investigation. The gate was recommissioned and is giving good service.

The Spillway Gate

The spillway gate installation at Dundreggan dam consists of two radial gates and one bottom hinged flap gate. The radial gates are 8.25m high and 8.69m wide. The skin plate radius of the gates is 8.24m and the height of the trunnions above the sill level is 5.79m. Each gate has a mass of 27.46 tonnes and the gate load, when the upstream water level is at the crest of the gate, is 3150KN. The skin plate assembly is supported by two braced gates arms per side, Fig. 1. The gate arms are tied by horizontal girders. The skin plate is stiffened by vertical beams riveted to the skin plate.

Dams 2000, Thomas Telford, London, 2000.

The flap gate is located between the two radial gates. It is 3.05m high and 6.1m wide. The function of the flap gate is to discharge compensating flow and floating debris.

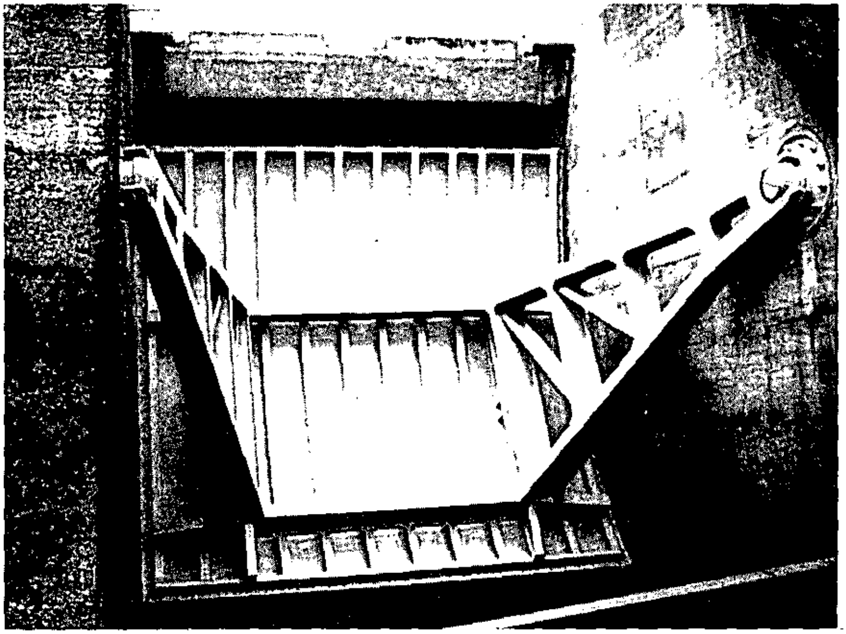


Fig. 1. Dundreggan radial gate

Gate Vibration

Following a review by Scottish Hydro-Electric – influenced by the collapse of a spillway gate at the Folsom Dam in California – the left bank radial gate was taken out of service during the summer of 1998 to install new trunnion bearings of the self-lubricating type. The Folsom Dam gate collapsed due to friction caused by corrosion on the loaded side of the steel trunnion pins. This caused failure of a bracing member of the gate arms (Bureau of Reclamation, 1996). Inspection of the gate at Dundreggan while it was behind stoplogs revealed multiple fatigue cracks in the web of the vertical skin plate stiffener beams. Movement of the joints occurred between the upper gate arms and their horizontal tie beam and sheared a bolt.

The gate had been subjected to severe vibration at low openings. Both radial gates are operated by electric motor driven hoists. Originally, gate movement had been controlled by manually actuated push buttons. The radial gates were converted to operation by programmable controller in 1979. As soon as the upstream level exceeded a predetermined value the programmer responded by opening the radial gate in small steps. The second radial gate was run off the same programmable controller. Compensation water and freshet flows were probed through the left hand radial gate by remote manual pulse signals. A minimum initial gate opening, sufficient to avoid gate vibration, had not been programmed into the control algorithm.

The sill beam had been constructed with a triangular cut out so that the sill seal in the gate would be perpendicular to the contact face when the gate was in the shut position. It may have been assumed, erroneously, that the flow under the gate would rise due to sill beam geometry. Possibly because of this assumption, the gate lip had been extended in the form of a shoe to deflect the flow downwards, Fig. 2.

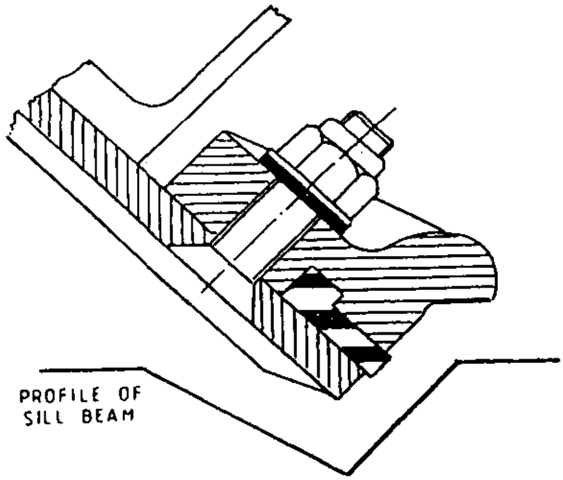


Fig. 2. Original gate lip

Gate vibration was due to this feature. The flow separated at the bottom edge of the skin plate and reattached to the extension of the lip in the downstream direction, forming a trapped shear layer. At larger gate openings, the variable pressures caused by flow attachment and detachment are less, and gate vibration is probably damped out by side seal friction. The cycle of eddy shedding is shown in more detail in Fig. 3.

Fatigue Fractures

The vertical skin plate stiffener beams consisted of two T-sections with welded spacer plates forming the centre of the web of the resultant beam. The spacer plates were not continuous, resulting in rectangular holes, Fig. 4.

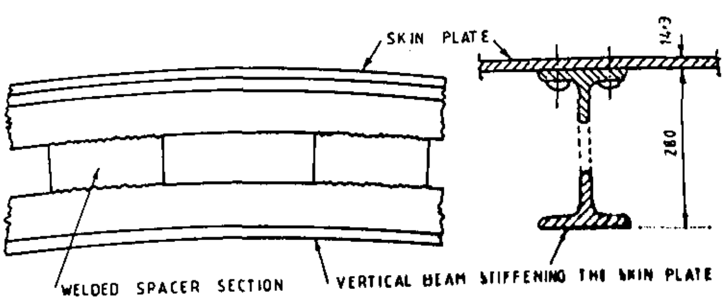
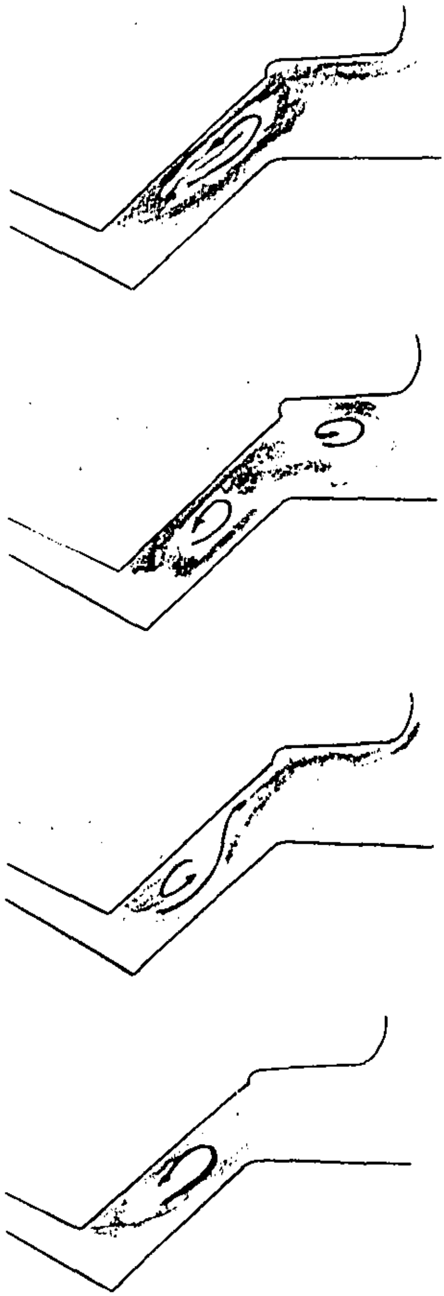


Fig. 4. Original vertical skin plate stiffener beam



The flow is separating at the bottom edge of the skin plate and is forming an eddy. The centripedal effects associated with this large eddy generate low pressures which are likely to cause maximum downpull on the gate for this condition.

The eddy has grown unstable as indicated by its distorted form. The downstream half is about to be shed into the wake while the upstream half tends to contract and migrate upstream.

Small eddy has formed near the separation point. The mainstream has re-attached to the the gate further downstream. The higher pressures generated by this re-attachment are likely to cause minimum downpull on the gate.

The eddy expands across the lip as it captures vorticity from the free shear layer prior to shedding. The cycle is then repeated.

(reproduced from the report by Dr. J. D. Hardwick)

Fig. 3. Flow conditions at the gate lip at low openings

The fractures originated at the corners of the rectangular holes in the webs of the vertical stiffener beams, Fig. 5.

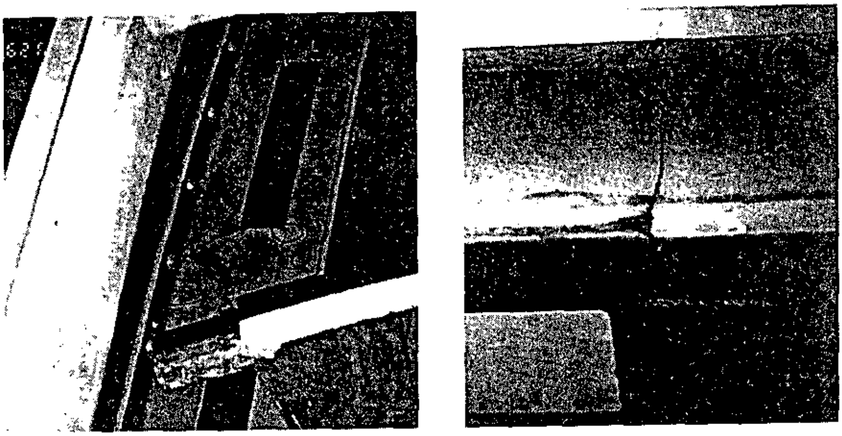


Fig. 5. Fractures in web of stiffener beams

One fracture of the downstream flange of a vertical stiffener beam occurred as a result of a crack propagating from the corner of a rectangular hole. The cracks in the web and the fracture of the uppermost vertical stiffener beam were consistent with fatigue failure due to flexing. Most cracks had corroded, indicating that they had occurred over an appreciable time.

Sections were cut from the webs at the location of cracks and were subjected to metallographic examination. This confirmed that the mode of failure was fatigue cracking evidenced by a transgranular, unbranched nature. Tests showed that the material was similar in chemical composition and tensile strength to weldable structural steel to BS 4360 Grade 43 D, apart from slight impurities of nickel and copper.

The gate incorporates tie beams between the gate arms on the right and left sides. These are bolted to horizontal beams. One bolt had sheared and others were loose. Bolt holes had elongated and bolts and bolt holes were corroded; there was also evidence of relative movement of connections.

Fatigue Crack Distribution and Stresses

The length of cracks and their distribution varied. The majority of cracks were located at or just below the transverse beam tying the upper gate arms, and at or just above the lower transverse beam tying the lower gate arms.

The concentration of cracks corresponded with the maximum nominal shear stress in the web of the vertical stiffener beams when the gate was subject to a hydrostatic load at reservoir retention level, which is 190mm below the gate crest. The calculated maximum nominal shear stresses in the webs were about 65% of the permissible values. The actual stresses were considerably higher due to the stress concentration caused by the rectangular holes and poor welding of the inset plate. The stress concentration factor was assessed at 3 (Peterson, 1974). This resulted in actual shear stresses within the range of failure due to alternating stress.

The gate was surveyed but no other failures were discovered. A detailed structural analysis was carried out, which showed that the original design was based on generous safety factors and that the gate would withstand a considerable overload. With few exceptions, working stresses were below the fatigue limit of weldable structural steel of equivalent grade.

Remedial Work

The lip of the gate was modified by removing the projecting shoe shaped section, Fig. 6.

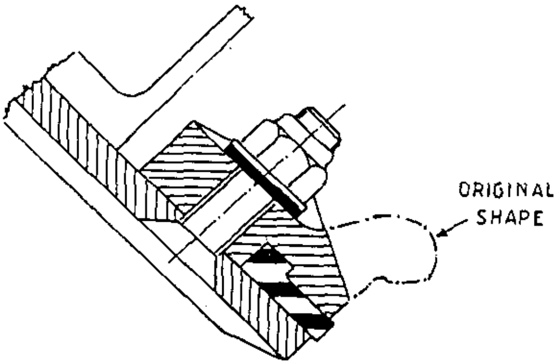


Fig. 6. Modified gate lip

This ensured clean flow separation and avoided reattachment of flow at the projecting section.

The downstream flanges and the webs of the vertical stiffener beams were cut off close to the leg of the upstream T-Section. New webs and flanges were welded on. The webs were continuous without any holes, Fig. 7.

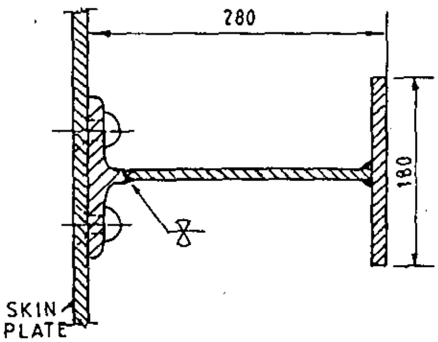


Fig. 7. Replacement webs and flanges of vertical stiffener beams

The elongated holes at the bolted junctions between the gate arms and the horizontal beam at the upper gate arms were reamered to the next larger size. Bolts and nuts were replaced with larger size bolts of a higher strength grade. The gate was shot blasted and repainted.

Site Work and Quality Control

Following discovery of the cracked ribs the turnkey contractor (Alstom) was instructed to carry out approved remedial works. The site works were undertaken by MacKay & McLeod of Evanton as sub-contractor to Alstom. All areas were thoroughly blast cleaned and the affected ribs cut out down to the base of the webs. Coded welders were used to rebuild the ribs under the control of approved NDT specialists.

All work was carried out behind steel 'needles' (larsen piles) which provided a satisfactory working area. The gate programmable logic controller (PLC) was commissioned as a model to agreed theoretical flood hydrographs. The gate was then commissioned on site to simulate openings via the PLC, remote pulse and hand control. The open and close movements were restricted to 150mm.

Reinstatement of the Gate

When the defects in the gate were discovered, it was considered that incipient fatigue might have developed at other structural members. This would have necessitated replacement of the whole skin plate assembly. The examination of the gate and the results of the structural analysis showed that the replacement of the skin plate was not warranted.

The repair work, carried out to a high standard, was accepted as a permanent solution. Vibration of the gate has been eliminated at all stages of gate opening. Operating regimes have been adapted to allow alternating sources of compensation water release.

Sill Beam

On a first investigation of fatigue cracks it was considered that the triangular cut out in the sill beam might have to be filled to produce a level surface, because fluctuating pressures could occur in the upstream section when the gate started to rise. No instability was apparent in the model study and the sill beam was retained in its original shape. Following modification, site tests of the discharge release at low openings with both the original and adjusted sill detail have shown no detrimental effect on the discharge.

Conclusion

The design of the gate lip was the root cause of vibration. The requirement that the lip of a gate should have a short cut off point is important (Schmidgall 1972, Hart et al 1979, Vrijer 1979, Lewin 1995). Equally important is the avoidance of flow reattachment (Lewin 1995).

The occurrence of gate vibration should be reported by operators as soon as it is noticed and the cause identified and rectified. This is important even where vibration is 'driven' through.

TEDDINGTON WEIR RADIAL GATES

Introduction

Teddington Weir is the upstream limit of the tidal river Thames. In 1989 the National Rivers Authority, now the Environment Agency, started reconstruction of the weir. New radial gates of the overflow and undershoot type were installed. The gates can be operated in the overflow mode when they act as weirs or as conventional undershoot radial gates.

On commissioning of the first gates they vibrated during discharge under the gates. The sill seal was formed by a timber block, a feature which had previously been used on Thames weirs with smaller gates of this type.

The gate vibration was due to intermittent flow reattachment at the timber sill seal. After demonstration in a hydraulic model study, the gate lip was modified to provide a sharp cut off. The installed gates were fitted with the redesigned lip and the additional gates were altered in a similar manner. The gates subsequently provided good service.

The Weir Control Gates

The gates are 4m wide and 2.7m high with a skin plate radius of 4.2m, Fig. 8.

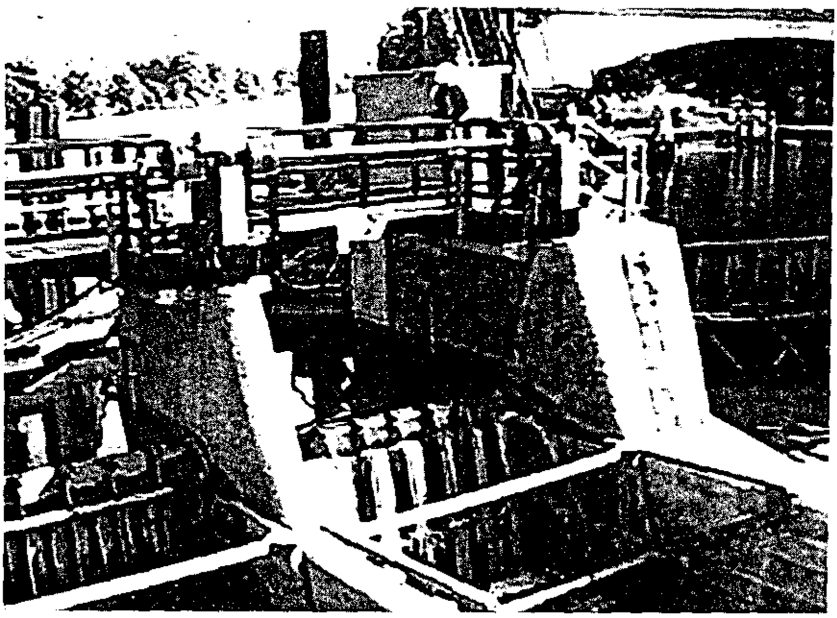


Fig. 8. Teddington weir gates

Flow breakers are fitted to the crest of the gates to prevent vibration due to fluctuating low pressures under the nappe during overflow conditions. The gate sill seals were formed by a timber block, Fig. 9. This sealing arrangement had been used with apparent success at other weir gates on the Thames. However, these gates were significantly smaller than those installed at Teddington.

Gate Vibration

On commissioning, the first two gates vibrated during discharge under the gate. Vibration was estimated to be in the region of 5-7Hz and could be observed at the gate, the hoisting drum drive shaft and the chain drive of the hoist shaft. An attempt was made to improve the flow under the gate by fitting a curved metal fairing section, but this did not stop the gate vibration. Vibration was due to flow separation and reattachment at the timber block which formed the seal. The process was demonstrated in a hydraulic model study at Kingston University. After the model was modified so that the gate lip provided a sharp cut off, the discharge under the gate was steady.

Remedial Work

The gate lip was altered to provide a sharp cut off, Fig. 10, and the section was redesigned to prevent a trapped shear layer below the bracket mounting the seal where eddies could form. At the same time, the approach flow to the weir crest seal was faired by introducing a section above the seal. After these alterations the gates performed satisfactorily during underflow conditions.

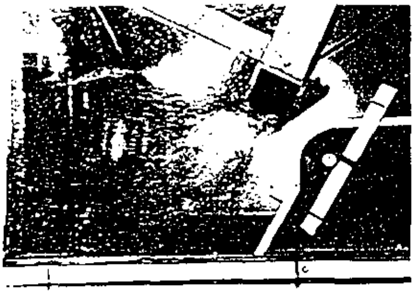


Fig. 9. *Timber block sill seal*

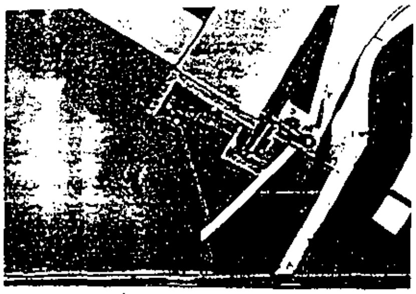


Fig. 10. Modified sill seal

Conclusion

Timber blocks were used as sill seals on a number of weir gates on the Thames. It was also the practice of the former GLC at tributaries of the Thames within the GLC region. The gates were small in width and height, thus the damping effect of the side seals was high in relation to the exciting forces caused by flow reattachment. It was the practice at Thames Weirs to use Sorbo rubber for side seals. Sorbo rubber has a significantly higher coefficient of friction than natural rubber or Neoprene because of its macro structure. This created a hoist capacity problem at the gates at Teddington Weir. In smaller gates it had increased damping. Based on previous experience, it must have been assumed that the use of timber sill seals was good, or at least reasonable, practice.

Hydraulic secondary forces increase with size of the control structure and applied head, therefore some hydraulic phenomena will not be significant until there is a change in size and imposed load.

Guidelines to avoid gate vibration due to flow reattachment require that there should be a sharp cut off at the point of discharge (Lewin 1983 & 1995). Structural members should not be located close to the gate lip to prevent flow reattachment, and the geometry at the upstream side of the gate lip should not create a trapped shear layer (Lewin 1983 & 1995).

TORRUMBARRY WEIR CONTROL GATES

The Weir and Gate Installation

The basin of the River Murray covers the States of New South Wales, Victoria, South Australia and Queensland. It drains 70% of the Australian Continent. The uppermost weir on the Murray, originally constructed in 1923, is at Torrumbarry. It incorporates a lock. Following a major piping failure of the weir apron and the decision to reconstruct it, a new weir was built alongside the existing lock, which was retained. The old weir had been controlled by an unusual arrangement of moving trestles operated by a steam driven winch.

The new weir incorporates six radial gates of 11m span, 6.25m high with a skin plate radius of 9.3m, Fig. 11.

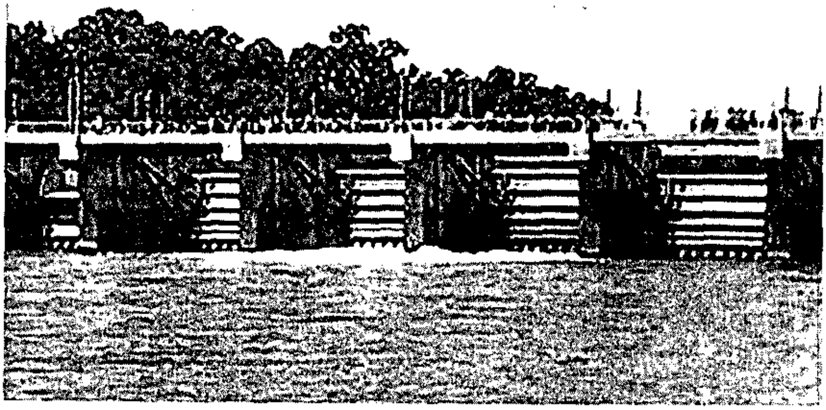


Fig. 11. Torrumbarry weir

The gates at the abutments have overflow flaps to assist in clearing floating debris, which under flood conditions includes substantial logs. The gates are operated automatically by programmable logic controllers (PLCs) with manual push button control as a standby. (Goulburn Murray Water, 1997).

The skin plate assembly is supported by two braced arms per side. These are of structural hollow sections. The horizontal members stiffening the skin plate are formed of trapezoidal closed sections and the lower sections are faired to prevent the lodgement of floating timber. As a result, the skin plate structure is unusually stiff in comparison with a construction using open girders to reinforce the skin plate.

Gate operation, as well as raising and lowering of the overflow flaps, is by oil hydraulic cylinders.

Gate Vibration

During the first flood season after the weir was put into operation in September 1996, gate vibration occurred. At that time the gates were partially open, with the upstream water level at 6.05m above the sill and the downstream water level 5.37m. When the gates were open between 3 and 4m, vibration was observed. From measurements at site, the motion appeared to be mainly fore and aft, with smaller rotational motions and even smaller *transverse motions.* The main frequency was approximately 10Hz. The length of the piston rods of the hydraulic operating cylinders appeared to be unchanged as the gates vibrated. As the gate was submerged, the vibration frequency fell due to increase in added mass. To ascertain the relationship between gate opening, predicted exciting frequency and predicted natural frequency of the gate, a graph was plotted, Fig. 12.

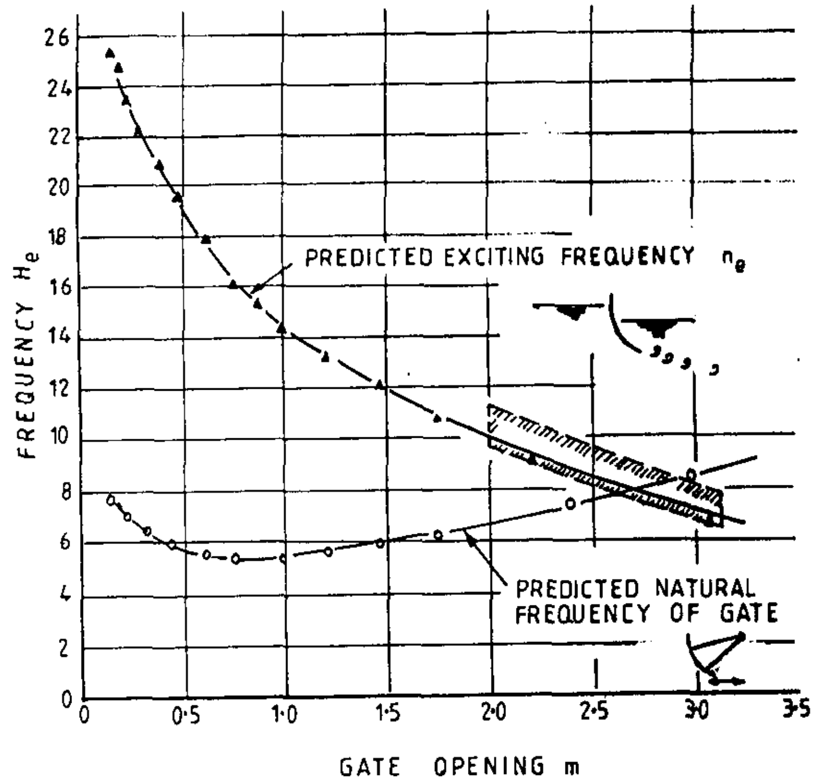


Fig. 12. Torrumbarry weir gates – gate opening versus frequency

An equation was postulated:

$$\frac{n_1}{n_2} = \sqrt{\frac{m_2}{m_1}}$$

where
n_1 & n_2 are the frequencies of vibration at different submergences 1 & 2
m_1 & m_2 are the total mass vibrating at submergences 1 & 2
total mass is mass of the gate plus added mass of water vibrating with the gate

Limited site prototype data supported this equation. It was assumed that the gate vibrated in the same mode in all cases.

A hydraulic model study was carried out at Imperial College by Dr. David Hardwick. The model was to a scale of 1:25. The most likely source of excitation was considered to be eddy shedding from the lip of the gate. The eddies shed from the lip were photographed and analysed. The results suggested a linear variation of the average exciting frequency with the velocity along the free shear layer.

Observations at the weir showed a range of vibration frequencies of 9.9-11.0Hz. This variation reflects the influence of a varying added mass and the tendency of the eddy shedding to lock into the natural frequency of the gate. The peak excitation occurred when the natural and exciting frequencies coincided but the vibration persisted when the natural frequency was 5.9% below the exciting frequency and 11.2% above.

The field and laboratory observations pointed to an excitation of the lip of the gates by pressure pulsations associated with the shedding of eddies.

A number of possible solutions were investigated. Structural modifications were considered unrealistic, and reducing the stiffness of the gate lip to lower the natural frequencies would have required a model scaled to elastic similarity in order to be representative of full scale behaviour. Dr. Hardwick suggested breaking up the regularity of eddy shedding. Flow breakers to be attached to the upstream lip of the gates were designed and tested in the laboratory. Observation of the capability of the flow breakers to impede the regularity of eddy shedding was promising.

Initially one gate was modified by bolting ten flow breakers to the upstream face of the gate lip spaced 600mm apart, Fig. 13.

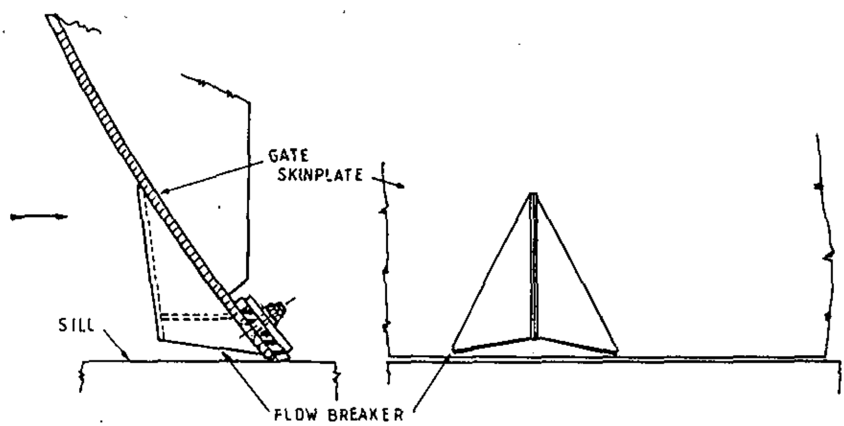


Fig. 13. Torrumbarry weir gates – flow breakers

This was successful, and the other gates were then similarly modified.

Conclusion
The gates had been checked at the design stage for possible vibration using a Strouhal number of $^1/_7$. This is the only published figure and is for a flat plate. The frequency of a vortex shed from the lip of the gate was calculated on this basis. It substantially underestimated the exciting frequency. The Strouhal number does not appear to be applicable to radial gates and may not be correct for vertical lift gates.

The natural frequency of vibration of the gate was also underestimated at the design stage. For investigation of gate excitation due to eddy shedding it is probably only necessary to consider vibration in the fore and aft direction. To compute the exciting forces on a gate due to eddy shedding at the lip of the gate, the frequency of eddy shedding at different gate openings and submergence has to be established. The tendency of a gate to regularise unsteady exciting forces must also be taken into account. This suggests that there should be a significant difference between the frequency of eddy shedding and the natural frequency of the gate, taking added mass into account.

At the time of preparation of this paper, research is in progress at Imperial College to investigate whether relationships can be established for estimating the frequency of eddy shedding at the lip of a gate.

ACKNOWLEDGEMENTS
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Teddington Weir Radial Gate: With grateful acknowledgement to the Environment Agency, Flood Defence Operations, South East Area – Thames Region.
Torrumbarry Weir Control Gates: Published with the kind permission of Goulburn-Murray Rural Water Authority and the Murray-Darling Basin Commission, Australia. Thanks are due to Dr. David Hardwick who carried out the hydraulic model study at Imperial College, and suggested the solution to the vibration problem.

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Deterioration of pre-tensioned bars retaining radial spillway gates leading to failure and loss of tension - a case study

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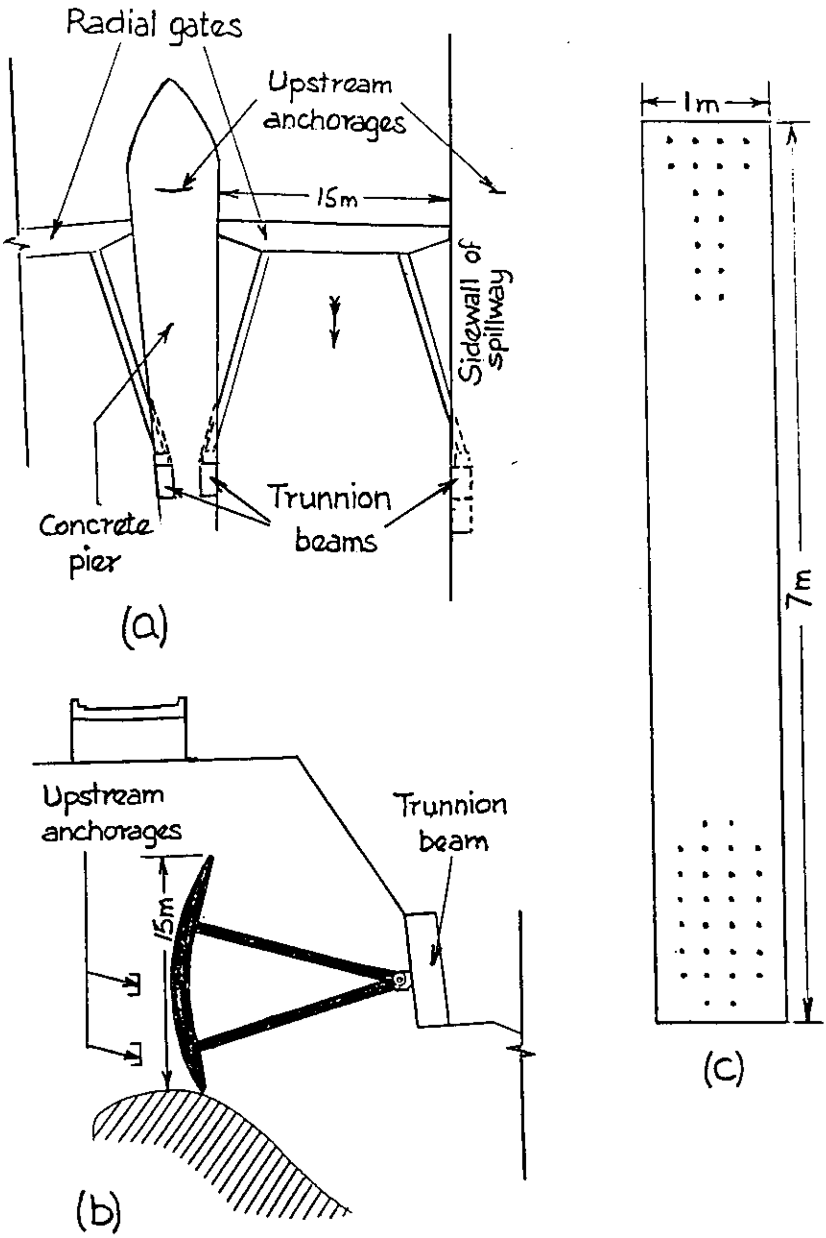
SYNPOSIS. The 12800 m^3/s spillway of the dam covered by this paper is controlled by radial gates. The thrust on the gate trunnions was transmitted to steel box beams which are tied to the concrete piers of the spillway monolith by pre-stressed steel bars. During safety reviews of the dam some of these bars were seen to have failed and others were suspected of having lost tension, either partially or completely. Tensile tests confirmed the loss of tension in some bars. Attempts to extract some bars which were known to have failed demonstrated attachment within the structure not intended in the design. The paper reviews the deterioration observed over time, as well as the unexpected attachments found, and describes the measures taken to ensure the continued function of the intact bars remaining, allowing for future deterioration, as well as to extract and replace failed bars.

INTRODUCTION

The spillway of the dam (which is overseas and whose identity has been withheld at the request of the client) is divided into three channels each controlled by a radial gate 15 m wide and 15.55 m high, measured vertically from the plate on which the bottom seal bears to the top edge of the gates. The full capacity of the spillway would be realised when the reservoir level is significantly higher than the top of the closed gates, but at that time the gates would be fully open and the drawdown curve of the water surface would be below the bottom edge of the gates. The thrust on the gate trunnions is transmitted to steel box trunnion beams, two per gate, one on each side of each spillway channel (Figure 1). The beams are vertical, except for a slight upstream tilt, and each takes thrust from one gate only. They are 1 m wide by 2 m deep, in an upstream-downstream direction, and 7 m high. They were each tied to the upstream zone of the concrete piers of the spillway monolith by 44 pre-tensioned steel bars per beam: a total of 264 bars in the whole spillway. The bars are approximately 19m long, and 26 mm nominal diameter, and were originally pre-tensioned to 30 tons force which, if transferred to the upstream anchorage, would place the load where there is sufficient concrete further downstream to carry the load in compression. Design drawings of the bars show that they were to be sheathed by "jute-served bitumen". Removal of the side panels of the trunnion beams showed that, where the bars pass through the open space inside the beams, they were sheathed by a corrugated ferrous material much of which had corroded away. The bars pass through holes in the

Dams 2000, Thomas Telford, London, 2000.



NOT TO SCALE

Fig 1 (a) Half-plan of spillway
(b) Section through radial gate and spillway crest with concrete pier in background
(c) Location of anchors on downstream face of trunnion beam

plates forming the upstream face of the trunnion beams, which bear on the concrete of the piers, and from that point are fully surrounded by the structural concrete of the piers which appears to have been cast around the bars, with or without sheathing.

OBSERVATION OF FAILED BARS

During a safety review of the dam in January 1998, by engineers from Binnie & Partners (Overseas) Ltd (a member of the Black & Veatch group), some of the bars were seen to have failed in tension, having broken ends protruding loosely from their anchorages. Others were shown to have failed or lost tension because their ends could be moved sideways by hammer blows. It was suspected that there were yet more which had lost tension at least partially, although this could not be readily seen.

The failure of 3 bars had been noted in a previous safety inspection in 1986, and more were noted as failed, or having reduced tension, in 1989/90 by a specialist contractor who was engaged on other works on the gates. Remedial work on the anchors was not within that contractor's terms of reference so he tightened up the downstream anchor nuts on certain bars and reported the failures he had observed.

FORCES IN ANCHOR BARS

The original force tying the trunnion beams to the piers, due to the pre-tension in the bars, was 1320 tons force per beam and thus 2640 tons force per gate. The trunnions were located below the mid-point of the beams and 16 of the anchorage points of the bars were grouped together towards the top of each beam and 28 grouped towards the bottom, so that the centre of gravity of the anchorages was at the level of the thrust from the trunnions.

The horizontal water thrust on each gate is 1884 tons force when the gate is overtopped by 0.3 m (the original design criterion). The restraining force in each bar to resist this would be 21.4 tons, if the force is equal in all 44 bars of each trunnion beam. Thus originally the factor of safety against movement of the trunnions, which would occur if the thrust of the water on the gates caused the load in the bars to exceed their pre-stress, was 2640/1884, i.e. 1.4. However this factor of safety had clearly been reduced by failure of some bars, and by any loss of tension which had occurred in others. Furthermore the centre of gravity of the restraint provided by the bars would have moved away from the centre of gravity of the anchorages because there was either limited restraint or none at some anchorages. Displacement of the centre of restraint from the centre of the anchorages means that thrust from the trunnions will cause unequal loads in the bars, so that if the reservoir level was high enough the load would exceed the pre-stress in some bars before it does in others. In that case the total water thrust on the gates, at which movement of the trunnions occurs, would be less than the total of the pre-stress remaining in the bars restraining that trunnion beam.

Breakage of the bars, as opposed to movement, would not occur until their ultimate tensile strength was exceeded, although an element of permanent extension could be expected if the stress were to exceed 75% of their ultimate strength. Tests carried out on three samples of broken bar gave ultimate tensile strengths of 59, 54, and 51 tons respectively with typical elongation after fracture of 1.4%. The modulus of elasticity was 212000 MPa.
The reservoir records show that in previous years the reservoir was sometimes allowed to rise above the 0.3 m surcharge criterion. One time at least, the resulting thrust on the gates, even without considering any transient surcharges due to waves, exceeded the load which would have caused movement of one gate, if at that date the failures and loss of tension, found in the anchor bars of one of its trunnion beams in 1998, had already occurred.

CONTRACT FOR REMEDIAL WORKS

An Engineer Procure and Construct (EPC) contract was awarded to Black & Veatch (UK) Ltd for remedial works. This allowed the Binnie & Partners engineers who were involved in the safety appraisal to be directly involved. The advantages of this method of proceeding, and the progress which was achieved, are described in Mason et al (1999). The contract also included remedial works to the gates themselves, which were jamming. Those works are not described in this paper but are described in Hallowes et al (2000).

OPTIONS FOR REPLACING BROKEN BARS

Three options were identified for replacing broken bars. The first was to install one piece bars similar to those being replaced. The second was to install shorter lengths of bar joined by couplers. The third was to install steel strands similar to those used for ground anchors.

One piece bars (19 m long) were considered impracticable. Firstly, they would have needed to be supported on a suitable alignment for insertion into the holes from which the broken bars were removed. This would have required extensive scaffolding founded on the walls between the spillway chutes which fall away towards downstream. Secondly the original bars were aligned on a curvature of slightly over 100 m radius within the piers, and the curved holes might have caused excessive resistance to insertion of, or damage to, replacement bars. Thirdly one trunnion beam was inset into the side wall of the spillway which retained the adjacent rockfill embankment; it would not have been possible to align single piece bars downstream of that beam unless slots were cut into the side wall. To do that would have been difficult and costly, and could also have compromised the structural integrity of the wall. Also, transport of such long bars to site would have presented significant problems.

Shorter lengths of bar joined by couplers presented no transportation problems, and the diameter of bar adequate to take the 30 ton pre-stress load was less than the 26 mm original bar. However, the outer diameter of the couplers would have exceeded the diameter of the holes. It would have been possible to ream out the original holes but this would have been time consuming and costly and it is likely to have involved cutting through reinforcing bars within the piers.

Multi-wire steel strands of 18 mm diameter, of guaranteed adequate strength to allow a "locked-off" load of 285 kN (29 tons force) could be supplied. Even with a plastic anti-corrosion sheathing their diameter was less than that of the existing holes and with grease inside the plastic they would be double corrosion protected. They presented no transportation problems and allowed a workable installation method, even for the trunnion beam which was inset into the spillway wall. Therefore, use of steel wire strands was selected as being the only practicable system for replacing broken anchor bars. The strands were anchored by taper wedges in wedge blocks reacting against anchor plates at each end and tensioned using hydraulic jacks. Using grease and end caps with gaskets, the anchorages also were double protected against corrosion. The extra amount that the strands had to be pulled through, to allow sufficient relaxation for the wedges to lock, defined the maximum load which could be relied on. It was decided that a slight reduction of pre-stress (to 29 tons force from the original 30 tons in the bars) would be acceptable. However the test certificates for the strand actually supplied showed a slightly higher strength than the guarantee and this allowed strands to be locked off at near 30 tons.

The extensibility of the strands under a given load is four times that of the bars. So long as the pre-stress load in the strands and bars is not exceeded by the distributed thrust from the gates, this difference will have no practical effect, because the extensions of both strand and bar at that load are already locked in. If the prestress load should be exceeded and movement initiated, then the lower modulus of the strands will mean that they will not take up additional load as quickly as the bars, and therefore the bars will take up more load than they would do if all the anchors consisted of bars. Similarly there will be more movement caused by any given overload. This means that having some strands will reduce the ultimate factor of safety of the anchorages slightly. However the rule of operation of the gates should be, as it was before, such as to make sure that the pre-stress load of the anchors is not exceeded, because once any movement occurs there is a risk of the gates jamming.

TESTING OF EXISTING BARS

All bars which were not seen to have failed were tested to 30 tons force applied by a hydraulic jack at the downstream end. After these tests had been carried out the scope of the work was defined as follows:

Failed bars:	18
Bars to be replaced:	20
Bars to re-tension:	10

The number of bars needing replacement exceeded the number failed because the downstream ends of two had been broken off just beyond the anchor nuts, leaving insufficient thread to attach a jack for testing or re-tensioning if needed. The behaviour of these two bars, when they were cut to de-tension them prior to removal, showed that they did still hold a significant tension, even though it could not be measured. The number of failed bars was one less than had been suspected previously because one bar which had been without tension was able to take a 30 tons load.

ACCESS

The upstream anchor points for the bars were located in chambers within the piers between the spillway channels, the lowest being in chambers at the bottom of shafts 23 m deep. There was a higher chamber for the top 16 bars and a lower chamber for the remaining 28 bars of each trunnion beam. These locations were confined spaces. Safe entry and working required gas monitoring and forced ventilation in addition to task lighting and provision of top men to prevent materials being dropped down the vertical access shafts.

Five of the six trunnion beams were mounted above platforms on the concrete piers between the spillways. Access was by permanent ladders and there were drops in excess of 15 m into the adjacent spillway channels from the platforms. The first task during the main works was installation of handrailing around them to enable safe working. Additional steel platforms were fabricated for access to the top 16 anchor bars on the trunnion beams. A footbridge constructed from prefabricated parts was erected over one spillway channel, to provide access to the sixth trunnion beam which was set into the side wall of the spillway. However until the bridge was installed, access to that area was obtained by abseiling down the side face of the spillway from a point some 15 m above the working area. Staff wore safety harnesses whilst working in this location.

REMOVAL OF BROKEN BARS

The broken bars had to be removed from both upstream and downstream ends. Some of the bars that required removal had readily accessible threads that could be cleaned up and connected to the hydraulic jack via threaded couplers. At the upstream end, however, some of the anchorages were recessed into the concrete wall of the anchor chambers and there was often little or no thread exposed. A method which worked well initially was to weld adaptors, to which could be attached a bar which fitted within a hydraulic jack, to the anchor nuts and then the hydraulic jack was used to pull the bar. After extraction of approximately 600 mm of anchor bar, the jack was fitted with three jaw tapered clamp pullers to grip the anchor bar. Successive lengths of about 2 metres had to be cut off by angle grinder because of the limited size of the chambers. However after the first few successes it was found that the adaptor could not be welded sufficiently strongly to pull some of the broken bars.

By cutting away the concrete around the upstream anchorages, the ends of the bars could be gripped and then sufficient force could be used to extract them. However the maximum force which had to be used at that stage to extract the end of a bar known to be broken, because the other end was loose and had been extracted, was 32 tons.

RE-TESTING EXISTING BARS

As the 32 tons force required to extract a part of one broken bar was in excess of the nominal pre-stress of an intact bar and also higher than the force used to check the integrity of the bars, this prompted a re-assessment of the testing process. It was concluded that there could be broken bars that had not been identified by the previous tests which were limited to 30 tons. It was possible that some bars were broken, and would move at a force close to 30 tons and would therefore have almost no safety factor. It was therefore decided to re-test all anchor bars that were not known to be broken, this time with a load of 37.5 tons. This figure was picked as being 75% of 50 tons which the tests on samples indicated as a low estimate of the ultimate tensile strength of the existing bars, and therefore 37.5 tons was the highest load that would not be likely to incur a permanent set in the bars. A four stage testing procedure was carried out as follows.

Stage 1. The hydraulic jack was attached to the bar and the load increased until the anchor nut just lifted off the anchor plate and the jack load recorded. The load was then released.

Stage 2. A load of 37.5 tons was applied, held for a short period of time and then released. A number of anchor bars broke during application of this load. Where this occurred the load at the point of failure was recorded.

Stage 3. A load of 32 tons was applied and held during the re-tightening of the anchor nut. This load was selected to allow sufficient extension of the bar for the nut to be re-tightened and to leave 30 tons force in the bar once the tension was released and the seating losses taken account of.

Stage 4. The load was increased until the anchor nut just lifted off the anchor plate and the jack load was recorded. The load was then released. Where the load at lift-off was under 30 tons, steps 3 and 4 were repeated until an acceptable result was achieved. Where the load was too high, a load was applied that lifted off the nut, the nut was loosened and the process was repeated until an acceptable result was achieved.

During the higher test loads it was noted that a number of bars were stretched further each time the load was applied. This movement of the bars with respect to the surrounding concrete was generally accompanied by concrete dust being extruded at the interface between the bar and the concrete against which the trunnion beams bear. Some bars were checked for extension against load, applying the modulus for the bars obtained from the laboratory tests on samples. The limited extension under load in some cases appeared to demonstrate that the bar was actually bonded part way along its length rather than being free all the way to the anchorage in the chamber at the upstream end. It is believed that further stretching of some bars under repeated application of the higher loads resulted from progressive detachment of the bars from points along their length where they had previously been bonded to the surrounding concrete. Thus parts of some bars were being loaded fully for the first time. In some cases, the ferrous sheath surrounding the bars near the downstream end was seen to have been penetrated by concrete grout. It is possible that this occurred elsewhere along the length of some bars creating a bond between the bars and their sheaths which in turn were held by the surrounding concrete, contrary to the apparent intention when a sheath of "jute-served bitumen" was specified. Alternatively it is possible that a continuous sheath was not installed.

One bar was so severely restrained that it could not be extracted. It withstood 37.5 tons load, but was extended irreversibly and the thread was not adequate to allow tightening to make up for that additional extension. Therefore it was cut preparatory to extraction. However it could not be moved despite making special arrangements to allow first two jacks (one pulling from the upstream end and one pushing from the downstream end) and then three jacks (one pulling from the downstream end and two pushing side by side from the upstream end) acting together. The maximum force applied was a total of approximately 80 tons. From the length that the bar extended when pulled from the upstream end it appeared that the point of fixity was close to the downstream end of the pier. Some of the concrete surrounding the bar at the downstream end was cut away in an attempt to free the bar, but the fixity was beyond reach and it was decided that the damage that would be caused by further removal of concrete was not justified by the benefit of having 44 rather than 43 fully tensioned bars or strands in the anchorage. The factor of safety for the forces restraining the gate remained adequate. Therefore the broken bar was left in place.

TIGHTENING SLACK BARS

Under-stressed bars were re-tensioned at the downstream anchor point. First the thread on the bars was cleaned with a wire brush and then re-cut using a die of the original thread size (M27x2).

A puller bar with a coupler threaded to match the anchor bar was attached to the anchor bar. A hydraulic jack was then placed over the puller bar at the same time as a short slugging ring spanner. An attachment block was threaded onto the end of the puller bar, and the hydraulic jack was stressed until the test load was achieved. The load caused the bar to extend and the attachment nut to be lifted off the anchor plate on the trunnion beam. The nut was turned by hammering the slugging spanner until it was tight on the anchor plate. The load was then released on the hydraulic jack and reapplied. This process was carried out until the nut did not lift off on application of the test load.

REPLACEMENT OF FAILED BARS BY STRAND

Once the pieces of a failed bar had been extracted the following method was used to install the replacement strands.

Stage 1. The ties holding the strand coiled for transport were cut on top of the dam. The strand was then held by two ropes and lowered to the appropriate trunnion beam platform.

Stage 2. A nose piece was attached to the strand king wire and the strand was inserted through the downstream face of the trunnion beam and fed through the hole until it reached the upstream chamber. Throughout this process intercom communications were maintained between the teams working in the upstream chamber and those at the trunnion beam.

Stage 3. Upstream and downstream anchor plates were fitted over the strand. These plates had locating tubes which were tight fitting on the strand outer sheath in order to minimise the risk of corrosion damage to the strands.

Stage 4. The upstream and downstream anchor blocks and taper wedges were fitted and the stressing jack installed downstream.

Stage 5. A small load was applied that pulled the upstream taper wedges tight. The alignment of the upstream anchor plate was checked, and then the upstream team left the chamber.

Stage 6. The jack was stressed to the datum load, reset and then stressed to the final load that took account of lock-off losses. The jack pressure was released rapidly to effect lock-off. During this process measurements were taken that allowed strand slip at lock-off and strand extension to be calculated.

Stage 7. The jack was stressed and the lift-off load was recorded.

Stage 8. The excess strand was removed leaving a preset length that allowed for re-tensioning in future. Preservative grease was applied to the strand and the anchor blocks both upstream and downstream. Finally, sealing caps and gaskets were installed over the strands and bolted to the anchor plates.

FINAL SCOPE OF REPLACEMENT AND RE-TENSIONING

As a result of the tests with the revised procedure at higher loads more unsatisfactory bars which needed replacement were identified. Nine bars broke at 37.5 tons or loads between 30 and 37.5 tons, indicating that they had deteriorated to the point that they were not far from failing under the normal pre-stress load. The final tally of replacement and re-tensioning work is given below:

Bars under-tensioned – retensioned to correct value:	204
Bars at correct tension:	18
Bars overtensioned – retensioned to correct value:	3
Bars replaced by strand:	38
Failed bars which could not be extracted:	1

One effect of this further round of testing was to increase the factor of safety of the remaining bars as they have all now been tested to 1.25 times the pre-stress load. This should have the effect of reducing the number of further failures in the near future since those which were close to failure should have been eliminated.

OBSERVATION OF CONDITION OF BARS

Observation of the broken surfaces of failed bars showed fan shaped markings extending across the bars from a point or area of initiation on one side. The surface of all bars was corroded, although no loss of section was visible to the naked eye except where the bars passed through the inside space of the trunnion beams. It appears likely that the failures originated in corrosion pits on the surface of the bars.

Noticeable corrosion had occurred within the trunnion beams. The corrugated ferrous sheathing had rotted away on almost all the anchor bars on which it was originally installed. This is considered to be due to the fact that water was able to enter the trunnion beams, leading to very high humidity inside the beams during hot weather. Corrosion inside the beams was particularly significant because it meant that the thread there was often lost and thus did not allow tightening of a bar which had stretched.

All debris within the trunnion beams was cleaned out, all surfaces including the original bars were repainted to inhibit further corrosion and holes were drilled to allow any water entering them to drain out in future.

The threaded ends of the bars protruding from the anchorages on the downstream faces of the trunnion beams, are unprotected and had suffered minor corrosion. However this was much less than the corrosion which had occurred inside the beams, presumably because of the dryness of the atmosphere when it is not raining.

UNUSUAL INSTALLATION PRACTICES

During the extraction of bars a number of instances of unusual installation practice were found. There were signs that some bars had been welded to steelwork within the piers. This was presumably to assist in holding the bars in position while the concrete of the piers was poured around them, but it had the effect of creating a permanent bond between the bars and the piers and thus negating any protection against bonding provided by sheathing. It is also likely to have compromised the strength of the bar in the area of the weld.

One of the anchor bars was found to be broken, but to be welded to the inside face of the downstream plate of the trunnion beam, possibly so that it would withstand a test load. Some bars which required considerable force to initiate withdrawal from the downstream end, were found to have pieces welded to them in line with the upstream face of the trunnion beams where they contacted the concrete pier. One failed bar was found to have a welded joint; surprisingly it had failed elsewhere and the weld remained intact.

THE EFFECT OF BONDED BARS

In view of the difficulty of extracting some of the bars which appeared to be bonded along their length, it is worth questioning whether this was always worthwhile. If all the bars were bonded close to the trunnion beams they would put the concrete of the piers into tension which would be undesirable. However it probably would not matter if occasional bars are bonded close to the beams provided they can withstand the pre-stress load without movement. It seems reasonable to say that if the bar which could not be extracted were still anchored to the trunnion beam and bonded to the concrete along its length, instead of there being no stress at all in that bar as is now the case, the situation of that beam would be slightly improved. However in the event of the pre-tension in the bars being exceeded, bars which are bonded close to their downstream end, and thus have a shorter stressed length, will be more vulnerable than the remainder, because any given movement will raise the stress in them more.

CONCLUSION

Gradual deterioration has been observed in the anchor bars at this dam, and it is probably typical of a process which is proceeding at other sites. Lack of provision for replacement of the bars is awkward but it was not insuperable on this site given the availability of modern high strength materials. Any new installation should have double protection against corrosion as was provided for the replacement strands in this case.

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A guide to the Reservoirs Act 1975

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SYNOPSIS. In 1999 the authors put together the material that is being published by the Institution of Civil Engineers as 'A Guide to the Reservoirs Act, 1975'. This paper introduces the Guide, describes the sources of information used, explains its aspirations and highlights a number of points that have emerged during its compilation. The objective of making the guide intelligible and useful to non panel engineers is seen as important and the paper explains how this affected the structure and contents. The paper is intended to promote discussion on reservoir safety issues.

INTRODUCTION

In 1995, the Institution of Civil Engineers (ICE) issued an explanatory document entitled 'Information for Reservoir Panel Engineers', which was issued free to all panel engineers. In 1997, the ICE issued a questionnaire to all panel engineers asking for feedback on the operation of the Reservoirs Act 1975 (the Act), following which discussions were held with government concerning possible changes to the Act (Sims & Parr, 1998). For a number of reasons, although possible legislation was identified in outline, it was not possible to proceed with the proposed changes.

Following this, the Reservoirs Committee of the ICE, in conjunction with the Department of the Environment, Transport and the Regions (DETR), decided to update 'Information for Reservoir Panel Engineers'. Brown & Root were appointed to carry out this work in February 1999, under the direction of a Review Group, effectively a sub-committee of the Reservoirs Committee, chaired by Dr J A Charles. The Guide to the Reservoirs Act 1975 (the Guide) was produced in two draft versions; with the final version submitted to the ICE in September 1999.

PURPOSE OF THE GUIDE

The purpose of the Guide can be simply stated. It is to provide guidance on the application of the Reservoirs Act 1975 reflecting the current views and practice of the dam engineering profession.

In meeting this purpose it is intended that the Guide will fulfil the following principal objectives:

- Encourage high standards of safety in reservoirs
- Promote the proper application of the Act by all parties involved

- Encourage a consistent approach in the application of the Act, and
- Provide a reference document for use by all who have an interest in reservoirs.

The Guide is designed to be of use and interest not just to the panel engineers who have a defined role in the Reservoirs Act but also by staff of the Enforcement Authorities, owners of reservoirs and those intending to construct reservoirs.

The Guide is particularly aimed at providing guidance in situations where Enforcement Authorities, undertakers and panel engineers have experienced difficulty in applying the requirements set out in the Act. Such statements are intended to reflect the consensus view of users of the Act. However it should be noted that the Guide does not have the force of law.

It must also be emphasised that the Guide is not prescriptive and that all those involved in the implementation of the Act are required to use their professional judgement in the discharge of their professional responsibilities in accordance with the Act.

PREPARATION OF THE GUIDE

The first draft of the Guide used two main sources; the results of the questionnaire in 1996 to identify areas where the profession felt that a commentary on a particular issue or Section of the Act would be helpful, and 'Information for Reservoir Panel Engineers'. A meeting was held with representatives of an Enforcement Authority to ascertain areas of difficulty that they had encountered in applying the Act. There were also informal discussions with panel engineers and other professionals involved in the application of the Act.

There were four meetings of the Review Group, and the draft of versions of the Guide were circulated to the DETR, the Scottish Office, and members of the Reservoirs Committee, including representatives of Enforcement Authorities and owners of reservoirs.

This consultation process identified areas where there were varying opinions, and a lack of consistency by those involved in the application of the Reservoirs Act. The results of the consultation were consolidated into the Guide as the considered view of the majority of the profession. This commentary was then reviewed, and in some cases amended, during the discussions of the drafts with the Review Group.

In some instances, where alternative views were widely adopted by the profession the DETR stipulated what position should be taken.

CONTENTS OF THE GUIDE

The Guide is divided into 5 sections as follows:

Part A Background to the history and administration of the Act. It also includes flow charts to direct the parties involved in the implementation of the Act to the sections of the Act which are of particular relevance to them.

Part B Detailed commentary on the Act following the text of the relevant section of the Act

Part C A description of the content of all Statutory Instruments relating to the Reservoirs Act issued to date together with the full text of the Statutory Instruments which are still in force.

Part D Guidance on issues related to reservoir safety, including identifying technical guides.

Part E Appendices including checklists for various reports under the Act.

FLOW CHARTS

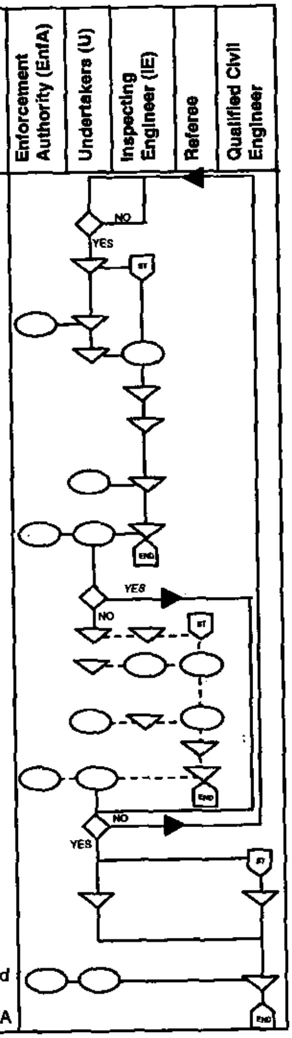
In reviewing the results of the questionnaire and holding meetings with a representative of an Enforcement Authority it became clear that some form of overview or summary of the main processes and procedures in the Act would be helpful. This would allow users to identify quickly the relevant sections of both the Act and the associated Statutory Instruments. To facilitate this flow charts of the main procedures and processes in the Act were prepared, showing the parties involved and the actions required by each party. Each chart was fitted onto one A4 sheet, to maintain the concept of an easy to use summary.

Six flow charts are included in the Guide, three covering the Supervision, Inspection and Construction or alteration of reservoirs and the other three relating to the activities carried out by Enforcement Authorities. Symbols were used to differentiate activities, decisions, and the receipt or supply of information. Symbols for the start and end of the involvement by a panel engineer were also used, to clarify the confusion that sometimes occurs over this, particularly the involvement of an Inspecting Engineer after completion of an inspection. The chart for inspection of reservoirs is attached here as Figure 1.

It is intended that these charts should be used both as a checklist to refresh the user's memory of the relevant sections of the Act, and also to provide information when dealing with an activity that is less routine (for example enforcement of the Act). It is envisaged that a copy of the charts could be pinned onto the office wall for quick reference. It would also prove particularly useful in discussions with owners of small reservoirs, who are often less familiar with the application of the Act.

CHART 2 - PROCEDURE FOR PERIODIC INSPECTION AND SUPERVISION OF MEASURES IN THE INTERESTS OF SAFETY
(Schematic only; refer to Reservoirs Act for details)

Section of Act	Statutory Instrument	Activity description
10(2)		Is inspection due? • Date for next inspection reached? • Inspection required for any other reason?
10(1)		Appoint Inspecting Engineer (IE) and check validity of their panel appointment
21(4)		Inform Enforcement Authority (EnfA) of appointment of IE
21(5)		Supply information
10(3)		Carry out visit and prepare report
10(4) & 11(2)		Decide matters to be watched by Supervising Engineer, and monitoring of records
10(3)		Issue report
10(5) 20(4)	1986-468	Issue Certificate, with copy to EnfA. Where there are recommendations in the interests of safety, then a copy of the report must go to the EnfA
19(1)		Are recommendations of report accepted?
19(2)	1986-467	*Appoint Referee within 40 days of report*
19(3)	1986-467	Undertakers submit written statement of *grounds for complaint*
19(3)	1986-467	IE makes written observations
19(3)	1986-467	Carry out visit *and investigate complaint*
19(4)	1986-468	Issue certificate
10(6)		Are any measures required in the interests of safety?
10(6)		Appoint qualified civil engineer
10(6)		Carry out measures required in the interests of safety, under supervision of qualified civil engineer
10(6)	1986-468	Visit to check that measures have been carried out satisfactorily, if so issue certificate that works have been carried out, with copy to EnfA



Legend:

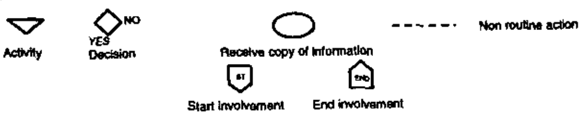


Fig 1. Example of one of six flow charts in the Guide

PART D GUIDANCE ON ISSUES RELATED TO RESERVOIR SAFETY

In planning the structure of the Guide, it was decided that in addition to the commentary on the Act, it would also be valuable to comment on a number of related issues. In broad terms this would comprise extending the contents of Information for Reservoir Panel Engineers (ICE, 1995), to provide information relevant to all users of the Act. This Part of the Guide would explain what information exists and how to find it, rather than reproducing the information.

The first section deals with general responsibilities, including citing Rule 3 of the ICE Rules for professional conduct and pointing out the various duties of undertakers, including that he will pay his panel engineers!

In the second section, the various pieces of legislation and issues related to Health and Safety are summarised, including the phone number and web site of the HSE and a summary of procedures for dealing with confined spaces. Other organisations that those applying the Reservoirs Act may need to contact are described, including the Environment Agencies and planning authorities. The need for consultation and complying with conditions that may affect SSSI and Ramsar sites are also noted.

The existence and scope of the technical guides and the reservoir safety research programme by the DETR is described, including other technical references that may be relevant to reservoir safety and performance. Attention is drawn to the role of the Supervising Engineer as a key element of the observational approach to the safety of old embankment dams. The need for continuing professional development of both panel engineers and other professionals involved in the application of the Act is emphasised, and the various meetings and publications of the British Dam Society and the International Commission on Large Dams noted.

TWENTY COMMON QUESTIONS (AND ANSWERS) ON THE APPLICATION OF THE ACT

Some of the areas of the Act which required considerable discussion to achieve a consensus or where guidance was required to achieve consistency of application of the Act are described below. The clause of the Act which deals with the issue is indicated in brackets against each question.

<u>How does the Reservoirs Act define safety?</u>

The Act does not define safety. It provides a legal and administrative framework for the construction and management of reservoirs in a manner which reduces to an acceptable level the risks associated with escapes of water from reservoirs. There are references to safety in Sections 16(1) and 16(2) of the Act, which refer to a reservoir that is unsafe and to the Enforcement Authority having the powers to take immediate action "to protect persons or property against an escape of water from the reservoir". From the above, it could be argued that safety is not defined in relation to the integrity of the structure forming the reservoir but in relation to its effect on the public, both persons and property.

Should road and railway embankments come within the ambit of the Act (Section 1(1))?

Some road and railway embankments may create bodies of water in flood conditions. The capacity of any drainage culverts through the embankment may not allow sufficient water to run-off in certain weather conditions and can result in the embankment temporarily retaining more than 25,000 cubic metres of water above the natural level of the surrounding ground. Section 1(2) of the Act suggests that, to constitute a reservoir, water must be artificially retained, and that for any case not involving lakes or lochs it must be intended to make some use of the water. Since the water retained by a road or railway embankment is not retained by design or for any particular purpose, this accidentally retained water is not a reservoir in the intended meaning of the 1975 Act. However, it should be recognised that road and railway embankments may pose a hazard if overtopped or if water is accidentally retained against them and should be designed to withstand the water forces imposed.

Can a reservoir whose capacity is reduced to below 25,000 m^3 by deposited sediment be removed from the Act (Section 1(1))?

The status of deposited sediment is not defined. Section 1(1) of the Act provides two tests for a reservoir to be a large raised reservoir; "it is designed to hold, or capable of holding more than 25,000 cubic metres of water". These tests have caused concern in relation to the registration of reservoirs, which have been partially filled with sediment, which is now so dense that it cannot flow.

These two tests are considered to be alternatives. Where it can be shown, by whatever means, that the reservoir was designed to hold more than 25,000 cubic metres of water, then the degree of siltation is immaterial; the reservoir must be regarded as falling within the ambit of the Act. Only where there is no clear evidence about the design capacity should the second test come into play.

A reservoir which was designed to hold more than 25,000 cubic metres of water must be registered under the Act. Its status can be reconsidered under Section 13 of the Act, *Discontinuance of large raised reservoirs*, but only as a result of alterations to reduce its capacity below 25,000 cubic metres that have been designed, approved and supervised by a qualified civil engineer. It is unlikely that the natural process of siltation can be considered to have been designed or approved and supervised.

How is the Act applied where the reservoir has multiple owners (Section 1(4))?

A number of cases have arisen where a reservoir has multiple owners and for example, the water is owned by one party, the crest road by another and the downstream face of the dam by a third. In this instance all of the parties involved are considered to be Undertakers. It is the responsibility of the local authority under Section 2 of the Act to identify all of the Undertakers.

The Undertakers may choose to share the costs of measures required in connection with the Act amongst the owners of the various parts of the reservoir. There have been instances of difficulty in securing agreement among Undertakers, for example, where access or works are required to parts of the reservoir which are not owned by the principal Undertakers who are carrying out the works. Provided all parties have been identified as Undertakers by the local authority, the powers under Section 17 of the Act can be used by the Enforcement Authority.

How does devolution affect the application of the Act (Section 2(1))?

From 1 July 1999, the Secretary of State's Reservoirs Act functions have been devolved to the Scottish Parliament and the Scottish Executive has the administrative lead there. From the same date the functions in relation to Wales were devolved to the National Assembly for Wales and its officials have the administrative lead there.

Under the Reservoirs Act 1975, the Secretary of State has functions in relation to:

- The oversight of Enforcement Authorities
- The appointment of qualified civil engineers
- The making of Statutory Instruments under the provisions of Section 5 of the Act

All three executives will continue to work closely with one another to preserve as far as possible the coherence throughout Great Britain of reservoir safety arrangements under the Act.

When should a construction engineer be appointed (Section 6(1))?

A construction engineer is appointed under three circumstances:

- Where a new large raised reservoir is constructed
- Where an abandoned reservoir is to be restored to become a large raised reservoir again
- Where an existing reservoir is altered to increase its capacity

A construction engineer is not appointed in relation to carrying out measures in the interests of safety or other rehabilitation works even where these are major construction works.

<u>Who appoints the construction engineer (Section 6(1))?</u>
The Act does not state who employs the Construction Engineer but the wording of the certificates in SI 1986 No. 468 indicates that the Construction Engineer is appointed by the reservoir undertaker or the Enforcement Authority (in the event of a default by the undertaker).

In recent years design and construct contracts have been used for service reservoirs and design, construct and operate contracts have been suggested. In such cases particular care is necessary in making arrangements for the appointment of the Construction Engineer, bearing in mind that the Construction Engineer has responsibilities to the Undertakers and the community after the completion of construction for a minimum of three years. A design and construct contractor is unlikely to be the reservoir undertaker but a design, construct and operate contractor could be the undertaker.

<u>When should the issue of a final certificate be delayed (Section 7(3))?</u>
The Final Certificate should be issued only after a minimum period of three years has elapsed following the issue of a Preliminary Certificate. One of the purposes of the three year period between the start of filling of the reservoir and the issue of the Final Certificate, is to enable the Construction Engineer to observe the performance of the reservoir as impounding proceeds. Unless the reservoir performance has been observed under these conditions it is unlikely that the Construction Engineer can be satisfied that the reservoir is sound and satisfactory.

It therefore follows that the Construction Engineer will normally delay the issue of the Final Certificate in accordance with Section 7(4), if the reservoir has not been filled within the three year period following the issue of the Preliminary Certificate. In the case of flood detention reservoirs, and some storage reservoirs which never fill, normal practice is to issue a Final Certificate with a schedule of matters to be watched when it does fill.

<u>Can inspections be performed for only one element of a reservoir (Section 10(2))?</u>
When recommending an inspection under Section 10(2)(c), the Supervising Engineer's concerns may relate to only one particular aspect of the reservoir. There is no stated provision in the Act for inspection to be limited to a particular element of the reservoir. While the Inspecting Engineer may only consider it necessary to look at this particular aspect in detail he should, nevertheless, consider and refer to all aspects of the reservoir in his report. Unless the engineer has carried out a full inspection the date of the next inspection should not be altered from that given in the previous inspection report.

When should recommendations be made in the interests of safety (Section 10(3))?

Recommendations made in the interests of safety place a legal obligation on the Undertakers to see that they are carried out. As discussed in the preamble to the Act, there is no clearly stated definition of safety given in the Act. From the references to safety made in the Act, it could be argued that safety is not defined in relation to the integrity of the structure forming the reservoir but in relation to its effect on the public, both persons and property. This would suggest that measures should only be recommended in the interests of safety if they are necessary to prevent an escape of water that poses a threat to persons or property.

Measures which may be recommended to preserve the integrity of the reservoir structures or for good maintenance practices, which are not recommended in the interests of safety, should be clearly identified separately. Such measures are not legally enforceable under the Act although the report should identify the possible consequences if timely action is not taken.

Who certifies that recommendations made in the interests of safety have been put into effect (Section 10(6))?

Section 10(6) states that measures recommended in the interests of safety should be put into effect under the supervision of a qualified civil engineer. In this context the qualified civil engineer is a member of the appropriate inspecting panel (see Section 4). He can, of course, delegate day to day tasks associated with the work but he retains overall responsibility. It is usual for the Inspecting Engineer under 10(1) above to be retained to supervise the measures in the interests of safety but Undertakers may, if they wish, appoint another qualified civil engineer on the appropriate panel to carry out this work. On completion of the recommended measures, a qualified civil engineer from the appropriate inspecting panel should issue a certificate stating that the measures have been carried into effect.

What are 'Matters to be watched by the Supervising Engineer' (Section 10(4))?

The Construction Engineer in the annex to the final certificate (Section 7(5)), or the Inspecting Engineer in his report (Section 10(4)) shall include a note on matters that the Supervising Engineer needs to watch (pay particular attention to). This is important, as under Section 12(2) it triggers the requirement for the Supervising Engineer to produce an annual written statement.

What is 'The form of record' (Section 11(1))?

This is defined in Section 11, and has the purposes of

- bringing together key information about the operation of the reservoir that would be valuable in the event of any problem that may affect reservoir safety.
- acting as a diary of the life of a reservoir in recording behaviour, maintenance, problems and steps taken to resolve those problems.

Who decides what should be recorded in the reservoir record (Section 11(2)?

Statutory Instruments 1985 No 177 and No 548 state what information should be included, these being 'prescribed' by the Secretary of State. However, Section 11(2) states that the Construction or Inspecting Engineer should give directions on the interval and manner of recording information, and this is an important duty of the Construction Engineer or Inspecting Engineer. This direction should give due and careful consideration of the purpose of the monitoring.

What is the annual statement by the Supervising Engineer (Section 12(2))?

This is defined in Section 12(2), although no format is given in the Act. The Guide includes as Appendix E5 a set of headings that suggest the information that should be included. The annual statement will be one of the key sets of information that the Inspecting Engineer looks at when carrying out his inspection, and as such it should be a complete record of all matters relating to the behaviour and safety of the dam over the year it covers. The Guide draws attention to the observational method for the safety evaluation of old dams in Part D7. The annual statement forms the written record of this process.

How often should the Supervising Engineer visit, and who decides this (Section 12(2)?

The Act does not prescribe the frequency of visits. However, in 1985 the DOE issued a note on the functions, duties and powers of the Supervising Engineer; this note is included in Appendix E3 of the Guide. This note states that the number of visits a year is a matter for the Supervising Engineer to decide. It also notes that the Inspecting or Construction Engineer may suggest a frequency and times of the year (although they are under no obligation to do so) and that the Supervising Engineer may wish to bear this in mind. The commentary on the Act suggests that a Supervising Engineer's visits to a reservoir are commonly twice a year and it is unusual to be less than once a year.

What is the difference between the discontinuance and the abandonment of reservoirs (Sections 13 and 14)?

Sections 13 and 14 of the Act cover the discontinuance and abandonment of reservoirs respectively. Discontinuance consists of rendering a large raised reservoir physically incapable of holding a capacity greater than 25,000m^3. Abandonment comprises measures to secure the reservoir so that it cannot fill with water above the natural level of any land adjoining the reservoir.

Discontinuance would be used when a reservoir is either completely removed, or when it is altered such that its capacity is reduced to below the threshold of the Reservoirs Act. Abandonment could be used when a non-impounding or service reservoir is to be taken out of active use by sealing the pipes bringing water into the reservoir, but not structurally altered. Abandoned reservoirs still remain on the register of reservoirs, require the continuous appointment of a Supervising Engineer and periodic inspections by Inspecting Engineers.

What is the procedure for the reference of disputed recommendations to a referee (Section 19)?

This is covered by Section 19 and Statutory Instrument 1986 No 467, a copy of the latter being given in Part C of the Guide. It is important to note that there are several time limits in this procedure. For example, if the undertakers are unhappy with the inspection report they must let the inspecting engineer know and appoint a referee in agreement with him within 40 days of the date of receipt of the report. If they fail to agree on the referee, they have 50 days from the date of receipt of the report to refer the matter to the Secretary of State.

What do you do when the wording of a certificate does not match the situation you are dealing with (Section 20(1))?

The DETR have stated that the wording of the certificates has been prescribed by law, and should not be altered. Where the wording of the certificate does not cover the situation encountered, but the difference is minor, the certificate may be signed and the difference drawn to the attention of the Undertakers and Enforcement Authority in an annex to the certificate. However, where the difference is such that the certificate would be incorrect, or seriously misleading, then a letter to the Undertakers and Enforcement Authority would be sent in place of the certificate. The letter should give a statement by the engineer on the matter, using wording similar to the certificate, including why the form of words on the certificate does not allow the engineer to sign it.

When should advice given by a Supervising Engineer to the undertakers be copied to the Enforcement Authority (Section 20(4))?

Section 20(4) of the Act lists documents that have to be copied to the Enforcement Authority by the various engineers. For Supervising

Engineers, this includes any advice to the Undertakers which recommends any action relating to reservoir safety, including a breach of Section 11 (recording of water levels etc). It does not include the annual statement.

FUTURE IMPROVEMENTS TO THE GUIDE

There is no room to be complacent about dam safety in the UK, as every year passes the stock of British dams gets one year older. The purpose of the Guide is to assist in ensuring that British dams continue to have a good safety record. The Guide will therefore need periodic review and updating, both because of changes in legislation and because of developments in technical and other issues related to the application of the Act. Devolution of Scotland and Wales is already changing the way the Act is administered. The public perception of acceptable hazard is also changing with time. The authors therefore encourage constructive comments on the Guide, which should be addressed to the Reservoirs Committee at the ICE.

CONCLUSIONS

It is clear from the authors' experience of the preparation of the Guide that the Reservoirs Act 1975 is not always applied consistently. In order to promote a more consistent approach, wide dissemination of experiences is vital and this should be encouraged to take place, particularly at the regular British Dam Society conferences, Supervising Engineers Forum and Inspecting Panel meetings.

The publication of *A Guide to the Reservoirs Act 1975* brings together the consensus views of the profession and it is anticipated that this will be a valuable document which will be widely consulted by all who have an interest in reservoirs.

This Guide should be updated in a few years time and all parties involved are encouraged to send constructive comments on the guide to the Reservoirs Committee at the Institution of Civil Engineers.

ACKNOWLEDGEMENTS

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Reservoir safety and quality management

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SYNOPSIS. In the context of reservoir safety, quality should be taken to mean delivering the required high level of safety without excessive cost. The publication of the series of engineering guides, including the recently prepared *A Guide to the Reservoirs Act 1975*, should encourage a more formal application of relevant quality management principles to reservoir safety. While this is beneficial and can be developed further, a simplistic attempt to make reservoir safety fit an existing quality management methodology based on customer satisfaction should be avoided.

INTRODUCTION

"Every dam, regardless of its size, is in some degree a potential menace to everything below it. There is nothing so relentless in its immediate destructiveness, so uncontrollable and deadly as a huge volume of water suddenly released." (Hinderlider, 1932). These are the opening remarks in a paper on *"Necessity for, and penalties for lack of, supervision"* presented at a symposium on the public supervision of dams which was held in New York in 1930. In the same year reservoir safety legislation was introduced into Great Britain in the form of the *Reservoirs (Safety Provisions) Act.*

Since 1930, there have been no dam failures in Great Britain which have caused loss of life, although there have been a number of failures involving breaching of embankments and there have been many serious incidents. Wright (1994) reported that since the *Reservoirs Act 1975* came into force in 1985-86 no catastrophic failures are known. With such a good safety record over a period of 70 years, it could be questioned whether any improvement in reservoir supervision and periodic inspection is needed. However, the safety record does not indicate that catastrophic failures are no longer possible, but rather that continuing vigilance is required.

The question of whether any change in reservoir safety procedures are required has to be examined against the increasing expectation that a risk free environment can and should be provided. Although complete freedom from risk is not possible, the climate of public opinion cannot be ignored. Any failure or well publicised serious incident is likely to have major repercussions for owners of large dams. Generally, risk is reduced by expenditure which dam owners incur and, consequently, they may view tolerable risk differently from the downstream population. Where there is a possibility of casualties, the public may want the elimination of all risk

Dams 2000, Thomas Telford, London, 2000.

irrespective of cost. Dam owners are a major beneficiary of reservoir safety, but the public has the principal interest and, while neither party would want a dam to fail, their interests are not identical.

The concentration of ownership of large dams in a few major companies has important implications. The commercial context within which reservoir safety is delivered is changing, particularly in the relationship between the panel engineer and the owner of the dam. While some owners of large dams select inspecting engineers solely on merit and negotiate rates on a time basis (Martin, 1998) others have introduced fixed fees and fee competition for some reservoir inspections (Milne, 1996). Hay (1996) has claimed that this latter practice threatens the independence of inspecting engineers and their ability to exercise care and skill impartially. Hinks (1996) suggested that major owners should discontinue fee competition based on lump sums for category A dams, as defined in the floods guide (Institution of Civil Engineers, 1996). Inspecting engineers may be challenged where the owner considers that there is excessive conservatism; the 1975 Act provides a procedure for this, but there are other less transparent ways in which this can be done.

It is desirable to achieve greater uniformity of approach so that dams and reservoirs presenting a similar hazard to public safety meet similar safety standards and that successive inspections of the same reservoir are carried out to the same standards. The concept that prevention is preferable to detection and correction implies that high hazard dams should be upgraded to current standards even where no defects have yet been manifested; recommendations on this matter should come from a current DETR research contract concerned with embankment instability and internal erosion.

The study described in this paper was carried out as part of the DETR Reservoir Safety Research Programme. The principal objective was to determine the scope for the explicit application of total quality management (TQM) concepts and systems to the conduct of reservoir supervision and periodic inspections by reservoir panel engineers. It has been concluded that a simplistic attempt to make reservoir safety fit existing quality management methodology will not be helpful, but that there could be significant benefits in more formally applying relevant quality management principles to reservoir safety procedures.

TOTAL QUALITY MANAGEMENT

Dictionary definitions of quality refer to concepts such as *"that which makes a thing what it is"* and *"kind or degree of goodness or worth"*. In common understanding, quality means the inherent properties of something or some measure of its worth and has tended to imply good or high quality. In an industrial context, quality has acquired a related but specialised meaning of fitness for purpose or use. This has been expressed more fully as *"the totality of features and characteristics of a product or service that bear on its ability to satisfy stated or implied needs"* (BS 4778; Part 1, 1987). Quality is seen as meeting customer requirements, as conformance to requirements rather than "goodness". The management of quality, including quality control and quality assurance systems, has developed rapidly since the 1970s. Ideas based on the Japanese quality management culture and on the work of a number of specialists in the western world have been widely publicised. A succession can be traced through developments in the 1960s and 1970s, to total quality management, a business philosophy concerned with customer satisfaction, in the 1980s and 1990s.

There was a perception in the 1980s that the poor quality of products was having a detrimental effect on the performance of British companies and against this background total quality management was adopted by many organisations. Hellard (1993) stated that it has been forecast that by the end of the 20th century there will be only two kinds of commercial operations, one which has embraced and practises total quality management and the other which will have gone out of business! Although large and successful business organisations which have successfully adopted TQM are often cited, Dale (1994) suggested that senior management is prone to exaggerate commitment and that in some cases *"less progress has been made than indicated by the tone of the presentation and associated publicity"* .

Total quality management has been defined in various ways, for example:

- *"A management philosophy embracing all activities through which the needs and expectations of the customer and the community, and the objectives of the organisation are satisfied in the most efficient and cost effective way by maximising the potential of all employees in a continuing drive for improvement."* (BS 4778: Part 2, 1991)
- *"Total quality management is meeting and exceeding the customer's expectations by continuously improving all processes, goods and services through creative involvement of all staff."* (Juran et al, 1979)

These definitions are broad and incorporate diverse activities including management auditing, product liability prevention, reliability engineering, human resources management and failure modes and effects analysis. The following concepts give an indication of some of the common elements of TQM:

- Managing a business by providing a fundamental strategy which permeates the culture of the organisation.
- Concern with fitness for use, with insistence on compliance and conformance rather than on elegance, with meeting agreed customer requirements effectively and efficiently.

- Prevention is better than detection, it is important to get it right first time and the performance standard is zero defects, although mistakes should be viewed as improvement opportunities.
- Quality management auditing extends the process of auditing beyond the conventional monitoring and reporting of the financial control, status and position of a company into all aspects of a corporate operation.
- Continuous improvement requires culture change and represents a permanent on-going activity.
- Universal participation and commitment within the organisation is required from top management downwards, teamwork is essential and there is a need for training and education.
- External suppliers and customers should be integrated into the process.

HEALTH CARE

The principal application of total quality management has been in businesses which supply products to customers. Widening the field of application to providers of services where the service and the customer are clearly defined has presented no great conceptual difficulties. However, not all services are readily amenable to TQM and when the service being provided involves health and safety there are major difficulties. It may not be easy to identify the "customer".

The perceived need to promote economy, efficiency and effectiveness within the health service has led to attempts to apply total quality management. There are many different aspects to health care including medical, clinical, clerical, hotel, community and management services (Hawkes, 1992). Particular services are provided for individual patients, but a more general service of health and welfare is provided for the community.

There are major differences between the provision of health care and commercial undertakings and industrial processes. Patients are not the same as customers in a market place and the measures of customer satisfaction are substantially different.

- Risk to life will often be a prominent feature; the consequences of failure may be fatal.
- The patient is very dependent on expert and highly specialised advice, the validity of which he may have few means of checking.
- Ill health and worry may render a dispassionate decision by the patient improbable.
- Medical facilities do not closely resemble a production line; highly specialised equipment may be required which is very expensive and difficult to justify on a cost-benefit basis.
- The type of staff and their inter-relationships are different from those in an industrial context; while teamwork is vital, the individual expertise of specialists and consultants is far more important than in many industrial situations.

In competitive markets companies need to improve quality in order to stay in business, but this is not the case in a health service provided by a monopolistic and bureaucratic system in which the patient does not directly pay for the services received; the providers of the health care are accountable to those providing the funding. It has been argued that TQM can be adapted to most health care settings subject to a new definition of quality based on the extent to which the actual care provided as part of an operational service measures up to the best care possible in terms of both efficiency and access to the service (Davies, 1992).

RISK MANAGEMENT

The provision of a quality environment with an emphasis on compliance with requirements should mean that the risk of non-compliance is maintained at an agreed and acceptably low level. There is, therefore, an important link between quality management and risk management.

Quantitative risk assessment (QRA) is a technique for assessing the frequency of an unwanted event and its measurable consequences in terms such as number of fatalities or cost of damage. QRA has found application in the nuclear, chemical, offshore, defence, marine and automotive industries. However, there are important differences between chemical and nuclear plants, for example, and reservoirs impounded by embankment dams (Charles, 1997; Charles et al, 1998).

An alternative approach to risk management can be based on failure modes and effects analysis (FMEA) or failure modes, effects and criticality analysis (FMECA). These methods of reliability analysis identify failures with consequences which affect the functioning of a system, thus enabling priorities to be set (British Standards Institution, 1991). Draft guidance on the environmental risk aspects of the Control of Major Accident Hazards (COMAH) Regulations makes reference to FMEA (Environment Agency, 1999). Sandilands et al (1998) have described the use of FMECA for the dam based hydro schemes of Scottish and Southern Energy. The use of a structured qualitative risk assessment built upon FMEA has been advocated for tailings dams and waste impoundments (McLeod and Plewes, 1999). The guide to risk management for UK reservoirs (CIRIA, 2000) should be relevant to the framework within which the panel engineer reports risk, including the application of FMECA, and to the way in which safety standards should be related to acceptable risk.

RESERVOIR SAFETY

The application of total quality management to health care has illustrated some of the difficulties that will be encountered with reservoir safety. The ideology of quality management typically assumes that there is a product or service, a provider and a customer and the basic concern is with customer satisfaction. In applying such ideas to reservoir safety, the service can be considered to be reservoir safety, but the identity of the provider and the customer are not so obvious. Total quality management is essentially concerned with organisations and the reservoir safety regulatory system involves a number of different individuals and organisations with distinct roles. Reservoir safety in the form of a tolerably low risk of failure is provided by a number of parties:

- qualified civil engineer (to advise on safety),
- undertakers (with duties as owner or operator),
- enforcement authority (to ensure compliance with legislation).

In addition to the above, the government has a role as legislator.

Certain facets of total quality management are relevant to reservoir safety. For example, quality management recognises that the prevention of defects is better than their detection and a philosophy of continuous improvement is advocated. Other concepts, such as "just-in-time", are less appropriate in situations involving public safety and the emphasis on teamwork in TQM does not easily relate to the undivided responsibility of the inspecting engineer. Furthermore, reservoir safety is a low probability/high consequence scenario which does not naturally fit into the TQM model.

The transfer of total quality management to the reservoir safety environment is not a straightforward exercise and, as in health care, there is a need to redefine quality. It is proposed that in the context of reservoir safety, quality should be taken to mean *delivering the required high level of safety without excessive cost.* This definition is closely related to the ALARP (as low as reasonably practicable) principle often quoted in connection with risk tolerability and the BATNEEC (best available technology not entailing excessive cost) principle used in the remediation of contaminated land.

In applying the principles of quality management to reservoir safety practice, the cycle of improvement practised in TQM is helpful. This normally involves the following:

- Establish customer needs (in this case, safe reservoirs not incurring excessive cost).
- Define the quality required to meet those needs (standards of safety, including safety of individual components that may cause a loss of safety, and the severity of natural hazards that may lead to a lack of safety in reservoirs).
- Set procedures that make it possible to consistently attain the quality required.

Consensus is desirable concerning the tolerable level of risk. If one panel engineer considered that the acceptable risk of failure of a particular dam is 1×10^{-5} per annum and a second panel engineer decided to accept a higher level of risk, say 1×10^{-4} per annum, the difference in tolerable risk could have major expenditure consequences. If the undertakers use the second panel engineer, the most likely outcome for the panel engineer and the undertakers is that all is well and costs have been significantly reduced. However, from a national and public safety perspective this may not be acceptable. Table 1 of the floods guide (Institution of Civil Engineers, 1996) provides a widely accepted relationship between acceptable risk and downstream hazard.

Safety standards are needed which are compatible with tolerable risk. This is relatively straightforward for the threat posed by floods and earthquakes but is not so simple for the hazards of embankment instability and internal erosion. There may be differences in the technical opinions of qualified civil engineers which can only be addressed by improved understanding of the problems and better training of qualified civil engineers.

UNDERTAKERS

It might seem appropriate for quality management to be focused on the undertakers since they have the ultimate responsibility for reservoir safety and employ the inspecting and supervising engineers. The continuing safety of old embankment dams is heavily dependent on an observational approach (Johnston et al, 1999) and the programme of surveillance for high hazard dams may include the undertakers' personnel visiting the site several times a week or even daily. Quality management procedures clearly have a major role in this area.

One difficulty with an approach centred on reservoir undertakers lies in the diversity of these undertakers.

- Some 50% of reservoirs under the Act have undertakers with more than 20 reservoirs. Many major water supply companies with large numbers of reservoirs have professional engineers on their own staff who oversee their reservoirs.
- Some 25% of reservoirs under the Act have undertakers with only one reservoir.

The former category of undertakers is likely to have a highly developed quality management system, but a quality management system suited to the former category is unlikely to be relevant to the latter category.

It is the responsibility of the undertakers to arrange that both inspections and measures recommended in the interests of safety are carried out, and of the enforcement authority to ensure that they are done.

ENFORCEMENT AUTHORITY

Under the 1975 Act, nearly all technical duties are carried out by the qualified civil engineer and enforcement duties are mainly of an administrative and legal nature with only a small technical content. A basis for quality management is provided by three charts in *A Guide to the Reservoirs Act 1975* which illustrate the following procedures of an enforcement authority:

- the interaction of the enforcement authority with panel engineers and undertakers in routine reservoir safety activities (chart 4, page 10),
- the enforcement of appointments of panel engineers (chart 5, page 11),
- the enforcement of measures in the interests of safety (chart 6, page 12).

If, in due course, enforcement duties in England and Wales are transferred from local authorities to the Environment Agency, this could lead to an enhanced enforcement role with more uniform procedures.

QUALIFIED CIVIL ENGINEER

Although undertakers are responsible for the safety of the reservoir, the technical cornerstone of the British approach to reservoir safety has been the independence, integrity and experience of the qualified civil engineer (panel engineer) who takes an unambiguously defined individual responsibility for activities performed under the Act. The British reservoir safety regulatory system is based on inspections by independent qualified civil engineers rather than by a government inspectorate. However, the concept of the independent professional consulting engineer is currently under pressure and the independence of the panel engineer is not a matter which should be taken for granted. Since the supervising engineer may be an employee of the undertakers, it is particularly important that the inspecting engineer is, and is seen to be, independent.

Demonstrable independence is important for public accountability. In the changing environment in which reservoir safety is implemented, it could be questioned whether the definition of "independent" given in section 10(9) of the Act, which ensures that the inspecting engineer is not an employee of the undertaker and was not the construction engineer or connected to the construction engineer as partner, employer, employee or fellow employee, is adequate. Practical independence requires professional integrity and inspecting engineers must carry out their duties without any improper pressure being applied with regard to the commercial consequences of requiring safety measures to be carried out. The appointment and re-appointment procedures to which panel engineers are subject need to ensure that professional standards are upheld.

With a system based on the experience, skills and integrity of qualified civil engineers acting in an individual capacity, appropriate qualifications and training for these responsibilities are of fundamental importance. The system for regulating reservoir safety has developed over a seventy year period and is largely unstructured with regard to training. The 1975 Act simply refers to "civil engineers" and does not specify any particular qualification such as membership of the Institution of Civil Engineers (ICE).

It is reasonable to require that applicants for panel membership have appropriate knowledge and experience together with technical qualifications which would provide eligibility for chartered membership of a relevant engineering institution. The vast majority of panel engineers are chartered engineers who have successfully completed first degrees in civil engineering. They should, therefore, be well qualified in the basic scientific and technical disciplines on which reservoir safety depends. WTI provides courses suitable for panel engineers, but on the job training predominates.

The series of engineering guides covers most of the subjects relevant to reservoir safety with which the panel engineer should be familiar. Of particular importance are the floods guide (Institution of Civil Engineers, 1996), the embankment dam guide (Johnston et al, 1999), the concrete and masonry dams guide (Kennard et al, 1996), and the seismic guide (Charles et al, 1991) and application note (Institution of Civil Engineers, 1998).

Continuing professional development (CPD) has been defined as the systematic maintenance, improvement and broadening of knowledge and skill, and the development of personal qualities necessary for the execution of professional and technical duties throughout one's working life. Guidelines for CPD have been prepared by the ICE and it is recommended that every member maintains a continuing professional development record. The record can include, amongst other activities, attendance at technical meetings organised by appropriate professional bodies and at training courses.

The appointment and re-appointment processes for panel engineers should be used to raise standards. Requirements for training should be tightened and the need for CPD made more explicit. Applications for re-appointment should include descriptions of how the applicant has kept up with relevant technical and professional developments. A CPD form could be required to be submitted with all applications for re-appointment to reservoir panels, the form to cover the five year period of the appointment which has just expired.

Although the CPD terminology may be relatively recent, the underlying concern is not. More than 35 years ago, Mr Julius Kennard commented as follows: " *... a human being, individually, in his right senses, when concerned with his health and condition and in need of a check-up, would not dream of consulting a retired medical practioner who was no longer up-to-date in his particular field or perhaps did not have access to the special equipment required to conduct the necessary examination and tests. Yet those very persons, collectively as a water authority, would often entrust the examination of a reservoir to a retired engineer who might have ceased to practise for anything up to twenty years, ...* ".

In the application of TQM to panel engineer activities two background factors need to be kept in mind.

- The British regulatory system can be regarded as a system of self-regulation in that the ICE Reservoirs Committee is composed predominantly of inspecting engineers on the All Reservoirs panel and, the appointment of qualified civil engineers is, therefore, largely in the hands of other qualified civil engineers.
- The inspecting engineer and the supervising engineer of a reservoir are engaged by the undertakers whose interests are not identical with those of the general public.

It is important that there is transparency, consistency, traceability and accountability in the activities of panel engineers. Quality management procedures can assist in fulfilling these requirements.

INSPECTING ENGINEER ACTIVITIES

The *inspecting engineer* is concerned with the periodic inspection of a reservoir. Indeed, the 10 year inspection by a qualified civil engineer is the centre-piece of the British reservoir safety system. It is generally considered that the letter of Mr Edward Sandeman to *The Times* on 4 December 1925, following the recent failure of dams in Scotland and North Wales, stirred the government into action with the result that the Reservoirs (Safety Provisions) Act was passed in 1930. Mr Sandeman drew attention to the report made by the Select Committee on the Waterworks Bill, 1865 (Sessional Paper 401) after it had considered the evidence concerning the failure of Dale Dyke on 11 March 1864 and the various recommendations of the Committee including that *"As large reservoirs have been allowed to decay and to become dangerous, periodical inspection should be made of them."*

The term "inspection" focuses attention on a site visit and suggests a walk-over of the site looking for evidence of defects and deterioration, but the inspection may cover many other matters such as flood calculations and slope stability evaluation. Thus the inspection should be viewed as a major safety evaluation comprising a desk study, site inspection, report preparation and recommendations for measures to be taken in the interests of safety. To assist panel engineers to take a comprehensive and consistent approach, it would be beneficial to integrate into the system some more general approach such as failure modes and effects criticality analysis (FMECA).

The determination of the required quality or level of safety to meet standards established through experience and practice is the responsibility of the inspecting engineer who is the key technical authority for reservoir safety. An attempt to impose a quality management system which was too rigid and prescriptive might not be helpful. It would not be appropriate to require the inspecting engineer to follow a procedure set by others and it is unlikely that a procedure could be devised that would be sufficiently comprehensive to ensure that inspecting engineers fully utilised their experience and knowledge of large numbers of dams, including design, construction, maintenance and performance.

The undivided responsibility of the inspecting engineer encouraged the development of an individualistic approach which in the last few years has been modified by a number of developments. In particular, the publication of engineering guides and the checklists included in these guides have tended to produce a more uniform approach. The wide use of such guidance should encourage more consistent standards for levels of safety across the population of UK dams and ensure that all aspects of reservoirs are covered.

The legislative framework should aim to ensure that decisions concerning safety are made objectively, consistently and with proper regard for uncertainty. More can be done to provide a framework within which inspections are carried out and encourage best practice. Quality control procedures and a more standardised format for reporting with check lists and tables to aid consistency should be developed.

The activities of the inspecting engineer have been illuminated by the recently prepared *A Guide to the Reservoirs Act 1975* (Institution of Civil Engineers, 2000). An outline procedure for periodic inspection and supervision of measures in the interests of safety is provided by chart 2 (page 8) and a check list for inspections is included as appendix E4 (page 195). The procedure and the check list provide a basis for quality management of inspecting engineer activities. Guidance on the reports required under Section 10(3) of the 1975 Act has been prepared and included as an annex in the report to the DETR on the application of TQM to reservoir safety (Charles, 1999); this expands on the outline in appendix E4 of *A Guide to the Reservoirs Act 1975* by giving guidance on the detail that may be provided under the various headings. Copies of this draft guidance will be available at the Conference and comments will be welcome. It is intended that, following appropriate revision, the guidance will be published in *Dams & Reservoirs*.

Some inspecting engineers now include a caveat in their inspection reports drawing attention to the status of the inspection and the report (Hay, 1996). The legitimate extent of disclaimers and qualifications to inspection reports needs to be examined. To point out that the site visit was on a specified date and that the inspecting engineer cannot be responsible for features not observable on that date and which have not been reported to him, seems not unreasonable. For the inspecting engineer to state that the overall stability of the dam has not been considered does not seem reasonable unless the reasons why the panel engineer considered that this was unnecessary are clearly stated.

SUPERVISING ENGINEER ACTIVITIES

The work of the supervising engineer ensures that there is professional supervision of the reservoir in the interval between inspections, when material changes that could affect safety may become apparent. The role of supervising engineer was an innovation of the 1975 Act, but its utility has been amply demonstrated. Although the technical ability required for a supervising engineer is inferior to that required of an inspecting engineer, the former has a continuing and continuous association with, and responsibility for, a particular reservoir which the latter, who is appointed only to carry out an inspection, does not. While some supervising engineers are mainly employed as such and supervise twenty or more reservoirs, for others reservoir supervision is only a very minor part of their duties and they supervise only one or two reservoirs. In the latter situation it is important to ensure that there is an adequate continuing involvement with reservoir safety.

An inspecting engineer may suggest in the inspection report the frequency and times of the year of the supervising engineer's visits. However, once appointed, frequency of visits by the supervising engineer is a matter for that engineer to decide, bearing in mind adequacy for safety purposes and any changes in circumstances and statutory duties. Factors to be considered include size, type and age of reservoir and dam, previous history, proximity of dwellings and the possibility of new developments downstream, degree of surveillance and standard of maintenance achieved by undertakers, and, finally, the minimising of expense for the undertakers as far as is compatible with these other factors.

It would be beneficial to further develop quality control procedures. Chart 1 in *A Guide to the Reservoirs Act 1975* (page 7) outlines the procedure for monitoring and supervision and a check list for the annual statement of the supervising engineer is given in appendix E5 (page 197). These provide a suitable basis for the quality management of supervising engineer activities. One possible development would be to require that the annual statement to the undertakers is copied to the enforcement authority.

AUDIT OF PANEL ENGINEER ACTIVITIES

In the report of a committee of engineers and geologists, appointed by the District Attorney of Los Angeles County, on the failure of the St Francis dam on 12 March 1928 it was concluded that *"A sound policy of public safety and business and engineering judgement demands that the construction and operation of a great dam should never be left to the sole judgement of one man, no matter how eminent, without check by* *independent expert authority, for no one is free from error, and checking by independent experts will eliminate the effect of human error and insure safety."* (This report was not published but is quoted in the Proceedings of the American Society of Civil Engineers, October 1929, p 2162)

Auditing is a prominent feature of TQM and the introduction of more explicit quality management procedures should be accompanied by more formal and transparent audit procedures. Although statutory reports are not required to have a third party check, where an inspecting engineer recommends measures in the interests of safety, the Act permits aggrieved undertakers to refer their complaint to a referee. This provision in some cases might prevent expense on unnecessary remedial works, but is unlikely to enhance public safety.

Although not a legal requirement, some internal quality assurance procedure is likely to be in place where a panel engineer is a member of a large firm of consulting engineers; this should ensure that reservoir inspection reports are reviewed independently by experienced staff for content, consistency and clarity. This type of internal procedure is more difficult to arrange for the panel engineer operating as an independent consultant. However, most inspecting engineers submit drafts of their reports to the undertakers and the supervising engineer for comment. While such comments do not have to be followed by the inspecting engineer, the process does provide a means of checking factual correctness, content, consistency and clarity.

High standards are vital and the utility and propriety of external audits need to be addressed. Where independent review boards are appointed, their brief should be clearly defined; there is much in favour of limiting their brief to ensuring safety and excluding issues of cost-effectiveness. If the enforcement role in England and Wales is transferred to the Environment Agency, some type of auditing could form part of the enforcement duties.

CONCLUSIONS AND RECOMMENDATIONS

The reservoir safety regulatory system could benefit from some modifications in order to maintain and enhance the current high safety standards. The ideas of quality management, suitably adapted should prove useful. While there are features of modern quality management that can be adapted to reservoir safety and made more explicit in reservoir safety procedures, public safety rather than customer satisfaction is the key requirement. Consequently, there are aspects of total quality management which do not readily fit into the reservoir safety regulatory system. In the context of reservoir safety, it is proposed that quality should be defined as the provision of the required high level of safety without excessive cost. This involves determining the tolerable level of risk and developing safety standards which are compatible with that risk.

The reservoir safety regulatory system involves undertakers, panel engineers and enforcement authorities. The inspecting engineer should remain the key technical authority for reservoir safety and an attempt to impose a quality management system which was too rigid and prescriptive would be unhelpful. Nevertheless, more can be done to provide a framework which will encourage best practice in inspecting engineer activities and *A Guide to the Reservoirs Act 1975* provides procedural charts and check lists. In the changing environment in which reservoir safety is implemented, the independence of the inspecting engineer needs to be preserved. The supervising engineer has a continuing responsibility for a reservoir and some enhancement of the role of the supervising engineer would be beneficial. Improved guidance should be given on minimum levels of experience and qualifications for panel appointment. Requirements for training should be tightened with re-appointment procedures more closely linked to continuing professional development.

The appointment of the Environment Agency as the enforcement authority in England and Wales should strengthen enforcement and promote more uniform standards in inspections and reports. Some form of third party auditing, either by the Environment Agency or by a more widespread use of review boards, could be introduced.

The work of the ICE Reservoirs Committee is focused primarily on the dam-reservoir system, but in a failure event the downstream valley system will figure largely. The two elements of reservoir safety should be closely linked and quantitative risk assessment brings together the dam-reservoir system and the downstream valley system as two component parts of the analysis.

There are a number of routes to change:

- Revision of primary legislation is the most radical route to change and, currently, there is some prospect of modifying the Reservoirs Act.
- Statutory instruments include certificates that panel engineers are required to complete and to some degree, therefore, they control the reporting of panel engineer activities. This could be developed further.
- Revision and expansion of the engineering guides provides a means of improving quality management. The technical guides are complemented with *A Guide to the Reservoirs Act 1975* which has procedural charts and check lists promoting quality management.
- The meetings, conferences and publications of the British Dam Society and ICE can be used to promote best practice.

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Risk management for UK reservoirs

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SYNOPSIS. A CIRIA research project has reviewed risk management associated with UK reservoirs and prepared a guidance document summarising best practice at the present time. This paper summarises the study and outlines the risk assessment methodology developed. The application of the methodology, and its relationship with reservoir safety legislation and various guidance documents is discussed.

INTRODUCTION

The application of risk assessment methodology for use on UK reservoirs was first suggested in 1982 by the House of Lords Select Committee on Science & Technology. A pilot risk assessment was carried out for an earth dam in north west England which was described by Parr and Cullen (1988). A number of difficulties were encountered in this exercise, most notably the fact that there were insufficient statistical data on dam failures and other incidents to enable probabilities to be assigned.

Because of this shortage of statistical data, the Building Research Establishment (BRE) developed a computerised database containing information on all 2500 or so dams that are subject to the provisions of the Reservoirs Act 1975. This database includes information on matters such as dam construction, problems, investigations and remedial works (Tedd *et al*, 1992).

A CIRIA research project has developed an initial approach to risk assessment for UK reservoirs. The study commenced in 1997 and CIRIA appointed a consortium led by RKL-Arup and including HR Wallingford, EQE International Ltd, the University of Newcastle and BRE to undertake the detailed research for the study. The objective of the project was to produce a guidance document for reservoir owners, panel engineers, regulators, insurance companies and other stakeholders concerned with reservoir safety. The project proceeded in four stages:

1. Assessment of industry requirements and prioritising of risk assessment and management/maintenance tasks.
2. Analysis of the hazard and risk posed by UK reservoirs.
3. Examination of the role of hazard and risk assessment in the management of UK reservoirs.

4. Preparation of a guide on hazard and risk assessment and their use in the management of UK reservoirs.

Stage 1 involved a literature review, consultation with stakeholders and others through a questionnaire and workshop, and preparation of a Position Report. The principal conclusions and recommendations from Stage 1 were as follows:-

- The application of risk assessment could help to improve reservoir safety in the UK and it should therefore be welcomed.
- A relatively simple and easily understood risk assessment methodology would be preferred which is cheap to implement. Full probabilistic risk assessments using fault trees etc were not desired, although a simplified approach may be appropriate in some cases.
- Hazard indexing would be useful in identifying the potential consequence of failure and in the classification of reservoirs.
- There was concern about the availability of data on dam 'incidents' including failures and 'near misses'.
- The UK should take note of approaches to risk analysis in other countries, but should develop its own methods applicable to its own unique stock of dams.
- A consistent approach is needed to the preparation of emergency plans and inundation maps.

INTENDED APPLICATION

A number of guides have been prepared in recent years to assist engineers carrying out inspections and safety works associated with the Reservoirs Act 1975 (eg. for floods, seismic risk etc). The risk assessment techniques developed are designed to highlight areas where current engineering standards may not be sufficient to ensure safety and/or issues that current engineering approaches may not have addressed adequately. The CIRIA report (CIRIA, 2000) provides a method that may be applied to any reservoir falling within the provisions of the Reservoirs Act 1975, including non-impounding and service reservoirs.

It is anticipated that risk assessments are likely to be used as a tool for owners of several reservoirs to rank the reservoirs in order of risk and hazard, and possibly assist them in prioritising any works needed. Risk assessments are only likely to be carried out for safety reasons on individual reservoirs that pose a significant hazard to life and property downstream. It is not anticipated that risk assessments will be carried out on many small reservoirs where the existing inspection and reporting procedures under the Reservoirs Act 1975 should be sufficient to maintain an acceptable level of reservoir safety. It is not considered likely that an Inspecting Engineer will automatically require a risk assessment to be carried out as part of, or following, his inspection.

Specific benefits of the application of risk assessment procedures to reservoir safety include:

- prioritising the implementation of safety recommendations and remedial works
- prioritising maintenance
- planning a surveillance, monitoring and instrumentation strategy
- identifying possible failure modes requiring detailed investigation and analysis
- checking that all hazards at a reservoir are systematically identified and considered
- preparation of emergency plans for dam operation and interaction with emergency services
- identifying the financial risk associated with the failure of a dam
- providing comparison with hazards in other industries
- avoiding complacency in respect of dam safety.

It should be realised that there are potential drawbacks resulting from the implementation of a risk assessment/risk management system and these include:

- cost of undertaking the risk assessment, particularly if the system is too complicated
- cost of implementing measures resulting from the risk assessment (eg. remedial works, additional surveillance/monitoring etc)
- incomplete knowledge of matters affecting the performance of the dam.

It is also possible that risk assessment could be used to modify legislation to move from a situation based on retained volume to one based on risk and hazard.

RISK CRITERIA

Society does not have a logical view on acceptable levels of risk: it is reasonable to assume that accidents killing the same number of people are equally as acceptable or unacceptable. There is, however, a tendency to be more concerned about accidents involving a high number of fatalities such as air crashes, when in fact statistically air travel is far safer than road travel. The perception of risk is also influenced by factors such as whether the risk is undertaken voluntarily (eg. rock climbing) or is imposed by others (eg. construction of a nuclear power station). It is therefore difficult to set rigid risk criteria for all industries or situations. The approach generally adopted within the UK risk industry comprises a three tier system:

1. An upper-bound on individual (and possible, societal) risk levels, beyond which risks are deemed unacceptable.

2. A lower-bound on individual (and possible, societal) risk levels, below which risks are deemed not to warrant regulatory concern.
3. An intermediate region between the above bounds, where further risk reduction is required to achieve a level deemed 'As Low As Reasonably Practicable' (ALARP).

Target values of maximum risk are published by the HSE (1989, 1996) for various hazardous industries and these are generally in the range of 10^{-5} to 10^{-7} (one in 100,000 to one in 10,000,000 per year). The most appropriate standard to apply to reservoirs would appear to be that for land-use planning (HSE, 1989) where the target level of individual risk is 10^{-6} (one in 1,000,000 per year).

RISK ASSESSMENT

The two extremes of risk assessment are an engineering judgement approach (fully qualitative) and full-probabilistic assessment (fully quantitative). Between these two extremes lie a range of semi-quantitative approaches that build on the strengths of both systems when data and assessment budgets are limited but consistency and reliability are equally important. One of these, the Failure Modes, Effect and Criticality Analysis (FMECA) approach was adopted because it provides the flexibility to deal with varying levels of knowledge regarding the performance and reliability of different dam components. A similar method has been used for UK dams by Scottish Hydro-Electric (Sandilands et al, 1998).

The FMECA approach developed involves the use of a Location, Cause, Indicator (LCI) Diagram which shows various elements within a dam and how they may contribute to possible failure of a dam. An extract from a typical LCI Diagram is shown in Figure 1. The full LCI diagrams include items associated with the spillway and inlet/outlet works together with blank boxes for the user to add other elements relevant to the dam being analysed. Each element on the LCI Diagram is allocated scores between 1 and 5 for the following three factors:

Consequence: how directly is this element related to failure of the dam? (1 low, 5 high).

Likelihood: what is the likelihood of failure of this particular element? (1 low, 5 high).

Confidence: what is the confidence in the reliability of the predictions for consequence and likelihood factors? (5 low, 1 high).

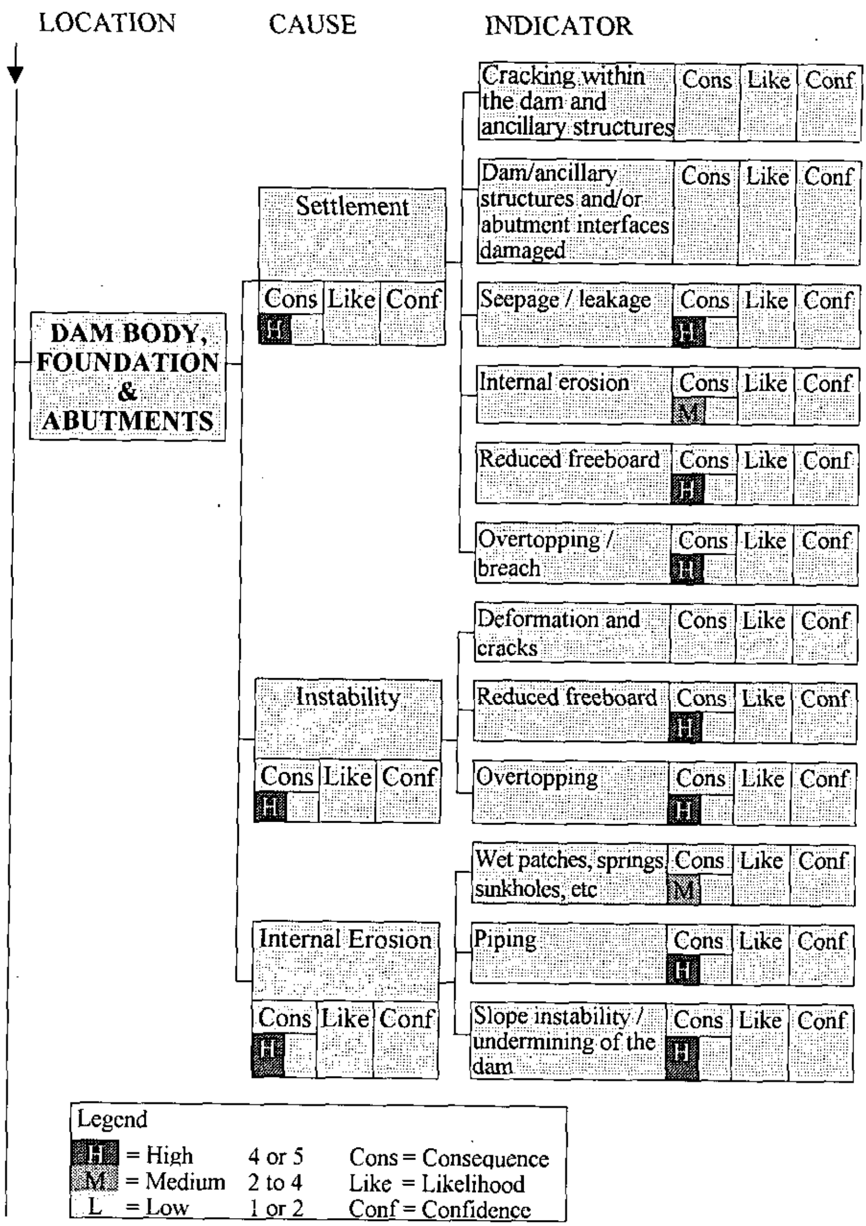

Fig 1: Extract from a typical LCI diagram (embankment dams less than 15m high constructed between 1840 and 1960)

The 'Criticality' score for each element is calculated from:

Criticality = Consequence x Likelihood x Confidence.

Each element on the LCI Diagram will therefore have a Criticality score between 1 and 125.

For elements with the highest Criticality scores, mitigation measures to reduce the level of risk may include the undertaking of works to reduce the consequences or likelihood of failure of an element, or the investigation of an element to improve the confidence score. The most appropriate response will depend upon the nature of the element and the relative score values. Where an element has a high confidence score, therefore, (ie. poor knowledge of/confidence in the consequence and/or likelihood scores), consideration should be given to undertaking investigative works to reduce the uncertainty of the predictions. Similarly, where the consequence and likelihood scores are high, consideration should be given to undertaking works to reduce the consequences, likelihood, or both.

By prioritising elements with respect to the product of the consequence and likelihood scores, and the confidence score, it is therefore possible to prioritise both investigative works and capital works to reduce the risk posed by the dam.

DATABASE ANALYSIS

Information on the historic performance of UK dams was analysed to provide guidance on allocation of 'Consequence' scores on the LCI Diagrams. Data was analysed from three sources:

1. The BRE database of UK dams. This was prepared under a DETR project and contains information on some 2700 dams in the UK that come within the ambit of the Reservoirs Act 1975: details are given in Tedd et al (1992). Basic details such as name, location, capacity and type of construction are recorded together with information on problems, investigations and remedial works at a proportion of the dams.
2. A database held by A I B Moffat at the University of Newcastle. This database contains over 800 entries associated with incidents at dams in the UK which are, or would have been, subject to reservoir safety legislation. The classification includes information relating to whether there was failure or an incident. Data on the general dimensions of the dams and their performance history including significant remedial/upgrading works, where known, are also included.
3. A database held by Dr A K Hughes at RKL-Arup. This database contains more than 700 occurrences of overtopping to dams worldwide. A subset of UK dams details those which have overtopped causing damage and in some cases failure. The database gives data on the general dimensions of

the dam, date of construction, etc, as well as notes detailing information relating to the overtopping event.

In order that consistent conclusions were reached using the three databases, a common approach was developed to describe incidents that were reported: four types of incident were defined:

- 'Failure' involving a major uncontrolled release of a significant proportion of retained water.
- 'Category 1 incident' which involved immediate emergency action or drawdown.
- 'Category 2 incident' which caused serious concern and/or involved significant investigation/remedial action.
- 'Category 3 incident' which involved the observation of indicators causing serious concern.

Information was sub-divided into various categories based on year of construction, as given in Table 1.

Table 1 : Sub-divisions of dams/reservoirs by age

Dam/reservoir type	**Age categories**		
Embankment dams	Pre-1840	1840-1960	Post 1960
Concrete/masonry dams	Pre-1960	Post 1960	
Service reservoirs	No age category		

The dates of the embankment dam categories were based on significant changes in construction techniques, for example the 1840-1960 period covering the central puddle clay core era. The embankment and concrete/masonry dams were also split into three height categories:

- less than 15m
- 15m-30m
- greater than 30m

Although this categorisation is somewhat arbitrary, it was considered that different height categories were appropriate because certain types of incident varied with height: a particular example is settlement and loss of freeboard, which becomes more significant as height increases.

The numbers of each type of incident (eg. settlement) for each dam age/height category were established and this enabled guidance to be given on the relationship between the type of incident and dam failure. This was done using a consistent rules-based decision matrix to allocate 'low', 'medium' and 'high' consequence estimates.

The databases were also used to identify the critical issues associated with each dam type. A typical list is given in Table 2, for embankment dams constructed between 1840 and 1960.

Table 2 : Incidents at Embankment Dams constructed between 1840-1960

Incident type	% of all recorded incidents
Settlement	18%
Slope instability	10%
Seepage/Leakage/Internal erosion	30%
External erosion	6%
Overtopping/breaching	4%
Obstruction of flow	1%
Valve/gate failure	3%
Inadequate overflow capacity	24%
Ancillary damage	3%
Other causes	1%

IMPACT ASSESSMENT

An impact assessment procedure was developed during the CIRIA study to determine potential flood water levels and their impact on the community. This includes a method that allows the rapid estimation of potential flood levels. It should be stressed that the results gained from this flood level estimation technique are not considered to be as robust as dam-break simulation through computer modelling. In any application of the impact assessment procedure then the best available data should be used (including the outputs from dam-break analyses if available).

Once an outline of potential flood levels has been mapped either from the 'quick' method or from dam-break modelling, the impact on the following seven key areas should be considered:

- Residential properties
- Non-residential properties
- Transportation infrastructure
- Recreational sites
- Industrial sites
- Utilities
- Agriculture/habitats

Each of the above impacts should be considered in turn and be allocated a potential level of impact from 0 to 4 (0 low, 4 high). Loss of life should be considered separately and an estimate made of People at Risk (PAR) associated with the first four impacts listed above. Studies in the United States have shown that distance from the dam, and hence evacuation/warning time, is a key issue affecting loss of life in dam-break events, with the majority of fatalities occurring within 5km of the dam. Separate factors are therefore applied to estimate the potential loss of life for near (less than 5km) and far (5-30km) valleys.

The impact score and PAR values are combined to provide a single measure of impact for the area downstream of a dam. This enables the impacts for different reservoirs to be easily compared. In addition, by multiplying the highest Criticality scores (or Consequence x Likelihood or Confidence scores) obtained from the FMECA for various dams by the relevant impact score, it is possible to compare element risk between different sites. If, for example, different elements at different dams have identical criticality scores, then priority should be given to the element at the dam with the highest impact score.

EMERGENCY PLANNING

A risk assessment should assist in the risk of dam failure being reduced to an acceptable level. Despite all the effort put into minimising the risk of failure, there will always be a chance, albeit extremely small, of a dam's failure. Where dams pose the potential for significant loss of life and damage to property, it is advisable to have plans to deal with such an unlikely event.

A contingency plan should embrace all aspects of organisation and the procedures to be followed and include comprehensive details of likely scenarios, communities and lives at risk, warning times, lines of communication, sources of expert advice and evacuation procedures, availability of support services and all other essential considerations such as evacuation rates to avoid gridlock, feeding, shelter and transport. The responsibilities of the owner/operator and of the various emergency services should be clearly defined, together with a 'chain of command'. It must be recognised that mass evacuation of people requires well laid out plans and procedures to be carried out with a clear decision structure in place to facilitate management of the emergency. The plan should identify key indicators which would trigger an evacuation. There is the need to be aware that by opting for mass evacuation one civil emergency (the dam failure) is being replaced by another (mass evacuation) and that neither is without risk to life and property.

The extent of potential inundation and flood damage following a dam failure is usually determined by dam-break modelling and the preparation of inundation maps. Dam break modelling has now been carried out for a substantial number of dams but the reasons for undertaking such work have varied: in some cases it has been to determine the flood category of the dam, in others for emergency planning purposes. The scale and quality of inundation maps has also varied considerably: in some cases the scale has been so small that it is difficult to see which properties are likely to be affected. Recommendations on the preparation of inundation maps were included in a report by Binnie and Partners (1991), although these recommendations are not being universally applied. New guidelines on dam-break modelling and the preparation of inundation maps are likely to result from the CADAM (Concerted Action on Dam-break Modelling) study being funded by the European Commission.

The question of whether to inform the public of the existence of inundation maps and contingency plans is a matter of some debate. Reservoir owners are generally reluctant to release them publicly because of the risk of adverse reaction from the public and the media: they could, for example, cause unwarranted stress or adversely affect property values. Most owners are, however, prepared to release such information on a confidential basis to Emergency Planning Officers and the emergency services. This is a matter which requires further discussion between reservoir owners, Government Departments, the Environment Agency and Emergency Planning Officers etc.

CONCLUSIONS

The risk assessment methodology which has been developed during the CIRIA study should assist reservoir owners to rank their dams in terms of risk and hazard, and help to prioritise investigatory and remedial works. The methodology has only been tested on a small number of reservoirs to date. It is anticipated that the methodology will be reviewed following more widespread use, and it may be appropriate to publish an additional guidance note. The updating of databases to include information on dam incidents is an ongoing process, and personnel involved in reservoir safety are encouraged to provide information (confidentially if desired) on incidents.

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Safety issues at small reservoirs

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SYNOPSIS. This paper reviews recent experience with safety issues on small reservoirs in southern England that are registered under the Reservoirs Act but in general do not pose a significant threat to public safety. These dams are often old and therefore not constructed to modern standards. The reservoirs are also frequently developed in cascade. Over 50% of the 60 dams considered in this paper have longstanding seepages and 20% have been overtopped. The paper also describes works that have been carried out on these small dams to meet modern flood standards, and the approach that has been taken to contain and monitor seepages. The paper concludes by noting the need for a sufficient level of engineering advice, even for small scale works, to ensure the success of the rehabilitation measures undertaken at small dams.

INTRODUCTION

The Reservoirs (Safety Provisions) Act 1930 introduced the requirement that all reservoirs with a storage capacity of five million gallons (22,700m^3) or more, above the lowest natural ground level, should be subject to periodic independent inspection to ensure that they were safe. The Reservoirs Act 1975 made improvements, including the introduction of the role of the Supervising Engineer, and the qualifying reservoir size was metricated to 25,000m^3.

This paper considers safety issues on 60 small reservoirs, mostly in southern England, which fall under the Reservoirs Act where the authors' company (Brown & Root) provide, or have provided, the Supervising Engineer.

DEFINITION OF SMALL RESERVOIR

There are various definitions of small that may be used, some relating to the capacity of the reservoir and others relating to the height of the dam. The classic textbook, Design of Small Dams (USBR, 1974), uses the definition of the height above the stream bed of less than 50 feet (15m). To qualify for the ICOLD register of large dams the dam must be 15m above the lowest foundation level. However, dams between 10 and 15m in height may be included if they also exceed one of the following criteria: length of crest 500m, reservoir capacity 1Mm^3, maximum flood discharge of 2000m^3/s or if the dam had difficult foundation problems or is of unusual design.

Dams 2000, Thomas Telford, London, 2000.

In the United Kingdom (UK) reservoir safety legislation states that 'large raised reservoirs' are those with a reservoir capacity greater than 25,000m^3. This was defined following the failure of Skelmorlie dam in 1925 with a reservoir capacity of 24,000m^3 and height of 5m, killing 5 people. The main parameters of the other dams that have failed in the UK with loss of life are summarised in Wright (1994). It is of interest to note that the references quoted by Kennard (1998) suggest that a contributory factor to the failure of Skelmorlie was the release of water from storage in a quarry upstream.

Proposals for a change in the Reservoirs Act (Sims & Parr, 1998), although not implemented, used a reservoir capacity of 100,000m^3 as being the minimum size of reservoir which no longer automatically warranted continuous supervision, in the meaning of the Act. This would only be implemented where the Inspecting Engineer indicated that there was no risk to public safety. It is of interest to note that the latter proposal would potentially have affected approximately 45% of the dams on the Building Research Establishment (BRE) Register of British dams (see Figure 1). The equivalent percentile for height is 7m (Figure 1).

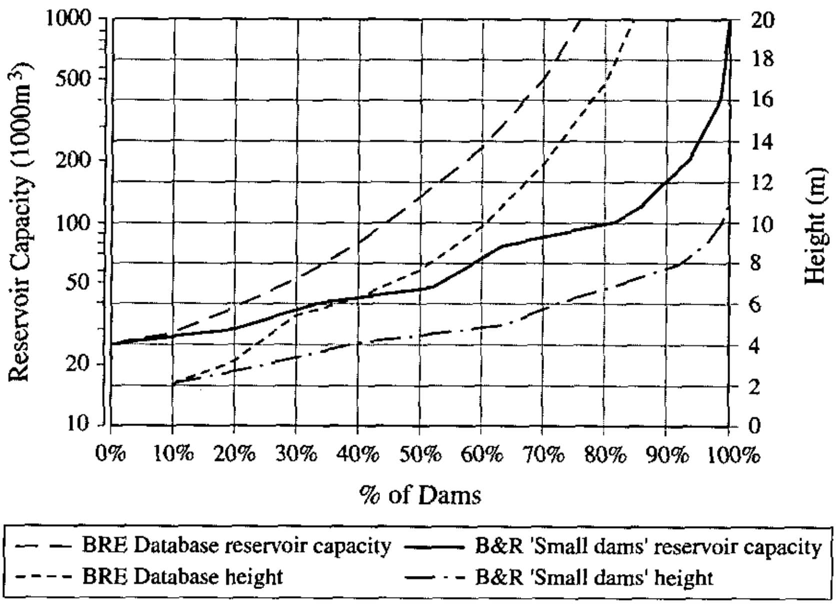

Fig. 1. Physical characteristics of reservoirs in the UK

An alternative approach to using the physical dimensions is to consider the significance of failure of the dam, in terms of its potential impact on the public. The dam category for the application of flood, wind and wave standards is well established (ICE, 1996). The term "small" could be used where the dam is Category C (where a breach would pose negligible risk to life and cause limited damage) or D (no significant threat to life or property), in the sense that the impact of a breach on the public would be small.

Figure 1 also includes the distribution of dam height and reservoir capacity for the reservoirs considered in this paper (referred to as B&R small dams), whilst Figure 2 shows the distribution of dam category in terms of potential hazard to life and property downstream (ICE, 1996). Eighty percent of the B&R small dams considered have a reservoir capacity of less than 100,000m^3 or are less than 7m high, whilst 80% of the dams are Category C or D. The B&R small dams are embankment dams, apart from two old masonry dams.

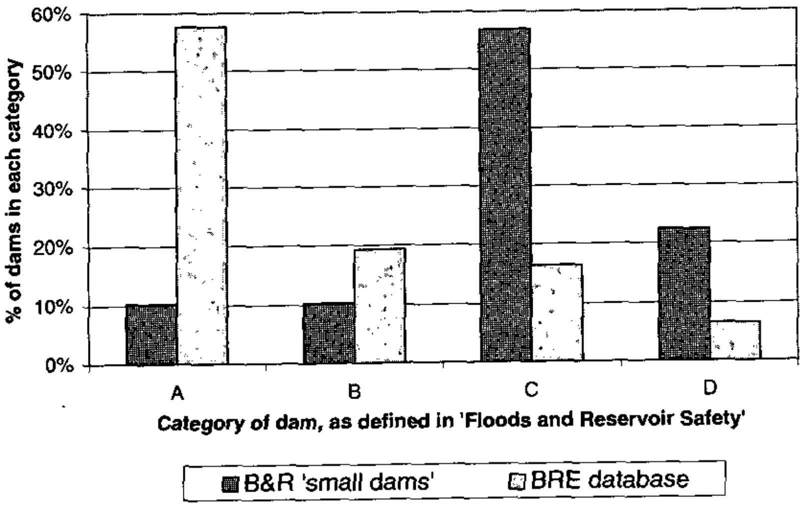

Fig. 2. Dam category (Floods & Reservoir Safety, ICE, 1996)

AGE OF DAMS

The age distribution of both the dams on the BRE database and the dams considered in this paper is shown on Figure 3. The figure also shows a number of key dates in the development of the engineering of dams in the UK, as defined by Binnie (1987), Skempton (1989) and Glossop (1961).

It is noted that only 35% of the dams on the BRE database and only 10% of the B&R dams were built after 1940, when soil mechanics was used in design and modern compaction plant was used for construction. It is also noted that dams built before about 1750 would be expected to be homogenous with no clay core or foundation cut-off trench. The standard of design and construction of older dams would be expected to be below that of more recent dams, noting that it is likely that the B&R small dams would have a greater than average need for maintenance and remedial works.

Many of the older dams discussed in this paper were developed in cascades frequently including dams with a reservoir capacity below 25,000 cubic metres, the threshold for registration under the Reservoirs Act. Before 1770 dam builders would not exceed about 7m height (Skempton, 1989), as a result of the limitations imposed by contemporary engineering methods and equipment. Thus to fully develop a river several dams had to be built in cascade. An example of a cascade and some of the problems it poses are discussed in the section on flood capacity.

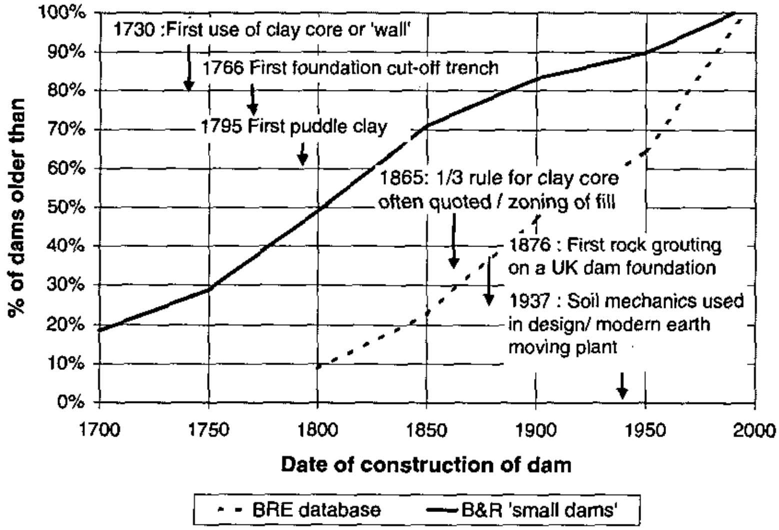

Fig. 3. Age distribution and the historical development of UK dams.

OWNERSHIP AND VALUATION OF SMALL RESERVOIRS

Many of the small reservoirs are owned by small fishing clubs, private individuals or charities, to whom the cost of reservoir maintenance is a major burden. Small dams are frequently not insured for their replacement cost, although they are insured for public liability.

There are a number of ways in which a reservoir may be valued: as the replacement cost of the dam, in business terms of turnover and profit of the business sustained by the reservoir, as the market value or as the insurance value. A commercial fishery could have a market value of between £10,000 and £25,000 per acre for the water and say 30 to 50% of this for the surrounding land. A small reservoir used as a fishery with a surface area of around 5 acres (2 ha) might be valued at around £150,000 to £200,000 including its surrounding shore. The replacement cost could be of the order of £250,000. The annual rental from such a fishing lake is likely to be well below £5,000.

It is clear that rehabilitation works costing say £100,000 would represent a large percentage of the market value and replacement cost and could not be funded from fishing income. A similar problem arises with site investigation where the cost is likely to be a minimum of £10,000 and can only be justified if savings in the cost of rehabilitation works of greater than this value are likely to result.

TYPICAL PROBLEMS

The condition of the dams when first seen by a Reservoirs Act Panel Engineer was highly variable. A few were extremely well maintained, the majority had a minimum of maintenance carried out and some had no maintenance at all. It is common for the downstream face and crest to consist of semi-mature woodland; the downstream face may therefore be initially completely overgrown and inaccessible. Although a low level outlet is sometimes present, it is unusual for this still to be functioning, such that it is normal to have no permanent facilities for emptying the reservoir.

It is rare to have any form of construction records or drawings, other than those prepared during the first inspection under the Act. Similarly it is unusual to have any topographic survey, the only survey data being that on the published Ordnance Survey maps.

Typical problems encountered at these small dams were as follows:

- Settlement and erosion of the crest by pedestrian or vehicular traffic resulting in a loss of flood freeboard
- Inadequate spillway capacity
- Seepage emerging on the downstream face or at the toe. This was rarely much more than a steady trickle.
- Standing water and brown staining at the downstream toe.

The first two problems are discussed in the section on flood capacity and the latter two in the section on seepage. Embankment slope stability has rarely been a problem, whilst wave erosion is normally treated as an ongoing maintenance item.

FLOOD CAPACITY

Assessment of the design flood

'Floods and Reservoirs Safety: an engineering guide' (ICE 1996) proposes four dam categories for use in the assessment of the appropriate design flood. These categories are based on the potential hazard to life and property downstream. For a small reservoir, particularly less than 5 metres high, it is sometimes difficult to assess with any certainty the threat to life that might result from a breach, because of the lack of detailed survey and the limited height of the flood wave and depth of inundation. The normal way to assess this for a large reservoir would be to carry out a dambreak analysis, including survey downstream, but the expense of these would rarely be warranted for a small reservoir.

In densely populated southern England it is unusual for there not to be a minor road or footpath a short distance downstream of the dam, which are features suggesting classification as Category C (where a breach would pose negligible risk to life and cause limited damage). Hence Category D of 'Floods and Reservoir Safety', which poses no significant threat to life or property, is unusual in southern England.

It is normal to rely on the Rapid Method in the ICE Guide for estimating flood capacity. However, difficulties arise in applying it to small reservoirs, which often have small catchments (less than $5km^2$) with no blue line on the 1:25,000 OS map (perennial stream, used to define mainstream length) and are in a cascade of reservoirs. Where comparisons have been carried out with a full analysis, the rapid method commonly proves to be slightly conservative.

One of the initial problems experienced is that of persuading the owner that inadequate flood capacity poses a real risk of damage and potentially failure of the reservoir. He will frequently maintain that the reservoir level has never approached the level of the embankment crest. However, on further probing or discussion with his staff, situations when the embankment has been overtopped are often recalled. Any evidence of overtopping also helps to persuade the owner that there is a potential problem. Evidence of overtopping of the embankment has been found in 20% of the reservoirs referred to in this paper. This has ranged from repairs and evidence of reconstruction of a breach, through deep erosion gullies on the downstream face to shallow gullies and deposits of brushwood washed over the crest.

Types of remedial works

The types of remedial works used to provide an increase in spillway capacity fall into the following broad areas: crest raising, reinforcing the crest and downstream face to permit overflow, lowering of the maximum retention level (MRL), construction of a new spillway or construction of an auxiliary spillway. In many cases where reservoirs are small no specific allowance is made for wave freeboard in addition to the flood freeboard.

Crest raising poses a number of difficulties on small reservoirs but can be a cost effective means of increasing the spillway capacity where the embankment is relatively short, the crest is sufficiently wide and access along the crest is only for pedestrians. Crest raising may consist of increasing the full width of the crest or the provision of a bund of minimum width normally upstream of the crest path. The main difficulties encountered, particularly where the owner carries out the work himself or employs an inexperienced contractor, are:

- Ensuring that organic material is removed from the formation prior to commencing the earthworks
- Keeping mature trees alive when fill is placed around the trunks. This has been successfully carried out by providing a gravel drainage zone around the trunk.
- Ensuring that impervious material is used for the raised portion .
- Ensuring that an allowance is made for subsequent settlement and erosion of the crest fill.

Example of a cascade of reservoirs

As mentioned earlier, cascades of reservoirs are common features of the B&R small dams. One example which comprises five reservoirs is illustrated in Figure 4. Three of the reservoirs are under the Reservoirs Act; namely Lower Pond and the two fishing lakes. The two upper reservoirs, built in 1860, are retained by earth embankments about 4 metres high with the spillway comprising a weir formed by stoplogs discharging into a culvert through the embankment. The capacity of Upper Pond was just below 25,000 m^3, the threshold of registration under the Reservoirs Act, and showed obvious signs of previous overtopping with erosion gullies on the downstream face. The capacity of Lower Pond was estimated as 29,500 m^3.

Immediately downstream of the Lower Pond the stream discharges into a small Mill Pond. Immediately downstream of the crest of this pond a large mill house and adjacent cottage are built across almost the entire width of the valley. The ground floor level of these buildings is about 1.5 metres below the crest of the Mill Pond embankment. There is also a pair of semi-detached cottages on the eastern side of the pond embankment with their ground floor at the pond embankment crest level. The spillway from the Mill Pond passes adjacent to the mill house and comprises a narrow weir discharging into a 0.65 metre diameter pipe.

Downstream of the mill the stream runs alongside two further lakes in a man-made channel with a capacity of approximately 0.5 m^3/s. A concrete diversion structure with stoplogs can divert streamflow into the lakes when required. Immediately downstream of the lakes the stream passes under a footpath and then under various roads before entering a major river 4 km downstream.

The principal difficulty surrounded the selection of the dam category in accordance with 'Floods and Reservoir Safety'. Even though the Lower Pond dam is only 4 metres high, the presence of the houses directly across the flood plain only 200 metres downstream suggests that lives could be at risk in the event of failure of the dam. The capacity of the reservoir was estimated to be 18% of the volume of the 1,000 year flood and thus, based on the guidelines of Floods and Reservoir Safety, the additional damage

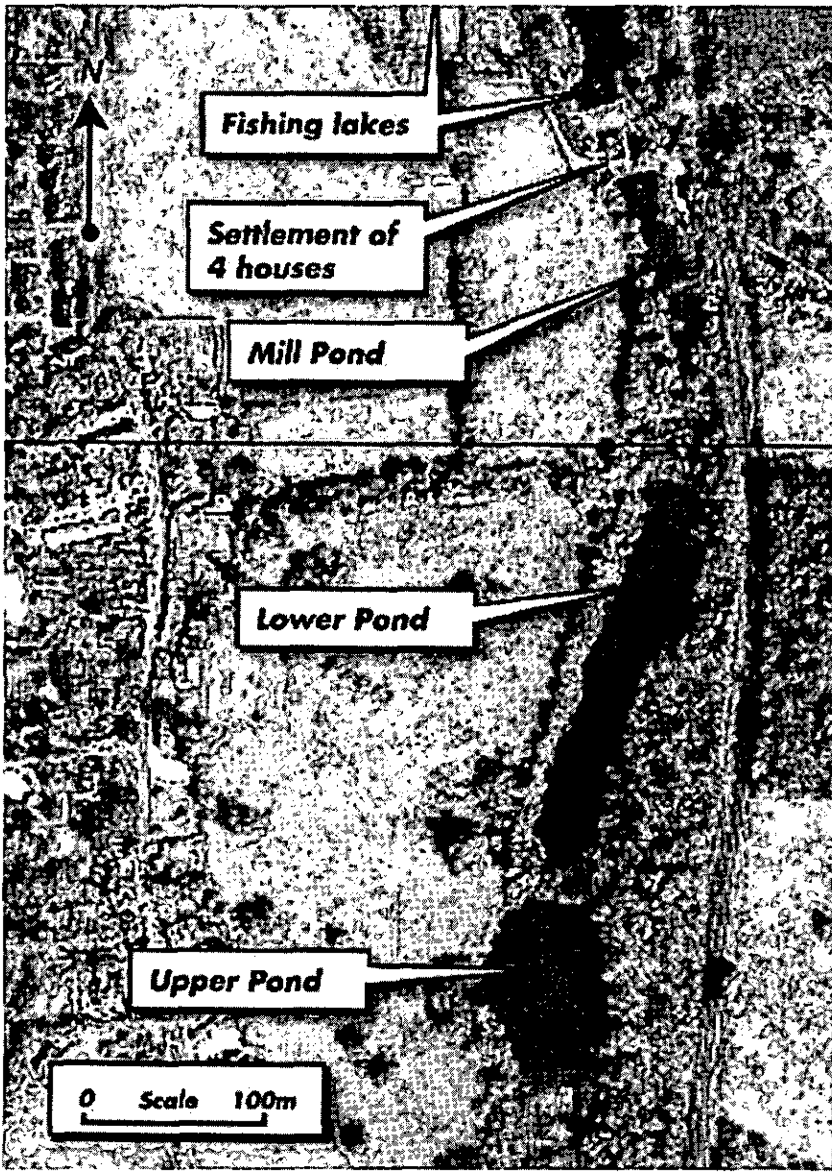

Fig. 4 Aerial view of upper part of cascade of five small reservoirs

caused by the release of the flood wave arising from a breach would be significant. Thus despite its low height the dam was considered to be Category A. However care had to be taken in considering the recommendations as any significant increase in the spillway size for discharging normal flows could have an adverse impact on flooding at the house downstream since the overflow capacity at the mill pond is very small. The proposed solution was to reduce the reservoir level in Lower Pond to provide additional flood storage and to leave the existing spillway arrangements unchanged but reinforce the embankment crest and downstream slope to enable it to withstand overtopping flow.

It was also considered that if the Upper Pond embankment failed due to erosion caused by overtopping flow the resulting flood wave could cause the Lower Pond to fail even with the proposed reinforcement measures. The recommendation for downstream face reinforcement was therefore extended to also cover the Upper Pond embankment , which has the same owner as the Lower Pond.

SEEPAGE AND INTERNAL EROSION

Assessing the problem

Seepages vary from the ground normally being damp, through standing or slow flowing water with brown staining (precipitation of ions in the water, or bacteriological action), to boils or other vigorous flows. Boils and vigorous flows are obviously an immediate problem requiring action. However, discernible flows are in many cases longstanding and stable, such that immediate action is not necessarily warranted.

Investigation and assessment of seepages is normally carried out at the time of the first appointment to a new reservoir, and reviewed at each subsequent Section 10 inspection. This includes assessing whether seepage is caused by factors such as groundwater or whether it represents seepage from the reservoir, and if so whether it is through the foundation or embankment. It is carried out by a combination of visual observation and a desk study (Charles et al, 1996). The latter comprises a study of published geological maps and memoirs and contacting local sources such as hydrogeologists at the Environment Agency (EA) and local water companies.

Seepage is a safety issue where it is considered to be causing, or may cause, internal erosion, or may lead to slope instability. Where a seepage is small and reported as longstanding and stable, and there is no sign of any suspended fines in the flow then the approach normally adopted is the observational approach, as described in Chapter 5 of Johnston et al (1999). Where there is significant flow then in adddition the installation of a filter is normally recommended.

When using the observational approach it is essential to make regular visits over a period of time. This is the Supervising Engineer's role, although on small dams it is necessary to rely on the owner's staff for reporting on seasonal (quarterly or monthly) behaviour.

Where head differences allow, v-notch weirs are installed to quantify seepage and establish whether the base flow changes with time. Inclusion of a small chamber upstream of the v-notch provides a receptacle for deposition of sediment. However, it is important to set the invert of the weir at the correct relative elevation to ensure that seepage does not go round the weir. Local opposition can even be encountered to this simple installation. The Inspecting Engineer's recommendation to install a seepage measurement weir downstream of a 450 year old dam in West Sussex met local opposition on the grounds that the seepage at the toe of the dam was a historic medicinal spring.

Over 50% of the dams considered in this paper have some form of longstanding issue of water, or seepage, at the toe or elsewhere on the downstream side of the dam, which in most cases has been stable and has been monitored using the observational approach. At a number of these dams it has been feasible to install a v-notch weir, or use some other quantitative method of assessing seepage (e.g. a measuring jug and stopwatch). The most common seepage is probably foundation seepage, where the original design either had no cut-off, or an inadequate cut-off, and seepage is occurring in relatively stable foundation strata. Where changes in seepage have occurred which have required action, this has most often been associated either with seepage along, or in the vicinity of, structures, or with tree roots. An analysis of the location of seepage at these dams is given in Table 1.

Table 1 : Location of visible seepage at B&R small dams

Location of seepage	%
No seepage	28 %
Ground downstream of toe	18 %
Downstream toe/ toe ditch	40 %
Embankment face/ crest	14 %
Adjacent to structure	19 %
Through structure (i.e. masonry)	4 %

Note: The total is greater than 100% as some dams have visible seepages at more than one location.

These percentages are not surprising when viewed in relation to the age, and thus form of construction of these dams. Modern dams will incorporate internal filters and drains which are designed to intercept, collect and where practicable measure seepage. The great majority of the B&R small dams would have had no such internal filters and drains.

Repair techniques

The common techniques for dealing with seepage are summarised in Table 2. Case histories using techniques to cut off the seepage for UK dams are well documented in Charles et al (1996) and Johnston et al (1999). Very few examples of the use of drainage have been documented.

Where seepage flows give cause for concern over safety the most frequent approach on small dams, at least as a first stage, is to construct either a toe drain or filter blanket at the downstream end of the seepage. This technique is significantly cheaper than other techniques of containing seepage. The cost of a toe drain or filter will typically only be a few thousand pounds, whilst a positive cut-off will be 10 to 100 times greater than this.

However construction of low cost drainage measures on a small scale is not always successful. Reasons for this include:

- Insufficient definition of the works required
- No site survey to enable the works to be clearly defined
- No engineering supervision of the construction
- Works carried out by an inexperienced contractor, owner or volunteers
- Insufficient attention paid to detailing.

Table 2 : Summary of methods of controlling excess seepage

Approach	Technique	Comparative Cost	Case history
Cut-off flow at upstream end, or on dam centreline	Slurry walls (single or double phase)	High	-
	Excavate and backfill shallow trench with clay	Medium	1
	Sheet piling	Medium	4
	Grouting (injection or jet)	High	4
	Upstream watertight element (blanket)	Varies	-
Control at downstream end	Foundation drainage & filters (Kennard, 1988)	Cheap	2 and 3
Lower reservoir level		Cheap where outlet structure exists	3

Selected case histories for small dams, which illustrate some of the problems encountered, are given below.

Case histories

1. Old Bury Hill lake, near Dorking, is a 5m high Category C dam built in 1780 retaining a 80,000m^3 capacity reservoir. Although the crest width of the main embankment is generous at 12m, this reduces to about 1.5m on

the left abutment (where the lake was formed by excavation). The whole dam is heavily overgrown with trees and there have been two occasions when new seepages have occurred in the left abutment area, flooding the public footpath at the downstream toe. It is considered that these seepages are probably due to seepage along decaying tree roots. On the first occasion the owner, on his own initiative, excavated a shallow trench along the crest, filling this with clay excavated locally, which cured the seepage. The second incident was discussed with the Supervising Engineer and similar remedial works agreed.

2. Epsom Pond is a 3m high Category C dam built in the Middle Ages, breached between 1843 and 1867 for pasture and the breach section then rebuilt by volunteers in 1978-80. The ground at the downstream toe of the embankment is wet due to foundation seepage, varying from boggy in the summer to a pool of standing water in the winter. The overall dimensions of the damp area are 18m long and up to 4m from the toe. An inspection in 1986 recommended various works, including raising the crest, spillway improvements and a toe drain installed along the toe discharging via a v-notch weir in a brick chamber into the spillway outfall channel. No topographic survey was available, with the recommended works shown in sketch form. However, the contractor installed the invert level of the v-notch at ground level, and consequently there has never been any flow over the v-notch. The performance of the dam continues to be satisfactory in that the seepage is stable and there is no sign of material in suspension.

3. Philimore lake at Winkworth Arboretum is a 6m high Category C dam built in 1880 retaining a 28,000m^3 capacity reservoir. It has a long history of seepages along the downstream toe and abutments, which are thought to be predominantly due to the regional groundwater level in the foundation strata, the Hythe Beds, being above the crest of the dam. The Hythe Beds are a local water supply aquifer, part of the Lower Greensand.

In August 1997 a new seepage was noted by the owner's staff in the upper part of the embankment, in the vicinity of the bottom outlet culvert. The leak may be due to "hydraulic separation" at the outlet structure, this being the loss of positive soil pressures between the embankment fill and outlet pipe, allowing water to seep along the interface. The reservoir was temporarily lowered and a filter blanket comprising gravel overlying geotextile placed over the leak. However, on refilling the reservoir, although initially the blanket was successful, the leak soon reappeared above the top of the filter blanket, at about a metre higher than its original level. The reservoir was then lowered again, a lowering of 50mm being sufficient for the leakage to cease. The reservoir is currently being held at this lower level, pending further investigations. The leak remains dry, although at times of flood when the reservoir level rises the leakage restarts.

4. This is a 5m high dam built in 1850 and retaining a 50,000m^3 capacity reservoir. In the mid 1960's the dam failed due to erosion at the location of the low level outlet (possibly due to overtopping), resulting in the emptying of the lake. In the late 1960's the embankment was repaired by the army, using a 14ft deep sheetpile wall over a length of 30ft to repair the breach. In 1975 a sinkhole appeared immediately upstream of the sheetpiling. The inspection recommended digging a trench 5ft deep and 2ft wide upstream of the sheetpiling to investigate the cause of the sinkhole. Several seepage paths through the sheetpiling were identified and were sealed by grouting. The trench was then backfilled with well compacted clay. The subsequent performance has been satisfactory.

CONCLUSIONS

The sample of small dams in southern England considered in this paper is generally over 70 years old, and has a median age of 200 years. Thus the majority were constructed before modern design standards were developed and modern construction plant was available. Many of these dams were developed in cascades of reservoirs, often including reservoirs which have a capacity below the threshold of the Reservoirs Act. This may be because a height of about 6m represented the limiting height for dam technology at the date of construction.

For the sample of 60 small dams referred to in this paper over 50% have some form of longstanding seepage at the toe and 20% show signs of, or are reliably reported as, having been overtopped in the past. Although a median age of 200 years suggests a certain robustness, the performance of the majority is at the limit of what would now be considered acceptable, and the risk of an incident is probably greater than for larger dams. Nevertheless because of their smaller height and reservoir capacity they pose less risk to the public and are commonly Category C, where a breach would pose negligible risk to life and cause limited damage.

The owners of small reservoirs tend to be private individuals or small fishing clubs or charities, to whom the cost of reservoir maintenance is a major burden. Nevertheless these reservoirs have a substantial amenity value. Affordable methods used to ensure their longevity frequently involve small scale works such as crest raising and drainage measures. These measures are frequently implemented by the owner with little professional supervision and are sometimes unsuccessful. The success rate would be improved with additional engineering input to adapt typical details to the site conditions.

The challenge of small reservoirs to the professional dam engineer is to assist the reservoir owner to understand the value of timely practical engineering advice in maintaining the integrity of his asset and maximising the value of maintenance and repair works.

ACKNOWLEDGEMENTS

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Emerging concepts in project risk management: some innovative suggestions and solutions for the dam industry

M W GOOD. Aon Project Risk Advisors

SYNOPSIS. This paper describes some of the newer concepts currently emerging from the Risk Management & Insurance industry and describes how those concepts can bring benefits to the dams Industry. This is of particular importance in privately financed projects, which are likely to be the norm going forward.

The goal of this paper is to encourage the concept that earlier and greater attention to 'Matters of Risk' will produce financial benefits for project sponsors through shortening the time to financial close and by reducing the overall cost of financing.

These benefits will be delivered through the use of the Project Risk Advisory concept which has been developed by Aon Risk Services in response to the demand for fresh ideas in the financing of projects and the mitigation of associated risks. The ideas presented have emerged in the main from practical experiences encountered in South America on a number of hydroelectric projects in which we have been involved.

BASIC PHILOSOPHY – PROJECT RISK ADVISOR CONCEPT

The role of the Project Risk Advisor is that of 'Trusted Advisor on Matters of Risk'. This is based on the premise that risk is not the enemy, risk is a business opportunity. The Project Risk Advisor's function, therefore, is to define, package and price those opportunities in a completely seamless and transparent fashion. This will facilitate financing and help ensure completion on time, on budget.

The Project Risk Advisor concept focuses in the first instance on the risk allocation process. It develops from there into a continual ongoing process taking a holistic view of project risk. Through earlier entry into the project development process the Project Risk Advisor can become one of the originators of risk capital for dams and other projects.

By departing from the traditional role of the Insurance Broker or Insurance Advisor into an expanded Consulting/Advisory role, the Project Risk Advisor becomes more of an underwriter of the development process rather than a vendor of insurance products as has been the case in the past. This shift can only come about through closer working relationships with the project principal.

Dams 2000, Thomas Telford, London, 2000.

Primary Roles

Primary roles of the Project Risk Advisor are –

- Risk Identification & Analysis
- Risk Allocation Consulting
- Financial Engineering
- Financial Advisory & Sourcing of Risk Capital

The challenge for the Project Risk Advisor is to strike an optimal balance between risk and reward. While born out of the insurance brokerage business, the Project Risk Advisor will focus on the broader risk issues related to projects, including those risks traditionally considered 'uninsurable'. More traditional aspects of insurance remain the purview of established construction insurance professionals.

CHANGING PRIORITIES AND CHALLENGES

In today's changing market conditions, the structure of the Dam and Construction industries continues to evolve. Today's equivalent of the former General Contractor, now often the Engineering Procurement & Construction (EPC) contractor in the project structure is either obliged or, in many cases, may prefer to take an equity stake in the project as part of the financial engineering. Private finance requires lump sum fixed price turnkey contracts raising many 'completion risk' issues. Most projects are carried out by single project companies (SPC), whose financiers have limited recourse in the event of failure. Global demand for new and replacement infrastructure, combined with the desire of most Governments to exit the infrastructure business, has created enormous concession opportunities. This provides a challenging contrast to the situation, not so long ago – particularly in the dams business, where Governments financed large civil engineering projects which were constructed under very traditional project structures and contractual conditions.

Risk Managers Viewpoint

From the point of view of the Insurance industry, especially brokers, the task was to respond to the Risk Manager of a successful bidder carrying out a contract, usually, under FIDIC Conditions but possibly with some amendments. In general, the challenge was to go to the Global Insurance Markets and put together the most competitively priced package that could be found for a programme of insurance which matched as precisely as possible those conditions. From a professional standpoint, this was not a huge intellectual challenge but that was the insurance broker's role in the project business. Meanwhile the Employer (usually a Government body) frequently remained liable for the 'excepted risks' which are often the largest in value.

In today's marketplace and without the backstop of a government employer the Risk Manager often finds his position somewhat different. All too often in recent times, he has found that a more adequate job description for his position would be –

- Manager of those risks our commercial people took on in order to win the project and,
- Could not then pass on to anyone else – **but me.**

This is clearly a position in which no Risk Manager should ever find himself; there needs to be continual consultation with those Risk Management & Insurance professionals whose job it will be to provide solutions to risk. This process needs to commence at the conceptual stage and is best managed through a Project Risk Advisor.

PROJECT RISK ADVISOR CONCEPT

What has emerged is the development of a new risk advisory function which goes far beyond the traditional role of the insurance broker in the project business: that of the Project Risk Advisor. His role is to guide his clients through the minefield of risk advising them how to take advantage of the business opportunities that 'risk' presents.

The Project Risk Advisor concept can only flourish if <u>all</u> parties to the project have the same common agenda (as they, indeed, should) of -

- Completion on time
- Completion on budget

Everyone should be using their best endeavours to achieve these goals. It should be 'us against the work and not against each other'.

PROGRESSION OF WHO BUYS INSURANCE

Over recent years one of the fundamental changes in the insurance market is 'buyer progression' in terms of which party contracts the insurance programmes.

<u>Contractor Controlled Insurance Programme (CCIP)</u>

In previous times most forms of contract required the contractor to arrange a series of specified insurances under a Contractor Purchased Insurance Programme (CCIP). The prime constraint was that the insurances should be placed with an acceptable insurance market.

Owner Controlled Insurance Programmes (OCIP)

Thirty years or so ago there started to emerge a concept that it might be better if 'The Owner' of the works or, in those days 'The Employer', purchased the insurance programme under an Owner Controlled Insurance Programme (OCIP). This change was to a great extent fuelled by the experience on the Tarbela Project in Pakistan and the difficulties which arose in securing recovery from differing insurers under differing contracts for some of the losses that arose on that project.

The OCIP has gained considerable following in recent years, not least, because of the private finance initiative. The theory, at least, is that if the lenders interest is to be fully protected by some form of Advance Loss of Profits (ALOP) or Revenue Stream Protection Insurance (RSP), that insurance must be placed in conjunction with the Physical Damage (Contractors All Risks/Builders Risk) Cover. This makes the Owner a more appropriate policyholder than the contractor.

While splitting up the placing of these covers remains undesirable due to potential conflict of interest issues, it is not impossible to structure this differently today.

Lender Controlled Insurance Programme (LCIP)

The question arises in today's market as to who is 'The Owner'. In the case of a well known bridge project a special project company with paid-up capital of £1,000 was the vehicle through which this £350m project was developed.

The providers of the main financing package in this and many other cases may feel that they have greater entitlement to the sobriquet 'Owner' than the holders of the rather modest equity in the SPC. This has resulted in lenders taking a much greater interest in the insurance programme. They, to an extent, are now flexing their muscles saying that they wish to have a much more direct involvement in the insurance programme, even to the extent of promoting Lender Controlled Insurance Programmes.

Combined Financial Advisory & Risk Management

There is, indeed, a strong case to argue that money and risk go hand in hand. Why not, therefore, move a step ahead even beyond the Lender Controlled Insurance Programme and combine Financial Advisory and Risk Management services into a single discipline at one overall fee. The concept is sound and the benefits may be considerable.

Construction Industry Reaction

In the meantime, in USA at least, possibly elsewhere as well, a debate currently rages over who should control the Risk Management and Insurance function in major projects.

A section of the Construction Industry holds the view that the insurance broking fraternity have deserted their traditional client base – the general contractor – and run off with project owners. As a result, brokers regularly now find themselves in conflicts of interest caught between their owner client base and their contractor client base. The contractors also feel that owners are failing to take advantage of the contractors often vast experience of Insurance and Risk Management problems on many similar projects, experience which could in the event of problems be key to the success of the venture.

Now with the emergence of the Lender Controlled Insurance Programme, the general contractor is finding himself even further removed from the process. The fact remains that there is only going to be one insurance programme for the project and that all participants, be they Owner, Contractor or Lender are going to have to rely on that single programme for their protection.

Who should manage the insurance programme? It seems clear that, in the same way that risks must be borne by the party best able to manage and absorb the risk, the insurance programme should be managed by the project participant best able, and most experienced, at handling such matters on a day to day basis. That, in the vast majority of cases, will be the EPC Contractor. So, what is the solution to this conflict? What can be done to get all parties to sign up on the common 'risk agenda'. One possible solution is to have a Project Risk Advisor.

PROJECT RISK ADVISOR CONCEPT

As stated earlier, the concept of the Project Risk Advisor (PRA) is that he works as the 'Trusted Advisor on Matters of Risk'.

He works for the project and NOT for the individual participants. The role is to ensure complete transparency in the management of risk and, as mentioned earlier, the concept relies on the parties having no hidden agenda from each other in this area.

Project Risk Advisor Benefits

The benefits that the Project Risk Advisor will bring are essentially –

- To reduce time to financial close
- To reduce long term cost of money

Through transparency, the Project Risk Advisor will reduce confrontation and will ensure that internal conflicts are carefully managed. Time and money will be saved.

At a typical risk allocation meeting on a privately financed project, each project participant is regularly accompanied by his attorney, and his insurance broker. What usually develops first is a legal exercise in passing risks between each of the attorney's clients without reference to practicalities or capabilities for efficient management of risks. The second scenario is that the lender's insurance consultant imposes suites of insurances on the project as a condition of financing. Both of these add cost and confrontation setting the wrong initial atmosphere for project success.

The Project Risk Advisor concept avoids, through its transparent approach, the need for everyone to have their own insurance advisor. This will, undoubtedly, reduce cost, conflict and the time involved.

The fundamental key is that the Project Risk Advisor displays in a transparent template all the risks facing the project. It clarifies who bears those risks, what mitigation plan should be adopted, what insurance is available and most importantly, what risks can which project participant sensibly and readily assume. This enables project participants to view this as a business opportunity and not simply an exercise in insurance procurement.

BALANCING RISK AND REWARD

Thus the Project Risk Advisor has the task of assisting the parties in balancing risk and reward. As mentioned earlier, Risk is a business opportunity. Risk is often wrongly confused with uncertainty. Uncertainty is simply undefined risk. If the Project Risk Advisor can assist in successfully defining the risk the result will be the successful financing of the project.

The Basic Process

The process which enables the Project Risk Advisor to produce the solutions involves considering and advising on Risk Identification, Risk Analysis, Risk Allocation and Financial Engineering.

The goal of the Project Risk Advisor will be to make your project more attractive to lenders and equity investors by reducing the downside financial risk and improving the overall viability of the project.

SELECTING A PROJECT RISK ADVISOR

As with the emergence of any new concept there are no precise guidelines or parameters which fully define the role of the PRA or the skills he/she needs to possess. The individual, or company, selected certainly needs good commercial judgement backed up with significant practical experience, diplomatic and communication skills are essential, engineering, legal and insurance knowledge are also necessary as is a willingness to think 'outside the box' when it comes to 'matters of risk'.

NEW MITIGATION PRODUCTS AND CONCEPTS

The secondary purpose of this paper is to highlight some of the new risk mitigation products and concepts which are becoming available both from the insurance markets and from the capital markets. These are all still in the development phase and most are untried in the dams and hydro business. Several have been used in other types of power project.

Blended Risk Solutions

There is a move towards more integrated or blended risk solutions. These will result in a more comprehensive package of protection than the traditional set of different policies providing limited but specific cover addressing different areas of risk.

The convergence between the banking industry and the insurance business is progressing quite rapidly. Much greater consolidation between these previously separate financial activities can be anticipated in the coming decade. The linking of capital market investors with traditional insurance underwriters produces a new potent force with a considerable appetite for risk – provided it is properly defined and adequately priced.

This new breed of player and the emergence of the alternative risk transfer (ART) market will provide risk solutions previously unknown in either industry. The value of these products lies largely in their ability to enhance the credit-worthiness of the project &/or the SPC and which will lead directly to the Project Risk Advisor's goals of lowering the cost of and speeding up the financing.

There are available a wide range of creatively devised financial products which can be applied to formerly 'uninsurable' risks. Ultimately, the scope of and the extent to which these products can be used will become more fully defined but as they emerge their availability and implication is limited only by the imagination of the parties involved. Here, just a very modest selection are mentioned which have particular relevance to hydropower projects.

Cost Overrun Protection

Of the new products coming from the marketplace probably the single most important product as far as the dams industry is concerned is the development of cost over-run insurance for tunnelling and foundation work.

This is by no means an off-the-shelf product, nor is it readily available, but it is possible to cap the tunnelling cost risk through the insurance market. Of the uncertainties facing dams projects the risk of encountering something unforeseen in the tunnels remains the greatest concern of lenders. Anything which can be done to assuauge this concern and guarantee costs within certain parameters is certain to be hugely welcomed.

Regulatory Risk

Another new product recently to emerge from the market is a wide ranging Political or Regulatory Risks form of cover. This is designed to provide an indemnity in the event that any changes occur to the 'level playing field' perceived at conception of the project during construction and into commercial operations.

Revenue Stream Stabilisation

Numerous new mechanisms have been developed for the protection of project revenue. These vary from Loss Stabilisation Programmes to sophisticated Capital Equity Puts (Cat-E-Puts), the detailed description of which is beyond the scope of this particular paper. These products can be used in the hydropower business to deal with the hydrology risk and might also have application in the merchant power market, which is now developing in many countries. Again, the function of all these products is to make the project more attractive to lenders.

CONCLUSION

A role has been defined in which the residual risks inherent in dam projects, be they geotechnical, hydrological market or construction management, can be transparently allocated into financial vehicles with the objective of making projects more economically viable.

Developments in the British national dams database

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SYNOPSIS. The formation of a national dams database commenced in 1987 at the Building Research Establishment and the database has been progressively expanded and updated over the last 12 years as part of the DETR Reservoir Safety Research Programme. It contains information on some 2650 dams which impound the 2500 reservoirs that come within the ambit of the Reservoirs Act 1975.

INTRODUCTION

In his report to the Secretary of State for the Environment on the failure of Carsington dam, Coxon (1986) recommended that consideration be given to centralisation of records of certification and inspection. He commented that the Building Research Establishment (BRE) appeared to be well suited for this activity. Although no regulatory changes were made to implement this proposal, in 1987 BRE commenced forming the national dams database as part of the DETR Reservoir Safety Research Programme.

Some work on hazard and risk assessment for an old earth embankment dam had been carried out prior to the formation of the BRE dams database and this led to the conclusion that existing data were inadequate to quantify the risk of dam failure. In his report on this application of quantitative risk assessment to reservoir safety, Cullen (1990) recommended that the retrieval and collation of information on dam incidents should be continued with the co-operation of owners and engineers to expand the database now formed.

The BRE dams database contains information on the stock of dams whose reservoirs come within the ambit of the Reservoirs Act 1975. As there is no formal requirement to supply information to a central agency, the information has been acquired from a number of sources including responses to questionnaires, published information and private communications. Despite this limitation the database contains some information on virtually every dam in the UK. Information is available to parties with a legitimate concern with the safety of dams, subject to issues of confidentiality. Many engineers have availed themselves of this facility and have contributed much to the database. A report can be created on any dam giving basic details such as height, dam type, location, and problems and remedial works together with any published references.

Dams 2000, Thomas Telford, London, 2000.

The initial objectives of the database were as follows:

- To provide a register of dams that come within the ambit of the Reservoirs Act 1975.
- To provide background information for the DETR's Reservoir Safety Research Programme.
- To identify research needs.
- To assemble data on dam failures and incidents to allow some form of risk assessment to be carried out.

Tedd et al (1992) described the structure of the database and presented the results of some initial searches of the incident and remedial works data. In 1994, a Register of British dams was produced (BRE, 1994) and a bibliography of British dams was published in 1996 (Charles & Tedd, 1996). The bibliography within the database now contains more than 700 references and is updated through the annual review of publications relevant to reservoir safety (Charles & Tedd, 1999). Searches of the database have been undertaken in connection with the preparation of a number of engineering guides.

Increasingly there has been a requirement for the database to provide data to undertake some form of risk assessment. It has been used recently for data analysis for the CIRIA Reservoirs and Risk project (Hughes & Hewlett, 2000).

POPULATION OF DAMS AND RESERVOIRS

Data on the development of dam construction with time based on the distributions of dam height and reservoir capacity were published in the Register of British dams. With devolution, it is of interest to know how many dams are within each country. Table 1 summarises the distribution of dams in terms of basic types: embankment, concrete/masonry and service. Embankment dams predominate in all countries but there is a much larger percentage of concrete/masonry dams in Scotland. In addition to the dams in Table 1, there are data on historic failures and on 173 reservoirs that have been discontinued or abandoned. There is also information on 50 dams in Northern Ireland and 13 dams on the Isle of Man and the Channel Islands.

Table 1. National population of dams

Country	No. of dams	Percentage of total population			
		All	Embankment	Concrete/ masonry	Service
England	1705	64	57	5	2
Scotland	752	28	20	7	1
Wales	196	8	6	1	1
Total	2653	100	83	13	4

The development of UK reservoir capacity and number of reservoirs with time is shown in Fig. 1. It shows the effect of the older dams that were generally low structures with relatively small reservoir capacities and the influence of constructing much larger reservoirs in the 1950s, in particular the construction of the hydroelectric schemes in Scotland where concrete dams were commonly built to impound very large reservoirs such as Quoich with a capacity of $382 \times 10^6\ m^3$.

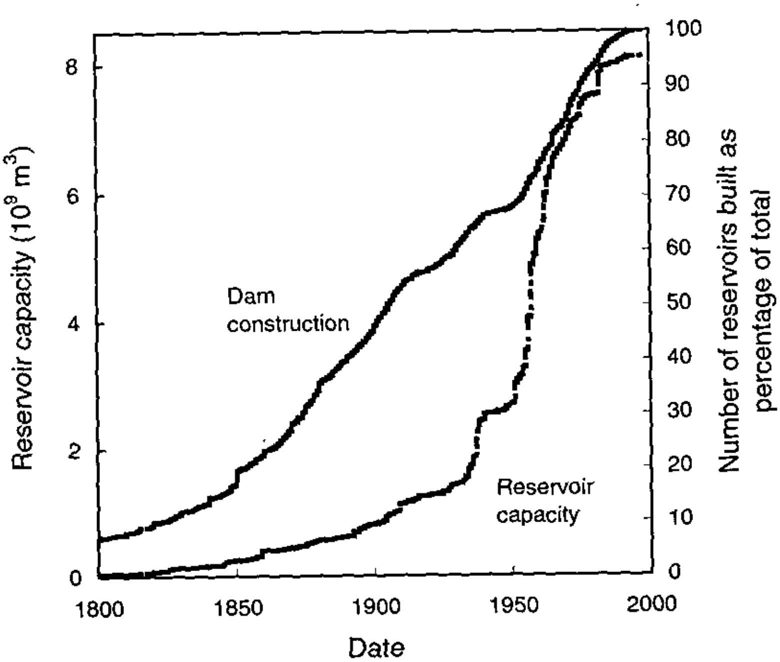

Fig. 1. Increase in UK reservoir capacity and number of reservoirs with time

DAM OWNERSHIP AND HAZARD

Statistics about dam height and reservoir volume can be compiled from the data in the Register of British dams, but it is more difficult to obtain statistics on the potential effects of dam failure. These hazards do not correlate well with dam height and reservoir volume as they depend critically on the location of people and property downstream of the reservoir. The four dam categories defined in the ICE Floods Guide (ICE, 1996) can be used to provide a broad indication of downstream hazard and are given in Table 2. However, it is clear from the data held on the database that reclassification on successive reservoir inspections is not uncommon.

Table 2. Dam category (after, ICE 1996)

Dam category	Potential effect of a dam breach
A	Where a breach could endanger lives in a community
B	Where a breach (i) could endanger lives not in a community or (ii) could result in extensive damage
C	Where a breach would pose negligible risk to life and cause limited damage
D	Special cases where no loss of life can be foreseen as result of a breach and very limited additional flood damage would be caused

Information on dam category is available for about one third of the dams in the database. The percentage of these dams that fall in each of the four dam categories is shown in Table 3 in relationship to ownership category.

Three categories of owners have been defined as follows:

L Owners of more than 20 reservoirs
M Owners of between 5 and 20 reservoirs
S Owners of fewer than 5 reservoirs

More than two thirds of the available data relate to dams belonging to the category L dam owners whereas these owners account for only half of the total number of dams. Consequently the percentages in Table 3 may not be fully representative of the total dam population.

Category L owners are mostly water supply companies, together with Scottish and Southern Energy for hydropower dams and British Waterways. Some 35% of the dams are owned by category S owners and approximately 600 of those owners have only one dam, representing approximately 25% of the total stock of British dams.

Table 3. Downstream hazard in relation to reservoir ownership (all figures are percentages)

Dam Category	Ownership category		All
	L	M&S	
A	70	25	56
B	18	20	19
C	10	36	18
D	2	19	7

Category A and category B dams together make up more than two thirds of the total dam population and represent those reservoirs where there is a significant risk to life and or extensive damage to property. The majority of category A dams are in ownership category L. Some 70% of dams in ownership category L are in dam category A. In a sub-sample of 60 small embankment dams in the south of England, Gosden & Brown (2000) found that only 10% were in category A and nearly 80% were in dam category C or D.

CLASSIFICATION OF INCIDENTS AND REMEDIAL WORKS

Problems and remedial works have been classified in the database according to their seriousness as shown in Table 4. The definition of the severity of a given problem is necessarily subjective and the study of the remedial works can give a further indication of the severity of the original incident. In general, the more serious a problem or remedial works, the more likely it will be in the public domain.

INCIDENTS AND PROBLEMS AT EMBANKMENT DAMS

The statistical analysis of historical data on the frequency and severity of problems found in dams of a range of ages and dimensions can be used to give an indication of the probability of failure of a dam by a number of different modes. Information on the recorded incidences of a number of selected problems has been analysed to determine the usefulness of the current data in assessing the risks posed by embankment dams.

Charles et al (1998) undertook an analysis of a subset of 700 British embankment dams which are at least 10m high to study the occurrence and seriousness of internal erosion. The data indicated that nearly half of the incidents occurred in the early years during first filling of the reservoir. This was followed by a long period when the frequency of reported incident was low with some increase in problems in the very long term.

Table 4. Problem and remedial works classification

Problem classification:	Remedial works classification:
1. *Failure:* Uncontrolled release of water resulting in death or damage downstream.	1. *Total reconstruction* following major incident
2. *Serious incident involving emergency action or drawdown:* Large settlement or leakage involving emergency action to draw down the reservoir.	2. *Major works* following serious incident
3. *Incident causing concern, major investigation and remedial works:* Incident or change in previously observed behaviour to cause major investigations and remedial works.	3. *Works carried out in the interests of safety*, eg floods works
4. *Symptoms causing concern:* symptoms causing sufficient concern to involve an inspecting engineer and an increase in surveillance and monitoring of instrumentation.	4. *General maintenance*
5. *Design limitation:* Inadequate flood capacity, scour capacity, emergency drawdown capacity, slope stability based on reassessment.	5. *Raised to increase capacity not safety*

Skinner (2000) has undertaken a more detailed analysis of incidents using the same subset of embankment dams. The number and date of reported problems in the database, for this subset, is shown in Fig. 2. The number of reported problems has been normalised against dams in existence at that date. Figure 2 shows that the number of reported problems is dominated by internal erosion and for all types has increased significantly from the first half to the last half of the 20th century. A number of factors may have affected this increase: increase in the publication of reported incidents, increasing age of dams and increase in required safety standards. Some 77% of the problems reported as an inadequate overflow have been categorised as design limitation, category 5 in Table 4. The database indicates that 70 % of those cases were category A dams under the floods definition.

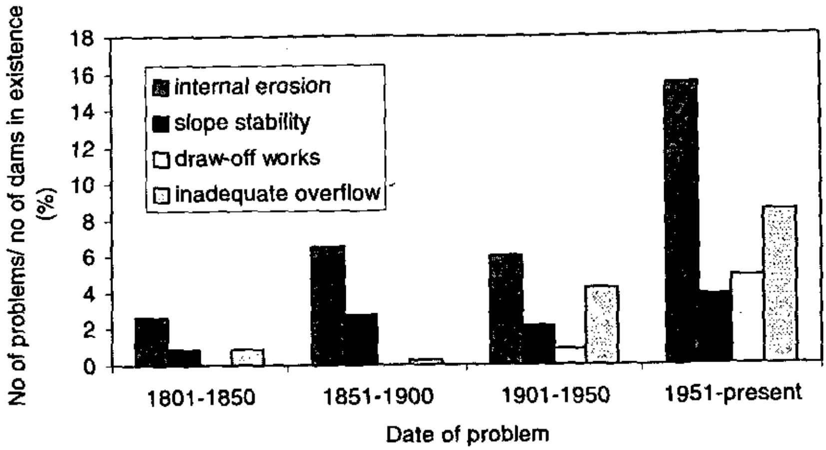

Fig. 2. Distribution of problem type with date

REMEDIAL WORKS AT EMBANKMENT DAMS

Remedial works are undertaken either in response to an incident or deterioration of a dam or because of an increase in the required safety of a dam following new guidance. The database has been structured to take account of these various types of remedial works. Upgrading of spillways to allow the dam to carry the design flood in accordance with current guidance has accounted for a significant proportion of the remedial works. This has had an impact on the instances of failure due to overtopping, which have decreased over the same period (Wright, 1994). The use of historical data on overtopping to analyse the current probability of overtopping occurring is not valid because of the improvement to overflow works. Fig. 3 shows the frequency of overflow improvements with time.

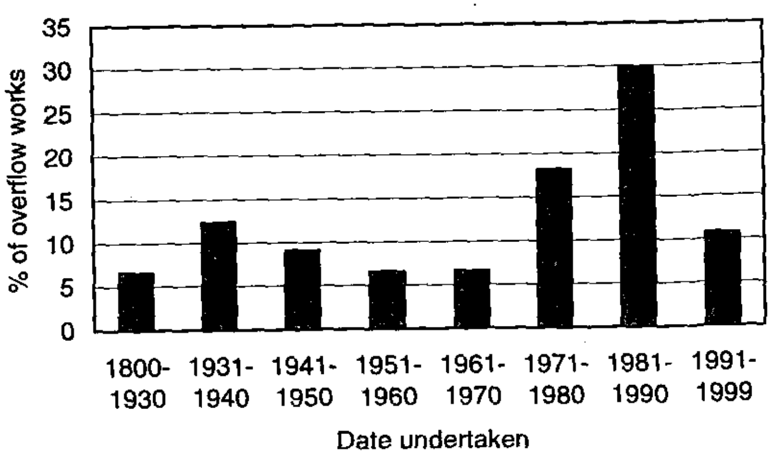

Fig. 3. Improvements to overflow works with time

as a function of dam category. Remedial works to control leakage may also improve slope stability. Between 70 and 90% of the remedial works have been undertaken at category A and B dams.

Table 5. Remedial works as function of type and dam category

	No of Works	Percentage distribution of problem type with dam category			
		A	B	C	D
Leakage	137	65	23	11	1
Slope instability	95	52	26	20	1
Draw-off works	225	64	33	2	1
Overflow works	255	62	21	14	3

Whilst the rate at which major problems are reported seems almost constant or reducing since legislation has been introduced, the reported incidence of problems is increasing. This may be as a result of improved surveillance and reporting and a greater understanding of geotechnics. There has been an increasing number of remedial works involving slope improvements in the last decade.

Most reported remedial works associated with draw-off systems have been carried out on dams over 50 years old and 60% of these are associated with repairs to the valves or pipework. One of the most common recent types of repair has been relining of the old outlet pipes which were often cast iron and may have become corroded.

The increase in reported remedial works since 1930 may be due to the changes that have taken place in dam ownership. There have been changes in the way in which dams have been managed and supervised; from a reservoir keeper with a gang of workmen making repairs on a regular but unreported basis, to a more structured system of supervision and reporting. The knowledge of the risks posed by incidents has increased during this time, and that willingness to accept risk has reduced.

CONCLUSIONS

The reservoirs of the United Kingdom represent a significant asset to the country and it is appropriate that there is a national database. The database has been used by a number of organisations for research purposes and in assisting many engineers concerned with safety of individual dams. It has provided a register of British dams and an extensive bibliography. Data should continue to be gathered on major and minor incidents and remedial works in order to both maintain our current level of awareness and increase the usefulness of the data.

ACKNOWLEDGEMENTS

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Environmental assessment of reservoirs as a means of reducing the disbenefit/benefit ratio

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SYNOPSIS: Successful applications for new reservoirs in the UK are now rare; reasons for this usually include strong opposition to issues arising from the scale of environmental change. Ideally mitigation of environmental impacts (ecological, cultural, landscape or socio-economic) should replace 'like' with 'like'. Where this is impossible, the 'quantity' of mitigation should significantly outweigh the losses. This paper reviews alternative approaches to mitigation, including more strategic evaluation of county/regional environmental priorities, deficiencies and sensitivities, thereby highlighting opportunities for reservoir developers to make a significant long term contribution to environmental quality. Examples refer to mitigation for existing and proposed reservoirs.

INTRODUCTION: CHANGING PRIORITIES

Successful applications for planning consent to construct new reservoirs in the UK are becoming rare phenomena. The growth in UK and European environmental legislation, reflected in national and local planning policy, presents a daunting challenge to any water company seeking to identify sites for new reservoirs, or even wishing to extend existing reservoirs. Whilst many planning constraints apply to development in general, thereby increasing the competition for a diminishing supply of available land, the scale of land-take and the particular geographical and topographic requirements of reservoirs impose additional constraints upon site selection. One could speculate about which of our earlier UK reservoirs would have gained consent under current environmental legislation.

Planning authorities determining reservoir applications now require evidence of the 'need argument', and also evidence that other options, for example leakage reduction and inter-catchment transfer, have been fully explored. For example, the proposed Broad Oak reservoir, for which no application for consent was ever submitted, was considered likely to be refused on the grounds of issues relating to water demand. It has been suggested that Kielder reservoir in Northumberland was built about 20 years earlier than necessary, demand for water in the Tyne/Tees area having diminished during the design period.

Dams 2000, Thomas Telford, London, 2000.

Many areas now occupied by reservoirs are protected through statutory designations, such as Areas of Outstanding Natural Beauty (AONB), Sites of Special Scientific Interest (SSSI) and Scheduled Ancient Monuments (SAMs). In most cases these designations came into being after construction of the reservoirs; indeed in some cases the increase in nature conservation interest has derived directly from the presence of the reservoir. The numerous examples of this process include Abberton reservoir, constructed in 1938 in largely arable farmland with very little nature conservation interest, three kilometres inland from the Essex coast. This site is now a SSSI, an internationally important Ramsar wetland site and Special Protection Area because of its outstanding waterfowl populations. Many other large reservoirs, for example Kielder and Rutland, have become regionally important centres for recreational activities such as watersports, angling, bird watching, and walking (Bridle & Sims,1999).

In other countries, very large reservoir schemes, such as the Aswan dam in Egypt, or the Kariba dam on the Zimbabwe/Zambia border, have resulted in enormous economic benefit, but also in some cases significant environmental impact and social upheaval, involving displacement and resettlement of huge numbers of people. Today the environmental framework within which site selection for reservoirs takes place overseas is either legislation equivalent to that in the UK, or in many cases the environmental requirements of funding agencies. More rigorous socio-environmental assessment has been undertaken for recent overseas schemes, such as Ghazi Barotha reservoir and hydro-electric scheme (now under construction), which was subject to extensive re-routing to avoid features of particular cultural significance (Binnie & Clifton, 1999). A recently proposed scheme, Ilisu in Turkey, has also been subject to detailed environmental assessment.

The need to balance immediate human requirements with the sustainability of the natural environment, especially where internationally important wildlife or archaeological resources are involved, is also recognised by funding agencies. A large-scale east-west waterway, proposed for EU funding as part of the TransEuropean Transport Network, and likely to involve damming of the Vistula river in Poland, has provoked a strong reaction from conservationists. The World Wildlife Fund has urged the Polish government to delay the scheme until a full environmental assessment has been undertaken, and, given Poland's application for EU accession, to consider the need for compliance with the EU Habitats Directive (World Water and Environmental Engineering,1999).

UK ENVIRONMENTAL ASSESSMENT

In the UK, the approach required for environmental assessment of reservoir schemes is prescribed in the Town and Country Planning (Environmental Impact Assessment) Regulations 1999, an amended version of similar regulations issued in 1985. This amended legislation now specifies a requirement for a formal Environmental Statement (ES) for reservoirs with storage capacity in excess of 10 million cubic metres. A reservoir exceeding one hectare in area may require a formal ES if its environmental impacts are considered to be potentially significant. In practice, this amendment to the regulations makes little difference; almost any reservoir that comes within the Reservoirs Act (1975) is likely to generate such interest that a planning authority would almost certainly require an ES.

Whilst the environmental impacts of reservoirs will clearly depend upon their size, location and the habitats and land-uses to be replaced by a large expanse of water, there are a number of impacts which are characteristic of reservoir schemes prior to the formulation of any mitigation measures. These are summarised in Table 1.

MITIGATION

The process of environmental assessment should identify impacts, whether direct or indirect, adverse or beneficial, temporary or permanent, and evaluate their geographical scale and significance. It should also indicate the scope for mitigation of these impacts, and then assess the likely residual impact taking account of the effects of the mitigation measures. Where impacts cannot be mitigated acceptably, the losses incurred by the reservoir scheme must be weighed against its expected benefits.

Wherever possible, mitigation measures must seek to replace "like" with "like". Examples of this concept may include re-creation of amenity facilities such as playing fields; sometimes it is possible to dismantle Listed Buildings and re-construct them elsewhere under suitable professional supervision. Other features are more difficult to re-create; whilst some wildlife habitats such as ponds or plantation woodland can be created elsewhere, even these semi-natural habitats take time to acquire ecological and landscape value equivalent to that of those lost. For other habitats it is, by definition, impossible to re-create their complex ecological structure in the short-term, for example Ancient Woodland (defined as having had continuous woodland cover since at least 1600 AD). A number of experimental projects have translocated the soils and litter from Ancient Woodlands to suitable alternative sites, recognising that these are the elements of the ecosystem which best represent the ecological potential of the habitat. Nevertheless it will take many years before their success can be evaluated.

Table 1. Characteristic Environmental Impacts of Reservoir Schemes.

Impacts	Disbenefits	Potential Benefits
Soils/agricultural land	Significant loss. Reduction in silt provided to fields by flooding	Irrigation potential. Reduction in soil erosion caused by flooding
Ecology: • Species	Loss of terrestrial species	Potential gain for birds, invertebrates, amphibia
• Terrestrial habitats	Significant losses: irreplaceable in same location	None
• Aquatic habitats	Loss of small areas Sometimes reduced variation in d/s flows. Reduction in flooding of wetland habitats dependent upon it	Gain of large area Sometimes d/s flows maintained during droughts
Landscape	Depends on existing landscape value	Potential gain if well-designed
Microclimate	Loss of terrestrial microclimate extremes	Possibly milder climate around large reservoir
Minerals	Sterilisation of moderately deep mineral deposits	Potential for extraction of shallow mineral deposits
Pollution	Possible water quality problems due to stagnation. Temporary siltation d/s during construction; reduction in silt d/s during operation	Sometimes improvement in water qualilty due to retention of silt in the reservoir
Infrastructure/traffic	Potential disruption of routes	Access routes may be improved
Airfields	Birdstrike hazard	-
Human settlements	Possible need for resettlement	Improved amenity value longterm
Recreation	Depends on existing recreational value	Potentially very significant gains: watersports, angling, birdwatching, footpaths, cycleways, etc
Flood defence	-	Flood protection for human settlements and activities

Reservoirs themselves take many years to develop their full ecological potential. As described earlier, Abberton Reservoir took several decades to achieve its current international status as a wildfowl reserve. As the environmental resources of reservoirs develop over time, so can the potential conflicts between them, unless appropriately zoned or otherwise managed. It could be argued that Abberton has not yet fulfilled its multi-functional potential. While it is a very pleasant feature in the flat Essex landscape, a significant opportunity for visual enhancement was missed at the time of its construction. A well-designed scheme of tree planting, at that time, requiring a small additional land-take, would by now have created a regionally notable landscape feature. Priorities were different then!

Possibly the relative visual blandness of Abberton has helped limit its appeal as a tourist attraction, and so left it relatively undisturbed for waterfowl, though its size would probably ensure that enough of its area is sufficiently distant from the shore to avoid disturbance of waterfowl. However further potential conflicts, as well as benefits, are likely to arise from current consideration of works to soften/naturalise the banks. Whilst this would undoubtedly provide visual enhancement, and in time improve the habitat for waders as well as waterfowl, it would also increase the amenity value of the reservoir. Public access would then need to be limited to certain areas to avoid disturbance of the birds.

The "sustainability" concept, brought to the fore in 1992 by the UN Earth Summit Conference in Rio de Janeiro, strengthens the case for reservoirs in a number of respects. The original purpose of reservoirs is usually related to water supply, hydropower (a sustainable power supply) or flood defence. However the development over their lifespan of other uses, including nature conservation and recreation, is a characteristic which should be taken into consideration when assessing their long term environmental impact.

It is usually difficult, when providing mitigation of environmental impacts, to replace "like" with "like" exactly, and to predict the outcome reliably. It is therefore appropriate for mitigation measures to provide a greater quantity of the feature concerned than that which will be lost through the proposed scheme. In some situations the mitigation measures may themselves create unacceptable impacts; for example habitat creation attracting birds in the vicinity of airports is generally unacceptable to the Civil Aviation Authority (CAA) due to the perceived increase in bird-strike hazard.

A further consideration is that it is always technically feasible, if appropriate, to demolish dams, and in time, unless significant siltation has occurred, possibly to restore the reservoir bed to its previous (or other) uses. However in some instances where this has been considered, the ecological and/or landscape value developed by the reservoir during its life span has often been an argument for either wholly or partially retaining the reservoir.

When an option study was undertaken to review possible future uses of Dowdeswell reservoir, located in the Cotswolds Area of Outstanding Natural Beauty (AONB) and Site of Importance for Nature Conservation (SINC), Severn Trent Water decided to lower the impounding level in order to retain the landscape, ecological and fisheries value of the reservoir.

Conversely, where increased water demand indicates a need for more water storage in a given area, but no new sites are available due to strong planning constraints, the option of raising the level of a reservoir may be the least environmentally damaging option. While the existing land-uses to be affected would require the normal rigorous assessment, this option may also give the greatest enhancement of existing nature conservation, landscape, recreational and fisheries resources.

Adverse impacts upon river flows, sediment loads and ecology downstream of the reservoir have often been cited as grounds for objection to schemes. However in the UK the concept of compensation flows, ensuring that riparian owners downstream of a reservoir continue to receive water for their existing uses, has long been recognised. In the nineteenth century such flows, often in excess or their current requirements, were discharged from reservoirs to mill owners downstream. More recently the Department for International Development (DFID) has provided funding to assess the value of making large releases of water from dams to reinstate downstream flooding of wetland resources and associated rural livelihoods. Production of the resulting guidelines forms part of the UK contribution to the World Commission on Dams, set up by the World Bank and the World Conservation Union (IUCN) and due to report in June 2000 (DFID, 1999). Such a catchment-wide approach sets an important example indicating the scale of assessment and mitigation appropriate to addressing the socio-environmental effects of reservoirs.

A STRATEGIC APPROACH TO MITIGATION

The scale and site-specific impacts of environmental change associated with reservoir construction undoubtedly mean that for some schemes, where losses are significant and irreplaceable, the disbenefits outweigh the benefits and the proposals will, and should, be rejected. There are other small proposals where it is possible to provide localised impact-specific mitigation outweighing the losses, and thereby an overall benefit to the environment.

However, where there are no environmental resources of such national or international significance that they should be protected at all costs, a large reservoir scheme can actually create a mechanism for a substantial county, regional, or even national or international environmental benefit. Funding may be released for a strategic review of the particular environmental resources to be affected, both at a wider scale geographically and over a longer period of time than the present-day distribution of the resource in the immediate vicinity of the reservoir. The review would then identify short- and long-term threats, sensitivities, deficiencies and priorities at this scale. Mitigation measures could then be more usefully focused to optimise their value to the conservation of the resource concerned. A number of examples of a more strategic approach to mitigation, taking opportunities for environmental benefit on a wider scale, are outlined below.

A good example of a strategic approach to mitigation was provided, not for a reservoir scheme, but for a proposed theme park granted consent for development upon a degraded portion of the Inner Thames Marshes SSSI in the Outer Thames Estuary. The mitigation package for this development, if it had been implemented, would have released £12 million for improved management of much larger areas of grazing marsh than that to be lost. These were located not only immediately adjacent to the site, but throughout the Thames estuary.

Formulation of a mitigation package for the proposed Broad Oak reservoir, which would have resulted in the loss of 17 hectares of Ancient Woodland included assessment of the distribution of this habitat and the threats to it at a county level. This indicated that fragmentation of once extensive woodland into smaller blocks was not only reducing its area, but threatening the survival of certain rare species, such as the heath fritillary butterfly, which, being relatively immobile, cannot move between blocks and so needs large areas of woodland. The mitigation package for Broad Oak therefore included proposals to purchase additional land (though by present-day standards a rather meagre amount at just over 6 hectares) in strategic locations where it would be possible to reconnect the fragmented blocks of woodland. The benefit would therefore be much greater than that associated with the actual area planted.

In the UK, National and Local Biodiversity Action Plans are well on the way to providing invaluable guidance on the status of the rarer species and habitats, and recommendations for their enhancement and management. Strategic mitigation measures, possibly well away from a proposed reservoir site, may represent the most beneficial compensation for losses resulting from reservoir construction, and may be provided either instead of translocation schemes or in addition to them.

Mitigation for a proposed flood alleviation reservoir in the midlands, which would have a localised impact upon an ecologically important river, is likely to include measures to reduce the amount of nutrient-rich sediment entering the river from arable land. These would comprise a silt-trap and a number of "green dams" comprising belts of willow trees well upstream of the works. The flood alleviation works themselves would adopt the usual measures to prevent siltation of the river; these additional silt-trapping measures therefore represent indirect compensation for localised disturbance of the site.

Similarly, a proposed reservoir site may have cultural heritage resources which are considered by statutory consultees to be of a kind which do not require preservation in situ, but which can be excavated and recorded. In these circumstances consultees may welcome proposals for further investigation of similar resources in other parts of the county, or provision of long term storage and exhibition facilities, as indirect compensation for the resources to be lost. At Rutland Water, the original site of Normanton Church, built in 1826, would have placed it below the level of the proposed water line. It was raised above the water level on a pier of stones, and now houses an exhibition on the works involved (Rutland on line, 2000). Where even more significant archaeological finds come to light during excavations, facilities to exhibit these to the public can not only provide compensation for the impact, but possibly generate revenue to support environmental initiatives.

The CAA is a statutory consultee for any developments within thirteen kilometres of an airport. Reservoirs usually attract large flocks of birds which are a potential birdstrike hazard for aircraft. Whilst human safety is clearly paramount, a strategic study of bird nesting and roosting sites and diurnal migration routes between them over the wider area around a proposed reservoir site can sometimes provide valuable evidence of the role the reservoir is likely to play at this scale. It may be that the bird routes would link the proposed reservoir only with sites in the opposite direction to the airport, and thereby present no more risk than prior to its construction.

In support of the environmental benefits of reservoirs, a historical review of the distribution of large water bodies in the UK would confirm that in the past they were much more abundant than today in many parts of the country, notably the Fens of East Anglia. The fenland meres provided the waterfowl and fish which were the main food-source for thousands of people. Clearly we cannot turn back the clock, and we need to take account of the increase in human population, development of industry and general demand for water. However it could be argued that in many instances the creation of reservoirs marks a return to a more "natural" landscape, a point often overlooked in the development of County Landscape Strategies. Nature conservation and fisheries resources may also be returned to a standard more characteristic of the time of the fenland meres.

This paper does not attempt to address the issue of financial cost-benefit analysis of reservoir schemes. However it must be acknowledged that the regional significance and long term economic benefits of the development of recreational assets are often considerable, and should be taken into account in determining reservoir applications.

Finally, funding agencies and frequently national legislation will now demand that any large overseas schemes necessitating large scale resettlement of human communities ensure that such schemes maximise opportunities to progress regional or national social and economic development plans. Mitigation may include not only re-housing and provision of infrastructure, education, health and welfare facilities, but also appropriate management of change in the context of full awareness of cultural implications and needs.

CONCLUSION

The environmental and social impacts of reservoirs are usually significant, and often extend over considerable areas, time spans and issues. These impacts cannot be played down: they must be identified and evaluated in accordance with the relevant legislation and "best practice". Where the perceived benefits in terms of essential water resource management are considered to outweigh the environmental losses, the most acceptable route for gaining consent is likely to be through a strategic approach to mitigation of these losses. This requires consideration of all the pressures upon the affected environmental resources over a much wider area, and of the opportunities which can be taken up through funding from the reservoir developer. The mitigation package formulated is likely to draw upon a range of different measures in combination, including some of those described above and many others specific to particular locations. If rigorously assessed and appropriately funded, both reservoirs themselves and mitigation for their effects can still make a significant contribution to environmental quality.

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Environmental implications – benefits and dis-benefits of new reservoir projects

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SYNOPSIS. The evaluation of reservoir projects has both suffered and benefitted from environmental objections. There has been over-reaction to these objections, resulting in the stalling of developments needed to satisfy human needs. Environmentalists increasingly appreciate that uneconomic small projects ill-use land, water and financial resources. Promoting authorities realise that reservoirs must be made acceptable to all people affected by them. The process of overcoming problems rather than abandonment of wanted projects will be advanced by developments in technology. Examples are the construction of large reservoirs underground using fast, low cost boring techniques and/or adopting disused mining caverns as reservoirs.

INTRODUCTION.

Environmental objections to the construction of new reservoir projects have gained exponentially in strength and momentum since the 1950s, largely because it became widely realised that projects were being built without sufficient understanding of their long term effects on the environment and because local inhabitants were unable to avoid losing their homes and livelihoods, without acceptable provisions for their future. In the 1970s, governments and development authorities increasingly required that the environmental impacts of proposed projects should be assessed and that the assessments should be considered by the authorities and if necessary should receive governmental approval before the projects could proceed. As a result of the controversy over the impacts and benefits arising from dam projects, the World Commission on Dams (WCD) was set up to resolve these issues, with goals including the assessment of alternatives, the preparation of a framework for decision making and the development of internationally acceptable criteria, guidelines and standards. Twelve Commissioners were selected to represent and assess fairly all sides of the issues. Substantial funding has been provided but effectively only two years have been available for studies and deliberations before producing balanced reports, which are due by June 2000, on these vast subjects (Bridle, 1999). The reports and recommendations will be advisory and not legally binding. Regrettably it cannot be expected that the findings will be acceptable to many protagonists because feelings have run high, beyond the limits of

Dams 2000, Thomas Telford, London, 2000.

balance and reason, into the realms of emotion. It is hoped, however, that the facts available will be tabled and the issues will be clarified, enabling a more enlightened and universally acceptable approach to reservoir development in future. The Authors believe that the time for this is ripe, because of evidence which will be summarised, of the growing appreciation of the benefits of reservoir projects and the realisation that objections to them can be overcome with knowledge, good will, ability and finance. Perhaps the foremost example of a project in operation which illustrates the tortuous route to mature assessments is the Aswan High Dam. The occurrence of some extremes of nature such as seismicity cannot at present generally be predicted in advance, while others can, with some reliability (for example, tides). Thus it should be said that this paper for 'Dams 2000' is not looking ahead 100 years, much less 1000 but takes and attempts to illustrate an overview of the present trends which seem to be in evidence. Despite these limitations, in this context it can be said that the impacts of a dam and reservoir extend beyond the feasible life of the project to the feasibility of its renewal or abandonment and/or its replacement. So far in the future, the present cost of this may be of little economic consequence and unforeseen technological, social and political developments before then may make speculation unreal but when they occur, the impacts on the environment may be major.

ENVIRONMENTAL TRENDS

The Aswan High Dam

This is one of the large dam projects the development of which has been widely criticised in recent times. The dam, in Egypt, was completed in 1968, to Russian designs after proposals for funding by Western countries had been withheld, which had led to the Suez crisis. Major objections, both well argued and emotive, were to the potential inundation of temples on the Nile, to deprivation of flood plains below the dam of fertile silt deposited during annual floods, to the disturbance and displacement of people including nomads, to the potential causation of earthquakes and to the decimation of the sardine population of the south eastern Mediterranean etc.. A total of some 100,000 Nubians were displaced, almost equally from Egypt and the Sudan, the Egyptian Government attempting to improve their social, cultural and political lives when resettled, as summerised by Faed (1993). Despite the early formidable objections, Gasser and El-Gamal (1994) reported that the project has made a tremendous contribution to the economic and social development of Egypt. It has protected Egypt from high floods during the seventies and saved Egypt from devastating droughts in the eighties and averted a disastrous famine in both the seventies and eighties. The dam has affected the regime of the Nile, resulting in a lowering of both water and bed levels. As a result, Esna Barrage has been replaced, a new navigation lock at Naga Hammadi Barrage has been built and many navigation bottlenecks at bridges crossing the river, and along the river course are being solved. Thus it can now be said that adverse environmental impacts have been dealt with or survived and that the project has been justified and needed for its beneficial impacts. Had the development been stopped, human suffering could not have been avoided and economic and social development would have been sacrificed (Cotillon, 1993). It must be said that the high Aswan dam still poses major environmental impacts such as loss of delta, loss of nutrition to fish, high grounwater levels, disease and ground salination.

One lesson appears to be that, in the past, protests have been not only successful but necessary in achieving some of the objectives of those protesting, while the protesters have not been called upon to contribute to the potentially heavy cost of delays in implementation of the project and of delays in creation of the benefits which their protests cause. A current example of this is the Sardar Sarovar Dam Project (SSDP).

Sardar Sarovar Dam Project (SSDP)

The SSDP is a multipurpose (irrigation and power) project on Narmada river initiated by the State Government of Gujarat, India, in October 1988 through a State owned company known as the Sardar Sarovar Project. The project was designed to produce 2450 MW of power from two power stations which were to have been commissioned by 1995. However, the project suffered considerable delays due to environmental, ecological, social and other related issues including legal wrangles. Moreover, the financial assistance originally granted by the World Bank and other funding agencies was withdrawn as a result of an independent review commissioned by the World Bank on the Sardar Sarovar Projects in 1991 (Morse and Burger 1992). Although the review report dealt with the highly controversial aspects of resettlement and rehabilitation of displaced persons and the environmental impact assessment of the projects, the authors believe that the major factor which influenced the funding organisations' decision was the conflict between the so called environmental pressure green groups and the Government of India. Undeterred by the decision of the funding agencies, the Government of India proceeded to continue the project construction with its own resources in the interest of the development of the country in spite of fierce agitation by a section of environmentalists opposing the project on issues related to ecology and resettlement of displaced persons. After prolonged agitation by a peoples movement known as Narmada Bachao Andolan (NBA)- meaning Save Narmada Agitation, the dispute went to the Supreme Court of India in April 1995. In February 1999, the Supreme Court ruled that the dam construction be continued and the court also directed a panel headed by a retired judge to review the progress of implementation of the resettlement package.

The design height of the dam is 146.5m but its present height stands at 80.30m. The 1,250 MW (5x250MW units) canal head powerhouse has been completed in the meantime but it cannot be commissioned until the main dam is raised up to a height of at least 100.63m for the purpose of power production. The 1,200 MW river head powerhouse had a setback due to a dispute in 1992 between the project company and Japan's Sumitomo which was to supply the six 200MW generating units. The dispute has been resolved and the six units will be supplied to enable the powerhouse to commence producing power from December 2002. The construction of SSDP started ten years ago and so far nearly 12 billion US dollars has been spent on the project and cost is escalating day by day due to the delays caused by environmental issues and obstructive tactics of oustees backed by Non-Governmental Organisations (NGOs). The authors have no doubt that NBA leaders and controversial authors on the subject of Narmada dam realise the consequences of their actions when they switch on their modern household gadgets in their comfortable homes.

India has a massive hydro potential of about 84,000 MW and hardly a quarter of its potential has been harnessed so far. India also has a massive population of about a billion mouths to be fed in addition to providing the comfortable lifestyle of modern living. The major issues confronting the hydro development in India are those of Environmental Impact Assessment (EIA) of projects and resettlement of displaced persons. The EIA aspect of projects is a cause for concern in most developing countries, including India. However, the authors strongly believe that part of the estimated cost of a project should be that of providing, for any persons to be displaced, a living and lifestyle which is at least equal to that which they are to lose, as well as a margin to compensate the disturbance to their lives and for their co-operation in change. To this end the authors were pleased to note that the Government of India has taken steps to deal with the problems preventing the development of multi-purpose resevoirs by implementing public hearings on proposed projects, appeals procedures to resolve conflicts related to approved projects and preparation of guideline manuals for EIA of projects based on experience gained by other countries together with effective legislations dealing with resettlement and rehabilitation procedures for hydro development in the country. The authors fervently hope that these measures will resolve at least some of the environmental issues and encourage resettlement of oustees so that a positive and constructive contribution can emerge from the affected persons, NGOs and other environmental green groups for the betterment of the country.

Three Gorges dam

At the time of writing, construction of the Three Gorges dam is proceeding and the first of 26,700MW units are programmed to be commissioned in 2003. Monumental environmental impacts and funding restraints are being tackled by China, in the face of international doubts. China plans to increase its hydroelectric power capacity from 70,000MW in 2000 to 125,000MW by 2010, surely more than a sign that in that vast country the value of hydropower is deeply appreciated. The Authors have no aspirations to probe any aspect of Three Gorges here in depth, but feel that it is appropriate to wonder if that dam will follow an analogous path to that of the Aswan High dam from abuse to heartfelt praise.

ALTERNATIVES TO LARGE RESERVOIRS

It has been widely propounded that 'small is beautiful' and that therefore 'micro', 'mini' and 'small' hydropower projects have environmental advantages over projects of economic optimal sizes. The common sense conclusion is that there are viable roles for hydropower projects of all feasible sizes, depending on the need and the demand (which govern load factors) and on tailoring the means of generation to the site. However, it is being increasingly realised that small reservoirs cannot provide the regulation and efficient use of water resources which can be obtained from large ones. To achieve the same output as one large reservoir, a complex of smaller reservoirs would be required, covering an aggregated area of land much greater than that of the equivalent large reservoir and needing much higher run-off. With regard to flood control, small reservoirs have little effect in attenuating extreme floods and, therefore, increasing the number of small reservoirs in series in a valley gives little relief from large floods.

In 1980, the World Bank found that the relatively high investment costs may make mini-hydro projects (under 1MW) uneconomical for village systems with low load factors. This conclusion was supported by a subsequent assessment sponsored by the British Overseas Development Administration, of 15 sites in 6 different Indonesian islands, with capacities up to 2.5MW. Six of the sites justified feasibility studies and of these, four were found to be economically viable. Of these, two have been recently completed with British Aid. In general, larger schemes do have the economic benefits which arise from scale and it is being realised that the environmental advantages of schemes smaller than 1MW can only be gained at the cost of finance which could have been used for other desirable benefits, such as health and education. Having said this, it has been estimated that an installed capacity of 421.7MW of small hydro power in Italy (with average installed capacities of less than 500kW) could save almost 3 tons of CO_2 emissions per kW per year compared with conventional fossil fuelled power plants and reduce other pollutants as well. In Portugal, 323.1MW (with average installed capacities of about 3MW) was estimated to be capable of saving almost 2.4 tonnes of CO_2 emissions per kW per year, so the potential savings in emissions by developing even small hydropower are impressive. Large schemes have their advantages over small ones but are not always feasible.

Should the possibilities of improved management and efficiency and the avoidance of wastage of water become exhausted, the main alternatives to building more large reservoirs for water supply are groundwater development and/or desalination of salt water, the former being limited and with environmental objections, and the latter being uneconomic except in situations of severe water shortage. Power and energy can alternatively be obtained by burning fuel to power turbines or from nuclear power or from renewable sources including solar, wind, wave and tides. There are serious environmental objections to all these alternatives, except perhaps to wave energy, for which, however, no widely feasible design has yet been developed.

Increase in agricultural food production, for which the demand for well regulated water supplies is much greater than for other purposes, can be achieved to some extent by use and development of higher yielding crop varieties and improved practices, including drainage of land and the use of fertilisers. The proportion of rain-fed crops can be increased where rainfall permits, but at the expense of utilising a greater area of land compared with double cropping under irrigation, to give the same yield. Water for irrigation can be obtained direct from rivers or from groundwater but the regulation of supplies to be available when needed throughout the year is sometimes only accomplished by the operation of large reservoirs.

The effects of developments in technology on environmental impacts

As indicated in the previous sections, the environmental pressure groups in the world now form a very strong lobby against further development of hydropower projects. Unfortunately, the actual and potential violations of the environment by hydropower projects have led to the build up of antagonism and suspicion by those seriously concerned about risks to the environment and social welfare of displaced people that hydropower project proposals conceal adverse effects. Under these circumstances, the authors believe that there are extensive opportunities to develop hydropower schemes from disused mines in the world without either causing damage to the environment or to the social welfare of humanity thus avoiding public outcry related to the development of conventional hydropower schemes.

Since about 1968, when the principles of underground pumped-storage were first expounded, the concept has been considerably developed and extended. The potential for use of disused or abandoned mines for pumped-storage has been recognised. The economic attractions of the underground concept are greatly enhanced by the utilization of an existing mine to house the lower reservoir, powerhouse and other associated equipment. There are considerable benefits to be had from a pumped-storage power plant, particularly for large integrated systems such as that in the UK. Pumped-storage facilities improve security and reponse to peak demand and fluctuations in load. Moreover, underground pumped-storage schemes place insignificant demand on existing water supplies and cause only modest interference with the environment.

The pumped-storage concept originated in the early part of the 20^{th} century. The early plants generated under heads greater than 150m and used two or more pumping stages. Then, as now, the economics of pumped-storage favoured high head developments. The development of the single stage reversible pump turbine (SSRPT) is the most important in the history of pumped-storage. There are now prospects for developing single stage machines for heads up to about 900m and higher. There is no doubt that these developments will provide a unique opportunity in the future to develop typical sites for high-head pumped-storage plants in disused or abandoned mines where upper and lower underground reservoirs could be located close to each other, but with a considerable height difference between them. Deep mines are the prime resource for underground pumped–storage mainly because of long vertical drops available for exploitation. Multiple working levels aid underground pumped-storage by allowing more choices of hydraulic head. Mines with regular working intervals offer more specific reservoir alternatives than conventional surface to surface or surface to underground reservoir sites. The optimum working levels can be developed to match the output required of the scheme and the demands upon the power system. Existing tunnels, vertical shafts and workings can reduce capital cost requirements of a utility scale pumped-storage scheme down to one half of the total cost of a comparable surface pumped-storage project. The savings come through reduced excavation cost and hence reduced requirement for funds used for construction, less time from project design to on-line operation and reduced environmental impact mitigation costs. However, the savings would depend upon the scale of modifications required within a specific mine and on the suitablity of geological conditions.

Generally all deep underground mines will have at least two air shafts into the ore body, one for intake of air supplies and ore removal, and the other for return air. Larger mines will often have three to five shafts for air and services which could be used for the water conveyance system for pumped-storage. Caverns can be used to house hydroelectric machinery and associated equipment and they can be interconnected by a system of access tunnels leading to the shafts.

The configuration of the lower reservoir is subject to a number of constraints, the most significant being geological, hydraulic and economic. The lower reservoir space which is not under high pressures can be achieved either by a system of tunnels or caverns in the case of limestone, slate or other similar mines, where excavated space can be substantial in size. The required volume of the lower reservoir will vary in relation to the developed head, installed capacity and planned generating cycle of the power plant. Many possibilities exist for the upper reservoir to be located within the upper layers of the mine. This might have aesthetic advantages for a pumped-storage scheme, with similar features to those of the lower reservoir. Alternatively, the upper reservoir could be located above ground level. If an artificial reservoir were necessary, a relatively impermeable basin could be selected with a suitable site for a dam or surrounding water-retaining embankments. The possibility of siting the upper reservoir in quarry workings could be considered if such workings are very close to the disused or abandoned mines.

The principles of the layout of the powerhouse and the transformer gallery, together with the necessary adits and galleries etc., do not significantly differ from those of an ordinary underground hydro-electric power station. However, it would be an advantage if as much of these cavern spaces as possible were incorporated within mine space. A compromise, with respect to economics, will be needed when considering the modifications of existing tunnels and the excavation of new caverns to accommodate the necessary equipment. Switching station and other ancillary works are not likely to differ very much from those of an underground hydo-electric power station of corresponding capacity. The cost of roads and buildings for the operating staff may be considerably less due to utilisation of those built for mining operations.

Emphasis has been placed here on the merits of utilising disused mine workings but the development of improved low cost, rapid excavation techniques by rotary boring will have increasing importance in providing facilities to create large underground caverns at economic cost, sited to take the best advantage of geological features and project design requirements.

ARE ENVIRONMENTAL OBJECTIONS TO HYDROPOWER BEING OVERCOME ?

Recent developments show that the high feelings initially aroused by environmental objections to hydropower and by mistakes and oversights in promoting reservoir projects to which entire projects have succumbed, have reached a stage of rational settlement. An example is the report in Water Power & Dam Construction (March 1999) that the Chilean utility Endesa has reached agreement with Chile's Conadi, the commission administering the indigenous people, on the Ralco hydro power project. The 570MW project comprises a 155m high dam on the Bio Bio river which would flood 5,200 km^2 of land currently occupied by about 500 Pehuenche Indians. Chile's environment agency Conama approved the project in June 1997 but the project was in limbo until agreement was reached with Conadi on the relocation of the Pehuenche. In recent years such agreements would have seemed impossible to achieve. Even in this case, the agreement was being disputed by three individual Pehuenche women. Conveying a similar trend, Water Power & Dam Construction (June 1999) reported that the United States District Court for the District of Idaho approved a settlement between the Nez Perce tribe and the Avista Corporation (formerly Washington Water Power). The Clearwater river runs through the Nez Perce reservation of approximately 750,000 acres of land in North Central Idaho. The tribe claimed that two dams had blocked fish passage in violation of the tribe's treaty fishing rights since 1855. The tribe had also filed a suit against Idaho Power Company involving three dams on the Snake river, in which case, too, the parties were able to arrive at a settlement. (Nevertheless the controversy on the future of dams on the Snake river in relation to the passage of salmon was still escalating at the time of writing with two Presidential candidates speaking against the breaching of dams). In the same issue of the same journal, it was reported that Japanese funding for the San Roque project on the Agno river in Northern Luzon in the Philippines had been postponed because local tribespeople's approval is a prerequisite to the US$1.19B 345MW project. Subsequently it has been reported that an independent panel assessing the impacts of the project has found deficiencies in the environmental impact assessment, which points to the importance of making correct and complete project assessments. Thus, although it appears that reason increasingly prevails, environmental settlements continue to be beset by persistent objectors.

THE CURRENT & FORESEEABLE FUTURE EVALUATION OF NEW RESERVOIR PROJECTS

General

Traditional methods of power generation from fossil fuels have created a legacy of acid rain, oil spills and despite billions of pounds spent on research, serious anxieties still exist relating to the disposal of nuclear waste. All of such issues need to be tackled. Most serious of all is the contribution to climate change due to greenhouse effects as a result of releasing carbon dioxide and other gases into the atmosphere. It is needless to stress that one of the greatest environmental threats facing the world today is global warming. Despite all the warnings and all the evidence, business continues as usual for many energy companies in the world to invest in power stations that burn fossil fuels and discharge millions of tonnes of carbon dioxide emissions. Under these circumstances, it is increasingly important to promote energy production from renewable resources which will not deplete the earth's resources or pollute the environment. Renewable energy is the energy of the future and therefore it is reasonable to conclude that the development of reservoirs of all sizes will continue - large reservoirs at an increasing rate as the environmental restraints imposed over recent years are reviewed. Impressive plans are in the pipeline, including, for example, current extensive hydropower development in Turkey.

Impact assessments

Much has been written on the environmental impacts of reservoirs and there is nothing to be gained by repetition here in the space available. A helpful review has been provided by Binnie (1999). Eighteen aspects likely to merit very careful consideration have been outlined by Edward Gosschalk (1995-1999). The Authors wish here to say only in the context of environmental implications of reservoirs that the impact which is often the most objectionable is the displacement of people from their homes by inundation by the reservoir or by construction of temporary works and infrastructure. There must therefore be sufficient provision for their future, in the form of a living and a lifestyle at least equal to that which they are expected to lose. Such persons may need new infrastructure and training for a new life. There must indeed be a margin in their favour, to compensate them for the disturbance to their lives and for co-operation in change, even if this is not justified by conventional theories of economics.

The need for a collaborative approach. Environmental assessments, just like relicensing in the United States (LaBolle,1999), require a collaborative process: to avoid the time loss and cost of disputes, more and more often opposing views will be heard and discussed and agreements will be reached in the form of discussions and negotiations starting at the stage of identification of the problems.

The training and purposes of engineers are to seek to improve on what has already been achieved. The assessment of the environmental implications of their work on reservoirs should always be subject to impact assessments of a satisfactory standard by suitably qualified scientists and specialists (including, religious specialists ? Most engineers are creationists and do not believe that evolution is entirely within the realms of science ?). However, in the end reservoirs must be judged to be beneficial overall to human purposes.

ICOLD guidelines recommended. It is desirable to carry out a preliminary environmental impact assessment at a very early stage, (during conceptual planning or prefeasibility) in order to bring the principal issues into focus and to gain a measure of the studies needed and provisions which may be necessary. When dealing with reservoir projects involving 'large' dams, it is always worthwhile to refer to relevant ICOLD bulletins, because they cover a wide variety of subjects in some detail. They are prepared by engineers of international standing in their subjects and carry an encouraging degree of international acceptance, through their approval by representatives of the member countries of ICOLD. The Authors believe that EIAs should increasingly follow the guidelines of ICOLD Bulletin No. 35, (1980). It comprises four parts, of which the last is a matrix which is intended to provide a means of listing and evaluating, even if only in qualitative or relative terms, the impact of individual dams and related construction work, on specific parts of the environment.

Columns in the matrix deal with the characteristics of actions involved in the possible impacts, with distinctions between the use for which water is destined, the type of action, the zone concerned, physical corrective action and institutional action. The rows deal with the effects on the economic, social, geophysical, hydrological, climatic and biotic environment.

Each impact is evaluated with the symbols provided, which introduce the concepts of relative importance, degree of certainty, duration and delayed effects. The completed matrix must always be accompanied by a written commentary, explaining and justifying the user's interpretations.

This Bulletin No. 35 gives an introduction and instructions for use of the matrix with examples. Under the heading ' General synthesis', it gives descriptions and explanations of the physical and biological effects of river development, and social effects. (Other ICOLD Bulletins come under the heading 'Dams and Environment', notably Nos. 37, 50, 65 and 86, 1981, 1985, 1988 & 1992 respectively).

The costing of impacts. The quantitative costing of impacts is an exercise swimming in uncertainty because many of the impacts depend on probabilities and are indirect social costs. The Authors believe that an objective should be to make provisions sufficient to ensure that extreme events such as droughts, floods, seismicity, failure of constructed works etc., do not result in more unfavourable impacts than could have occurred in nature had the reservoir not been constructed. Judgmental qualitative assessments will be required from authorities for the approval of financing, to which these authorities may not be accustomed. They should therefore be given every possible assistance in the form of qualitative appraisals. However cost estimates to determine 'the present value' of favourable and unfavourable impacts must be made on a probability basis as data input into the assessment of the economic feasibility of the reservoir. An axiom has been mooted that it would not be acceptable on moral grounds to put a monetary value on human life in calculations of damage. This principle would presumably apply to valuation of lives saved as well as to lives lost. In many cases the costing of provision and operation of emergency plans can in theory avoid the loss of lives due to extreme events but does not eliminate the need to value the benefit of lives saved. For economic evaluation the assistance of insurance companies may be enlisted for this.

CONCLUSIONS

The development of large reservoirs will continue with growing realisation of the importance and economic necessity of their benefits in the interest of people and the realisation that the objections to them can be overcome with advancement in technology. The economic attractions of the concept of underground pumped-storage can enhance massive beneficial effects without serious environmental objections.

The Authors believe that proper implementation of projects with objective assessments and realistic measures to resolve serious issues relating to environental impacts and resettlement of oustees will contribute to the emergence of positive and constructive responses from affected persons, organisations and other groups of objectors to the development of reservoir projects. It is fervently hoped that the recommendations of WCD on decision making criteria, guidelines and standards will enable a more enlightened and environmentally acceptable universal approach to reservoir development in the 21st century.

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The potential for future dam construction in the United Kingdom: appreciating the benefits and accommodating the impacts

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SYNOPSIS. Within recent decades an ever growing number of well informed, responsible, politically astute and vocal environmental pressure groups have become established. As a direct consequence, any major infrastructure project attracts intense, and more commonly negative, interest from the very earliest stage of inception. This is particularly evident in the case of dam development projects where the construction impacts are protracted and the final benefits often seen to be largely restricted to future generations. There is an urgent need to reconcile the aspirations of promoters and detractors if both environmental and public interests are to be assured responsibly. The benefits of dams are reviewed, associated environmental impacts considered and possible areas for compromise or further study are suggested.

INTRODUCTION

With the passage of time successful promotion of any new dam development project in the United Kingdom is becoming ever more difficult. Undoubtedly the importance attached to the accepted needs to conserve water resources, curtail wastage and manage demand each play their respective part in fuelling resistance. Good government certainly requires that due attention be given to these factors. However, when a residual shortfall in resources still remains there often appears to be an inherent preference for resource options other than dam development. Such decisions are often supported by claims that the alternatives represent lower environmental impact. The extensive number of geographically dispersed impacts associated with manufacturing the many elements comprising these alternative processes and securing energy to drive them is not always evaluated in great depth. Subsequent plant replacement considerations, and further similar environmental costs, need to be fully recognised. This is particularly so given that most of these process options tend to have a much shorter operational life than a reservoir based scheme.

Dams 2000, Thomas Telford, London, 2000.

A reason for preferring shorter life options is the uncertainty associated with reliably projecting water demand in the long term (many decades in the case of dam development). Each dam promoter may have differing conceptions of future water requirements applicable to the specific geographical, demographic and socio-economic characteristics of the area for which he has a statutory responsibility to maintain supplies. The particular capital development strategy will also be driven by wider commercial factors, but the general obligation to maintain a regulated level of supply continuity is incumbent on him. Water demand in the short term may be projected with considerable confidence and in the medium within tolerable limits. Such is far from the case for a design horizon ensuring the timely availability of a new reservoir 15 to 25 years after a commitment to promote. The reported implications of global warming, potential changes in demography, rising social expectations, and the desire to improve river environments may ultimately oblige us to make such projections notwithstanding the attendant uncertainty. Future interruptions to water supply, resulting from more extreme climatic events than currently provided for, are likely to become ever less acceptable to the public. Future inability to maintain the regulated level of supply continuity will have both an adverse commercial impact on the utility and result in a loss of public credibility. The likely need to re-model, renovate, reconstruct or replace existing water retention structures in the future will only exacerbate the risk situation. When major dam development works become necessary, and not if they will be needed, may prove the key factor.

For a dam promotion to prove successful in the future, there will be a need to reconcile the key areas of conflict between the promoters of such projects and those resisting the development and to restore balance in their respective arguments. Significant compromise by all parties will undoubtedly be necessary if the best interests of both the environment and the well-being of future generations are to be assured in similar measure. The issues presented hereafter are directed primarily towards embankment dam development in the United Kingdom although specific aspects may also be of an international relevance. The issues addressed reflect the marked changes in public reaction to the prospect of any extensive engineering development which have taken place since the last major reservoir was developed. This has influenced the planning process such that the submission of Environmental Impact Statements are an ever more frequent requirement. Many of the comments which follow will be self-evident to members of the dams fraternity. They are nevertheless included to provide a comprehensive coverage of the full scope of issues pertinent to dam promotion in a non-engineering environment.

BENEFITS OF DAM-CONSTRUCTION

Catchment Regulation

Impoundments and, to a lesser extent, bank-side storage reservoirs, provide valuable environmental enhancement through the attenuation of flood flows when the feed waters are in spate. Many dams are constructed specifically to achieve the purposes of protecting life and property, supporting navigable waterways or to feed irrigation schemes. The potential contribution to flood control should however be a consideration in the design of any dam-based scheme regardless of the principal objective and be reflected in the determination of reservoir volume, operational practices and, possibly, intake arrangements. Available dam freeboard and therefore the operating control rules will clearly influence the extent of any beneficial regulation which can be provided at a specific time. Any final design proposals should be fully integrated with the river management objectives of the responsible agencies. For off-river structures the capacity of the abstraction works is clearly the principal factor governing the level of control which will be achievable. Where an intake is sited local to such a dam and probably below the normal operating reservoir water level the capacity of the abstraction pumps will be the limiting factor for any flood attenuation. On the other hand adequate design capacity of the discharge conduits may allow an appreciable release rate to impact favourably on river flows at times of minimal natural run-off.

The storage volumes provided for many existing schemes include allowances for compensation releases, often from designated impoundments provided solely for this purpose. The more a catchment is over-reservoired with respect to supporting its prime purpose then increasingly severe drought conditions may be ameliorated. Such a situation effectively arises in the early life of any new reservoir development. The long service life of a dam demands the selection of a design horizon well into the future, if the environmental and social impacts accompanying its construction are to be justifiably incurred. Consequently there exists, in the initial decades of its life, a substantial surfeit of capacity over that needed to support the designed yield. Prudent management of this "excess" capacity affords the facility for conjunctive use of the storage to provide support in maintaining an enhanced downstream river environment during periods of both flood and low natural low flow.

River Water Quality Management

At times of low run-off the middle and lower reaches of many rivers through urbanised areas experience marked increases in the proportion of flow attributable to waste returns. Although these contributing discharges may fully comply with the prevailing effluent quality regulations their resulting impact on the receiving waters inevitably increases with decrease in the level of dilution. In the extreme, river flows may be almost wholly of re-cycled water. Any facility to augment the natural river flow with water drawn from a source of higher quality, be it chemical, physical or biological, can only serve to improve overall quality through dilution, quicker re-oxygenation of inflows and enhanced velocities of flow. The latter may benefit navigation by maintaining sediment carrying capacity.

Sustaining Navigation

Augmentation of river flow above a minimum desirable level helps maintain the continued use of the waterway for leisure activities, both on the water and along its margins. The former will be of particular importance where the watercourse represents an otherwise navigable link between one or more components of a canal system. Conversely any possible attenuation of flood flows within set limits will also be of benefit. Actual benefits achievable by regulation will be largely site specific and catchment dependent but will, nevertheless, facilitate extended periods of leisure use at best and improved quality of such usage at worst.

Availability of Strategic Storage

The retention of large quantities of raw water, which can be released under gravity on demand, provides a valuable contingency reserve which may be deployed to satisfy a range of emergency and strategic needs. Of principal interest to the undertaker may be instances of the unforeseen pollution of an alternative source of raw water. Appropriate integration of the supply infrastructure will facilitate the use of any surfeit of storage during the early life a dam to mitigate such a loss. During this preliminary period of a dam's life more severe drought events can be tolerated than ultimately anticipated. Even when the design horizon has been attained, the conjunctive operation of the structure can yield benefits in excess of those accruing from its independent management. The sizing of reservoirs should consequently reflect the contingency demands of emergencies experienced at non-related source works.

There are indications that the strategic importance of raw water reservoirs in a National emergency is becoming more recognised. The survival, largely unscathed, of Thames Water's Yuvacik dam in Izmit, Turkey, played a crucial role in supporting emergency recovery activities following the earthquakes in the locality last year. The resilience of competently designed, constructed and supervised embankment dams under such severe conditions was patently demonstrated. The probable intensity of seismic events in the

UK pale into insignificance in comparison but other emergency scenarios may be developed where the availability of water in quantity under gravity could prove invaluable in managing disaster recovery. The fact that the water would be untreated would not detract from the potential benefit for purposes such as fire-fighting or supplying emergency package treatment plants. The facility to transfer stored water via local water courses or strategic transmission apparatus could become equally critical. A major international conflict could give rise to a need equal to that resulting from a natural phenomenon.

In- Bank Water Quality Control

Most dams will be of sufficient capacity to retain water for extended periods and thereby provide a number of advantageous quality related functions. A large receiving basin may be managed not only to facilitate selective seasonal draw-off but also the depth below water surface from which this outflow is taken. The scope for varying each depends on the status of reservoir water level, demand on the resource, etc. but there remains a degree of freedom to operate the off-take to optimise the quality of the water abstracted. For an off-river pumped storage scheme, the quality of inflow may additionally be managed to ensure that abstractions are timed to coincide with the most favourable river water conditions. Once retained, natural biological changes occur to the organic matter in the raw water, with settlement of any resulting debris and inert particulate matter. This is further complemented when it is used as a blending facility for waters received from two or more separate sources having differing bio-chemical characteristics. When prudently operated a reservoir can consequently provide a valuable water quality enhancement function.

Energy Capture

All dams have in common the attribute of retaining potential energy. Some are developed purely to serve this function, as in pumped storage schemes, to accommodate peak loads on the electricity grid. Others are constructed to intercept catchment run-off for the sole purpose of harnessing its energy for hydro-power. Still others are developed as part of water supply schemes where the primary function is to regulate the seasonal hydrological cycle in keeping with the essentially uniform demand pattern. In contrast dams for irrigation schemes balance the hydrological and agricultural cycles whilst flood protection structures attenuate river flows within desirable limits. Although elevations and geographical locations are selected to optimise their specific functions, and these vary greatly, energy is invariably harnessed. Where the purpose is unrelated to the direct provision of power the potential head may alternatively be utilised for forward conveyance of the resource or to drive pressure/gravity treatment processes. The low head differentials at some river weirs are now frequently being harnessed using high-flow/low-head generation installations. In the absence of this hydraulic potential the energy would need to be generated by other means commonly involving the exploitation of non-replaceable fuels such as coal or oil. In contrast, the hydraulic energy is largely sustainable through seasonal replenishment of water stocks and is not in itself polluting in any way.

It is perhaps pertinent to note that for each 100Ml/d of water released at an altitude 50m lower than the reservoir level approximately 1MW of energy is available. This is equivalent to the burning of 3000t/annum of coal (or 2000t/annum of fuel oil) for generating a similar quantity of electrical energy at a conventional power station. This inherent source of energy is sustainable for the life of the reservoir asset. The contribution to energy conservation represented by the gross British dam volume of $8x10^6$Ml, and hence protection of the environment, is significant. Good management of the environment implies the exploitation of sustainable sources of energy in all but the most patently uneconomic circumstances, or where other environmental considerations should take precedence. This is particularly so in cases where least environmental impact will be experienced as measured over the whole life of the project. Wherever practicable sustainable energy should be substituted for that available from less desirable generation options, and alternative energy sources should be environmentally weighted in project appraisals. Such weighting should reflect environmental impacts local to the source of generation, at the site that the fuel is won, in its transportation to the power station and in the transmission of the power to point of demand.

Provision of Amenities

The value of our existing dams to the growth in water-based leisure activities is ever more visible as access restrictions are gradually relaxed. In the specific case of reservoirs dedicated principally to potable water supply, the increasing freedom of access is due in the main to enhancements in treatment processes. Many are now managed and stocked to support fishing, whilst these and others provide opportunities for sailing and canoeing in designated areas. The public's affinity with large bodies of water may be innate, but whatever the reason many people are attracted to them simply as a visual amenity. This is nowhere more evident than at Carsington Water where, for some, access for simple relaxation appears to be benefit enough. Rambling, cycling and picnicking within visual contact of the water appears to be of equal attraction where facilities are provided. Hides and similar viewing facilities have been provided for ornithologists at some sites and these attract both the specialist and lay-person in similar numbers suggesting, perhaps, an educational value. Habitats with restricted access may be created for encouraging the breeding of many species of fauna both common and in danger. A number of reservoirs have become sufficiently successful in attracting and retaining wildlife and/or rare flora that they are now designated as SSSI's.

Educational Platform

Combining, as they do, the aquatic, waterside and terrestrial environments, reservoirs are uniquely placed to provide an ideal educational setting for a wide range of natural history and engineering topics. The on-site leisure and sporting facilities noted earlier can represent useful potential attractions to mitigate the perceived mundane aspects of the learning activity. An astute blend of educational curriculum, out-door interests and presentation facilities may benefit environmental, conservation and safety awareness in the attendee whilst constituting a valuable interface with the reservoir undertaker.

Risks and Cost Factors

The combination of longevity and high capital cost of dams presents unusual difficulties to a balanced cost/benefit analysis in the conventional planning environment. Unusual levels of boldness and commitment are prerequisite if substantial financial investment is to be entered into which will principally benefit future generations. Conventional techniques for forecasting future needs are largely academic given the extended periods involved. The large contributions to deployable output, which such reservoirs may yield, limits the relevance of any determination of the date of initial need only to the decision to proceed. Commissioning of a major reservoir is likely to be at least 15 years thereafter and its half-life some 50 more years later. It is inappropriate to assign confidence levels to discount rates, demographic changes and demand projections for a design horizon some 75 years into the future. To suggest that it might be otherwise is at best misleading and at worst foolhardy. The rigid application of financial analysis techniques which are the norm today would have precluded the development of many of the existing dams on which much public reliance is now placed. Value judgements would appear more appropriate based on likely ultimate need and not precisely when this will come about.

Following the availability of a new reservoir, a large surfeit of deployable output will be available for many years. This constitutes an invaluable contingency against many risks, which would otherwise assume much greater significance, by providing considerable flexibility in resource management. Some have been specifically noted earlier but less tangible ones remain. The latter decades of the 20th Century saw major revisions to public infrastructure, not least the power and water sectors. The trend of change is unlikely to cease and may be mirrored in industry. Similar major changes in the agricultural sector are equally possible within the coming century, especially if the forecasts of global warming prove to be well founded. Such events would reflect the levels of change which have occurred since our predecessors commissioned the construction of dams 100 years ago and which have made the then unforeseen changes sustainable. Hopefully we accept a similar responsibility for good governance in the interests of the well-being of our successors.

ENVIRONMENTAL IMPACTS OF DAM CONSTRUCTION

The Context of Environmental Change

It is perhaps paradoxical that, at a time when we are continually being exhorted to embrace change, the environmental protection principle has tended more towards the view that any potential change in flora, fauna or visual impact is abhorrent. Moreover, assessments of change impacts are commonly now made based on ever decreasing evaluation periods. This reflects the modern attraction for short termism in planning but ignores the potential for natural environmental recovery and maturation in the medium to long term. Impacts are now commonly measured on the basis of effects over periods of just one generation. Such a time-frame is inconsistent with that required for the change processes commonly encountered in nature.

A meaningful evaluation of the "permanent" impacts requires a much more holistic approach and recognition that any initial impacts will themselves transform over time as subsequent environmental processes gain the upper hand and nature exerts her authority. It has to be remembered that many existing water storage structures which, if promoted today, would be vigorously resisted are now classified as SSSI's. Others, for which development consent was obtained despite considerable opposition, have subsequently attained sufficient environmental and amenity appeal that any suggestion that they be removed would cause a furore. With or without man's intervention, environmental change is irrevocable and will continue to be so. Much of the present UK fauna and flora are themselves earlier imports. These include the dog and cat, the useful quadrupeds of horse, cow, sheep and hog, whilst most of our fruit, vegetables, flowers and all poultry are originally of Asiatic origin. The concept of environmental change should not be the nub of any conflict but rather that any change overtly caused by man should carry an obligation to ensure that it is effected sympathetically.

Impact on Topography

The existing surface landscape is also largely a direct product of man's intervention over the millennia. This influence pervades over all but the highest of our hills and on many of these as well. Examples include development of the early bronze age mines of Great Orme in the 2nd millennium BC and contemporary field systems at Throwleigh Common, Dartmoor. Later came the extensive field systems of Romano-British origin and the Medieval strip fields such as those at Kilsby, Northamptonshire. The more recent clearances of crofting settlements in favour of sporting pursuits and consolidation of small holdings to permit better deployment of modern agricultural machinery are but some examples of this natural process of change. Change per-se should not be considered to be in the same vein as pollution.

Land Take

Loss of Agricultural Land The real value of agricultural land is now exposed to the influences of the UK's participation in the European Economic Community. The balance of both arable and livestock farming is now driven by Continental forces both market and regulatory. Major imports of a wide range of produce, once wholly home grown, now occur. The perceived value of land for agricultural purposes will inevitably continue to reflect these forces over the next century, the conservative life-span of any new reservoir, with concurrent changes in the old valuation rationale. The current approach to classification is more representative of the immediate post-war era when there was an acute need to be agriculturally self-sufficient. In instances where the loss of best and most versatile land cannot be avoided, it should be possible with today's technology to enhance off-site areas to superior classification in mitigation. Such practices have repeatedly proved feasible to our forebears, as evidenced by the polders of the Netherlands, terracing in the tropics and irrigation schemes in Israel and elsewhere world-wide many centuries (or millennia) earlier.

Dispossession It is most unlikely nowadays that mass evictions by way of compulsory orders could be a realistic stratagem for enabling reservoir construction. Promotion in the UK of a large dam in an area of significant development would almost certainly not be a viable prospect. The established consultation processes mean that large sections of the public, particularly the local residents, are represented from the earliest date of promotion. For the limited numbers of property-holders who will be affected, however, a degree of reconciliation to the development proposals will obtain by the time an envisaged Inquiry takes place. The consultation process will have acquainted them with the reality that the outcome rests on a host of factors in addition to land-take. Most agricultural concerns have become highly commercial enterprises for which financial implications drive the decision making process. By and large issues surrounding levels of compensation and mitigation arrangements will control their successful acquisition. Small holders and other local residents will be similarly motivated but may additionally favour relocation packages which enhance their original situation. The land purchase policy of the developer and his views with respect to factors impacting on public perception will influence how protracted this process is. Resolution of land issues is therefore one of open negotiation and cost rather than principle and might ultimately be determined by reference to the Lands Tribunal.

Visual Impact of Dam Structures

The direct environmental effects of the resulting inundation can be largely ameliorated by sensitive environmental planning during outline design. The creation of alternative wildlife habitats and proper husbandry of existing surface and subsurface artefacts each play a part. Even the prolonged nuisance of construction is of finite duration and can be significantly reduced with proper preparation, management and control. The visual impact, however, is enduring. The compact form and materials adopted to construct a concrete dam call for a sensitive blend of architectural form and engineering if a visually pleasing structure is to result. Fortunately the natural laws of classical physics invariably yield an aesthetic outcome if they have been well designed. The form of an embankment dam on the other hand demands a much greater footprint for similar height. Following its construction, there is a requirement to ensure its continuing structural integrity by monitoring internal conditions as indicated by embedded instrumentation, drainage systems etc. More important to the present review, however, is the reliance historically placed on the ability to support the surveillance activities through a routine of intensive and unhindered visual inspection. Any factors which serve to impede this ability to examine the external faces of the dam and appurtenant structures are normally considered undesirable in the interests of safety. Shoulders and crests are generally maintained free of large vegetation, sight-lines kept clear and the upstream slope protection systems for embankment structures laid to uniform grades and straight or, occasionally, gently curved horizontal alignments. It is the uniform geometry and lack of vegetative cover that lies at the root of the environmentalist's concerns in respect of visual blight. To resolve this conflict future dam designs will have to be developed which represent less visual impact than has been the norm. They should, preferably, also be seen to be landscape enhancements providing a host of differing habitats and amenities.

Landscaping and Vegetative Cover

Downstream Shoulders Environmental objection to the geometric regularity of dams is accompanied by a distaste for the lack of "natural" vegetation on shoulders. The engineer's dislike for trees and shrubbery as a visual obstruction is complemented by concern for the potentially adverse influence on dam performance. This relates to possible adverse effects on sub-soil moisture conditions, transient bulk density/strength values and the risks that root migration may present to the performance of internal impermeable cut-offs. Focused study and detailed design should facilitate the engineering of structures which could accommodate planting without adverse structural effects. External profiles could be adjusted to safeguard engineering interests and geotechnical analyses conducted to accommodate the attendant seasonal and longer term variations. This would require skilled integration of the engineering discipline with those of both the botanist and climatologist. Co-operative endeavour would permit truly representative soil conditions to be reflected in geotechnical designs and enable a fuller understanding of the slope stability effects. Collaboration between disciplines may also prove to be the catalyst in creating a closer integration of the engineering and environmental disciplines to mutual advantage. In parallel, cost considerations may have to yield to aesthetics with the provision of much greater irregularity in embankment profiles which can be visually enhanced by the planting regime.

Upstream Shoulders and Water Surfaces More particular attention would need to be given to treatment of difficult upstream faces, especially at the visually prominent water margins. All possible means of reducing the artificial appearance need to be investigated in greater depth than available at the present time. Asphaltic systems are available but leave a lot to be desired visually, particularly in their early life. Less intrusive slope protection systems which can simultaneously perform the rigorous wave protection role yet be able to accept suitable aquatic plant species should be investigated further. Work is in hand by one promoter to examine the feasibility of providing floating booms and "buoyant" islands tethered by anchor systems reflecting varying water levels. In addition, off-shore wave attenuating systems are under consideration which may possibly enable the use of embankment protection techniques better able to sustain vegetation albeit on a cyclic basis.

Freedom of Access

The shoulder enhancements outlined in the foregoing two paragraphs should also optimise the full amenity potential afforded by a dam. A fresh review of safety and water quality constraints might be undertaken to ensure that any access restrictions or prohibited water based activities are absolutely inviolate. Until the public at large begin to feel an affinity and sense of ownership for reservoirs in general, their ardent support for such developments is unlikely to be demonstrated overtly. Access must be the most important vehicle for promoting the level of alliance necessary to redress the present imbalance between recognition of benefits and the overstated impacts.

Surveillance Practice

A preference for straight, uniformly graded and unobstructed dam surfaces when undertaking visual surveillance operations reflects established inspection procedures. Conventional instrument-based survey techniques imply long clear lines of site. Whilst the structural monitoring procedures continue to demand this facility and unobstructed access to instrument locations such will remain the case. In light of recent developments in the fields of electronics and survey techniques, perhaps the need for these restrictions could be challenged. Major recent advances in the fields of Global Positioning by Satellite (GPS), Synthetic Aperture Radar Interferometry, Thermal Imaging and Doppler Ground Probing Radar suggest that a continuing dependence on visual assessment techniques and local access to instrumentation may be fast disappearing. GPS currently provides accuracies of horizontal and vertical survey control of 2-3mm and 4-5mm respectively. With the imminent commissioning of the EU's Galileo system these accuracies will be much improved. Strategically located permanent surface stations would enable 3D ground movements to be determined using GPS techniques. Accuracy will continue to increase as the technology becomes ever more sophisticated. The required sight cones of 15^0 to the horizontal could be maintained whilst allowing planting in close proximity. Similarly the development of telemetry systems based on fibre optics and the ubiquitous PC permit real-time monitoring of data supported by multi-validated alarm systems. *It is wholly conceivable that by the time* that next new British dam is commissioned technologies and programmes will be available to respond automatically to most structural emergency situations on a real-time basis. The condition monitoring systems may equally become able to initiate planned contingency measures albeit under the surveillance of appropriately qualified supervisors.

A POTENTIAL DAM PROMOTION STRATEGY

A proactive delivery process needs to be identified for winning over the general public, their political representatives and specific key environmental groups to the major advantages of dam based schemes as the preferred alternative to other resource development options. The strategy must keep in focus the environmental importance of energy conservation, the sustainability of reservoired schemes and their non-polluting attributes.

A proactive approach should be adopted for the identification of possible environmental enhancements and amenity provisions appropriate to dam projects. Current access constraints should be critically reviewed and ways to enhance amenity value identified.

Design techniques should be reviewed to determine the engineering constraints which would need to attend acceptance of shoulder planting regímes. This should be approached from the premise that no future embankment dam proposal is likely to be successfully promoted without a measure of vegetative cover. Techniques should be developed to soften the structural impact of the downstream face. All potential techniques for lessening/masking the adverse visual impacts of upstream and downstream shoulders should be investigated through a review of conventional geometry. Proposals must maintain freeboard and erosion resistance, and accommodate varying water levels. The upstream review should extend to include the feasibility of providing floating islands, booms, aquatic planting, protection systems etc. all with a view to masking the artificial topography created.

An academic research project could be commissioned to quantify the actual energy requirements for the manufacture, installation, commissioning and operation of competing water resource processes. Research needs to be undertaken to quantify the direct and indirect environmental impacts of winning fuels for power generation, the rates of depletion of these resources and the impacts of energy transmission.

Feasible methodologies for automatically monitoring the structural performance of dams should be investigated and alternative methods for accurately installing such systems for extended operational life explored. Technically appropriate proposals for, and risk analyses to, evaluate long-life instrument based reservoir surveillance systems should be developed. Current technology constraints and development needs should be identified.

CONCLUSION

From the perspective of the engineer the current promotional problems must be addressed as a matter of some urgency or there may be little potential for the development of new dams in the UK. The Qualified Engineer's future activities might otherwise become limited to a finite period of surveillance and/or maintenance of the current stock of dams. This would achieve little towards ensuring the future provision of essential water supplies now and well into the future. The public's best interests may be served by the initiation of dialogue at a high level between senior representatives of the dam fraternity, key players in the environmental field and Government representatives. The objective would be to identify agreed generic areas of concern surrounding dams as the preferred water resource development option and the identification of a long term strategy for their resolution.

Environmental impacts of dams: the changing approach to mitigation

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SYNOPSIS. The construction of dams to create reservoirs for drinking water, irrigation or hydraulic power started with the beginning of civilisation. However, the Industrial Revolution in Europe in the 18th and 19th centuries led to the demand for water to be a critical component of increasingly rapid development. In England and Wales the new industries and growing populations could no longer rely on shallow springs and wells, or rivers which were prone to low flows. Accordingly, locations were sought to store water to provide safe and secure resources to meet demand. This paper records continuing research into the approaches which have been adopted by dam engineers and their design teams to the location and construction of dams, frequently in environmentally sensitive areas and in the face of considerable opposition. The paper compares the design of reservoirs in the 19th century with those designed and proposed in the 20th century. The creative design approach which is currently adopted in England and Wales where land is scarce and population density is high can offer useful guidance to designers of new dams and reservoirs elsewhere in the world where enhancement of habitat and amenity will be an increasing requirement.

INTRODUCTION

The construction of dams to create reservoirs for water storage has been undertaken since the beginning of civilisation; the oldest, built about 3000 BC, has been recorded at Jawa in Jordan (Schnitter, 1994). Many are recognised as magnificent structures – one of the most famous is the Khadjoo dam at Isfahan dating from the 17th century. In Britain they had been constructed since the Middle Ages to create landscape features, to power mills or to provide water for canals. However, it was not until the nineteenth century when reservoirs were needed for public water supply that they were built in any number.

This 'golden age' of reservoir construction was long overdue. "At the beginning of the nineteenth century the population of Great Britain was 10.5 million and, although the Industrial Revolution had started some fifty years previously, there were practically no piped supplies of water". (Skeat, 1961). The Reform Act of 1832, together with the burgeoning public health movement led to a change in both the ability and need to meet the growing demand for water supply – and with it the use of water resources and their impoundment in reservoirs. In a crowded island community, the search for suitable sites away from centres of population and potentially polluting agricultural activities has led to the exploitation of our most valued landscapes and habitats. Today this results in considerable concern and opposition to the potential loss of these features and consequently engineers and their design teams go to great lengths to mitigate the predicted environmental impacts. This paper examines to what extent this approach is new, and, if it has changed, what have been the principal drivers for change.

In recent years, environmental assessment as a process has become a requirement for such development (the EC Directive on Environmental Assessment was implemented in England and Wales in 1988 and the amended Directive in 1999 – reservoirs are a Schedule 2 development in accordance Directive with the Town and Country Planning (Environmental Impact Assessment) Regulations 1999). This has formalised the need for consideration of mitigation in proposals for dam construction and reservoir creation as well as wide consultation requirements. However, before these latest specific requirements of legislation, other forces have played their part: we may call this developing process the 'Design Squeeze'. (Fig. 1)

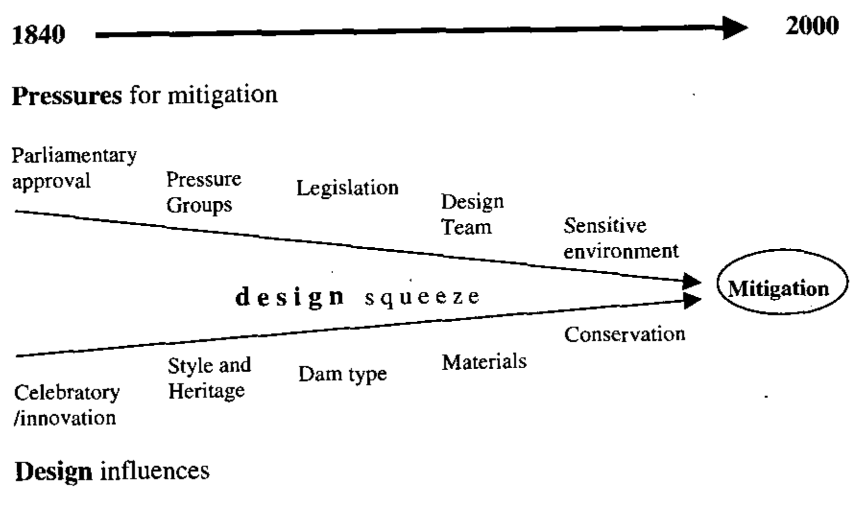

Fig. 1. The 'Design Squeeze' process.

It is interesting to note how much engineers responded to the environment in which such features were planned and comparison by means of case studies can assist in identifying how this has changed. (The word 'engineer', after all, is derived from the Latin 'ingenium' – a talent or skill).

Identifying the principal pressures and the design responses can inform regulators and designers elsewhere in their increasing need to achieve sustainable development with at least no net loss to the environment and communities, and offers opportunities for real environmental enhancement.

CASE STUDY 1: THE ELAN VALLEY

Up until 1870, water supply for Birmingham, (one of the fastest growing of Britain's industrial towns in the nineteenth century), was derived from rivers and shallow wells and boreholes; in the interests of public health the Corporation acquired the Birmingham Waterworks Company in 1875. During the period preceding this acquisition the Corporation instructed the engineer Robert Rawlinson to examine the existing water supply yields and potential new sources. He concluded that nearby sources were all too polluted or in catchments which were agricultural and heavily manured and recommended the development of the Elan and Claerwen catchments in central Wales some 75 miles from central Birmingham. These were "devoid of human habitation, mines and other sources of pollution ..." (Rawlinson, 1871). (Fig. 2).

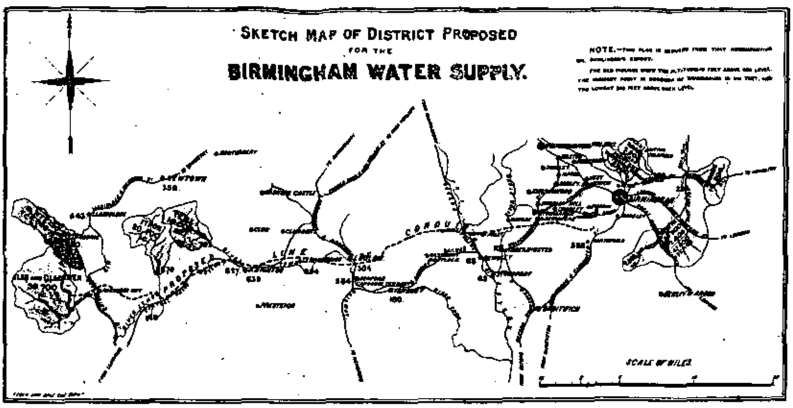

Fig. 2. Alternative catchments considered for Birmingham's Water resources (Rawlinson, 1871)

The scheme to construct six dams in the valleys was eventually presented to the House of Lords Committee in 1892 by James Mansergh who had also considered a range of alternative solutions. While Mansergh stated in his evidence that "the physical conditions ... have themselves settled the actual magnitude of the scheme" (Mansergh, 1892), one of his engineers R Eustace Tickell recognised the beauty of the place and its association with the English Romantic poet Shelley who had lived in the area with his first wife and described it as "... a solitude of mountains, woods and rivers, silent, solitary and old ...". Tickell, who became the engineer in charge of the Pen-y-Gareg dam, made sketches of the valleys prior to their flooding (Fig. 3), and described the area as "one of the most charming valleys in Great Britain, scenes which are soon to be lost for ever ... construction dooms many a picturesque spot ..." (Tickell, 1894).

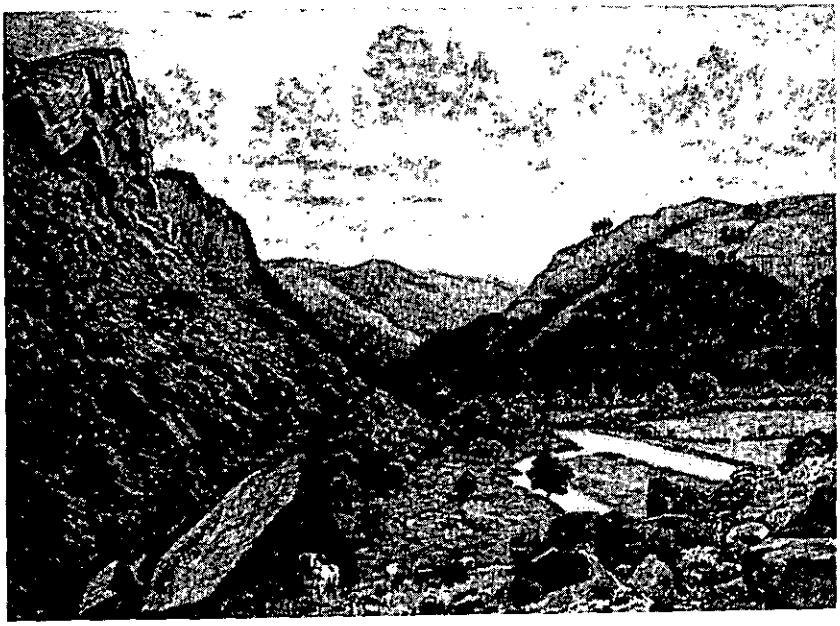

Fig. 3. Nantgwillt prior to reservoir construction (Tickell, 1894)

This provides a useful insight into the perception of the valleys by an engineer working on the scheme. Mansergh himself observed that "water will overflow from the reservoirs in picturesque cascades ... forming probably the finest waterfall in this country". (Tickell, 1894). (Fig. 4).

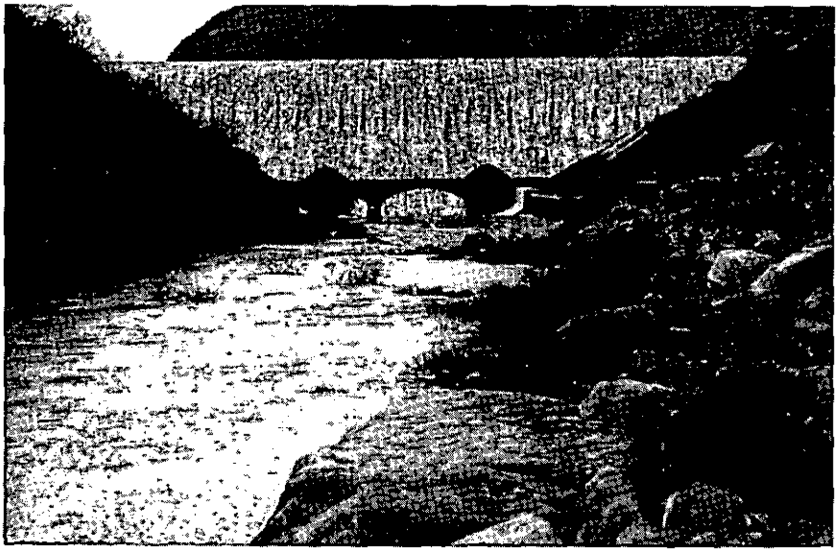

Fig. 4. "Probably the finest waterfall in this country" Caban Coch dam

The proposals attracted the attention of the Woolhope Naturalists Field Club* who visited the area during construction in 1896 and noted that "some (trees) on the higher ground will be spared to form here and there pleasing objects in the landscape bordering the reservoirs" (Moore, 1896).

Considerable opposition was mounted against the scheme when it was presented to the House of Commons – environmental concerns focused on compensation water to protect downstream fisheries. Access to hills on the watershed was proposed by Mr Shaw-Lefevre MP "the champion of the Preservation of the Commons" (Barclay, 1898) which was readily agreed to by the Corporation on second reading. Mansergh had actually proposed compensation water of 22.5 mgd and while the rod fishermen on the Wye claimed 40 mgd, the committee finally approved 27 mgd – a recognition that Mansergh, *of his own volition*, had indeed recognised the environmental issues before going public with his scheme. Mansergh did not have much time for the fishermen, stating "... there is very little spawning ground upon the Elan ... it is a sport rather than food of the people ... eating salmon is a luxury and fishing with a rod an occupation for the rich" (Mansergh, 1892). (There was no industrial use or navigation on the Wye and little water was taken for the towns on the river, whose populations were in decline at this time).

* Established in 1851 for the practical study of the natural history and archaeology of Hereford and adjacent districts.

After rigorous examination, the scheme received Royal Assent in 1892 and construction started in 1894. The provisions of the Birmingham Corporation Water Act 1892 required compensation water, provision of water to adjacent counties, access for the public "at all times enjoying air exercise and recreation" (BCWA, 1892) and protected rights for fishing. Telegraph and telephone wires were to be placed underground for the protection of amenity and the crossing of the River Tene by the aqueduct supplying the water to Birmingham was to be achieved by pipes in stone structures "in keeping with the scenery" (Mansergh & Mansergh, 1912). The Wye was crossed by siphon at considerable extra expense.

The faces of the dams, of which the first three were built by 1904, were faced with large blocks of local stone – reflecting the large boulders strewn along the river valley – rather than leaving them as bland concrete finishes. This again implies an understanding and recognition by Mansergh of the need to attempt harmony with the landscape. However the style of the masonry would be properly described as innovative rather than conservation (see Turner, 1987). Indeed while the Corporation was conscious of the expense, the style might be regarded as a celebration of their achievement and investment.

In the discussion of the paper describing the works, Walter Hunter declared that the scheme demonstrated that "the works of engineers could be carried out to harmonise with their situations and that the reservoirs of the Birmingham works had added to the beauty of the scenery" (Mansergh & Mansergh, 1912).

In 1937, and in the face of continual demand for water, the Birmingham Water Committee decided that the full economical yield of the Claerwen valley could be obtained by the construction of a single dam in place of the additional three smaller dams originally planned. A Bill was promoted in 1939 and while opposed – again principally for the effects on downstream flows – it received Royal Assent in 1940. The Corporation were anxious that the new work should harmonise and be in keeping with the high standard of the old dams. This new dam was designed as a gravity dam "curved in plan … for appearance and for architectural effect" (Morgan *et al* 1953). When tenders were invited in 1946, the Corporation selected the more expensive masonry faced design – despite stone supply problems and a shortage, in this post war reconstruction period, of skilled masons: 100 masons had to be brought to work on the dam from Italy.

Today the Wye remains one of the best salmon rivers in England and Wales. The Elan is a Special Protection Area in accordance with the EC Birds Directive and the estate managed by Welsh Water is a Site of Special Scientific Interest (SSSI).

CASE STUDY 2: CHEW VALLEY LAKE

The Bristol Waterworks Company had obtained its water supplies from springs in the Chew Valley in North Somerset since 1846: they were conveyed by means of an enclosed aqueduct to the Barrow Tanks south of Bristol. (There was a need to provide a 50 ft air vent at the high point on this 'line of works' and interestingly the engineers of the time disguised this vent by placing it within a stone obelisk – an early example of mitigation of visual impact.)

Despite continuing development of water resources, by the mid 1930s the demand for water in Bristol was increasing so rapidly that it was realised that a new source would be required and attention was again focussed on the Chew Valley. In 1934 gauging of the River Chew was begun to explore the possibilities of forming a reservoir in the valley ¾ mile upstream of the village of Chew Magna. It was determined that an embankment at this point 400 yd long and 42ft high could create a 1200 acre reservoir of 4,500 mg capacity. Accordingly a Bill was presented to Parliament in 1939. The Bill was opposed by, amongst others, the local authorities and riparian owners, principally to protect their interests and to ensure adequate provision of compensation water. However, Axbridge Rural District Council as the planning authority were also concerned at the injurious effects on the amenities, natural interest and beauty of the area. Although a relatively sparsely populated area, given its proximity to Bristol, a total of 17 cottages and 9 farms would be lost by the scheme – a total of 100 people to be rehoused. Royal Assent was granted in July of 1939 and the Bristol Waterworks Act 1939 authorised the Company to acquire land by compulsory purchase. However despite the loss of this valuable agricultural land all lands were acquired by private negotiation.

The outbreak of war delayed the start of construction of the scheme and when in 1945 water consumption began rising again Ministry approval was sought. However due to capital expenditure restrictions at this time it was not until 1950 that approval was given. By this time the proposed reservoir had developed into a regional scheme as a result of bulk supply agreements with Bath Corporation, West Gloucestershire Water Company and other Somerset water undertakings.

The loss of the fertile valley with its traditional farmsteads, manors and mills was viewed with concern by some and with resigned equanimity by others: the greatest concern was to record features and buildings before they were lost. In this respect it was fortunate that Bristol Waterworks official F C Jones was also an antiquarian and local historian. He realised the area's potential hidden treasures and every support was provided by the Company by means of excavation facilities. The Ancient Monuments Department of the Ministry of Works had been interested in the effects of the scheme since 1949 when it was apparent that a number of historic buildings would be lost. These included Walley Court, a medieval and Queen Anne building near to the proposed embankment, Spring Farm with a magnificent tithe barn and Stratford Mill an eighteenth century corn mill which was demolished and rebuilt at Blaise Castle Estate in Bristol. In 1953, the Ministry of Works, knowing that the valley had been inhabited from the Neolithic period, decided to embark on a systematic archaeological investigation prior to flooding of the valley. This revealed evidence of settlements from the Bronze Age, Iron Age and the whole Roman period with graves and numerous artefacts: the most significant find was a Roman writing tablet of wood with an ink inscription which had survived immersion in water for 1600 years.

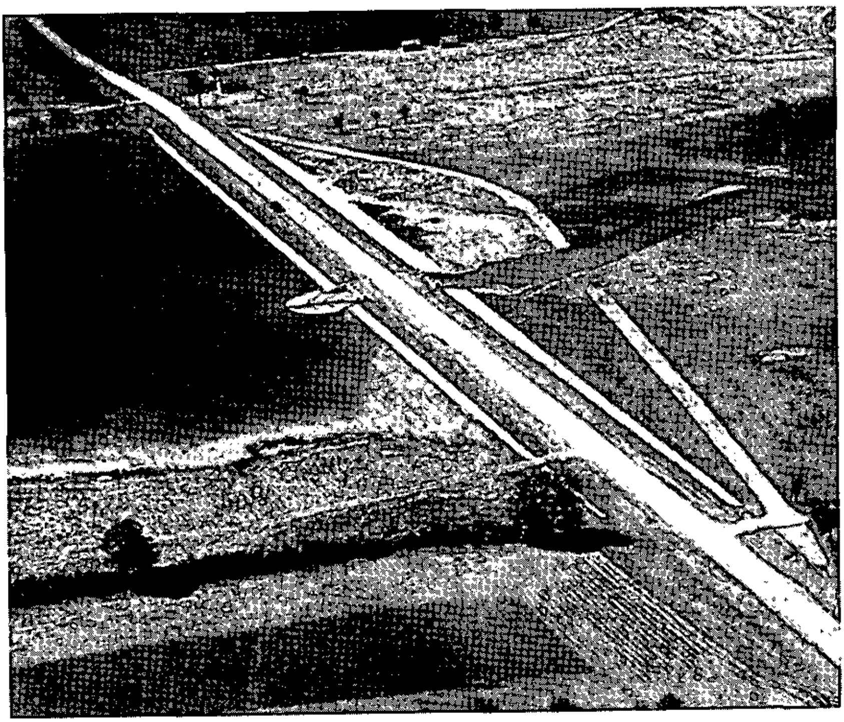

Fig.5. Herriotts Bridge Embankment (Farr, 1958)

The clearance works involved the loss of 70 miles of hedgerows and over 3000 mature trees which were replaced with young stock. The reservoir works involved the diversion of roads and the provision of new carriageways on embankments at various locations – at the southernmost edge of the new lake at Herriotts Bridge this formed 21 acres of very shallow water. The engineers decided that since "this would form an unsightly area when the water level dropped, it was decided to "pond up" this expanse permanently" (Picken, 1957). This area has now become a nature reserve managed by the Avon Wildlife Trust (Fig. 5). Conversely, at Herons Green to the west, the road embankment was made watertight to protect valuable farmland.

The Company realised that, given the reservoir's location near to Bristol it would soon become a visitor attraction and appear to have been concerned at the outset to ensure that it was managed accordingly. The amenity value was paramount and while fishing was encouraged it was not until 1967 that sailing was permitted. The Company's policy was "not merely to preserve the existing amenity but wherever possible to enhance it" (Picken, 1957).

In this respect the Company may have been encouraged by a clause in the Bristol Waterworks Act, 1917, for the construction of Cheddar reservoir which required it to take "all reasonable regard to the preservation of the beauty of the scenery of the district" (Bristol Waterworks Act 1917 Clause 37). The design and materials used for the various structures received special consideration and many trees and shrubs were planted around the reservoir – the architect Kenneth Nealon was retained to design various staff cottages as well as Woodford Lodge for anglers. While landscape architects had been required by Act of Parliament to be appointed to hydro-electric schemes in Wales by 1952, no landscape specialist was engaged on Chew Valley. However it was the first reservoir to be designed incorporating recreational facilities (Turner, 1987). Queen Elizabeth II commented at the opening ceremony on 17 April 1956 on the tree planting to heal the (inevitable) scars, the opportunities for wildlife as well as being a pleasant place for recreation. The report of the Queen's visit noted that the Bristol Waterworks Company "had aimed to make this a piece of lakeland scenery pleasing to see rather than simply a utilitarian reservoir" (Western Daily Press, 1956). Hundreds of thousands of trees were initially planted which was described as amenity planting in which the commercial value of the trees was of secondary importance. The amenity of the reservoir was greatly improved by Denny Island a high point in the valley and on which trees were planted in advance of the flooding in 1952, in anticipation of its creation and importance in the view (Fig. 6). However an additional artificial island planned for the lake was abandoned when the Ministry declined to fund the £18,000 cost.

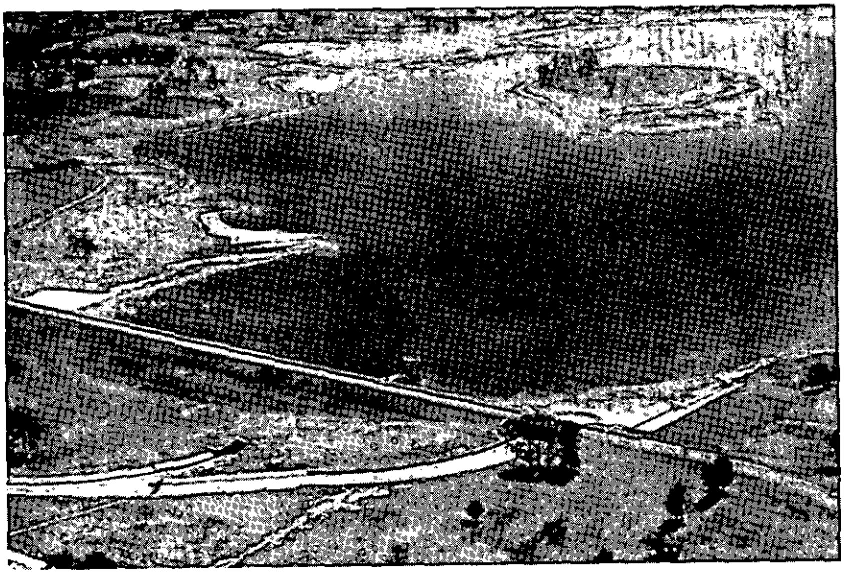

Fig. 6. Embankment dam and Denny Island (Far, 1958)

Fishing was encouraged and the lake was stocked with over 200,000 trout and facilities were provided for anglers. The fish were provided solely as an amenity – the "Company felt that as it had the privilege of controlling various large and suitable expanses of water it had a duty to provide good fishing" (Picken, 1957).

Today, apart from being scenically attractive and a mecca for angling, bird watching and sailing, the lake was designated as a SSSI in 1972 and later a Special Protection Area for birds under the EC Birds Directive. This has occurred because of the lake's size and its attractive position for migrating and wintering birds. The flooding of this lowland valley meant that natural and gently shelving banks with extensive shallows supporting a rich marginal vegetation were formed.

While Bristol Waterworks reservoirs have always considered amenity, (Blagdon Lake was stocked with trout in 1904) particular care was taken at Chew, both in its initial design and in its later operation and management. In this respect the 1966 Circular on the Use of Reservoirs and Gathering Grounds for Recreation (Ministry of Land and Natural Resources 1966) to promote easier public access to reservoirs, led to a great increase in recreational use.

CASE STUDY 3: THE AXE VALLEY WATER RESOURCES SCHEME

The purpose of this scheme was to provide water for public supply to meet the particular needs of East Devon in a rural area whose population is boosted by summer holiday visitors. It was proposed in 1992 by South West Water (following a series of studies which started in 1986) ie after the implementation of the EC Directive on Environmental Assessment and required planning consent in accordance with the Town and Country Planning Act 1970, as well as the Environmental Assessment Regulations 1988. An Abstraction Licence was also required from the National Rivers Authority (NRA) in order to regulate, abstract and store water for public supply. Approval for the dam design would also have eventually been required in accordance with the provisions of the Reservoirs Act 1975 for the construction of the reservoir.

In summary, the scheme proposed pumping water from the River Axe to a new treatment works from where it would be distributed for public supply. In times of low flows in the river, storage was to be provided by means of a pumped storage reservoir – which would also provide an alternative supply in the event of river pollution upstream of the intake. At such times water would be released from the reservoir to the treatment works by gravity via an interconnecting 4 km pipeline. Such releases would occur during late summer and early autumn and the reservoir would be refilled during the winter at times of high river flows.

Given the environmental sensitivity of the area, alternative sites for the reservoir were examined within the Axe valley with the environmental options being considered in parallel with engineering, geotechnical and hydrological studies. Alternatives to the Axe valley reservoir included groundwater development (Axe and/or Otter), Wimbleball pumped storage and Wimbleball raising were also considered.

The alternative sites were all located within the East Devon Area of Outstanding Natural Beauty (AONB), one of over 40 that have been designated in accordance with the National Parks and Access to the Countryside Act 1949. The objective of AONBs is to protect and enhance such valued landscape features, and to recognise opportunities for informal countryside recreation. In the case of the East Devon AONB the aim was to specifically protect the dramatic coastal scenery as well as the untouched rural hinterland of plateau heathlands, hills, river valleys and meadows. The designation required the local planning authority to consult with the then Countryside Commission on any development proposals within the AONB.

The design team included not only engineers but also landscape architects, ecologists and fisheries specialists since the promoters of the scheme recognised that the effect of reduced flows on fisheries as well as the effects on the ecology and landscape of the area was a primary consideration.

The preferred option was for the construction of a reservoir with a storage volume of 2850 Ml when full in a tributary valley of the River Axe. Whilst this location offered the greatest potential for integrating the dam and reservoir into the valley setting, it was adjacent to a SSSI notified by the Nature Conservancy Council (NCC) under the 1949 Act. The reservoir site itself was wholly agricultural and 30 ha of farmland would have been lost as a result of the scheme. To protect the SSSI, a backstop dam was to be provided which would limit the submergence area of the reservoir (Fig. 7). The main dam itself was to be earth embankment shaped to harmonise with the landscape, curved in plan with hedgerow planting to reflect the pastoral character of the area.

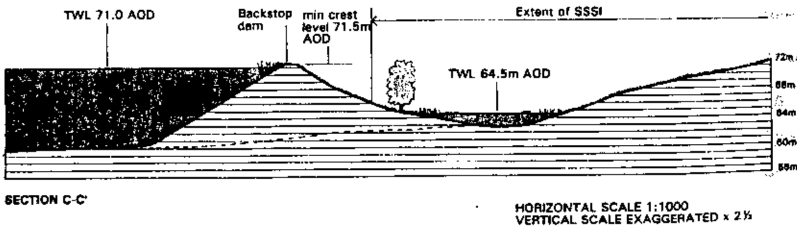

Fig. 7. Backstop dam for proposed Reservoir (SWWS, 1992)

The series of studies which were undertaken as a result of the EIA process concluded that the abstraction from the river would cause minimal impact on fisheries provided that measures were taken to protect flow conditions during critical times of the year for salmon and sea trout migration. These included operating rules to apply to the intake and limits on the amount abstracted between May and November. Careful consideration was given to the location and screening of the proposed water intake and treatment works as well as the routing of an underground pipeline to the reservoir – avoiding hedgebanks and using field entrances where possible.

Extensive consultation was undertaken as part of the scheme preparation and design. Conservation bodies – the Countryside Commission and the NCC – while generally opposed to the scheme, remained to be convinced that such development was in the national interest. However, the studies were carried out with their full co-operation and agreement. Landowners in particular were concerned at the loss of good quality farmland and the intrusion of a new reservoir, however sympathetically designed, into the relative seclusion of the valley; the potential increase in visitors was also of concern. A public campaign against the scheme was mounted through the local press and public meetings which gained considerable local support.

The NRA were principally concerned with the justification for the scheme in the context of water supply for east Devon as well as the impacts on river and tributary catchment environments. It was this holding objection from the NRA which prompted South West Water to review its strategy and promote an extension to the Wimbleball scheme instead. As a consequence the application and the Environmental Statement for the River Axe scheme were not submitted.

CONCLUSIONS

It is clear that the emphasis on providing mitigation for the environmental impact of dams has increased over the last hundred years.This has been due to range of factors:

- Increased public awareness and concern
- More regulation by statutory bodies
- Wider disciplines in design team
- Wider consultation required by legislation (especially EIA Regulations)

However this paper has demonstrated that prior to the above factors, dam engineers in the last century, at least in the case study examined, *did* consider the effects on the environment of their development: mitigation measures such as compensation water to ensure the maintenance of flow downstream of the dams were proposed without such conditions being imposed by the determination process. A response to the landscape was also demonstrated in terms of scale and use of materials. Post war, considerable efforts have been made to ensure that reservoirs, while changing the landscape character of an area can add considerably to habitat and amenity alike.

Supplementary dams can also be built to protect and enhance habitats. For earth dams a curved profile can be provided to follow contours and hedgerow planting across the downstream face can be in keeping with, and form an extension to, the local landscape.

Further studies are being undertaken as part of this ongoing research which will provide a schedule of mitigation techniques which have been developed in recent years.

ACKNOWLEDGEMENTS

The author wishes to thank Neil Whiter of South West Water and Colin Hunt of Bristol Water for their assistance in sourcing material for this paper. The views expressed are those of the author and do not necessarily represent those of the water companies mentioned in the text. The paper is based on and extends a poster paper presented at IAIA Glasgow 1999 and a paper subsequently published in EA magazine and reports on continuing doctoral research at the Department of Planning and Landscape at the University of Manchester.

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Environmental evaluation of reservoir sites

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SYNOPSIS. The planning of future reservoir sites is increasingly becoming dependent upon robust environmental analysis. European and National legislation requires the environmental impact of all potential sites to be assessed. Such assessments enable the comparison between various environmental impacts for each site. It is also relatively straightforward to compare similar environmental issues between sites. However, in order to undertake an overall objective comparison between potential sites scoring systems have been developed. Scoring systems should enable the positive benefits and negative impacts of all the sites identified in an environmental assessment to be quantified and compared.

INTRODUCTION

The comparison of the relative merits of alternative reservoir sites requires an integrated approach to engineering and environmental issues. This paper concentrates on the environmental element of this work.

The successful promotion of large projects such as reservoirs requires the undertaking of major consultation processes with both statutory and non-statutory bodies on the issue of a scheme's possible environmental impact, which may have both positive and negative effects. As part of this process it is necessary to demonstrate that the site chosen has been compared in a robust and impartial manner to all alternative sites.

The identification of the possible environmental impacts consequent upon the development of a reservoir normally follows guidelines set out in European and national legislation, including the requirements of the Town and Country Planning (Environmental Impact Assessment) (England and Wales) Regulations (1999). The environmental assessment will generally include the following issues:

- planning, including: countryside, built environment, housing, employment, retail, recreation, tourism, transport, minerals and waste issues;
- water quality, including surface and groundwater;
- land use and agriculture, including agricultural land classification and farm buildings;

Dams 2000, Thomas Telford, London, 2000.

- traffic and access, including: the use of existing roads, the need for new road construction, the protection of existing rights of way and bird strikes from nearby RAF bases;
- cultural heritage, including built heritage, archaeology and historic landscapes;
- natural environment, including: flora, fauna, habitat, designated sites and fisheries;
- landscape and visual;
- community, including disturbance, severance, amenity, housing, employment, air quality, dust and noise, and;
- other issues of specific relevance to individual sites or areas.

Once data on the above issues have been identified and impacts assessed for the alternative sites the results between these sites can be compared. In recent years various methods for comparing environmental impacts have been developed as part of the design, construction and operation of large scale civil engineering projects throughout the UK, such as major highway and railway projects, dams and reservoirs, waste treatment works and various waterway projects. Normally, the method for comparing alternative sites initially involves undertaking an evaluation of the magnitude and significance of the various environmental impacts. This includes:

- assigning a value to each of the site-specific environmental parameters e.g. sites of ecological significance, landscape designations, types of agricultural land, etc.;
- ascertaining the magnitude of potential impact on each environmental parameter;
- on the basis of the combination of value and impact magnitude, assigning levels of impact significance;
- application of appropriate mitigation measures, and;
- evaluation of residual impact significance.

Following on from the identification of the magnitude and significance of the possible environmental impacts a ranked matrix approach is often used to enable an easier comparison between a large number of sites and their associated environmental parameters. This process involves:

- the identification of the environmental parameters;
- defining criteria for the analysis of these environmental parameters, followed by the analysis itself;
- defining a universal scoring system to accommodate all parameters;
- for each site, assigning a score to each of these parameters within the matrix, based on the results of the data analysis;
- applying weightings to these scores where appropriate, and;

- summarising the scores for each option to allow like-for-like comparison.

The remainder of this paper describes a case study where this matrix approach and scoring system has recently been used to undertake a preliminary assessment of various reservoir options in order to compare the relative merits of each option.

CASE STUDY

In July 1997 Babtie Group were commissioned by Essex and Suffolk Water to evaluate the technical and environmental issues associated with seven potential reservoir sites in order to reduce the options to two preferred sites. It should be noted that the seven reservoir sites were required to store 20,000Ml of water. In addition, four of the locations were also considered for 50,000Ml capacity. In total this meant a comparison of eleven reservoir options.

The Study

In summary, the study looked at:

- seven possible reservoir locations, located across four counties which contained a diverse range of land forms, ecology, archaeology and land use;

- for each of the locations three main sources of environmental impact had to be considered: the construction of the main reservoir (including impacts of construction traffic and re-routing of local traffic up to filling of the reservoir); operation of the reservoir (once it had been filled); and construction and operation of associated pipelines, and;

- for each location, impacts on a wide range of environmental topic areas had to be considered. It was clear from the number of variables that needed to be compared that a rigorous objective methodology of comparison was required. Therefore, an overall standard was set using a matrix system.

The Matrix System

The use of a matrix system allowed for a variety of data to be compared, and provided a clear overall comparison of impacts. In total, the following three types of matrix were developed:

- A Type 1 matrix was produced for each of the reservoir locations and water storage sizes. The Type 1 matrix was used to indicate the overall level of impact as a score for each of the eight environmental issues (identified in the Introduction), which would result from each stage of the three phases of the scheme's development, i.e.:

a) construction of the reservoir and associated structures, up to and including filling of the reservoirs;
b) their operation (assuming minimal staffing maintenance), and;
c) construction and operation of the pipeline, and major road construction.

An average score for each environmental issue and its sub-categories enabled extremes (both positive and negative) to be reflected in the comparative stages of assessment.

- A Type 2 matrix presented a summary of the environmental impacts of the three phases of the scheme's development using the average scores for each environmental issue identified in the Type 1 matrices.

- A Type 3 matrix presented the summary information regarding the possible environmental impacts (from the Type 2 matrices), still considering the three stages of reservoir development. These matrices, allowed each of the eleven reservoir options to be compared with regard to their impact on each of the identified environmental issues. A ranking system was also introduced within the Type 3 matrices whereby the reservoir options were ranked from least damaging (1) to most damaging (7), according to the average scores for each environmental issue. An overall ranking between the reservoir options was then calculated from the final score for each environmental issue.

The Scoring System

In order to provide an objective comparison of the various environmental issues, a standard methodology framework was developed. A scoring system of 1-5 was selected showing both positive and negative impacts, as shown in Table 1.

Table 1: Scoring System

Score	Meaning	Example Definition
+5	Extreme positive	Improvement of Internationally important site
+4	Severe positive	Improvement of Nationally important site
+3	Substantial positive	Improvement of Regionally important site
+2	Moderate positive	Improvement of Locally important site
+1	Minor positive	Improvement of other areas
0	No impact	No positive or negative impact
-1	Minor negative	Loss or impact on other areas
-2	Moderate negative	Loss or impact on Locally important site
-3	Substantial negative	Loss or impact on Regionally important site
-4	Severe negative	Loss or impact on Nationally important site
-5	Extreme negative	Loss or impact on Internationally important site

The above scoring system was used as a guide for the development of each of the environmental issue methodologies. However, the grading system had to be adjusted according to the environmental issue, for example:

- a slightly impacted International site may be assigned a lower impact score than a completely destroyed National site;
- some topic areas did not have a designation, such as noise impact, in these instances the system was used more as a guide, although the methodology used was agreed prior to use, and;
- the full range of scores was not necessarily used for each subject area so that comparable effects could be recorded (for example it was not considered that the temporary impact of construction dust on a local community could ever correspond to the loss of an internationally designated site of historic or ecological importance).

We will now look at two environmental issues in greater detail to examine the key stages in determining and comparing the environmental effects of the eleven reservoir options and how these effects where incorporated into a scoring system. As part of this process an example of each of the three matrices has been included for one reservoir option. The first environmental issue is planning. The second environmental issue is the natural environment.

<u>Planning Issues</u>
The following key stages were followed to determine and compare the planning effects of the eleven reservoir options:

- identification of the relevant planning authorities;
- obtaining relevant development plans (structure and local plans) for each reservoir site;

- identification of any planning designations applicable within the study area, from proposal maps;
- consultation as to the likely promotability of a reservoir within the area;
- through consultation with various planning authorities the identification of major development proposals which were already permitted or the subject of a planning application;
- identification and listing of all development plan policies relevant to planning designations and development proposals identified;
- establishment of the significance of effects of any proposed reservoir upon planning designations and policies, and;
- ranking of the significance of effects on a scale of +5 to -5, in accordance with the following guide in terms of policies and plans (Table 2):

Table 2: Scoring System for Planning Issues

Meaning	Score	Example Definition
extreme +ve/-ve	5	Effect of international importance (for example, effects upon policies intended to protect World Heritage Sites and areas covered by European conservation designations)
severe +ve/-ve	4	Effect of national importance (for example, effects upon policies intended to protect Areas of Outstanding Natural Beauty, Sites of Special Scientific Interest, National Nature Reserves, Scheduled Ancient Monuments, Listed Buildings: Grade 1)
substantial +ve/-ve	3	Effects of regional/county importance (for example, effects upon policies intended to protect Areas of Special Landscape Value, Sites of Importance for Nature Conservation, Green Belt development policies, Local Nature Reserves)
Moderate +ve/-ve	2	Effects of local importance (for example, effects upon policies or plan designations relating to gaps between settlements, local plan housing/industry/retail or other development sites)
Minor +ve/-ve	1	Negligible effects (identifiable effects of little significance in relation to policies and plans)
No impact	0	No impact or irrelevant to policies or plans

Tables 3 and 4 show an example of how Planning issues for one 20,000Ml reservoir option were scored and progressed through the first two matrices. Table 5 shows the Type 3 matrix, and how the average scores for Planning issues at the three phases of the scheme's development where totalled and then ranked against the other 20,000Ml reservoir options to find the best environmental option, in this instance the example reservoir was ranked first.

Table 3: Example Planning Issues Type 1 Matrix - 20,000Ml Reservoir

Phase Factor	Construction of Reservoir	Operation of Reservoir	Pipeline Construction and Operation
Planning:			
countryside	-2	-2	-1
built environment	0	0	0
housing	0	0	0
employment	3	1	0
retail	3	0	0
recreation/tourism	-1	1	0
transport	-1	1	0
minerals and waste	-1	0	0
promotability	-3	3	0
Average Score	0	0	0

Table 4: Example Planning Issues Type 2 Matrix - 20,000Ml Reservoir

Phase Factor	Construction of Reservoir	Operation of Reservoir	Pipeline Construction and Operation
Planning	0	0	0

Table 5: Example Planning Issues Type 3 Matrix - Comparison of 20,000Ml Reservoirs

Site	Example Reservoir		Reservoir 2		Reservoir 3	
Factor	Score	Rank	Score	Rank	Score	Rank
Planning	1	1	-4	4	-5	6

Natural Environment Issues

The impact of construction and operation of the possible reservoir options on the natural environment was largely determined by the importance of the site. The level of importance of different sites is shown in Table 6.

Table 6: Level of Importance of Different Ecological Sites

Importance	Type of Ecological Site
International Importance	RAMSAR site SAC SPA Protected/scheduled species or habitat
National Importance	SSSI NNR Protected/scheduled species or habitat > 50 ha. Ancient woodland
Regional Importance	LNR Wildlife Trust reserve Important regional habitat 10-49 ha. Ancient woodland Designated non-statutory site
Local Importance	Local wildlife site Species rich hedgerow < 10 ha. Ancient woodland Tree Preservation Order Diverse waterways or wildlife corridors
Limited Importance	Few habitats of conservation value affected, or limited to common species or habitats. Lost habitat easy to re-create.

The table below indicates how these sites were scored, considering the likely level of impact (Table 7):

Table 7: Scoring System for Natural Environment Issues

Importance of site/ Level of impact	International	National	Regional	Local
Complete destruction	Extreme -5	Severe -4	Substantial -3	Moderate -2
Small loss	Severe -4	Substantial -3	Moderate -2	Minor -1
Disturbance	Substantial -3	Moderate -2	Minor -1	Minor -1

It should be noted that in some situations provision of a well designed area of open water can provide opportunities for habitat creation, and introduce valuable ecosystems to considerably increase the biodiversity of the area. In such cases, a positive impact was recorded.

An example of benefits to be gained in areas such as water quality, preservation and the relocation of flora and fauna, reservoir margin profiling and visual enhancement is the Roadford Dam in West Devon. For their work on this scheme Babtie Group gained a British Construction Industry Award for Environmental Excellence. Similarly, a designated Wildlife Heritage Site, consisting of a valuable wildlife habitat managed as a nature reserve, has been created at Moor Green Lakes, Finchampstead, Berkshire. This project dealt with the restoration of lakes created as a result of sand and gravel extraction in the valley of the River Blackwater.

Tables 8 and 9 show an example of how Natural Environment issues for one 20,000Ml reservoir option were scored and progressed through the first two matrices. Table 10 (Type 3 matrix) shows how the average scores for Natural Environment issues at the three phases of the scheme's development where totalled and then ranked against the other 20,000Ml reservoir options to find the best environmental option, in this instance the example reservoir was ranked first.

Table 8: Example Natural Environment Issues Type 1 Matrix - 20,000Ml Reservoir

Phase **Factor**	**Construction of Reservoir**	**Operation of Reservoir**	**Pipeline Construction and Operation**
Natural Environment:			
habitat	-1	2	0
flora	0	2	0
fauna	0	3	-2
designated sites	-1	1	-4
fisheries	0	1	0
Average Score	0	1	-1

Table 9: Example Natural Environment Issues Type 2 Matrix - 20,000Ml Reservoir

Phase **Factor**	**Construction of Reservoir**	**Operation of Reservoir**	**Pipeline Construction and Operation**
Natural Environment	0	1	-1

Table 10: Example Natural Environment Issues Type 3 Matrix - Comparison of 20,000Ml Reservoirs

Site **Factor**	**Example Reservoir** **Score**	 **Rank**	**Reservoir 2** **Score**	 **Rank**	**Reservoir 3** **Score**	 **Rank**
Natural Environment	1	1	0	2	-5	6

USE OF RESULTS AND SENSITIVITY ANALYSIS

Table 11 shows an example Type 3 matrix which compares three example reservoir sites against all the environmental issues considered. For each environmental issue separate methodologies were developed to generate the scores. These (Type 3) matrices allowed for a visual comparison to be made regarding the various reservoir sites that were under consideration. Using these matrices the preferred locations for both a reservoir of 20,000Ml capacity and a reservoir of 50,000Ml capacity were easily identified. In this instance it can be seen from Table 11 that the example reservoir was ranked first and would be the preferred location for the 20,000Ml reservoir option.

Table 11: Example Type 3 Matrix - Comparison of 20,000Ml Reservoirs

Site **Factor**	**Example Reservoir** **Score**	 **Rank**	**Reservoir 2** **Score**	 **Rank**	**Reservoir 3** **Score**	 **Rank**
Planning & promotability	0	1	-3	4	-5	7
Water quality	-4	6	-3	4	-5	7
Land use & *agriculture*	-2	1	-5	5	-5	5
Traffic & access	-1	2	-1	2	-7	7
Cultural heritage	-4	4	-3	3	-5	6
Natural *environment*	0	1	0	3	-2	6
Landscape & visual	-5	6	-3	2	-4	5
Community	1	2	0	3	-5	7

In order to test the objectivity of the methodology sensitivity analysis was undertaken. The sensitivity test analysed a number of assumptions made to ensure that particularly sensitive issues for certain sites did not unduly influence the overall results.

Four sensitivity tests were undertaken. For the first 'round' of testing the individual scores for certain environmental issues and their associated sub-categories were halved for every reservoir site and for both reservoir capacities. For the second and third 'round' of testing the total scores for four environmental issues which had the highest scores were halved for each reservoir site and size. These changes were then introduced to the three matrices and an overall comparison was made between the reservoir sites. The fourth 'round' of testing involved halving high scores for those environmental issues which were considered as potentially sensitive at each of the sites.

The sensitivity testing indicated that overall, there were no significant changes to the ranked order of sites as a result of the exercise.

As stated in the Introduction, the comparison of potential reservoir sites involves consideration of both the environmental and engineering issues. The results of the comparative analysis of environmental issues is then combined with cost information to identify the most favourable site.

CONCLUSIONS

This paper has described the range of environmental issues which are normally examined as part of the environmental assessment of a new or expanded reservoir site. Through the use of a case study the paper then examines one method for comparing levels of environmental impact between similar environmental issues, for two sizes of reservoir at seven different locations within England. Due to the high number of environmental variables that need to be compared a rigorous and objective methodology in the form of a matrix system and scoring methodology was developed in order to determine the best environmental option. Finally, the results of this process were subjected to sensitivity analysis to test a number of assumptions made and to ensure that particularly sensitive issues for certain reservoir locations did not unduly influence the overall results.

ACKNOWLEDGEMENTS

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A few problems with Central Asia's large dams

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SYNOPSIS The safety of the large dams in the catchments of the two main rivers of Central Asia, the Syr Daria and the Amu Daria, is causing international concern in the context of measures currently under way to save the Aral Sea. Based on safety assessments of ten of the dams by the authors, this paper describes particular features that reflect design and construction practice during the Soviet period, assesses some of the changes in operating conditions for the dams, including those resulting from the break-up of the Soviet Union, and categorises what are perceived to be the present deficiencies in safety standards. Emergency measures aimed at mitigating the hazardous condition of the dams are outlined.

INTRODUCTION

The stock of large dams located mainly within the catchments of the Syr Daria and the Amu Daria rivers, both of which drain to the Aral Sea, represents one of the most valuable assets inherited by the countries of Central Asia, following the break up of the Soviet Union. Investigations into the management of these dams, and the reservoirs that they impound, constitutes one of the components of the Water and Environmental Management Project (WAEMP) which is being implemented by the Executive Committee of the International Fund to Save the Aral Sea (EC-IFAS) under the Aral Sea Basin Program. The program is supported by a variety of donors, such as the Global Environment Facility (GEF) via the World Bank, the Dutch and Swedish Governments and the European Union.

Under the first stage of Component 'C' of the WAEMP, the authors' company was commissioned by the EC-IFAS Agency to inspect ten of the large dams, two in each of the Central Asian states, and to prepare dam safety assessment reports on each. The ten included Nurek, a 300m high rockfill embankment (believed to be currently the highest dam in the world) in Tajikistan, and Toktogul, a 215m high concrete gravity structure in Kyrgyzstan. All of the dams present a high level of hazard as they impound large storage volumes, and they are in most cases located above settled and densely populated communities. Seven of the dams are Class IV under the risk classification system given in ICOLD Bulletin 72, the others being Class III.

The locations of the ten dams are shown in Figure 1.

Dams 2000, Thomas Telford, London, 2000.

Figure 1 Map of Central Asia

SAFETY INSPECTIONS

The safety inspections comprised one-day visits to each of the dams, discussions with the principal staff responsible for operating the dams and examination of such drawings and reports as were readily available. Time did not permit a detailed inspection of the operating records, nor was much of the design and operating data easily accessible. The authors relied heavily on the work of a team of regional specialists, working closely with national teams in each country, to collect and evaluate basic information. A valuable insight into the operational problems of each of the dams was provided by the discussions with the dam operators. The only analytical work undertaken to date is the routing of original design flood hydrographs through the reservoirs.

Table 1 Details of the Dams

Name	Country	Type	Height m	Length m	Power MW
Akhangaran	Uzbek	Earth & gravel emb.	100	1,633	none
Bugun	Kazak	Earth embankment	21	5,200	none
Chardara	Kazak	Hydraulic fill emb.	28.5	5,300	4 x 25
Chimkurgan	Uzbek	Earth embankment	33	7,500	none
Kayrakkum	Tajik	Hydraulic fill emb.	32	1,202	6 x 21
Kopetdag	Turkmen	Hydraulic fill emb.	24	15,400	none
Hauzhan	Turkmen	Hydraulic fill emb.	14.3	35,000	none
Nurek	Tajik	Earth & gravel emb.	300	714	9 x 330
Toktogul	Kyrgyz	Concrete gravity	215	295	4 x 300
Uchkurgan	Kyrgyz	Earth & gravel emb.	37	2,900	4 x 45

COMMON FEATURES OF THE DAMS

The authors noted a number of features that are typical of dams in the region designed during the Soviet period, which are unconventional in the context of Western dam engineering practice and which have a bearing on their safety.

Spillways

Only a few of the large dams have conventional surface spillways, gated or otherwise. For control of floods, heavy reliance is placed on attenuating peak discharges in the reservoir, and on using low level outlets, including power outlets. Of the ten dams inspected, the only ones with spillways were Chimkurgan, Nurek, Toktogul, Uchkurgan and Kayrakkum. The spillway at Chimkurgan is ungated but is of small capacity. It is formed integrally with the valve tower (see Figure 2), discharging to the same conduits as the low-level outlet, and is protected by trash screens that render it liable to be somewhat ineffective under extreme flood conditions. The spillway at Nurek is controlled by twin 12m wide by 12.3m deep surface mounted radial gates and comprises a single 10m dia. tunnel 1,100m long in the left abutment, discharging by flip bucket back into the river below the dam. The two surface spillways at Toktogul comprise twin concrete chutes that pass over the top of the hydropower station and discharge by flip bucket to the stilling pond downstream of the dam. Each spillway chute is controlled by a single radial gate 10m wide by 9m deep. At both Nurek and Toktogul the flip buckets are angled at quite an acute angle so as to project the jet in the direction of flow of the downstream channel.

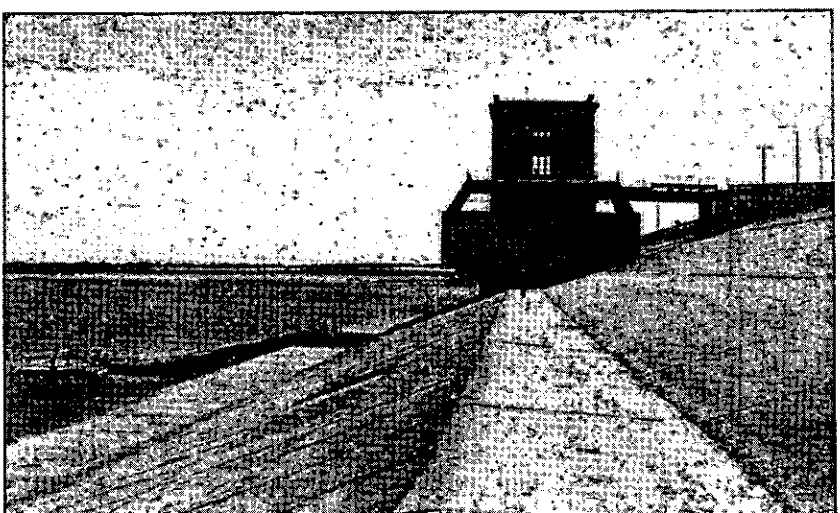

Figure 2 Chimkurgan Dam Intake Structure

At Uchkurgan, a single 12m wide gated spillway is located at the right hand side of the hydropower station. At Kayrakkum there are six spillway channels that pass over the power station, each controlled by 12m wide by 10m high vertical lift slide gates.

Low-Level Outlets

Because of their function to discharge floods, the low-level outlets are generally of large capacity, built either as multiple concrete conduits with an upstream valve tower, or, where the dam site is narrow, as a large bore tunnel excavated in one of the rock abutments. In the latter case, the tunnel is generally of 'D' shape, divided up the middle by a half-height wall designed to permit separate closure of each half of the tunnel for maintenance. Low-level outlets are generally at between one third and one half height in the dam, leaving a significant volume of dead storage in the reservoir. Often there is no bottom outlet so that the only facility for evacuating sediment from low level in the reservoir is through the power station outlets. In this regard Uchkurgan Dam, which is the lowest dam of the cascade of five dams in the Naryn gorge (Toktogul, Kurpsay, Tashkumyr, Shamaldysay and Uchkurgan), is unusual in that eight low level sluices are placed immediately below the intakes for the four hydropower units.

Embankment Construction

Embankment construction is generally conventional, comprising a central impermeable core zone with upstream and downstream shoulders of gravel or earth construction. Side slopes on the embankments are conservative and reflect the generally high levels of earthquake activity in the region. Four of the dams (Chardara, Hauzhan, Kayrakkum and Kopetdag) are constructed of hydraulically placed sand. These generally have much flatter side slopes (1: 20 - 35 in the case of Kopetdag and 1: 30 - 50 in the case of Hauzhan) but Chardara and Kayrakkum, which were two of the first dams built by this method, have side slopes of 1: 4 - 4.5. Where there is a shortage of suitable core material, core zones have been formed by grouting the gravel fill after it has been placed. The clay core in Akhangaran dam is steeply inclined, reflecting the fact that this dam was constructed in three stages, each stage extending the upstream face of the previous stage.

Embankment Slope Protection

Upstream slopes are normally protected against wave action by concrete slabs, (see Figure 2) usually placed insitu, but sometimes precast. The slabs vary in thickness but are normally underlain by a gravel and/or sand filter layer. Where there is a suitable source of sound rock, riprap is also used but the authors found cases where the size of stone or the thickness of the concrete slabs were much less than would be considered necessary under western design codes. This could account for some failures both of riprap and concrete facings. Downstream slopes are normally unprotected, or grassed which appears to be satisfactory in a region where annual precipitation is low and falls mainly as snow in the winter months. However, a few of the dams such as Bugun, which is constructed of compacted loess, have elaborate drainage collection systems on the downstream face to limit runneling in this highly erodible soil.

Power Outlets

Several unusual and innovative designs have been used in the layout of the hydropower stations. Mention has already been made of Uchkurgan where the low level sluice intakes are below the power intakes, with the sluices threaded between the sets at the same level as the turbine runners. It is probable that this layout was designed to produce some additional head for power generation when the sluices are discharging at full capacity. A similar concept has been adopted at Kayrakkum, but here the turbines and generators are encapsulated in individual chambers located immediately below each of the six spillway channels. To access each of the sets it is necessary to close the relevant spillway and lift an access hatch placed in the base of the spillway channel, using the same overhead crane as is used for handling the gates.

Probably the most unusual layout is that adopted for Toktogul. Here the gorge is too narrow for the four sets to be placed in line so that two of the sets are placed upstream of the other two, resulting in the draft tubes being stacked one on top of the other (see Figure 3).

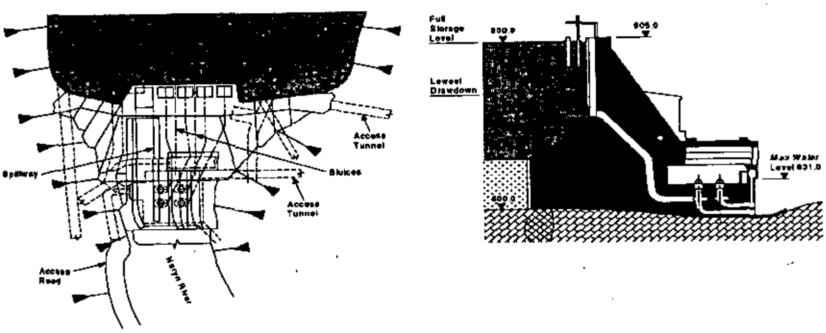

Figure 3 Toktogul Dam - Plan & Section

Hydromechanical Equipment

A conservative approach has been adopted in the provision of hydromechanical equipment, with generally a maintenance gate placed upstream of each control gate, and often a further emergency gate upstream of the maintenance gate, with a deck mounted gantry for gate handling. To dewater the section of conduit, or tunnel, on the upstream side of the main gates, the facility to place a stoplog, or emergency gate, at the upstream end is often provided. This appears to be an acceptable method of operation, even though the lifting gear is under the reservoir water for most of the year (see Figure 2).

The massive size of some of the gates (12m wide and weighing up to 125t) requires the provision of much larger gate gantries than would be normal on dams in the West. A typical example is that for Kayrakkum dam, where two gantries are provided, both having a lifting capacity of 250t.

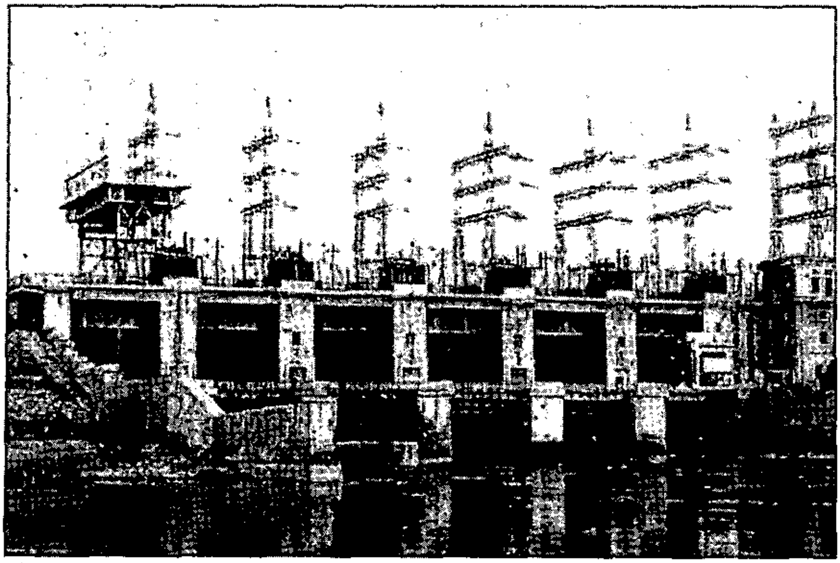

Figure 4 Kayrakkum Dam

DAM OPERATING CONDITIONS

Development of the large dams in Central Asia was conceived on the principle that operation would be primarily for the purpose of regulating river flows to meet irrigation demands throughout each of the catchments. The dams on the Syr Darya, principally Chardara, Kayrakkum, Toktogul (Naryn tributary) and Andijan (Kara Darya tributary), are reported to be capable of fully regulating all flows on that river. Nurek, with other dams existing and planned, would serve the same purpose on the Amu Darya. Hauzhan and Kopetdag provide online regulation for the Karakum main canal in Turkmenistan. Of the other dams inspected, Chimkurgan, Akhangaran and Bugun each serve local irrigation requirements. Only Uchkurgan, although originally developed for irrigation, is now used primarily for hydropower because the reservoir is almost completely filled with sediment.

Reservoir details for the dams that were inspected are shown in Table 2.

Table 2 Details of Reservoirs and their Catchments

Name	Catch-ment Area km^2	Mean Annual Precip. mm	Mean Annual Runoff Mm^3	Reservoir Storage Volumes			Mean Annual Flood m^3/s
				Dead Storage Mm^3	Live Storage Mm^3	Flood Storage Mm^3	
Akhangaran	1,290	810	720	13	185	N/A	175
Bugun	2,160	280	132	4	363	N/A	55
Chardara	174,000	350	17,011	1,000	4,700	800	2,053
Chimkurgan	5,590	470	800	50	450	50	161
Kayrakkum	136,000	350	16,411	2,140	2,600	800	1,784
Nurek	30,700	750	20,198	6,000	4,500	nil	2,345
Toktogul	52,500	360	12,087	5,400	14,100	N/A	1,593
Uchkurgan	58,200	380	13,529	40	16	N/A	1,579

Notes
1. Hauzhan & Kopetdag have only small local catchments and are omitted
2. Reservoir storage volumes quoted are before reduction by sedimentation
3. Mean annual flood discharges are for period of record at time of design

Following the break-up of the Soviet Union in 1991, the management of water resources in general, and the dams in particular, has created tensions between the neighbouring states. This is hardly surprising considering that the two largest dams, Toktogul and Nurek, which are important sources of hydroelectric power as well as being the main regulators for the two rivers, are operated by the state power companies of respectively Kyrgyz Republic and Tajikistan. Although the main area of dispute has been over the conflicting interests of hydropower and irrigation, the potential for unsafe operation of the dams, particularly in the event of a major flood, has tended to be overlooked.

Some of the other factors that are relevant in considering the safety of the dams under present circumstances and operating conditions are described below.

<u>Flood Estimation</u>

Floods generally occur in the period April-June and are the product of the combination of snow melt, glacial runoff and rainfall. Soviet design codes (SNIPS) require that dams are designed to accommodate flood discharges that are estimated by the statistical method. According to the codes, dams are ranked in four classes, depending on their size and potential risk, and design flood return periods are allocated to each class as follows: -

Class of Dam	Annual Exceedance Probability (AEP)	Return Period of Design Flood - Years
1	0.0001	10,000
2	0.001	1,000
3	0.01	100
4	not specified	not specified

In practice, outlet structures are designed using the exceedance probability hydrograph of the next lowest class but are checked for compliance against the exceedance probability hydrograph for the class to which they belong. Most of the dams inspected are Class 1 although a few are Class 2.

A particular problem with the statistical method of flood estimation is that the magnitude of the design flood can vary as the length of record extends. Consequently, the required design discharge capacity for several of the dams has gone up since they were first constructed, although there have been no funds available to implement the changes.

Flood Management

As noted previously, flood management for all of the dams depends on using part of the reservoir storage capacity to attenuate peak discharges. For example, Akhangaran Dam, which is designed for a peak inflow to the reservoir of 1,350 m^3/s, has an outlet capacity of only 400 m^3/s. Observance of operating rules that mandate the reservation of this flood storage capacity is therefore a key factor in ensuring the safe operation of the dams. Furthermore, during the Soviet period a central agency was responsible for fixing reservoir levels in advance of the flood, based on recorded snow conditions in the catchment and estimates of volume of runoff. Although the staff and facilities are still available to make such estimates, the fragmentation of the meteorological services has meant that there is now no single source of meteorological data, and, indeed, data collection has ceased in many parts of the catchments, particularly the remote mountainous areas.

Figure 5 Nurek Dam

Reservoir Sedimentation

Deposition of sediment is already a problem for the operation of several of the dams and the size of the problem will increase as the sediment deposits grow. There have been very few recent bathymetric surveys and although suspended sediment sampling is carried out at selected hydrometric stations on the rivers, the inter-relationship between sediment transport and deposition is not well understood. The currently available information for eight of the dams is shown in Table 3.

Table 3 Available Sediment Data for the Reservoirs

Name	Date of First Filling	Date of Lastest Bed Survey	Sediment Volume		Sediment as % of Original Volume	Catchment Sediment Yield t/ha/year
			At Time of Survey Mm^3	Current Estimate Mm^3		
Akhangaran	1966	1996	26	28.5	10%	8.5
Bugun	1963	None	-	39.5	9%	5.9
Chardara	1968	1977	581	1,859	31%	4.3
Chimkurgan	1961	1996	100	111	21%	6.6
Kayrakkum	1956	1969	313	1,059	24%	2.3
Nurek	1972	1989	1,840	2,862	27%	43.3
Toktogul	1965	none	-	584	3%	4.1
Uchkurgan	1961	1975	42	45	83%	3.9

The rate of sediment deposition in Nurek reservoir is causing particular concern, since the foot of the foreset slope is believed to be now within about 10 km of the power station intakes.

DAM SAFETY ISSUES

None of the dams is regarded by the authors as currently meeting normally accepted safety standards for dams of this magnitude and potential hazard. The concerns are categorised in terms of: -

1. Conceptual concerns
2. Operational concerns
3. Design issues
4. Financial constraints

Conceptual Concerns

These generally relate to Soviet era design practice, including the design procedures that were promulgated under Soviet design codes and standards (SNIPS). Particular examples are: -

- the statistical method of flood estimation, relying as it does on extrapolation from short periods of record, and whether this provides a sufficiently reliable estimate for the magnitude of extreme flood events,

- whether the size of flood adopted for design under the Soviet era codes is still appropriate,
- the assumptions made for passing the flood, particularly the acceptance of the use of the power station and other low level outlets,
- the extent of built-in redundancy in the hydro-mechanical equipment and whether there is sufficient capacity to allow for the possibility of equipment being out of operation when an extreme event occurs.

Operational Concerns

The concerns generally reflect the present circumstances where central control of the dams and their reservoirs is no longer practised. In particular there is a need to reach agreement between the neighbouring states for legislation to ensure the safe operation of the reservoirs, quite apart from agreements now being discussed for the sharing of use of the reservoirs. The legislation needs to take into account that the flood plains below the dams have been developed, and that the downstream communities have come to expect the dams to provide them with protection against flooding.

Another important operational concern is the need to verify the performance of hydro-mechanical equipment under design conditions by carrying out full-scale tests under controlled conditions. For example, the spillway and low level gates at Nurek have never been tested at full opening under full reservoir conditions, because of the damage that this would cause downstream.

For all of the dams, a strategy needs to be developed for managing the sediment in the reservoir, particularly as the foot of the sediment delta approaches the low level outlets of the dam.

Design Issues

Design philosophy during the Soviet period does not appear to have recognised that designs and design methods need to be reviewed and upgraded in accordance with changing world practice, particularly in respect of earthquake survival and flood management. The authors are particularly concerned by the procedures used for design against earthquakes. According to the Russian Seismic Standard, a seismic design coefficient (k_g) is derived for a site based on the Medvedev, Sponheuer & Karkik (MSK) earthquake intensity scale. The coefficients are derived based on a 1:500 year earthquake. The required minimum factor of safety in seismic condition is always greater than unity.

However, the current world practice based on the guidelines given in ICOLD Bulletin 72 is to assess dam safety against two representative design earthquakes that are as follows:

OBE	-	Operating Basis Earthquake, or "no damage earthquake"
MDE	-	Maximum Design Earthquake, or "no failure earthquake"

In addition, a particular deficiency is that little is known about the way that embankments will perform under earthquake shaking. It is well known that embankments constructed of hydraulically placed fill are particularly susceptible to liquefaction under earthquake conditions.

Financial Constraints

Expenditure on routine maintenance and replacement has been severely limited during the last 20 or more years to the point where there is now no assurance that equipment will operate as intended during emergency conditions, and reading and analysis of safety monitoring instrumentation has, in many cases, all but ceased. Many of the instruments are now no longer working.

EMERGENCY MITIGATION MEASURES

The authors have recommended immediate action under the WAEMP to replace defective monitoring instrumentation and improvements to the early warning and emergency preparedness procedures. A programme of surveys, investigations, inspections and studies, to improve basic knowledge of the dams in preparation for the design of such structural improvements as may be necessary, has also been proposed. These measures are all directed at reducing the present level of risk.

ACKNOWLEDGEMENTS

The authors would like to acknowledge the considerable support and assistance that was provided to them in Central Asia during the visits to the dams, in particular by the GEF Project Agency Component 'C' Director, Mr. Kurban Ballyev, and his assistant, Mr Alexander Enin. The authors also acknowledge the very valuable data and other information about the dams that was collected and made available by teams of experts in each of the countries, and by the regional specialists based in Tashkent.

Dam safety in Kyrgyz Republic

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SYNOPSIS. Prior to the disintegration of the Soviet Union in 1991 many large dams, including some of the world's highest, were constructed in what are now the Central Asian republics of Kazakhstan, Uzbekistan, Kyrgyzstan, Turkmenistan and Tajikistan. Funds for their maintenance have not, however, been available in recent years and this has given rise to serious concerns for their safety by the Governments concerned and by the World Bank. This paper describes ongoing work to achieve an acceptable level of safety for irrigation dams in Kyrgyzstan.

INTRODUCTION

The World Bank programme for the safety assessment and rehabilitation of dams in Central Asia started with a reconnaissance visit to the Kyrgyz Republic in 1997 in connection with the Irrigation Rehabilitation Project. This was followed by the appointment of Consultants (Temelsu/GIBB Joint Venture) at the end of 1997 to carry out safety assessments and to design rehabilitation works for selected irrigation dams. By early 2000 safety assessments had been carried out for seven dams and contracts for investigations or remedial works awarded for two of them. This paper describes significant design features and safety issues relating to a number of the more important dams.

COUNTRY BACKGROUND

Kyrgyzstan is one of the most attractive of the former Soviet Central Asian republics, and its capital Bishkek (population c.700,000) a pleasant city with tree-lined boulevards overlooked by permanently snow covered mountains. The country is bordered on the North by Kazakhstan, on the West by Uzbekistan, on the South by Tajikistan and to the East by China. With a population of some 4.5 million it comprises a land area of 198,500 km^2 of which over 90% is mountains. The average elevation of the country is 2750 masl and about 40% is over 3000 masl; of this three-quarters is under permanent snow and glaciers, giving some spectacular scenery.

About 80% of the discharge of the rivers in Kyrgyzstan derives from glacier melt-water and snowmelt, with the highest flows occurring generally in the period April - September.

Dams 2000, Thomas Telford, London, 2000.

The principal water resource of the country is the Naryn river which drains a catchment of 53,700 km^2, 27% of the area of the country, and flows into the Syrdarya river in the Fergana Valley, through parts of Tajikistan and Uzbekistan, into Kazakhstan and ultimately into the Aral Sea. Much of the north of the country drains to the Chui river, with a length of 220 km in the country and a mean flow of 70 m^3/s. The Naryn river is also the principal source of hydroelectric power in Kyrgyzstan with numerous large dams already constructed, including the Toktogul gravity dam (215 m), and there are plans for future dams to form an almost continuous cascade over its 500 km in the country.

Geologically, the Kyrgyz Republic is situated within a zone of young, seismically active mountains, and large earthquakes occur widely. In 1911 there was a magnitude 8.2 event about 100 km north-east of the Orto Tokoi damsite.

Glacial lake outbursts are hazards in rivers arising at high elevations and can give rise to flows greatly in excess of extreme rainfall generated events. For example, the 100 year return period flood on the Isfairamsai river 100 km west of the town of Osh is about 200 m^3/s, but in 1966 there was a glacial lake outburst giving rise to a peak flow of about 1700 m^3/sec. A similar event on the Sahri-Mardan river in 1998 is reported to have claimed a number of lives and to have destroyed irrigation infrastructure. Of the reservoirs studied that most at risk appears to be Papan where there are 160 km^2 of glaciers upstream.

Although data are scarce it is reported that glaciers in the Kyrgyz Republic are typically receding by as much as 5 metres per year. There is also some evidence that the percentage of total seasonal and annual precipitation occurring in daily rainfall events in the area has been increasing over the period 1935 to 1989 (Houghton et al, 1996). How, or whether, this should be taken into account in assessing flood safety is open to question, but it does suggest that a conservative approach is appropriate.

DAMS

Thirteen irrigation dams were included in the initial reconnaissance study, of which seven are included in the present stage of the rehabilitation programme.

The only concrete dam in the programme is the 86 m high **Kirov** hollow gravity dam in the northwest of the country near the border with Kazakhstan. The dam has twelve round-headed buttresses at 22 m centres with web thicknesses of 12 m. It is reported that when it was built the dam was unique in the Soviet Union and its performance attracted considerable interest in academic circles. The dam includes a crest spillway with a single radial gate and two low level outlet pipes built into adjacent buttress webs each controlled by a 2.2 m diameter cone valve (capacity 90 m^3/s each).

The dam was completed in 1975 and since that time has been operating with remarkably few problems. One of the cone valves is damaged, though still operating, but the other cannot be operated as it causes severe erosion damage to the embankment supporting the access road to the downstream toe of the dam. Proposed work includes the rehabilitation of the cone valves and of the monitoring instrumentation within the dam, and upgrading the instrument readout system.

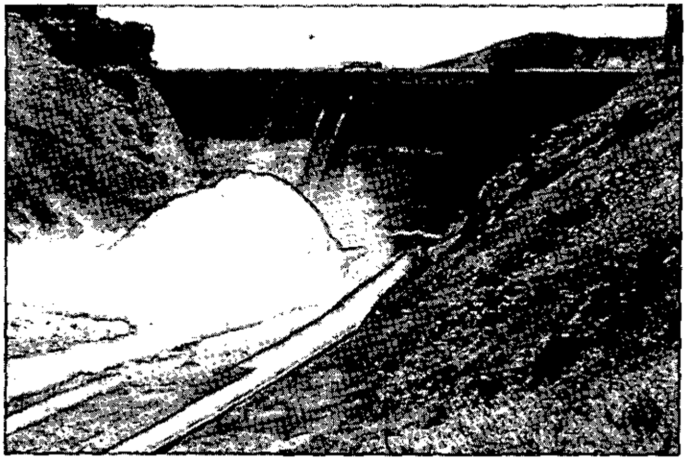


Figure 1: Kirov hollow gravity dam

Papan dam is situated in the south of the country about 20 km south of the town of Osh in a deep, narrow and extremely steep sided limestone gorge. There is a population of over 500,000 persons downstream. The dam is an embankment about 100 m high and 90 m long at crest level and impounds a reservoir of 260 Mm^3. It is constructed of widely graded alluvial sands and gravels taken from the reservoir area, with a cement/clay grouted core. Most of the fill was transported into the site through access tunnels.

Ancillary works comprise a 500 m long, 6 m diameter low level draw-off tunnel in the left abutment with two outlets, each controlled by two 2.75 m wide x 4.6 m deep gates in the base of an upstream draw-off tower anchored to a near vertical cliff. The draw-off tower also incorporates two 9 m long spillway weirs discharging into the main tunnel by way of a steeply inclined shaft. The maximum discharge capacity of the draw-off is 260 m^3/sec and in combination with the spillway the system is designed to pass a maximum total flow of 345 m^3/sec. To date the draw-off flow has never approached this value and has not been tested at full capacity, largely because it would cause serious flooding downstream. The spillway itself has never operated. In 1993 serious damage, thought to be due to cavitation, was sustained by the steel lining of the tunnel immediately downstream of the gates. This was repaired and the problem has not recurred. There are, however, vibrations which are measurable on the dam and which are attributed to operation of the draw-off at flows exceeding about 20 m^3/sec.

Flood routing studies have indicated that an extreme flood can only be controlled if the draw-off and spillway are both discharging at maximum capacity, and there is some doubt as to whether the system in its present state can be relied on to operate in this way for a sustained period. Clearly, if one outlet is out of action the ability to control a large flood would be seriously impaired.

When the reservoir level exceeds about 10 m below full storage level a series of well defined damp patches appear on the downstream slope, and drainage from the low level grouting adit increases abruptly from a few l/sec to over 100 l/sec.

There are very few functioning piezometers in the dam, but those which are still in operation indicate that the phreatic surface in the downstream shoulder is well above the design level, and rising. Back-analysis of the seepage pattern suggests that the grouting of the core zone has been largely ineffective. Although calculated hydraulic gradients in the embankment are not particularly high the grading of the gravel is such that it is probably not self-filtering, and the possibility that long term internal erosion (suffosion) is occurring under the influence of continuous seepage cannot be ruled out; this has important stability implications.

View Downstream

Upstream Face

Figure 2. Papan Dam

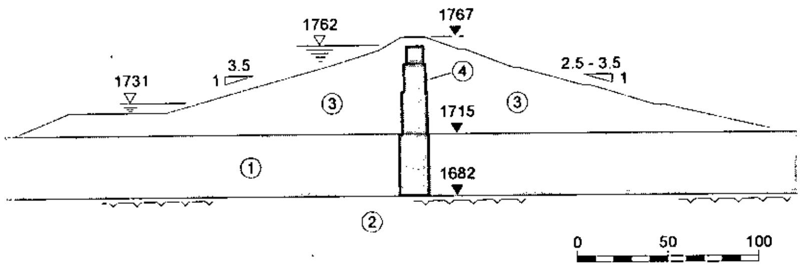
Rehabilitation works which are planned include investigations to study the fill and the grouted core zone, to be followed by modifications aimed at improving the watertightness of the core (probably by chemical grouting), reducing vibration at the draw-off gates and improving the instrumentation system, principally by installing additional piezometers. The purchase of a standby generator is an urgent requirement, to ensure that the gates can be opened in an emergency since the existing transmission line is vulnerable to rockfalls.

In view of the present problems and safety risks and pending completion of investigations and remedial works the water level in the reservoir is being restricted to 10 m below full storage level.

Orto Tokoi dam is a 52 metre high gravel embankment also with a grouted core zone, a surface spillway on the left bank and a draw-off tunnel through the right abutment. The draw-off tunnel incorporates twin 2.45 m x 2.75 m slide gates at the base of gate shaft near the upstream end. Downstream the tunnel bifurcates, the flow in each arm being controlled by a 2.2 m diameter hollow cone valve (total flow capacity 140 m^3/s).

Construction was completed some 40 years ago in the late 1950s and, apart from a major embankment grouting operation to reduce seepage carried out in 1960 – 63 has been operating satisfactorily ever since. However, there are a number of causes for concern in that the spillway (which it is understood has never operated) is in poor condition due to disintegration of concrete and a mudslide which has buried the flip bucket; the cone valves have deteriorated and there is possibly dangerous undercutting of the valve house. There is deterioration of the two upstream slide gates and the steelwork in the gate shaft, and there are defects in the draw-off tunnel concrete lining.

Planned rehabilitation works comprise refurbishing the cone valves, stabilising the foundations of the valve house, grouting sections of the draw-off tunnel lining and refurbishing and strengthening the spillway chute. The spillweir will be shortened from 40 m to 20 m to limit the outflow to the capacity of the chute (275 m^3/s) by exploiting the available reservoir freeboard to increase the flood storage.



① River bed alluvium
③ Sand/gravel fill
② Bedrock
④ Grouted zone

Figure 3. Cross-section of Orto Tokoi Dam

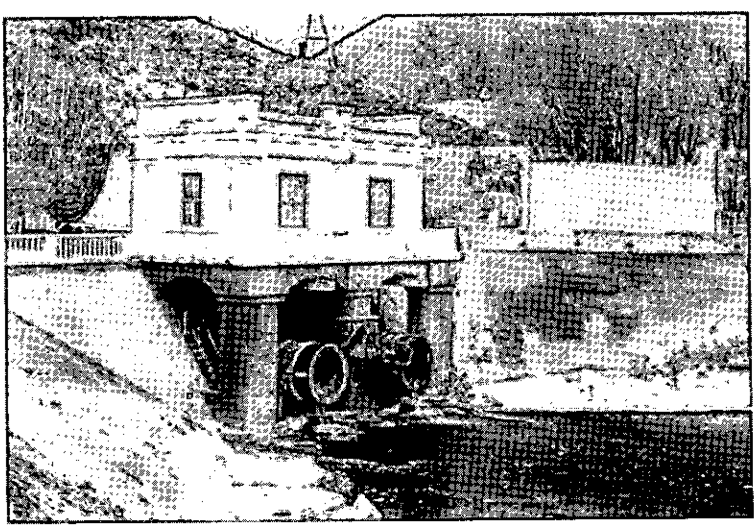


Figure 4. Orto Tokoi Dam – Downstream Valve House

Construction of the **Karabura dam** was started in 1985 but work was suspended shortly afterwards due to lack of funds; it is planned to complete it to a slightly lower height (50 m) and with a redesigned spillway. The partially completed dam has been overtopped by floods on several occasions but so far has suffered no serious damage.

Ala Archa (35 m) and **Spartak** (15 m) dams are both comparatively low homogeneous embankment dams constructed of sandy silt loess on loess and alluvium foundations. The loess material is highly susceptible to collapse on saturation and large scale trials were carried out on site in test pits before construction to ascertain the required depth of surface material to be removed to reach an acceptably stable material for the embankment foundations. Surface settlements of typically well over one metre were obtained on flooding the test pits with water. Neither Ala Archa nor Spartak dams have surface spillways, but for flood control rely wholly on their irrigation draw-offs, both of which require considerable repair work.

Bazar Kurgan dam (25 m), constructed in the mid 1960s, is a composite loess silt and gravel embankment, also founded largely on loess. It impounds an off-river reservoir and is fed by canal, though it does have a substantial independent catchment. Superficially the embankment appears to be sound, apart from the poor state of the concrete wave protection slabs on the upstream face. It does, however, have a long history of settlement problems, probably attributable to the loess foundations, and is reported to have suffered a washout of a short section on one flank shortly after commissioning. The electrical and mechanical equipment is clearly nearing the end of its useful life. The electrical gate actuators are all out of order and gate operation is wholly by hand. Following early reservoir leakage problems much of the reservoir bed was covered with a plastic membrane in 1975 which appears to have improved, though not totally cured, the problem. The dam does not have a surface spillway but relies on the irrigation draw-off for flood control, and to ensure the ability to control an extreme flood a reduction in the normal reservoir full storage level has been recommended.

DISCUSSION

On the basis of safety assessments carried out in dams associated with the Kyrgyz Irrigation Rehabilitation Project and dams in others of the former Soviet Central Asian republics certain general conclusions have been reached.

1) Design generally

   All the dams in the former Soviet Union republics have been designed in accordance with Russian Standards (SNIP). These standards are highly prescriptive and appear to have resulted in generally safe and sometimes innovative designs.

2) Seismic design

Seismic stability analysis is carried out in accordance with the Russian Seismic Standards, according to which a seismic design coefficient is derived for the site relating to an earthquake with annual exceedance risk of 1:500 (0.2%), which is reduced by a factor depending on whether permanent deformation is permissible. The minimum factor of safety by this method is required to be greater than unity.

The current approach in the west, however, (ICOLD Bulletin 72) is to assess the safety of the dam against two design earthquakes:

OBE Operating Basis Earthquake
MDE Maximum Design Earthquake

Where OBE ('no damage earthquake') has an occurrence risk of not more than once during the lifetime of the structure (>100 years), during which the structure should remain fully functional with a factor of safety ≥ 1.0, and MDE ('no failure earthquake') is the most severe ground motion which the structure should be able to withstand without catastrophic failure. For high risk structures an annual exceedance risk of 1:5,000 is recommended for MDE.

Independent assessments of the seismicity of the dam sites have been carried out and the dams have been checked against these criteria. Despite the very different approaches the results have been found to be comparable, with factors of safety of greater than unity for OBE, and tolerable deformations for MDE conditions.

3. Liquefaction

Russian standards do not appear to allow for any possible loss of strength due to liquefaction under earthquake shaking. In all cases where this may be a risk, therefore, soil testing for a liquefaction assessment has been recommended, to be followed by a review of the stability implications. This applies particularly to those embankments constructed on and of loess material.

4. Performance monitoring instrumentation

Basic instrumentation, normally comprising simple standpipe piezometers and settlement markers, is provided in all embankment dams, but it is found that after 15 – 20 years the majority of the piezometers have ceased to function.. Settlement measurements have generally ceased due to lack of survey equipment.

5. Spillways

It appears to have been the practice in the former Soviet Union to assume that all available draw-offs, including turbines in hydropower dams, could be utilized at full capacity for the control of floods. This is an optimistic approach which is unlikely to find acceptance in the west.

Of the seven dams inspected so far only three have surface spillways, and flood routing studies using independent estimates of the Probable Maximum Flood (PMF) have indicated that for all but one an extreme flood can only be controlled if all gates are in working order and can be operated for a sustained period at full capacity. To reduce the risk of overtopping in the event that some outlets are not operable it has been recommended that the provision of surface spillways should be studied.

6. Flood flows

Floods are mainly derived from snowmelt and peak flows are low compared with other areas of the world.

Table 1 gives key parameters for the largest of the irrigation reservoirs in the Kyrgyz Republic. Creager values (Ref. 2) for peak flood flows are calculated for the PMF inflows using the formula:

$Q = C \times 1.303 \times \left[\frac{A}{2.588}\right]^{\left(\frac{0.936}{A^{0.048}}\right)}$, where Q is the PMF inflow in m^3/s, C is the Creager Coefficient and A is the catchment area in km^2.

Reservoir	Reservoir Volume Mn^3	Dam Height m	Catchment Area km^2	Available Discharge Capacity m^3/s	PMF Inflow m^3/sec	Creager Coefficient C
Papan	260	100	2540	345	650	6.0
Orto Tokoi	470	52	5970	257+140	860	5.6
Kirov	570	86	8200	210+180	845	4.9

Table 1. Key Parameters for Large Irrigation Reservoirs in Kyrgyz Republic.

It will be seen that Creager values are remarkably low in comparison with values of 30 to 100 which would apply in much of Europe and North America (Creager et al, 1945).

7. Draw-down rates

All the dams inspected have irrigation outlets (usually a pair) of sufficient capacity to achieve a reservoir draw-down rate of at least 0.5 m/d (assuming no inflow), which is considered to be adequate for emergency draw-down. Checks carried out as part of the safety assessments indicated that this rate of draw-down would not endanger embankment stability. However, the operators are generally reluctant to use, or test the outlets at full capacity as it causes flooding downstream.

CONCLUSIONS

In general, the irrigation dams studied in Kyrgyzstan were found to be of sound design and robust construction, though a considerable amount of repair and maintenance, particularly of the instrumentation and the hydromechanical equipment is now needed. There is, however, some concern that the flood discharge criteria are based on optimistic assumptions regarding the use of gated draw-offs to control extreme events. Kyrgyzstan is a highly seismic area, and seismic behaviour is a matter of some concern, particularly where soils are susceptible to liquefaction

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Monitoring and planning mudflow control works following Mt Pinatubo eruption, Philippines

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SYNOPSIS. The 15 June 1991 eruption of Mt Pinatubo in the Philippines caused lahar and mudflows due to heavy rain falling on the volcanic ash filled rivers surrounding the mountain. Six years after the eruption, lahars still posed a great danger to life and property in the low-lying areas. During 1997, the Philippine Government initiated a major follow-on study of the lahar problems in the Pasig-Potrero River basin. This study included monitoring the lahar problem through one rainy season, and developing designs for works to control sediment. At a cost of US$ 200 million at 1997 prices, the resulting designs were economically justified.

INTRODUCTION

The Eruption

Mt. Pinatubo erupted on 15 June 1991. It was the world's largest eruption in more than half a century and probably the second largest of the 20th Century. The volume of volcanic debris that was produced is an order of magnitude greater than the volume generated by Mount St. Helens in 1980. The explosive eruption produced ash clouds that reached 40 km high (Fig.1), blasted 300 m from the mountain's original summit, and formed a crater that is some 2 km wide and 600 m deep. Eight major river basins: Pasig-Potrero, Sacobia-Bamban, Abacan, O'Donnell, Santo Tomas, Bucao, Maloma, and Gumain-Porac were filled with volcanic ash and debris. Secondary pyroclastic flows, some leaving deposits up to 10 km long, occurred for more than two years after the eruption.

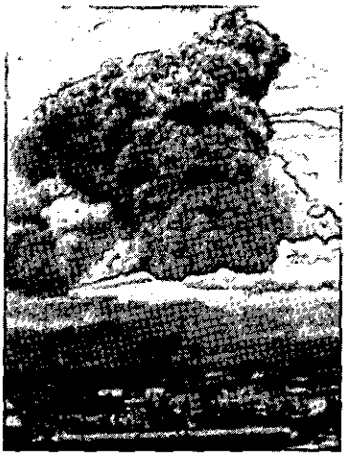

Figure 1 Pinatubo Eruption

The volcano affected global weather patterns and locally, caused devastation and misery through lahars and mudflows when heavy monsoon rains fell on volcanic ash recently deposited in the rivers valleys. The lahars posed a great danger and caused the loss of many human lives and property. The lush countryside in the surrounding areas was turned into an inhospitable grey desert.

Dams 2000, Thomas Telford, London, 2000.

Lahar

A lahar is defined as a rapidly flowing mixture of rock debris and water, other than normal stream flow, from a volcano. Lahars differ considerably from normal flow events due to the extreme sediment concentrations and increased erosiveness of the flow. Volumetric sediment concentrations of between 50 and 75% have been recorded. The flow resembles fluid concrete more than water. The high velocities, up to 10 m/s, can also cause the flows to super-elevate at bends by 2 m or more. Fluid bulk densities of debris flow slurries typically range from 1.8 to 2.3 g/cm^3, giving volumetric sediment concentrations in the range of 50 to 75 per cent. Fine grained debris flows in confined channels can travel along slopes of only 0.015. Deposition of fine grained debris flows can occur on slopes of between 0.006 to 0.02, (USACE, 1994).

A lahar was described graphically by Rodolfo, a local geologist, in 1996:

> 'Pulses or surges of muddy streamflow were the precursors of incoming lahars. These surges had minimal sediment concentrations, generally less than 15 per cent by volume. A gradual increase in flow depth and increased surge frequency signalled the onset of hyperconcentrated flows, accompanied by strong lateral erosion at typical rates of 0.5 to 3 m per minute and the formation of standing waves as high as 2 to 3m and 15 to 20 m long. As a hyperconcentrated lahar progressed in intensity and discharge, its standing waves started to break, roaring continuously like ocean surf and the flow became very turbulent and erosive. The flow no longer cut sideways so strongly, instead it cut downward into the channel bed. The material eroded from the bed became part of the lahar, making it more dense and energetic, so it could flow faster. The increased density and speed of the lahar allowed it to incise the channel more deeply. Velocities at this stage ranged from 3 to 6 m/s.'

Lahars can be triggered by several mechanisms, including intense rainfall, breaching of temporary lakes in the headwaters or landslide activity.

Foreign Assistance

Following the eruption, help was immediately offered by many countries including the US Government who funded relief efforts and studies into the eruption and methods of mitigating its effects, (USACE, 1994). Later, Japan International Co-operation Agency (JICA) funds were made available to continue the studies and begin rehabilitation. A Master Plan and Feasibility Study to mitigate the hazard posed by the lahars was completed in March 1996, and the Government of the Philippines applied for an OECF loan package from Japan for implementing the so called the Pinatubo Urgent Hazard - Mitigation Project. The package included data collection, survey and investigation for the 'Monitoring and Planning of Lahar/Mudflow Control Works in the Pasig-Potrero River Basin' project that started in 1997 and is discussed in this paper.

MONITORING OF LAHAR IN THE PASIG-POTRERO RIVER BASIN

The Project Area

Mount Pinatubo is situated approximately 100 km north-west of Manila in the Zambales Mountain Range on the west coast of Central Luzon (Fig. 2). It is one of a chain of volcanoes that constitutes the Luzon volcanic arc. The arc parallels the west coast of Luzon and reflects eastward dipping subduction along the Manila Trench to the west.

Figure 2 Location Map

Mount Pinatubo is a composite andesitic volcano constructed upon older sedimentary and ultramafic strata. Underlying older volcanic rocks consist mostly of andesitic agglomerates, tuff breccias, and tuffaceous sandstones interspersed with andesitic or basaltic flow rocks. Before its eruption, Mount Pinatubo was among the highest peaks in west-central Luzon. Its former summit consisted of the crest of a lava dome that rose about 700 m above a broad, gently sloping, deeply dissected apron of pyroclastic deposits. Older volcanic relics, including an ancestral Mount Pinatubo, lay south, east and north east of the peak. As a result, Mount Pinatubo was relatively inconspicuous. However, the mountain's lower flanks, composed mainly of thick pyroclastic deposits, were a testimony of past explosive episodes.

The Catchment

The Pasig-Potrero River drains the eastern slope of Mount Pinatubo as shown in the location map, Fig. 2. The river starts just below the summit of Pinatubo with deeply incised headwaters, and ends in a gently sloping alluvial fan that extends beyond the flanks of the mountain, across a multi-threaded delta area to Pampanga Bay, north west of Manila.

Alluvial fans form when rivers emerge from a confined mountain valley and spill onto a flatter unconfined area. Rapid deposition occurs as a result of the decrease in channel slope and flow velocity. This deposition can cause the channel to become perched above the adjacent ground, which can trigger sudden channel shifts or avulsions. Consequently, alluvial fans are characterised by unstable, multiple channels that may shift from one side of the fan to the other. Fan head trenching near the fan's apex occurs under certain conditions. This river bed degradation can occur during extreme rainfall events because of high channel velocities or erosive debris flows near the apex. Fan head trenching can also develop in response to a gradual decrease in sediment supply from the headwaters.

At the distal end of the fan, the river runs onto a low-lying alluvial plain and flows across the coastal marshes into Pampanga Bay. This lowland flood-plain is dissected by a complex maze of branching channels and is tidal. In this reach, the velocity of flow is low and therefore the capacity of the river to carry sediment above silt size is low.

Lowland alluvial plains typically occur in areas of very flat gradients and relatively low sediment supply. Rivers commonly develop a branching channel pattern, with one or two dominant channels and many smaller distributors. Channel shifting can occur as a result of local sedimentation at distributor junctions, which can result in the main channel being plugged with sediment, causing the main flow to shift into the smaller distributors. Consequently, the pattern of channel instability involves periodic abandonment of some channels and re-occupation and widening of others.

Hydrology

Rainfall in the Pasig-Potrero Basin is highly seasonal, with a pronounced rainy season from June through October at which time tropical cyclones are prevalent. These generally cause the maximum daily rainfalls in the Mount Pinatubo area and about 90 percent of the Pasig-Potrero Basin rainfall occurs during this Southwest Monsoon period. Data available from the Philippine meteorological service indicate that between 1948 and 1991, an average of 16 tropical cyclones per year affected weather conditions at various regions in the Philippines. Typically, four or five of these cyclones affected weather conditions around Mount Pinatubo. Daily rainfalls of over 400 mm are not uncommon (USACE, 1994).

The direct passage of a typhoon over the area typically causes the largest one-day rainfalls. Tropical storm Mameng, October 1995, with gusts of up to 120 kph, caused heavy destruction through flashfloods and landslides, affecting a total of 758,000 people. Two weeks after the storm the death toll was 91 with 93 persons missing. The hardest hit area was Pampanga where more than 15 million cubic metres of lahar from the Pinatubo volcano swamped the towns of Bacolor, Cabalantian, and San Fernando. The storm also caused 20-foot floods in Pampanga affecting 20 towns and 460,000 people.

During tropical storm Luming, 19-21 August 1997, intense rainfall lead to a debris flow on the Pasig-Potrero which resulted in 18 m of degradation of the river at the head of the alluvial fan and deposition over a wide area downstream.

Pyroclastic Flow Deposits

The greatest changes in the basin after the eruption were produced by the huge quantities of pyroclastic flow materials. These searing hot ash avalanches flowed down the eastern slopes, travelling at high speeds under the influence of gravity. The deposits extended as far as 10 km from the vent and caused major landscape changes by filling entire valleys and transforming steep, deeply dissected terrain into broad, gently sloping plains. Approximately 430 million m^3 of pyroclastic flow material was deposited in the upper basin of the Pasig-Potrero River. Most of this material was deposited, up to 200 m thick, in a canyon just below the crater.

During the rainy season, secondary explosions caused by rising groundwater encountering the still extremely hot rock, left craters up to hundreds of metres in diameter, and destabilised the valley fill deposits. The great thickness of the deposits has caused diversion of surface drainage and blockage of tributary valleys. This resulted in the formation of unstable lakes that failed by breaching, creating large surges, channel incision, and then deposition further downstream in the intensely farmed fan and delta regions.

One major change in the catchment was the capture of 21.3 km^2 of the Sacobia River Basin by the Pasig-Potrero River. This will mean that, even without the effects of sediment, the character of the Pasig-Potrero River will be changed with increased flow and energy. Given the large amount of unstable sediments remaining in the basin, the future occurrence of temporary blockages and sudden break-outs of debris-dammed lakes was considered likely.

Sediment

Sediment deposits tend to occur in discrete lobes, with a narrow U-shaped channel at the centre of the lobe. Individual lobes on streams draining Mt. Pinatubo typically range up to 400 m in width and from 5 to 8 m in thickness. This deposition pattern causes the fan topography to be relatively irregular, which can promote the development of avulsions during subsequent events.

At the start of 1997, surveys revealed the pattern of sediment deposition illustrated in Figure 3. At the head of the fan, the river had cut down up to 28 m and other areas had been raised by 18 m from 1990 levels. The pattern seemed to be one of down cutting at the head of the alluvial fan with the main sediment deposition zone moving downstream with time. However, a single major lahar event was capable of changing this pattern overnight.

The ash consists of 74% pumice and glass shards, 15% plagioclase and 9% hornblende with minor constituents of biotite, quartz, zircon, apatite and magnetite. Other eruption products included porphyritic, vesiculated white pumice and poorly vesiculated fine-grained grey pumice. The deposits are predominantly sandy in texture. The following size ranges are typical of the primary deposits:

Gravel (> 2 mm):	varies between 5 % to 30%
Sand (2 mm to 0.063 mm):	varies between 48% to 90%
Silt and clay (< 0.063 mm):	varies between 5 % and 22 %

Following major flow events in 1997, it was not uncommon to see cobble sized fragments of pumice floating down the river in the delta area. In other places pumice rich boulders, 2-3 m diameter, had been carried for several kilometres and left high up on otherwise sandy deposits.

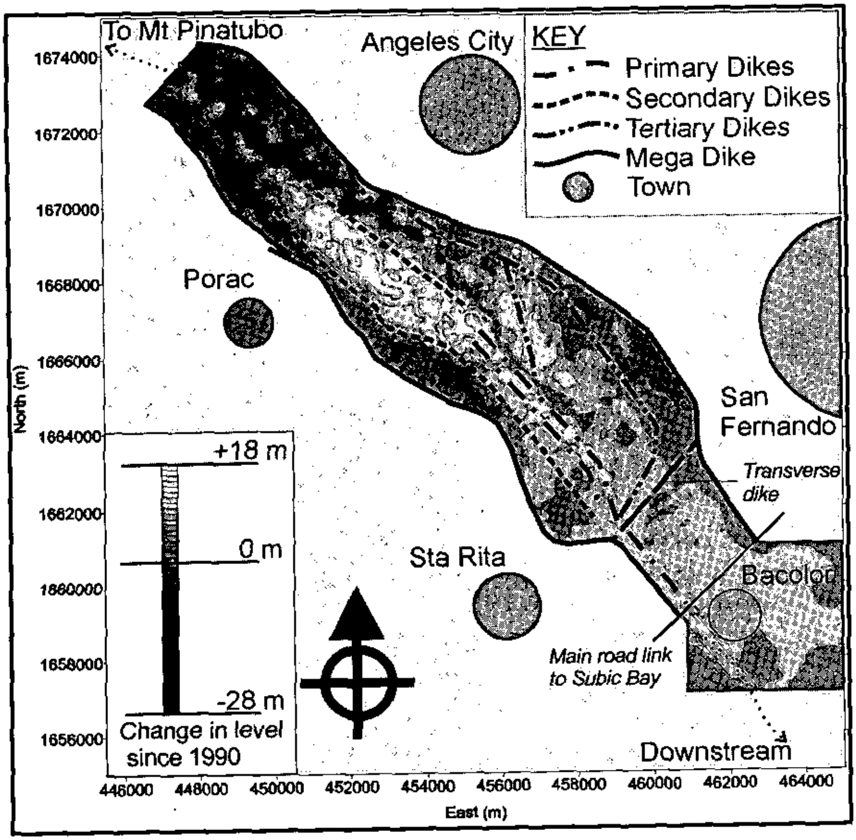

Figure 3 Sedimentation pattern within Mega Dike on the alluvial fan

EXISTING STRUCTURES

General

In order to assess the potential hazards to the communities close to the Pasig-Potrero River, the Study began with a review of the existing mudflow and flood control structures including structures from the pre-eruption period. Background data were collected, and then site inspections were carried out to determine the present condition, and likelihood of the restoration or rehabilitation of the structures for future use. The locations of these features on the alluvial fan are shown on Figure 3

Pre-eruption Measures

Pasig-Potrero River was an active system even before the eruption. Sabo dams (a term of Japanese origin that describes a debris retaining dam) had already been constructed to control sediment in the headwaters. River training works called the 'Primary Dikes' had been built to guide the river for 12 km between areas of farmland and conurbation at the downstream end of the fan to protect them from river bank breaches.

Post-eruption Measures

After the eruption, the Government and advisors from the various agencies struggled to keep the sediment engorged river in check. Through pressure of funds, limited time, and great distances to be protected, construction was at first limited to dikes constructed from the locally excavated lahar sand. Minimal compaction was carried out and no face protection applied. Despite this, the dikes were surprisingly effective until subjected to direct attack or undermined.

Between 1991 and 1992 the three sabo dams in the headwaters were washed out. One concrete dam, which had been constructed a decade earlier, had been undercut and left hanging between the valley sides 20 m in the air above the new river bed. Subsequently, deposition occurred in the area and, by 1997, the dam was buried up to its crest.

Between 1992 and 1996 a sequence of protection dikes were built, overtopped, and repaired or replaced. The space between the original 4 m high Primary Dikes was filled and these were supplemented by the 6 m Secondary Dikes designed to enclose and restrict the damaged area. During 1994 another set of enclosing dikes, the Tertiary Dikes, were required and other dikes were built between these to guide the flow away from sensitive areas. These dikes, up to 5 km apart, ran on two sides of a 13 km long section of the alluvial fan (Figure 3)

Mega Dike

After the 1996 rainy season, DPWH commenced construction of a 'final' outer dike, 8m high on average, confining the Tertiary Dike on the left bank. On the other bank, parts of the Secondary Dike and the Tertiary Dike were reinforced with concrete facing and raised to a maximum of 20 m. DPWH named this system of dikes the Mega Dike. It enclosed an area of 80 km^2. The strategy behind the Mega Dike design can be summarised as follows:

a) Construction of the dike itself to isolate the area already damaged;
b) Construction of a dike between the east and west Mega Dikes called the Transverse Dike, to form a 12 m deep pond during floods and thereby create an efficient sediment trap;

c) Excavation of a low flow channel to establish the route of the river and safely drain normal flows;
d) Utilisation of the main river channel to drain extreme floods;
e) River improvement and dredging of the lower Pasig-Potrero River.

The Transverse Dike was originally constructed with 3 spillway sections. The crests of these 10 m high structures were two metres below the main dike but they were constructed with three rows of six 750 mm steel pipes at intervals up the embankment. The embankment was constructed from lahar sand with a thin concrete skin. There was little in the way of seepage protection or energy dissipation provided, and in August 1996 the eastern spillway was breached causing severe damage downstream. At the start of 1997, the concept of forming a pond behind the Mega Dike had been dropped due to political pressure and two of the spillways were lowered to only 1 m above bed level.

HAZARD ASSESSMENT AND SEDIMENT RETENTION STRATEGY

Hazard Assessment

A hazard assessment was carried out taking into consideration the condition of existing structures and the behaviour of the Pasig-Potrero River. The categories used are described in Table 1 below.

Table 1 Risk categorisation

Level of Risk	Description
Low	Community protected by competent existing structural measures, river apparently stable in distant channel
Moderate	Flaws in existing structural measures, river apparently stable in distant channel
High	Flawed or no protective measures, normal river course close to or impinging on structures

The assessment of existing risk showed that 7 significant communities fell in the high category, 4 in the moderate category and 2 in the low category.

Hazard Management Strategy

A strategy for sediment retention and hazard management was developed which considered socio-economic conditions, hydrology, and river geo-morphology. Existing structures were utilised as much as possible in order to minimise the capital cost.

In the Headwaters, the ideal was to prevent sediment generation, monitor events, and warn downstream inhabitants about hazards. Seeding or planting of suitable vegetation on the pyroclastic flow deposit fields was considered an effective method of reducing sediment generated by sheet and gully/rill erosion. Planting can also contribute to environmental re-generation of the area. Species appropriate for re-vegetation are robust perennials with vigorous and extensive root systems. However, such quantities of seed would be required that this measure was not practicable. In addition, construction of sabo dams had to be deferred to the long-term, when conditions become more stable and construction possible. These structures are intended to control potentially damaging sediment or debris in three ways:

a) retain existing sediment in the river bed,
b) trap sediment being transported downstream in the river;
c) detain sediment temporarily during peak flow events, thus attenuating sediment load.

The Alluvial Fan within Mega Dike was already designated an area for sediment storage and the policy was to divert lahar into expendable unfilled areas, raise and improve Mega Dike to reduce the chances of failure. The Mega Dike concept, to retain the sediment in an efficient trap before it reached the delta area, was good but it had not catered for the volume of sediment being mobilised. It is important to match the predicted sediment load with the storage available and acknowledge that the bulk of sediment storage must start at the downstream end of the allocated area. In steep areas of catchment, the effective storage volume provided by a given height of structure is less than a more gently sloping area. Once a storage area is filled, it is possible to build a new retaining structure upstream but in-effective to build one downstream and hope for sediment to be carried the extra distance. No new sediment retention structures were planned above the Transverse Dike until storage on the lower end of fan is depleted, and Mega Dike has been provided with adequate freeboard.

On the Coastal Plain, a 25 km^2 upstream area, which had already been damaged and was largely uninhabited, was allocated for storage of fine sediment. A shallow collection area, or sand pocket, was formed within a surrounding dike in an area of low gradient. Channels were to be excavated and enlarged to improve drainage. Sumps, or in-channel basins, created by dredging below the grade of the channel were considered but ongoing maintenance costs and the requirement for disposal areas to accommodate the dredged materials made them unattractive.

STRUCTURAL DESIGN

General

Feasibility level designs were prepared for the proposed structures to meet the objectives of the Hazard Assessment. For small scale structures standard design details were used as much as possible to avoid repetition. The design of larger scale structures was based on the results of calculation, incorporating relevant design criteria. Wherever possible, local materials and DPWH standard details were used.

The structural measures considered included: sabo dams, dredging, channel excavation, levees, weirs to control the level of the bed, groynes or spur dikes. The materials available for bank protection included vegetation, rock, concrete sandbags, gabions, rubble concrete, and cemented sand and gravel (CSG).

The design of measures to protect and raise Mega Dike is described below.

South West Corner of Mega Dike

After the August 1997 lahar event that left deposits up to 12 m thick in the south-west corner of Mega Dike, softening, erosion, cracking, and slope movements were observed at the downstream toe. In addition, on the river side of the dike, longitudinal cracks of the order of 50 m long appeared in the concrete slope protection near the mud line. Appraisal of these signs of distress suggested that they were related to seepage of groundwater from the saturated sediments deposited on the upstream side of the dike, and settlement of the embankment fill on saturation.

This section of Mega Dike is the tallest, and therefore subjected to the highest hydraulic heads. The dike is of homogeneous lahar sand with a small geofabric protected cobble toe drain. After the August 1997 event, the hydraulic head across the dike rose to about 12 m due to the new, saturated sediment on the upstream side. There was no standing water immediately adjacent to the embankment.

During the first half of September 1997, seepage was observed on the downstream face of the dike, approximately 4 m above the toe, well above the toe drain. This was leading to softening and erosion of the embankment material.

A slide of the downstream dike slope occurred in the latter half of September. The plane of movement appeared to be confined to within the embankment fill, rather than in the foundation, but this could not be confirmed by any sub-surface investigation.

Seepage

The finite element freeware 'SEEP2D', developed by the United States Army Engineer Waterways Experiment Station, was used to back-analyse the dike from the observations of seepage. Permeabilities were assessed from particle sizes and uniformity, and possible future remedial scenarios checked.

It was not possible to reproduce the increased level of the phreatic line in Mega Dike when isotropic permeability conditions were considered. Even when the permeability of the cobble toe drain was lowered to model a blockage, the phreatic line did not rise higher than 1.5 m above the downstream toe.

Construction of embankments in layers often leads to anisotropic permeability, with higher permeability horizontally than vertically, as described by Fell et al (1992). Ratios of horizontal to vertical permeability of 10:1 are not unusual and they can be as high as 100:1. This condition was likely in Mega Dike with fill placed in thick layers to achieve rapid construction. Such layering was visible in exposed sections of the Eastern Mega Dike further upstream.

Anisotropic conditions were modelled assuming a ten to one horizontal to vertical permeability ratio. Under these conditions, the results showed that the phreatic line emerged at approximately 4 m elevation above the toe and agreed well with the field observations.

Stability

Scoping analyses of stability were performed to assess the downstream slope of the existing dike under steady seepage conditions. A high phreatic surface and the soil parameters shown in Table 2 were assumed.

Table 2 Material properties

Material	**c´kPa**	**ϕ'**	**γ_{wet} (kN/m^3)**	**γ_{sat} (kN/m^3)**
Lahar fill:	0	40°	17	20
Loose lahar fill:	0	30°	16	19
Foundation:	50	32°	20	22

Under these conditions, a factor of safety of 1.6 is achieved against sliding. A value of 1.5 is commonly required in design for these conditions and it was therefore concluded that high piezometric pressures, although a concern, were not the primary cause of the movement during September 1997. The movement was attributed to the softened fill.

Remedial Measures

Measures were proposed to provide adequate drainage to intercept seepage and carry it away without loosening and eroding the dike toe. The dike does not need to be watertight, but the seepage has to be controlled. It was recommended that a layer of granular filter material be incorporated into the dike to intercept the phreatic surface before it emerges on the downstream face. When the embankment is raised in the long term, this drain will be incorporated into the dike as a sloping chimney drain (Fig. 4).

A well proportioned granular filter was preferred to filter cloth. The lahar deposits are known to contain fine volcanic ash particles and there have been reports that the ash has migrated to the face of filter cloth, clogging the pores. In addition, the cloth could rip leading to clogging of the drain or an opportunity for internal erosion. Buried filter cloth can not be inspected nor repaired and should be avoided in critical areas.

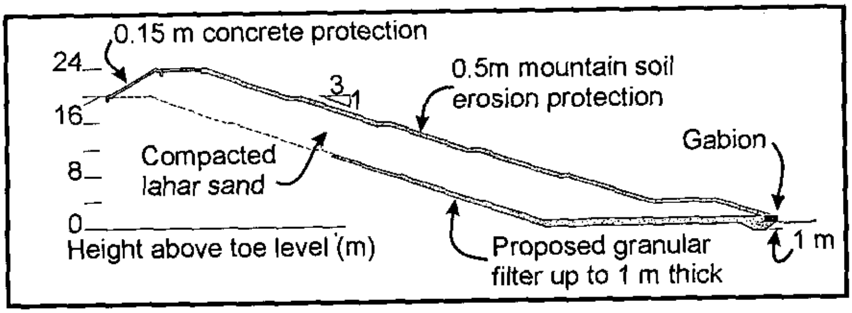

Figure 4 Mega Dike raising on downstream side of original embankment

CONCLUSIONS

The project was evaluated over a 30-year life and had an estimated net present value of approximately £50 million, the benefit to cost ratio was 1.45 with an economic internal rate of return (EIRR) of 19.4 per cent. The economic evaluation showed that the project was justifiable on financial grounds and it is difficult to see what choice the Philippine Government would have had given the political pressure to ease the suffering of the people of Pampanga and stop the environmental vandalism by lahar.

The plan developed provides an overall lahar/mudflow control but it will take time for implementation. The scheme is broken into four phases to meet funding restrictions and because of construction programme. The final design stage is currently underway. Urgent measures were sufficient to prevent a major breach of Mega Dike but the situation remains critically dependent on conditions in the Headwaters where lahars are generated.

ACKNOWLEDGEMENTS

During the 1997 monsoon season the author was seconded to Nippon-Koei on site in the Philippines where he was responsible for the planning and design for the structural measures described above. He would like to thank his Filipino, Japanese and Canadian colleagues for their support throughout this period.

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Leakage investigation and remedial works, West Dam High Island Reservoir, Hong Kong

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SYNOPSIS High Island Reservoir is formed by two main dams, East Dam and West Dam, and two subsidiary dams and was completed in 1979. On inspection of West Dam in 1993, it was found that leakage discharge into the gallery from the right abutment of West Dam had increased sharply, showing an upward trend. West Dam is a rockfill embankment dam with a central asphaltic concrete core. A leakage investigation was undertaken, and investigation and remedial works were carried out in November 1998 to April 1999 with the reservoir in service with about 20 m head at the leakage zone.

INTRODUCTION

West Dam is of rockfill construction, curved upstream in plan and it was completed in 1979. It is about 760 m long and 69 m high, with a central inclined asphaltic concrete core on a concrete core base, Fig.1. The foundation is of volcanic rock with a grout curtain. An internal drainage gallery is provided downstream of the core in the central section of the dam. Leakage discharge into the gallery from the right abutment was found to have increased sharply during an independent inspection in 1993 and the trend was still upward over seasonal high water levels.

The report of the independent inspection made the following safety recommendations:

(a) *Carry out a geotechnical investigation to assist in determining leakage paths associated with discharges observed into the gallery from Drain 19NA and adjacent drains,*

(b) *Carry out remedial works to seal leakage paths determined from the geotechnical investigation in (a) above, either in the asphaltic core or the bedrock.*

LEAKAGE STUDIES

An Advisory Service was instructed by Water Supplies Department (WSD) in September 1994 during the inspection covering review of the leakage

Dams 2000, Thomas Telford, London, 2000.

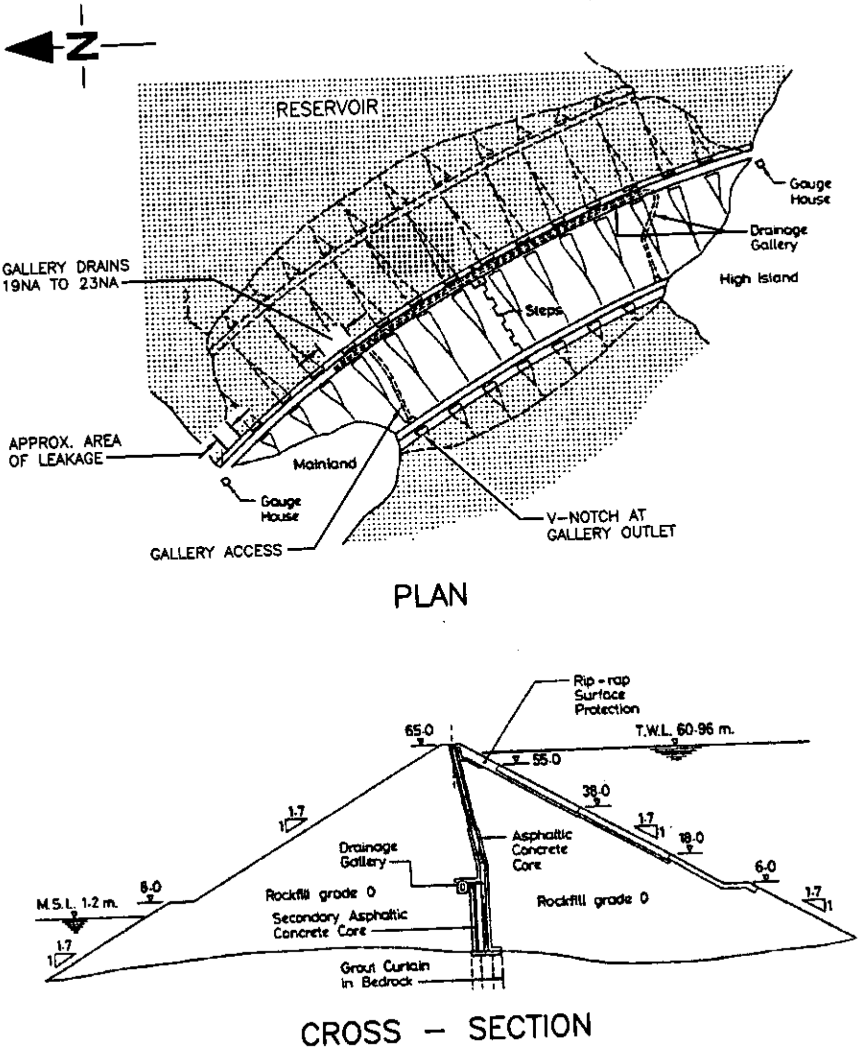

Fig. 1 Layout plan and section of West Dam

records and preparation of a report specifying and detailing recommended methods for carrying out investigation and remedial works. This task included preparation of outline drawings showing the works and temporary works required for their execution, and a programme of works and estimate of costs. A cross-section of the dam at the abutment is shown on Fig. 2. A plan and longitudinal section at the suspect area are shown in Fig. 3.

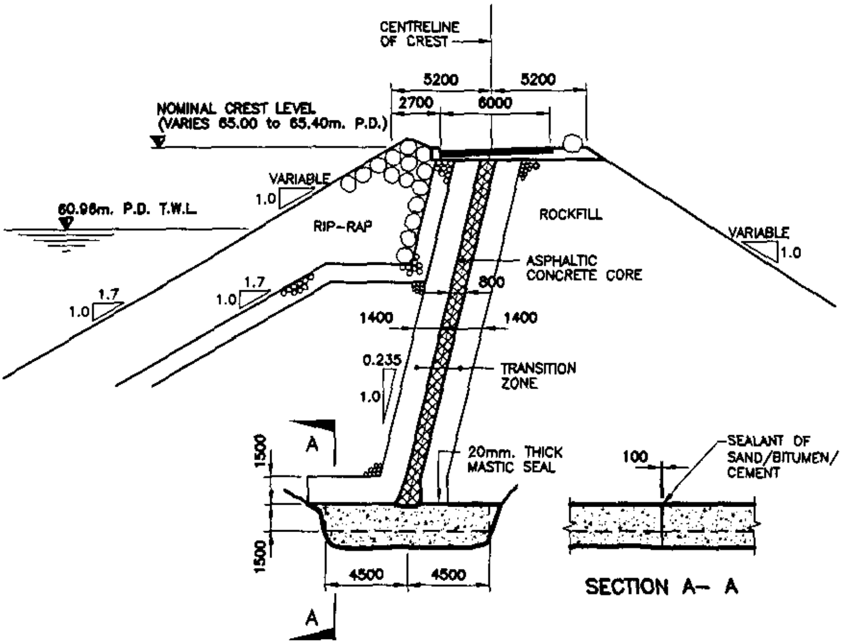

Fig. 2 **Cross-section of dam in suspect area**

The key findings of the Report on the above task with regard to leakage were as follows:

(a) *The most likely cause of leakage appears to be a defect in one of two transverse joints between the core base slabs at Chainage 193.1 or 202.25 m.*

The latter joint was considered to be more likely as its top and base levels coincided more closely with the apparent horizon where leakage started (about 43 m PD).

(b) *Other possible sources of leakage were at the asphaltic core/concrete core base interface, and through bedrock below the core base.*

It was considered most unlikely that there was leakage through the asphaltic core.

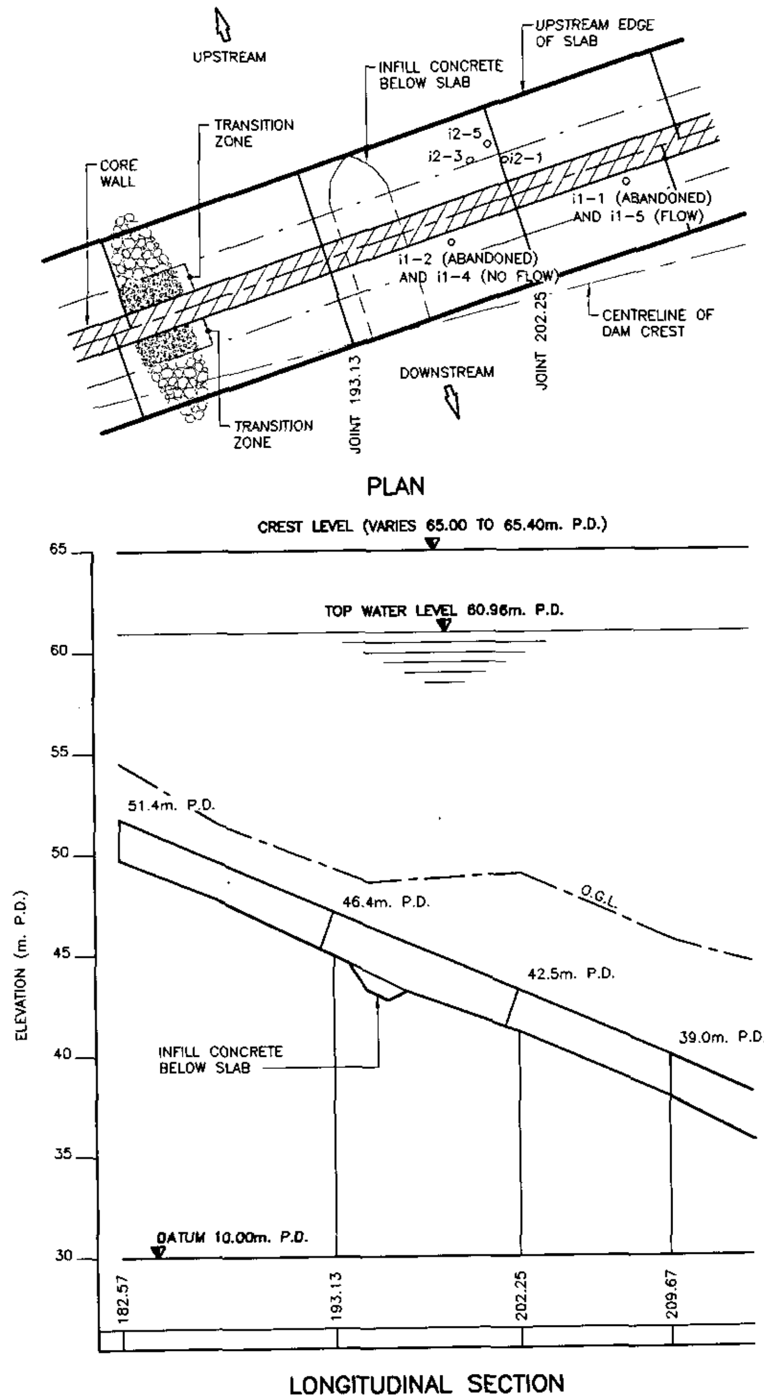

Fig. 3 Plan and longitudinal section of base slab in probable area of leakage

The contract for the investigation and remedial works was prepared by WSD Design Division, incorporating preparation work from the above Report. The Contract allowed for carrying out the investigation works in two stages followed by remedial grouting works. Drilling for both the investigation and remedial works was from the top of the dam through the transition zones on each side of the asphaltic core, and through rockfill upstream of the core. The contract was designed to have flexibility at both investigation and remedial works stages.

INVESTIGATION WORKS

General

The contract for the investigation and remedial works was awarded to Gammon Construction Limited and works commenced in November 1998 and were completed in April 1999. WSD Construction Division carried out the contract supervision with assistance from the Advisor.

Rainfall, reservoir water levels and drainage discharges were monitored during and subsequent to the contract. Drainage discharges are monitored at pipes within the drainage gallery (19NA-23NA) and at a V-notch chamber outside the gallery, Fig. 1.
The co-ordinates for the target locations of the investigation and grout holes were determined from the record drawings for the core base. The co-ordinates on the drawings were adjusted to take account of changes in the Hong Kong datum since original construction, and drilling locations on the dam crest were determined from the adjusted co-ordinates.

The investigation and remedial works were carried out under nearly full reservoir conditions giving a head of 17 to 18 m above the defective zone. There was very little rainfall during the period of the works (dry season).

The investigation works were carried out in two stages:

Stage 1 - Location of leakage area on the downstream side of the core.

Stage 2 - Confirmation of leakage paths from upstream to downstream of core.

Stage 1 – Investigation Works

Four holes were drilled by site investigation rotary drilling techniques with double tube core barrels, Fig. 3. Cores were recovered within the inner stationary tube of the core barrel.

The first two holes (i1-1 and i1-2), located downslope and upslope of the suspect joint at Ch 202.5, were drilled parallel with the core through the transition zone and were checked for alignment using the "Maxibor" system. Hole i1-2 intersected the edge of the core at its widened section near the base

and it was grouted over its full depth and abandoned. Hole i1-1 also intersected the core just above the concrete base slab.
The location of the hole i1-1 was checked using an inclinometer and this confirmed its location in relation to the core. It was then extended to establish the level of the base slab and investigate the condition of the core/slab junction. This was found to be watertight. Hole i1-1 was then grouted and abandoned. A decision was taken at this stage to adopt checking of alignment by inclinometer instead of the "Maxibor" system.

Two replacement holes (i1-4 and i1-5) were drilled to the same target locations as holes i1-2 and i1-1 respectively, allowing for drift found during the earlier drilling. They both intersected the core base slab about 0.5 m from the edge of the core, and were extended a short distance into the base slab.

A CCTV survey was carried out first in hole i1-4. The casing was located just slightly below the level of the concrete base slab at the start of the survey. The water level in the casing represented the saturation level in the transition zone, being just above the top of the core base. The camera was immersed below water level to check for any evidence of flow and there was none. The casing was then raised slowly to just above the top of the core base and water conditions in the hole allowed to stabilise. The flow conditions at and below the surface were inspected by camera and again there was no evidence of any significant movement including indicating flow after the application of dye.

A CCTV survey was then carried out in hole i1-5. The casing was again located just slightly below the top of the concrete base slab at the start of the survey. The water level in the casing, representing the saturation level in the transition zone, was about 0.4-0.5 m above the core base indicating a greater depth of saturation than at hole i1-4. The camera was immersed below water level and conditions indicating flow were not found. The casing was raised slowly to just above the core base and water conditions in the hole changed rapidly with a strong flow across the hole in a downslope direction. Dye was added to the hole and this confirmed the downslope flow. Traces of the dye were found at the discharge points within the drainage gallery (19NA-23NA) and in the V-notch chamber at the gallery outlet.

The results of the CCTV camera survey at holes i1-4 and i1-5 showed that the source of leakage was at or near the suspect joint at Ch 202.5 m. The casing in each hole was lowered to just below the level of the core base to allow monitoring of water levels and sampling in the transition zone during Stage 2 Investigation Works and Grouting Works.

Stage 2 – Investigation Works

Three holes (i2-1, i2-3 and i2-5) were drilled upstream of the asphaltic core for Stage 2, Fig. 3. Holes i2-1 and i2-5 were targeted to intersect the sloping construction joint in the base slab. All holes were drilled to about 5 m below the base into the underlying tuffs. Alignment was checked by inclinometer at 6 m intervals and a further alignment check was carried out when each drill hole was 1.5 m above the top of the core base slab.

Drilling was carried out by percussion top hammer methods with temporary casing (100 mm OD) to improve production. A 25 m deep hole was drilled in about one day compared with a production rate of 3-5 m per day with the rotary drilling site investigation technique used in Stage 1.

Cement/bentonite grout was injected into the transition zone above the concrete core base using the casing of investigation holes i2-1 and i2-5 prior to installation of the tube-à-manchette (TAM) sleeves. Pressure was built up within the casing and a substantial quantity of grout was injected, temporarily reducing the leakage by about 25-30%. The TAM sleeves were then installed in the sleeve grout to about 3 m above the concrete base. Part of the reduction in the leakage flow was fairly quickly re-established leaving an overall reduction of about 10%. The effect of sleeve grouting of the Stage 2 holes in shown on Fig. 4. Suspended cement particles were found in downstream investigation hole i1-5 during grouting demonstrating a connection across the core/cut-off.

Dye was also injected through the TAM sleeves in holes i2-1 and i2-5 at about the interface between the transition zone and concrete base and again traces of the dye were found in hole i1-5. Hole i2-3 was drilled after the grouting of holes i2-1 and i2-5. The casing to hole i2-3 was withdrawn to just above the concrete base slab and dye was injected through a pneumatic packer near the bottom of the hole. Dye was found shortly afterwards in hole i1-5 and at the drainage discharge points within the gallery. Installation of the TAM sleeves in hole i2-3 had little effect on the amount of leakage.

The Stage 2 investigations confirmed that there was a major leakage path at or near the suspect joint in the core base at Ch 202.5.

REMEDIAL GROUTING WORKS

General

Grouting was carried out as part of the installation of the tube-à-manchette (TAM) sleeves in the Stage 2 investigation holes. This was essentially sleeve grouting using cement (OPC)/bentonite mixes with single shot injection at the transition/base slab interface and this reduced leakage by about 10%. The Stage 2 investigation holes were adopted as holes for TAM grouting.

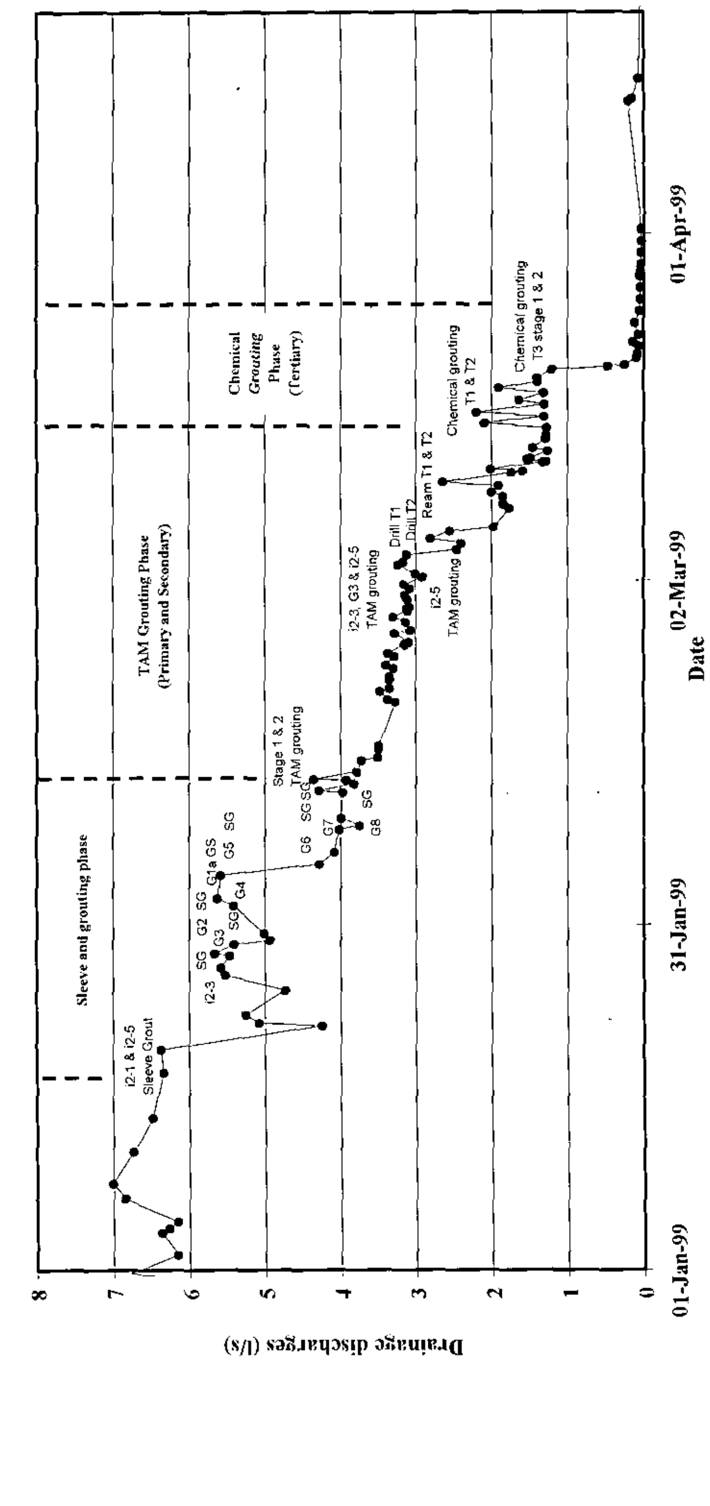

Fig. 4 Initial leakage and reduction in discharge at each grouting stage (Drains 19NA - 23NA)

The effect of sleeve grouting on leakage is indicated on Fig. 4. The largest impact on leakage in the G holes was during the sleeve grouting of hole G5 where 4,200 litres of grout were used. The overall leakage was reduced from about 6.3 l/s to 4 l/s after the sleeve grouting of both Primary and Secondary holes.

TAM grouting of the Primary and Secondary holes was carried out using Rheochem 650 superfine cement. A set quantity was injected at the sleeve unless the residual pressure at the top of the hole rose above 15 bar. Grouting of the Primary and Secondary holes resulted in a decrease in leakage to about 3 l/s. The reduction was small for the Secondary holes indicating that further reduction in leakage with TAM grouting using superfine cement was unlikely, particularly as the pressures were very high for the second stage of grouting.

Grouting of Tertiary Holes

The locations of three Tertiary holes T1, T2 and T3 were determined after grouting of the Primary and Secondary holes, Fig. 5. Holes T1 and T2 were located upstream of the core and were drilled and grouted in advance of T3. The latter hole was located downstream of the core. The effect on leakage of drilling and grouting the Tertiary holes is included on Fig. 4.

Holes T1 and T2 were advanced using percussion top hammer methods with temporary casing to about 0.3 m into the concrete base slab. The holes were then further advanced by rotary drilling using double tube core barrels with core recovery through the concrete base slab and about 5 m into bedrock. The holes were aligned to intersect the joint in the foundation slab and this was achieved in hole T2 where a bituminous filled joint was found. The bedrock below the concrete base slab was decomposed Tuff with closely spaced joints, dipping at 30°-70° and subvertical. The holes were reamed using percussion top hammer methods after coring.

Grouting of holes T1 and T2 was carried out using a chemical grout (Insta-Grout). Insta-Grout is a two component grout derived from synthetic oils. It has low viscosity and both the setting time and compression strength can be varied widely. The grout was designed to have a compressive strength of about 25 N/mm^2. The grout was injected into the holes in two stages through a pneumatic packer placed near the bottom of the holes (within the drill casing and concrete base slab). The holes in the concrete slab and bedrock were re-drilled after the initial stage to improve the chance of grout penetration. The grout was initially injected with set times of about 30 min that were gradually reduced to about 10 min.

Leakage was reduced during the drilling of holes T1 and T2 through the

Five grout holes (G1-G5 inclusive) were established around the Stage 2 investigation holes as an outer boundary, Fig. 5. The casing in Hole G1 sheared during drilling and this hole was replaced by Hole G1a. Three grout holes (G6-G8 inclusive) were established within the outer ring of G holes, Fig. 5. All G holes were drilled about 0.3 m into the concrete base and TAM sleeves were installed to about 3 m above the concrete base, except in G6. The drill casing in G6 sheared off at one of the casing joints and the hole could not be drilled to its full depth. Holes G1-G5 inclusive were adopted as Primary grouting holes. Holes i2-1, i2-3 and i2-5, and G7 and G8 were adopted as Secondary grouting holes.

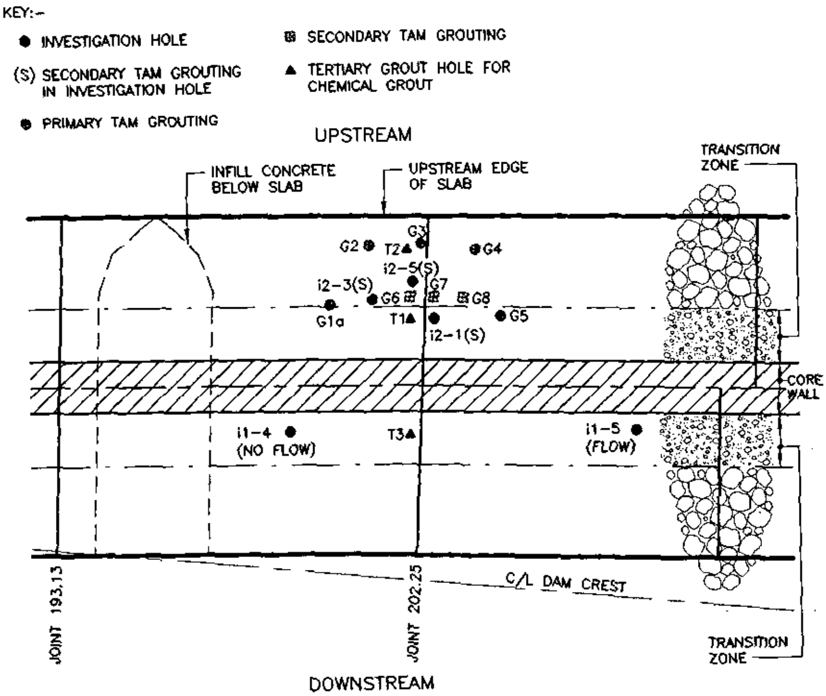

Fig. 5 Plan on core base showing investigation and grouting holes

Grouting of Primary and Secondary Holes

The grouting work for the Primary and Secondary holes was carried out in two main phases, sleeve and TAM grouting.

TAM sleeves were installed in all the G holes before TAM grouting was commenced. Grout was tremied into the holes prior to the installation of the TAM sleeves. The drill casing was withdrawn in each case to about 0.5 m above the concrete base slab and the hole was topped up with grout until it appeared at the top. Grout was pumped under a maximum pressure of about 10 bar through the swivel head of the drilling rig in holes G6, G7 and G8.

base slab, possibly due to obstruction of leakage paths. Grouting was terminated in holes T1 and T2 after the pressure rose sharply to about 50 bar. Chemical grouting of T1 and T2 reduced the overall leakage to about 1.3 l/s. The chemical grout takes in T1 and T2 are given in Table 1

Table 1 Chemical Grout Takes

Hole No.	Stage No.	Grout Take (litres)
T1	1	384
	2	938
T2	1	384
	2	0

An attempt to carry out chemical grouting in the Primary and Secondary holes as a follow-up to the chemical grouting of T1 and T2 was not successful as the pressure to break the sleeve grout was too high.

It was concluded that it would be difficult to reduce the leakage further by drilling and grouting more holes upstream of the core. It was therefore decided to proceed with the drilling and chemical grouting of hole T3 downstream of the core. The base slab and the bedrock to a depth of about 5 m below the slab in hole T3 were again cored. This hole was also aligned to intersect the joint in the base slab and a bituminous filled joint was found at about half slab depth, confirming the accuracy of the setting out and drilling. The bedrock was strong, slightly decomposed Tuff with very closely to medium spaced joints, similar to the rock conditions found in holes T1 and T2.

A CCTV survey was carried out in hole T3 before grouting and some ingress of water through the bedrock was evident. The first stage of grouting was carried out in two phases. The packer was set at the junction between the casing and the base slab to inject into the base slab joint and the bedrock. The initial grouting pressure was about 18 bar and this shortly reduced to 10 bar. Grouting was continued at the latter pressure for about 1.5 hours and the set time was gradually reduced from 20 to 5 min. After a break of about 40 min, grouting was continued for about 45 min again with a short set time of about 5 min. The total amount of chemical grout injected in the above two phases was 840 litres. Stage 1 investigation hole i1-5 was baled at intervals and traces of grout were found.

The second stage of grouting T3 was carried out on the following day using a grout with a set time of about 5 min. On this occasion, grouting was carried out for shorter periods with breaks. The amount of leakage fell sharply to about 0.5 l/s and grouting was stopped after injection of about 480 litres of Insta-Grout. The drainage flow in the gallery reduced to 0.07 l/s showing that the leakage had been sealed.

Chemical grouting in the tertiary holes (T1, T2 and T3) was successful in stopping the residual leakage. The grouting confirmed that the source of leakage was either local to or at the joint in the core base at Ch 202.5. The concentrated nature of the leakage indicates that the source was likely through the sloping joint in the base slab, although this may have caused erosion of the bedrock.

MONITORING OF PIEZOMETERS AND DRAINAGE DISCHARGES

Piezometers

Piezometers were installed in two holes upstream of the core (T1 and T2) and in two holes downstream of the core (i1-4 and i1-5), Fig. 6. The piezometers in T1, T2 and i1-4 were installed in the bedrock below the base slab to monitor pore water pressures in the foundation across the cut-off. The piezometer in i1-5 was installed within the concrete base to monitor the water level and drainage flows in the transition zone above the base. Drainage flows in the transition zone are also monitored at drain outlets within the gallery (19NA-23NA) and at the V-notch at the outlet of the right gallery, Fig. 1.

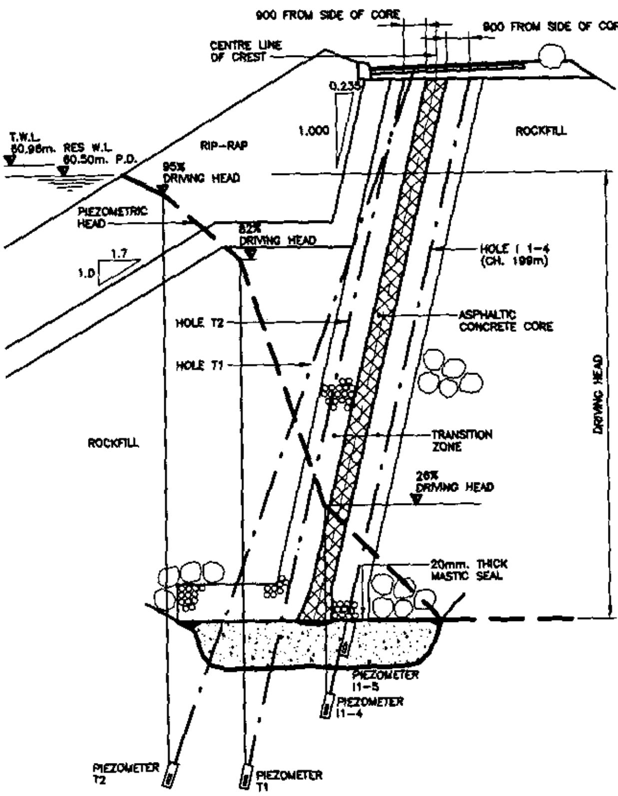

Fig. 6 Section showing monitoring results for piezometers

The bedrock was re-drilled in holes T1 and T2 before the installation of the piezometers. Hole i1-4 was extended from the concrete base by rotary drilling into bedrock for installation of the piezometer. The piezometer results and corresponding water levels following their installation (late March 1999) and at the end of November 1999 are given in Table 2.

Table 2 Piezometer Results

Date	RWL (m PD)	T2 (m PD) Tip 36.56	T1 (m PD) Tip 36.44	i1-4 (m PD) Tip 40.68	i1-5 (m PD) Tip 40.18
31.3.99	60.47	59.41 (94%)	56.95 (80%)	47.95 (23%)	40.17
29.11.99	60.98	60.03 (95%)	57.82 (82%)	48.51 (26%)	49.06
Change	+ 0.51	+ 0.62	+ 0.87	+ 0.56	+ 8.89

Notes: Percentage driving heads are given after the piezometer readings for T1, T2 and i1-4, and are indicated on Fig. 6.

The results between 31.3.99 and 29.11.99 are fairly consistent for piezometers T1, T2 and i14 with similar percentage driving heads. The results between 31.3.99 and 29.11.99 for i1-5 vary widely with a large increase in water level on the latter date. A water level of about 8.9 m in the transition zone above the top of the concrete base slab is not realistic, as there has been no change in the drainage measurement under dry weather conditions.

The high level in piezometer i1-5 indicates that fine material has blocked the response zone and flushing is required. The water level in piezometer i1-5 at the end of March 1999 following the grouting works was more or less level with the top of the concrete base slab, indicating minimal drainage flow/leakage.

Review of the percentage driving heads at piezometers T2, T1 and i1-4 confirms that there is an effective cut-off to seepage flow below the concrete base slab.

<u>Drainage discharges</u>

The total drainage flow in early December 1998 was about 6.3 l/s under dry weather conditions before investigation and remedial grouting works were carried out. The initial leakage and reduction in discharge at each stage of grouting are shown on Fig. 4. The flow dropped to about 0.07 l/s shortly after chemical grouting of tertiary hole T3 was completed (23 March 1999) and at the end of March 1999 it was about 0.05 l/s.

The flow at the end of November 1999 was about 0.04 l/s with the reservoir full and under dry weather conditions, confirming remedial grouting has been effective in sealing the major source of leakage.

CONCLUSION

The investigation works including CCTV surveys were successful in determining the source of leakage, and remedial grouting works including the use of superfine cement and chemical grouts were effective in sealing the leakage zone. The most likely cause of leakage at the right abutment of West Dam was failure of a bituminous filling to the joint in the mass concrete base slab below the asphaltic concrete core that may also have resulted in erosion of the underlying tuffs. The cost of the investigation and remedial works was about HK$3 M compared with a tender value of about HK$5 M.

ACKNOWLEDGEMENTS

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New technologies to optimise remedial works in dams: underwater installation of waterproofing revetments

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SYNOPSIS. Ageing dams experience deterioration leading to water infiltration. Underwater installation of a waterproofing revetment, negating the need to dewater, can be a cost effective alternative for seepage control. A new technology for underwater installation of a waterproofing synthetic membrane, developed in Europe and tested in the United States by the US Army Corps of Engineers, has been applied on an arch dam in Northern California, significantly reducing rehabilitation costs. The paper reports on the life cycle cost analysis of various alternatives, and details design and installation of the waterproofing membrane that was selected.

INTRODUCTION

Ageing concrete or concrete faced dams can be seriously affected by water seeping through the concrete from the reservoir, either along joints oO through the body of the concrete. In cold climates, or at high elevations, the water freezing on the downstream face of the dam expands in the concrete pores, causing pieces of concrete to break loose and separate from the mass of the concrete. Older dams, generally constructed of more porous, poorer quality concrete, or built before the advent of the use of air-entraining agents, are more affected by this condition. Dams subject to frequent freeze-thaw cycles, and thin arch dams where the short flow path allows the seepage to more readily reach the downstream face, are particularly vulnerable.

If the reservoir can be completely dewatered a range of solutions to mitigate seepage and deterioration is available. In the event that total dewatering is not possible due to the characteristics of the dam and of the impounded reservoir, or it is possible but it implies serious economic, social and environmental consequences, the range of solutions is considerably restricted.

The impact that a rehabilitation project can have on the operation of the dam, and on the environment, is nowadays a particular concern for owners and communities involved in the exploitation of a dam. Hence, waterproofing methods that can be accomplished without completely dewatering the reservoir are of significant interest to dam owners.

Dams 2000, Thomas Telford, London, 2000.

SOLUTIONS TO CONTROL SEEPAGE AND DETERIORATION

Until a few years ago, the range of available solutions was the following:

- Upstream chemical treatment to waterproof the outer layer of the concrete using epoxy or other chemical, after sandblasting to prepare the surface. A low installation cost entails on the other hand long execution times, the need to dewater, the impossibility to seal new fissures, the need for frequent repair, and a high impact on operation and environment
- Upstream sealant materials such as epoxy pastes, clay and other membrane type blankets, cast in situ on the upstream face. A low installation cost entails time consuming surface preparation, the need to dewater, a quality affected by weather conditions, a high susceptibility to freeze-thaw, the need for frequent maintenance, and a high impact on operation and environment
- Upstream concrete layer. This solution, adopted when improvement of safety is required, entails long site organisation and construction times, long dewatering time, high costs, and high impact on operation and environment
- Upstream concrete-type seal such as reinforced gunite or shotcrete. This solution, which is not considered reliable in cold environments due to susceptibility to freeze-thaw, entails long construction times, long dewatering time, and high impact on operation and environment
- Upstream prefabricated synthetic geomembrane mechanically anchored and drained. The advantages of this solution are its long time efficiency, short installation times, minimum impact on the environment, and low cost. The feasibility of underwater installation was still in the research and experimental stage
- Internal grouting by chemicals or microfine cement. This solution, which does not require dewatering, and therefore has low impact on operation and environment, is reputed by grouting specialists as having a questionable efficacy. Costs are high
- Internal drainage by drilling from the crest. This solution, which does not require dewatering, and therefore has low impact on operation and environment, is effective only if all seepage paths are intercepted, and costs are generally very high
- Downstream drain layer protected by reinforced shotcrete. This solution, which does not require dewatering, and therefore has low impact on operation and environment, entails long construction times and high cost, is not totally efficient unless all seepage is intercepted by the drain, and the susceptibility of shotcrete to freeze-thaw will expose the drain to environment deterioration in a short time
- Downstream RCC buttress. This solution, which does not require dewatering and therefore has low impact on operation, has high environmental impact due to site organisation, and very high cost.

DEVELOPING DESIGN AND PROCEDURES FOR UNDERWATER INSTALLATION

Until a few years ago, when total dewatering of the reservoir could not be accomplished, only internal and downstream solutions were the viable options for total repair. In 1992, the US Army Corps of Engineers started exploring the possibility of adapting to underwater installation an upstream geomembrane system that had already proven its reliability in dry installations since the early 1970s. The underwater installation of the system was already at those times the object of experimental research in Europe.

In 1994, a two-phase research contract was awarded by the Corps to CARPI, the geomembrane speciality contractor who, after having conceived the dry system, was performing research on the underwater system, and to Oceaneering, one of the largest and most experienced underwater services companies in the world. The first phase of the study consisted in research on geomembrane materials, drainage materials, types of perimeter seals and bolting element and equipment, installation procedures. Large scale testing, and subsequent design refinements, led to a first phase solution. The second phase of the study further refined the solution and investigated its constructability and efficiency by a complete underwater installation on a reinforced concrete structure.

Fig. 1. Diver inspecting the system installed underwater during the Army Corps project. The conformation of the geocomposite to the substrate, and the absence of leaks, demonstrates an effective seal has been achieved

In 1995 and 1996, publication of the Final Reports (Christensen et. al, 1995, and Christensen et al., 1996) of the study by the Corps confirmed that the objectives had been attained, and that an efficient system for underwater installation of an upstream impervious geomembrane was available.

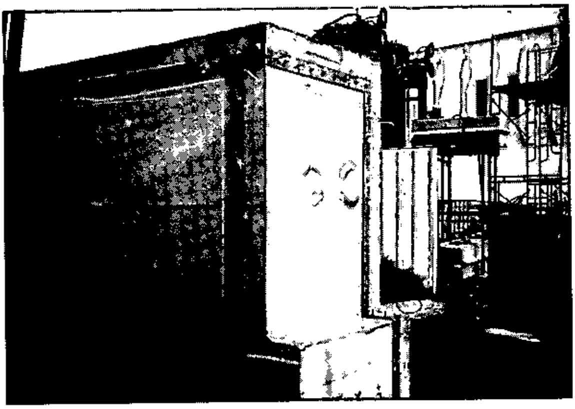

Fig. 2. The concrete structure with varying surface conditions to simulate conditions to be encountered on real projects. The system was tested for 15 days and held its seal at ambient pressure

The underwater system is an adaptation of the dry system that has been described by the Authors in previous conferences (Scuero & Vaschetti, 1998). The new concepts, in respect to the dry system, stay in the design for joining and anchoring adjoining vertical sheets, and in the installation procedures.

FROM RESEARCH TO SITE: LOST CREEK DAM

In the field, pioneer underwater installation of the system had been performed on a Portuguese dam in 1995. In 1997, the system developed during the US Army Corps of Engineers project was applied as a complete rehabilitation at Lost Creek dam.

Problems and investigated solutions

Lost Creek dam is a concrete arch dam, 36 meters high, with a crest length of 134 meters, situated at an elevation of approximately 970 meters a.s.l. in the mountains of Northern California. The dam, completed in 1924, forms a 6,969,213 m^3 storage and diversion reservoir of the Oroville-Wyandotte Irrigation District (OWID) Project on the South Fork of the Feather River and its tributaries. Lost Creek reservoir stores local run off and flows from the Sly Creek reservoir through the Sly Creek Powerhouse. Flows from Lost Creek reservoir are diverted through the Woodleaf Tunnel to Woodleaf Penstock and Powerhouse.

The deterioration of the dam, noted before the 1960s, by 1985 had resulted in loss of more than 1 foot of depth of concrete at the downstream side. The cause of the spalling and deterioration was the permeability of the joints, as well as of the concrete, which was somewhat porous. Investigations carried out from 1985 to 1994, to determine the condition and strength of the remaining concrete and stability of the dam, showed that the dam was structurally adequate under all operating and seismic conditions.

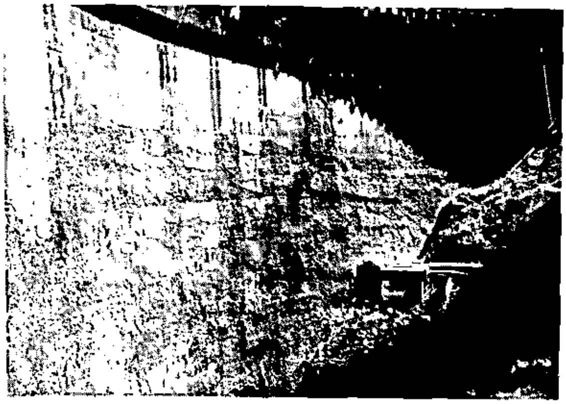

Fig. 3. The downstream face of Lost Creek dam deteriorating due to freeze-thaw, weakening the concrete to depths of one foot and more

A first study was performed to determine feasible methods for treating the dam, to evaluate related costs, and to obtain preliminary approvals by relevanr authorities. Because of environmental concerns, and because the reservoir acts as a forebay for Woodleaf Powerhouse, mitigation methods that would allow the reservoir to remain partially full received high ranking.

The investigated alternatives were
- upstream chemical treatment
- upstream chemical sealant
- upstream gunite or shotcrete
- upstream synthetic geomembrane mechanically anchored
- internal grouting
- downstream drain layer covered with reinforced gunite or shotcrete
- downstream RCC buttress.

<u>Alternatives selected for further evaluation</u>
The first study concluded that three alternatives were feasible and should be further evaluated, and feasibility cost estimates should be prepared:
1) downstream drainage system covered with shotcrete: a layer of geodrain material would be placed on the downstream face and covered with three inches of reinforced shotcrete. This design is similar to the method used to mitigate the concrete deterioration at Bowman South arch dam in Nevada (1995). The alternative could be accomplished without dewatering but, based on previous experience in cold climates, the shotcrete would be susceptible to freeze-thaw deterioration, resulting in short expected life of this alternative solution.
2) upstream synthetic geomembrane mechanically anchored to the dam: among several geomembrane systems evaluated, the most promising was a

PolyVinylChloride drained geocomposite mechanically anchored on the upstream face and left exposed. The method, which had documented satisfactory performance in a large number of applications accomplished in the dry since the 1970s, was the same for which underwater feasibility had been proven during the Army Corps project. All work could be done from the crest of the dam and from barges in the reservoir.

3) Downstream roller compacted concrete (RCC) buttress: on the downstream side, an RCC dam would be constructed, with a drainage system between the existing dam and the RCC, to intercept the seepage. Although not requiring dewatering, this alternative would have high environmental impact caused by quarrying, hauling traffic, construction, and some additional concrete deterioration would still occur.

Table 1. Comparison of costs of the selected alternatives

Alternative	Projected Price	Expected Life	Yearly Amortisation of Project Cost (4%)	Impact on Environ.
1. Downstream Drainage System Covered with Shotcrete	$2,083,000	20 years	$153,271	Low
1. Geocomposite System on Upstream Face	$2,053,000	50+ years	$95,568	Low
2. Roller Compacted Concrete Buttress	$4,569,000	50+ years	$212,688	High

The three methods described had differing benefits and therefore could not be directly compared based on cost alone. The Owner and their consultants concluded that Alternative 2, the upstream geomembrane system, offered the greatest benefits with respect to cost, and involved the least environmental impact.

Design of the system

The selected alternative was approved by the Federal Energy Regulatory Commission (FERC) and by the California Division of Safety of Dams (CDSD). The same team that had accomplished the Army Corps research project developed the detailed design of the system. The timing of the geomembrane installation was planned to coincide with the rewinding of the generator at the Woodleaf Powerhouse. This allowed significant drawdown of the reservoir without loss in power generation, resulting in reduction of dive depths, allowing longer time underwater per dive.

The system installed as a waterproofing liner is a composite membrane consisting of a 2.5 mm thick PVC geomembrane heat coupled during fabrication to a 500 g/m^2 polyester geotextile. To accommodate the need of underwater work, and to optimise construction schedule, the standard 2.10 m wide sheets are pre-assembled into 8 m wide panels, with custom-made lengths avoiding the need of transverse joints.

Adjoining PVC panels are joined by a specially designed assembly of two stainless steel profiles, which perform watertight connections between the panels, tension the panels to avoid slack areas, and act as vertical pipes to convey drained water to bottom collection. Along the entire submersible periphery, the PVC liner is secured to the dam face by a watertight seal made by compression with stainless steel batten strips. The watertight connection between adjoining panels, and the watertight fastening at the periphery, allow the construction one continuous impermeable PVC liner on the lined area.

A drainage system is designed behind the PVC geocomposite and its attachment system: the system consists of a drainage geonet, conveying drainage water to bottom collection in a drainage accumulator, and to downstream discharge by a drain pipe located in the existing lower outlet pipe.

The drainage system is designed to intercept water seeping from foundation, rain and snowmelt water from the crest, and condensation water migrating from the colder dam body towards the warmer upstream face. This should reduce the water content in the dam body, relieving internal uplift pressure.

Design of the monitoring system included control and monitoring of the discharged water, and a water level indicator installed behind the geomembrane through a conduit pipe just left of the bottom outlet.

<u>Sequence of construction</u>

The system was installed in the dry from elevation 3273' to elevation 3244', and underwater from elevation 3244' to elevation 3187'. Sequence of construction was the same in the dry and underwater. Turbidity of the water hampered photographic documentation of underwater construction.

Fig. 4. Installation of the drainage discharge pipe from the downstream face. After clearing of debris and silt (from a crane placed on the dive barge and by a dredge pipe) a drainage accumulator plate was installed over the exit hole at the upstream face, to provide a sump for collection of water from the drainage system

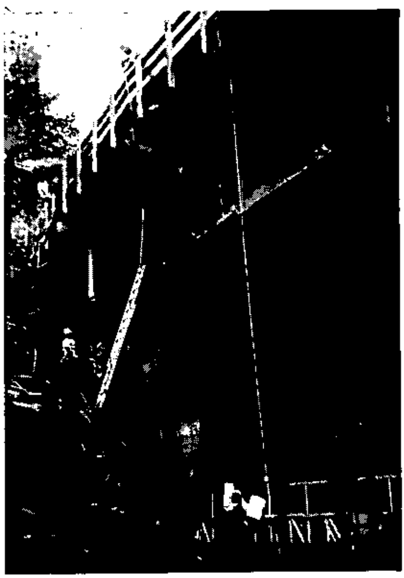

Fig. 5. After local repair of the concrete face with patches of geotextile or scrap geonet, or with epoxy mortar (underwater) and concrete or quickset cement or grout (above water), the geonet drainage layer was installed and anchored by impact anchors. The cross-diagonal grid pattern of the geonet provides free conveyance of seepage and condensation water to bottom. Ventilation holes prevented development of a vacuum behind the membrane

Fig. 6. The diver is installing the anchor bolts for the watertight perimeter seal at the air water interface at right abutment. Bottom seal was installed as close to the abutment ground surface as was feasible, to maximise coverage of the dam with the membrane. Anchor rods were set with chemical epoxy capsules

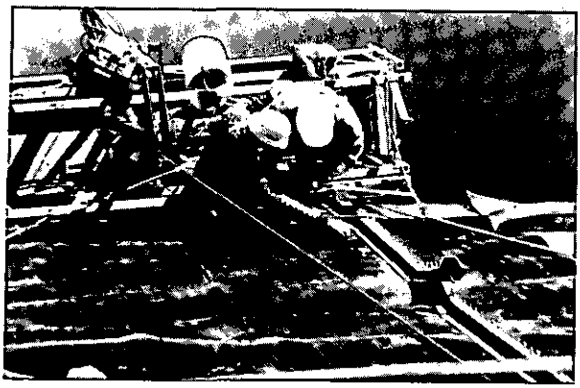

Fig. 7. The internal profile is installed over the drainage geonet. Guide wires ensured vertical installation, particularly for underwater installation, performed with visibility from nil to 2 feet. Anchorage methods used were chemical epoxy capsules (above water) and shallow or deep expansion anchors (below water)

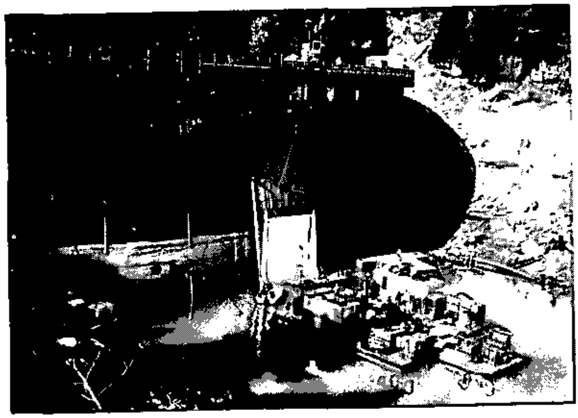

Fig. 8. The first prefabricated panel being deployed for alignment from the crest. Divers checked the position of the panel relative to the underwater profiles, to ensure sufficient membrane overlap along each profile.

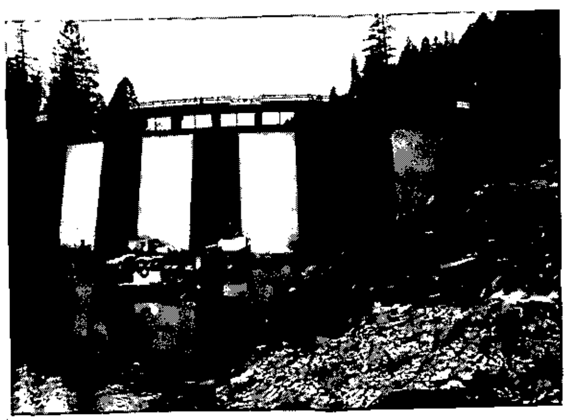

Fig. 9. The panels of waterproofing geocomposite were staggered to allow dry and underwater installation to proceed at separate locations, assuring safety of workers and divers. A piezometer conduit was anchored to the dam face along one of the panels. Water levels within the drainage layer can be measured from the bridge deck with a level indicator lowered down the conduit

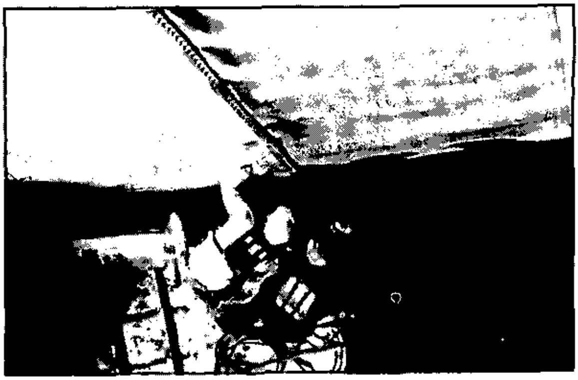

Fig. 10. External anchorage profile, with diver preparing to transit to underwater work. External dry profiles were covered by heat-welded PVC cover strips to prevent seepage along the set screw/hex coupler assembly. External underwater profiles were watertight connected to the internal ones

Fig. 11. The upper perimeter seal was installed just below the bottom of the concrete piers supporting the bridge deck

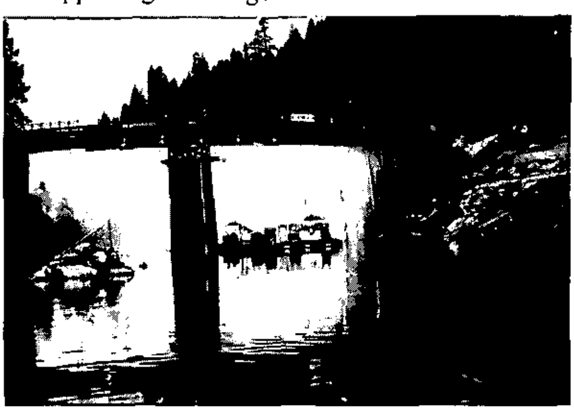

Fig. 12. Installation of water level monitoring system at panel 6. The installation of the waterproofing system was completed in less than 3 months. Total documented seepage is at present < 0.063 l/s

SEEPAGE PERFORMANCE

After completion of the geomembrane installation, the valve on the downstream end of the drainage discharge conduit was opened to drain the water trapped between the waterproofing membrane and dam face during installation. An underwater inspection verified that the membrane was tightly conformed to the dam face in all areas, and divers were unable to detect leakage of any kind at either the perimeter or the vertical seals. After initial fluctuation, as the water drained out of the geonet drain system and as the reservoir level rose, the seepage flow decreased, until at mid March 1998 the flow stabilised at a value of less than 0.063 l/s with the reservoir at spillway crest.

The water level indicator readings consistently show that there is no water standing in the geonet drain. The downstream face of the dam appears dry and no seepage through the concrete has been observed.

OWID will monitor the deterioration of the downstream face at a series of monitoring points, on a regular basis, to determine whether the deterioration of the concrete is continuing.

VALIDATION

The Lost Creek project has been granted the following USA awards:

- 1998 West Region Award of Merit from the Association of State Dam Safety Officials (ASDSO)
- 1999 Hydro Achievement Award for Technology Solutions from the National Hydropower Association
- Federal Laboratory Consortium Award of Merit from the U.S. Army Corps of Engineers.

CONCLUSIONS

The installation of a synthetic waterproofing geomembrane system can be accomplished underwater providing significant cost savings, and minimising environmental impact. The satisfactory behaviour evidenced at Lost Creek in two and a half year of exploitation indicate that a new technology has been made available in the field of dam rehabilitation.

ACKNOWLEDGEMENTS

The authors wish to thank OWID, in the person of Steve Onken, who made the Lost Creek underwater installation project become an important reality, and Richard Harlan, who first recognised the potentiality and applied in the field the new underwater technology, for the data provided on evaluation of alternatives and on seepage performance.

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Grouting of embankments at Cathaleen's Fall generating station on the river Erne

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SYNOPSIS. The Erne embankment dams were constructed in the period 1946-1949 to provide edge containment to a balancing reservoir required for the river Erne hydro-electric development. Over the life of the embankments grouting has been carried out from time to time to deal with leakages which have occurred. This paper discusses the characteristics of the embankment construction which render the dam vulnerable to internal erosion. The development of a significant leak recently, and the investigation procedure adopted to identify the route of the leak, are described. The extensive grouting programme, completed in 1998 to seal the leak, is also outlined.

INTRODUCTION.

The river Erne, at 97km long, is the second longest river in Ireland. The Erne rises at about 270mOD near Carrickboy, Co Cavan and flows in a north-westerly direction, entering the sea at Donegal bay downstream of Ballyshannon (Figure 1).

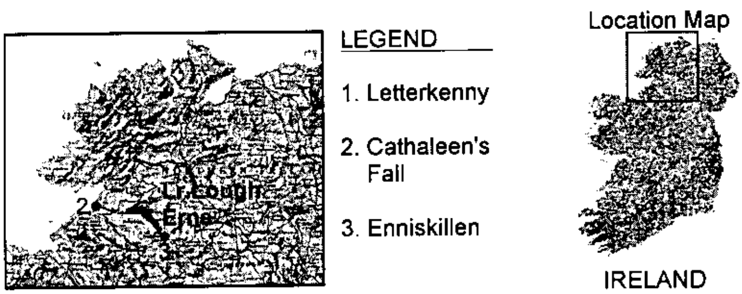

Figure 1. Site location

The river drops about 45m over the last 7 km of its course. It was this fall that was harnessed for the production of hydro-electric power with the construction of two generating stations on the Erne, some 5km apart, in the period 1946 to 1952. The stations at Cathaleen's Fall and Cliff incorporate mass concrete gravity dams with 30m and 10m heads respectively. A balancing reservoir, of approximately 2km^2 in area, is located between the two dams. The reservoir is enclosed on the southern side by 3 embankment dams with a total approximate length of 1.1km and a maximum height of 9m (Figure 2). The embankments are formed of clay with a concreted core trench through which extensive grouting of the underlying limestone was carried out during construction in the period 1946 to 1949. At the time of construction, the science of soil mechanics was in its infancy and Dr K. Terzaghi provided consultancy input (Ebrill, 1949) on the selection and testing of embankment materials. The embankment was generally constructed on top of the existing overburden after removal of topsoil. A typical cross-section through the embankment is indicated on Figure 3.

Recent technical reviews have identified the risk of internal erosion occurring below the embankment. The most vulnerable areas were seen to be at the interfaces between the original ground and the overburden, and between the overburden and the bedrock. In recent years the monitoring system had identified leakages which were increasing over time and instigating internal erosion. Following a review of the monitoring data a decision was made to commence a systematic grouting programme to halt the progress of internal erosion.

This paper outlines the geological setting, construction and history of the embankments and describes the investigations and analyses of the risks of internal erosion. The development of a recent significant leak and the investigation to identify the route of the leak is described. The extensive grouting programme, completed in 1998 to seal the leak, is also outlined.

GROUND CONDITIONS AT THE SITE.

Geological Sequence

The geological sequence at the dam site is known and understood from geological studies (Hazell,1947 and O'Brien,1949) and numerous site investigations carried out since the embankment dam was constructed. The general sequence in the Erne valley is as follows:

1. Glacial Till (generally 1m to 1.5m thick)
2. Ballyshannon Limestone (Carboniferous)(>20m thick)
3. Metamorphic Schist and Gneiss (Pre-Cambrian)

The Glacial Till at the embankment dam site comprises soft to firm silty sandy CLAY with some gravel and a medium dense coarse SAND with some gravel. The average SPT N value obtained during investigations in the material was 15.

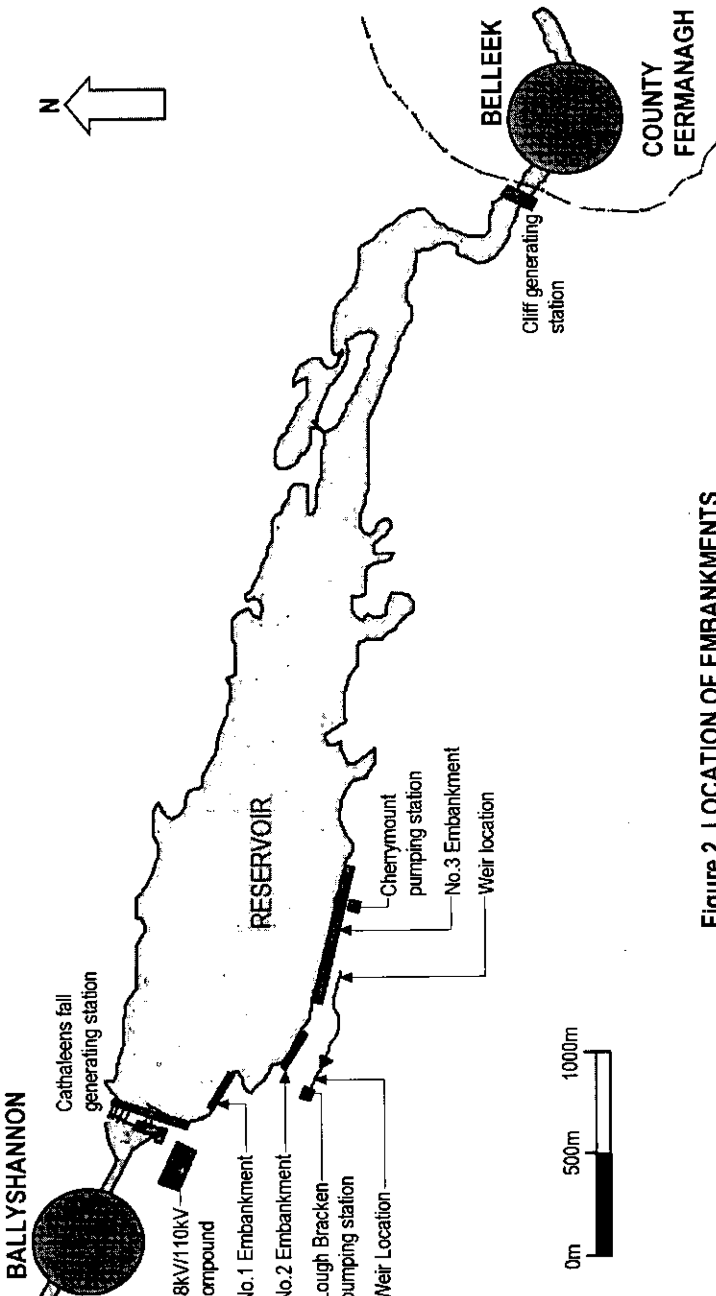

Figure 2. LOCATION OF EMBANKMENTS

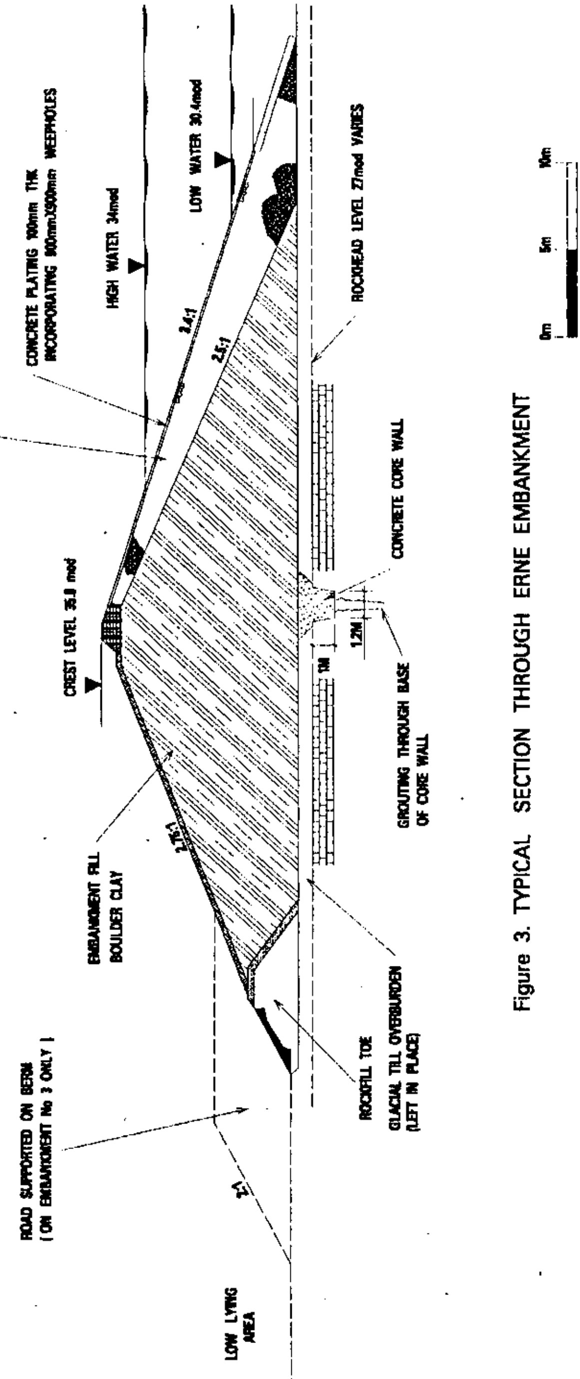

Figure 3. TYPICAL SECTION THROUGH ERNE EMBANKMENT

The Ballyshannon Limestone is a strong coarse-grained dolomitic limestone, revealed to be in a fractured state to the depths investigated (maximum approximately 6m below rockhead). The average measured RQD in the Ballyshannon Limestone at the site was 32%. A study of rock cuttings at the site of the power station, and an examination of rock cores obtained from below the embankment dam, suggested that water-carrying channels occur due to solution enlargements at the intersection of joints and bedding planes in the Limestone.

Little is known regarding the engineering properties of the Schist and Gneiss at the site but the depth of the pre-Cambrian rock is thought to be such as to preclude any influence by the material on the behaviour of the embankment dam.

EMBANKMENT DESIGN AND CONSTRUCTION.

Embankment Geometry

The overall contained length of 1.1km is provided by three individual embankments (denoted nos. 1, 2 and 3, see Figure 2), separated by elevated areas which provide natural containment. Approximately 57,000 m^3 of locally sourced fill was used in construction of the embankments. The upstream slope is formed at 3.4H:1V (Figure 3) and is faced with a concrete slabbing. The slabbing has 0.9mx0.9m square openings (weepholes) at intervals of approximately 7m, which provide pressure relief for an underlayer of stone (75mm nominal size) of variable thickness. The downstream slope is 2.75H:1V and incorporates a rock toe. A shallow core trench, concrete filled over most of its length, provides a capping to the single grout curtain that was provided to cut off seepage through the Ballyshannon Limestone near formation level. The core trench was excavated primarily through Limestone and in these areas grouting was carried out. At other locations the excavations revealed Glacial Till. Grouting comprised a series of primary, secondary and tertiary holes drilled to depths of 18m into the rock, primary holes being at 7.5m centres and at an angle of 30^0 to the vertical. The holes were raked in an attempt to intersect the joint planes. Anticipated zones of seepage were targeted for special treatment, and at one point holes 0.6m apart over a distance of 15m were required to seal off underground seepage. At locations where the core trench was in Glacial Till, the trench was filled with 'EarthFill' (term used in original site reports – assumed to mean reworked Glacial Till) and the junction between the Earthfill and the concrete made with galvanised steel corrugated sheeting.

Embankment Fill Characteristics

The embankment fill, as sampled during site investigations in 1988, was revealed to be a firm to stiff sandy silty clay with some gravel. Particle size distribution tests showed that the material comprised 30% to 40% silt and clay sized material, 25% to 30% sand and the remainder is gravel sized.

High densities, between 80% and 95% of optimum were recorded on samples on the fill.

Implications of Embankment Design for Internal Erosion

The earth dam was designed as a 'homogeneous' embankment with the water retaining function provided by the low permeability Glacial Till. The upstream concrete slabbing and stone underlay provide erosion protection from wave action in the reservoir. The rockfill toe on the downstream slope provides a buttress against potential slips as well as drawing down seepage flow lines away from the slope.

In contrast with modern dam design practice (Cedergren, 1989), little attention was placed on the vulnerability of the fill and foundation materials to internal erosion. Filter criteria were neither considered at the interface between the fill and the Glacial Till foundation material, nor assessed at the interface between the Till and the bedrock. At isolated locations along the dam, a relatively sandy overburden is in contact with weathered locally permeable bedrock. In these circumstances there is a vulnerability to suffusion of fines into the voids in the rock at locations where hydraulic gradients are high. The maximum gradients are likely to occur within the overburden material immediately downstream of the grouted zone in the bedrock (Innerhofer, 1993). At this location the head reduces from the reservoir level to the downstream ground level over a relatively short length. Thus, the presence of a grout curtain through the limestone beneath the centre of the dam, while providing a vertical barrier through the rock, does not provide a guard against internal erosion.

HISTORY OF EMBANKMENTS.

After flooding of the reservoir in 1950, a number of springs were noted downstream of the embankments. The springs were channelled into piped overflow monitoring locations, which to the present day provide indicators of variations in leakage from the embankments. The current monitoring locations are summarised in Table 1:

Table 1. Spring monitoring locations

Embankment	**Locations of Springs / Factors Influencing Flow**	**Typical Flow Range (l/s)**
Embankment No.2	Two springs located approximately 15m from the downstream toe. The flow is primarily influenced by reservoir level.	0.1 to 0.5

Table 1. (Continued) Spring monitoring locations

Embankment	**Locations of Springs**	**Typical Flow Range (l/s)**
Embankment No. 2	Flow is measured at a weir on a stream approximately 70m from the downstream toe (refer to Figure 2). The flow measured is heavily influenced by rainfall but, based on dry weather flow measurements and the sensitivity of flow to reservoir level, is known to contain a contribution from spring flows.	0.2 to 20
Embankment No.3	Five springs located at the downstream toe. The flow is primarily influenced by reservoir level.	0 to 0.1
Embankment No.3	Flow is measured at a weir on a stream at the toe of a road embankment , which forms a berm to the downstream toe of embankment no. 3 (refer to Figures 2 and 3). The flow measured is heavily influenced by rainfall but, based on dry weather flow measurements and the sensitivity of the flow to reservoir level, is known to contain a contribution from spring flows.	0.2 to 5
Embankment No.3	Two springs located at the downstream toe adjacent to a pumping station where a stream, obstructed by the embankment construction, is pumped into the reservoir	0 to 0.05

In addition to the monitoring of springs, an analysis is carried out on an annual basis of pumped volumes at the two pumphouses (Lough Bracken and Cherrymount) in the area of Embankments Nos. 2 and 3 (Figure 2). The pumphouses discharge into the reservoir water from streams which originally flowed naturally into the Erne, but whose courses were blocked by construction of the embankments. The annual analysis involves comparisons between the volume pumped and the rain falling on the small catchments which the streams serve. These comparisons give a broad indication of the proportion of the pumped volumes that are attributable to runoff and, therefore, provide an additional indication of leakage in the area of the embankments generally.

Initial Indications of Potential Internal Erosion

In the period 1966 to 1986, the dry weather flow in Spring No.2 (at Embankment No. 2) increased steadily from a rate of 0.1 l/s to 1.66 l/s. This spring is located close to the position of maximum embankment height and the flow had, on occasion, been observed to be carrying sediment. Furthermore, the flow was shown to be sensitive to changes in reservoir level. During the planned routine lowering of the reservoir in 1985, the flow varied between 1.2 l/s at a reservoir level of 33.7mOD, and 0.52 l/s at a level of 30.45mOD. The presence of sediment raised concerns regarding potential internal erosion in Embankment No.2. A drill and grout exercise through the embankment was carried out in 1986, in the vicinity of Spring no.2. Grout holes, 50mm diameter, were drilled using rotary open hole techniques to depths of 15m. A total of 3.92t of cement (incorporated in a 1/1 (by weight) water/cement grout) was injected but no decrease in spring flow was recorded. By 1988 the spring flow had increased to 2.06 l/s. A comprehensive borehole investigation, comprising cable percussion and rotary coring was instigated in the vicinity of the spring. The investigation confirmed the presence of a thin band of sandy Till overlying the Limestone bedrock below the embankment. A further drill and grouting exercise was carried out, injecting a total of 96.3t of cement (grout ratio as above). The spring flow was reduced by 90% as a result of the grouting exercise.

LEAK INCIDENT OF 1997/1998.

Indicators of leak

The weir on the stream near the downstream toe of Embankment No. 3 measures seepage flow through the embankment over a length of approximately 75m. The minimum recorded flow over the weir during the calendar years from 1983 to 1997 is shown on Figure 4.

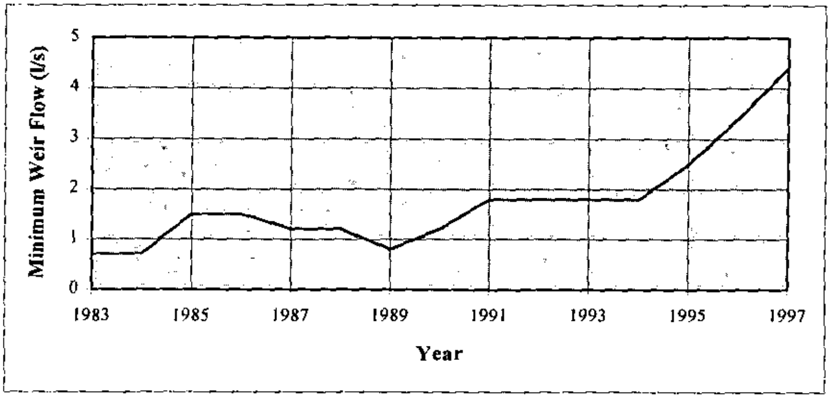

Figure 4. Recorded annual minimum flows at stream weir

Between 1991 and 1994 the minimum annual flow was 1.8 l/s. The flow increased steadily until 1997, during which year a minimum annual flow of 4.4 l/s was recorded. In September 1997 it was noted that the flow rate had reached 6.9 l/s following a dry period, and could therefore not be attributed to rainfall. Initial surveillance indicated a visible leak in the toe of the road embankment, which forms a downstream berm to the embankment dam at this location (Figure 3). It was decided to monitor the leak over the winter and to carry out a full investigation during the spring/summer of 1998. The investigation commenced in April 1998 and comprised:

i) dye injection into weepholes under water on the upstream slope
ii) drilling of a series of holes through the embankment on the upstream slope, which were then used for constant head tests to assess the permeability of the embankment fill and overburden.

Investigation Techniques

In April 1998, the reservoir level was reduced to determine the effects on flow at the stream weir (Embankment No. 3). The relationship between regressing reservoir level and flow in the weir is illustrated graphically in Figure 5 below (the peaks on the graph for the weir flow are attributable to rainfall). When the reservoir level had dropped to 32.5m, the weir flow reduced to 4.4 l/s. At this reservoir level the third row of weepholes, although underwater, were accessible. Emulsion paint was injected into the weepholes over a 45m length of embankment, via a pressure sprayer, to assess whether water was being drawn in. There was some indication of draw in the weepholes at three locations. Fluorescene dye was injected into these weepholes and a connection was established between one and the leakage on the downstream side. Dye was visible on the downstream side (beyond the toe of the embankment) approximately 3.5 hours after injecting into the weephole.

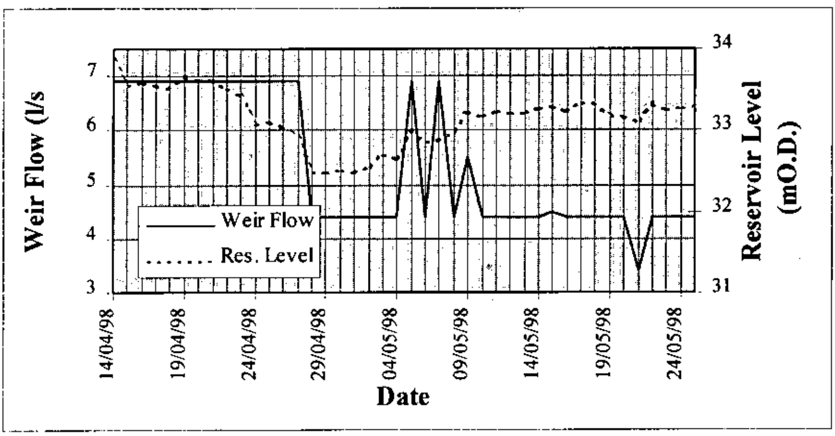

Figure 5. Relationship between reservoir level and flow in weir

A series of 33 holes of 50mm diameter, at 1.5m centres and 5.3m deep, was then drilled on the upstream slope of the embankment at level 35.35m. Some local erosion (clay in suspension) was visible on the downstream side after drilling two adjacent holes. A further 6 holes were drilled in the vicinity of these holes and all holes were then subjected to constant head tests. No significant water take was recorded. A further 10 holes were drilled to between 6.5m and 7m depth until a significant resistance to drilling was obtained. Construction records indicated this level to be at or just into original ground. Erosion at the downstream side was again visible subsequent to drilling and at the same position where erosion was first noted. These deeper holes were then subjected to constant head tests and a water volume in excess of 1.5 l/s was required to maintain a constant head at two adjacent holes, which were 0.7m apart.

Grouting Techniques

After observing the behaviour of the constant head tests it was decided to commence a grouting programme at one of the two adjacent holes, in an attempt to seal the apparent leak. Grouting was started on 26th May 1998 using a 10/1 water/cement mix followed by a 5/1 and a 2/1 mix. After injection of 7 tonnes of grout (by pouring under gravity using a bailing technique devised on site, the mix was thickened to 1/1 until a total of 11.2 tonnes of grout had been injected into the borehole. The borehole was then flushed and grouting suspended until the following day. On the second day, grouting commenced with a 5/1 mix and was thickened up rapidly to 1/1. After injection of 3 tonnes of cement the mix was again thickened to 2/3 (i.e. 100litres to 150kg cement). This was felt to be the thickest mix which could be safely injected without the risk of blocking up the borehole. Grouting continued using this mix, for the most part, over eight days with a total of 90 tonnes of cement injected. It was then decided to introduce fine sand into the mix and reduce the amount of cement. The sand content was increased gradually until 12% sand by weight was incorporated in a 1.5/1 mix. After a short period of time using sand in the mix, the flow of grout into the borehole appeared to be slowing down and the use of sand was discontinued. On reverting to a water/cement mix, the rate of grout uptake increased again. Sand was again included in the mix and the process repeated. On 12th June 1998 the borehole sealed and would not accept further grout.

The grouting operation had continued over a period of 18 days, during which flow over the weir was monitored. The low-lying area downstream of the road berm (Figure 3) was also kept under close observation. At the time of the grouting this low-lying area was flooded. The following observations were noted:

- Flow at the weir decreased daily, as grouting progressed, from a value of 4.4 l/s at the start to 0.2 l/s on completion of grouting (refer to Figure 6 below, noting that peaks in the weir flow graph are attributable to rainfall).

- The length of time between commencement of a day's grouting and the appearance of grout on the downstream side decreased progressively from about 6 hours on the first day to 10 minutes towards the end of grouting.
- No grout was seen to come from the area of visible leaks at the toe of the road embankment until the 5th day of grouting (and after 50 tonnes of grout had been injected) and then at only one of those locations.
- Other than at the leak noted above, evidence of grout downstream of the embankment appeared to come from the floor of the flooded area over a length of about 25m (rather than at the toe of the embankment)
- The total weight of cement grout injected through the selected borehole was 107 tonnes and this represents a set volume in excess of 60 cubic metres.
- In the 12-month period subsequent to grouting, the volume pumped at Lough Bracken pumphouse has reduced by approximately 50% when compared to the long-term average. This is an early indication of a substantial reduction in leakage flow through the embankments.
- The grouting exercise cost approximately £20,000. While embankment safety was the primary motivation for the exercise, a measurable economic benefit has accrued from the reduction in pumping costs which amounts to a saving of approximately £1000 per annum at current rates.

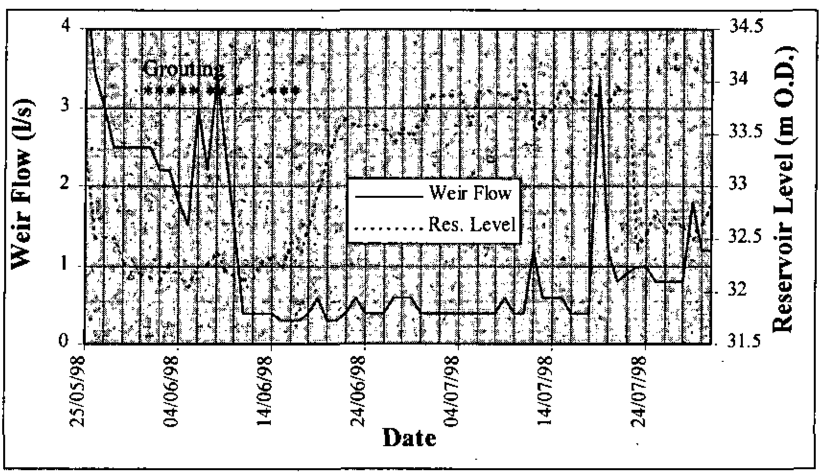

Figure 6. Response of flow at weir to grouting

CONCLUSIONS

The Erne embankments were constructed to the highest standards available at the time (1946-49) but the nature of the ground left in place at formation level has exposed a vulnerability to internal erosion. A grout curtain through the rock below the dam, while providing a barrier to water flow, results in high hydraulic gradients in the downstream overburden foundation material. Close monitoring of springs for increases in flow volume, and for the presence of fines in suspension, has provided an adequate means of alerting supervising personnel to potential problems. Observation of the behaviour of fluorescene dye and paint, when injected into weepholes on the upstream slope, has facilitated identification of the inlet region of leaks. The subsequent drilling of 50mm diameter holes using open holing techniques, in a systematic manner, has enabled leak paths to be determined. Grouting of these leak paths using a cement/water and a cement/sand/water mix with a progressively increasing viscosity, poured under gravity has produced measurable success in halting leakage and internal erosion. The emergence of grout beyond the downstream toe of the embankment supports the inference that the residual overburden material provides the primary path for leakage and potential internal erosion at the Erne embankments.

ACKNOWLEDGEMENTS

The support of the Chief Civil Engineer, ESB, Jack O' Keeffe, towards the preparation of this paper is gratefully acknowledged.

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Reconstruction of Lednock Dam crest

C C PASTEUR, Scottish and Southern Energy plc

SYNOPSIS. Lednock Dam is a mass concrete buttress dam completed in 1957. Statutory inspections and a programme of testing identified that the concrete forming the top 800mm of the dam crest was in poor condition, having been affected by water ingress and frost action. Localised repairs of the defective concrete over a number of years had proven to be ineffective. A decision was taken to reconstruct the dam crest using a modern durable concrete. This paper describes the design and construction of the reconstruction works.

GENERAL DESCRIPTION OF LEDNOCK DAM

Lednock Dam is situated in Glen Lednock, 5 miles north west of the village of Comrie in Perthshire. It was constructed to provide a storage reservoir for the St Fillans hydro-electric development, with a total catchment of 107 square kilometres and a capacity of 30.1 million cubic metres. The dam is 290m long at its top level of 354m O.D. and comprises thirteen mass concrete diamond-headed buttresses 15.2m wide at the head, and mass concrete gravity sections 38.1m long at the west side and 53.3m long at the east side. The central spillway 65.5m long is formed by concrete beams spanning across the downstream faces of five of the buttresses. The height of the dam is 37.4m. The general layout of the dam is shown in the photograph at figure 1.

INSPECTION AND ASSESSMENT

In accordance with the requirements of the Reservoirs Act 1975, the dam is inspected by an independent qualified engineer at ten-yearly intervals. The most recent Inspecting Engineer's inspection was carried out by Mr Alexander MacDonald of the Babtie Group in 1996. The Supervising Engineer, who is a member of Scottish and Southern Energy's (SSE's) Hydro Civil Engineering Department, inspects the dam twice a year.

The Supervising Engineer's reports had over a number of years indicated that the condition of the concrete forming the dam crest was deteriorating. This was evidenced by spalling of the top surface of the concrete and by efflorescence emanating from hairline cracks to the underside of the overhung crest. It was confirmed by the testing of concrete cores recovered from the dam crest.

Localised repairs to the worst areas of defective concrete had been carried out over a number of years as part of the ongoing maintenance of the dam.These repairs were generally not effective due to inadequate bonding to the poor quality concrete substrate, from which they became detached by the action of water and frost.

Fig. 1. Photograph of Lednock Dam

Following the 1996 inspection and core testing, the Inspecting Engineer advised that he considered that the structural function of the defective concrete was adequate for its duty, and that there was no requirement to replace it in the interest of safety under the Reservoirs Act 1975. He did however recommend that the problem should be addressed as a maintenance item before it had the potential to develop into a more serious issue. SSE were also keen to find a permanent solution in order to eliminate any risk of injury to its personnel or members of the public by being struck by falling debris, and to discontinue the requirement for expensive and ineffective piecemeal repairs. For these reasons it was decided to demolish and reconstruct the dam crest over its entire 225m length (excluding the spillway).

SITE INVESTIGATION

The dam crest was constructed using Class E concrete, a different mix to the Class F concrete used in the main body of the dam. In the Class F concrete only, 20% replacement of cement by fly ash had been adopted. The design mixes for the Classe E and F concretes are detailed in figure 2 below.

Class	Specification		Proportions Used by Weight				
	Min. 28 day strength N/mm^2	Max Agg. Size mm	Agg: Cement Ratio	Water: Cement ratio	Aggregate Sizes		
					76 to 38mm %	38 to 5 mm %	Sand %
E	19.3	38	7.7:1	0.6	0	68	32
F	19.3	76	9:1	0.6 to 0.64	34	34	32

Fig. 2. - Original Concrete Mix Designs

During 1996 a number of cores were recovered from the Class E dam crest concrete for testing. The testing programme was designed to determine the concrete compressive and tensile strength, density, excess voidage, cement content, original water/cement ratio and aggregate types, in order to assess its permeability and susceptibility to shrinkage and frost damage. A suite of 14 physical and chemical tests and analysis was carried out. The original design mix was compared to the test results. These identified the following:

- The compressive strength of the samples varied between $20.5N/mm^2$ and $39.5N/mm^2$. The considerable variation between samples indicates poor site quality control during construction.
- Distress due to alkali silica reactivity was not found.
- Shrinkage values varied between 0.029% and 0.065% (compared with a nominal value for normal reinforced concrete of 0.02%).
- Cement content varied between $148kg/m^3$ and $237kg/m^3$ (average $192kg/m^3$), compared with an original design value of $310kg/m^3$.
- Free water/cement ratio varied between 0.53 and 0.80 (average 0.68), compared to an original design value of 0.60.

The results indicate that that although the concrete had an adequate strength and good aggregate, it had a weekly cemented matrix (low cement content and high water/cement ratio). Additionally the wide range of results indicated that problems may have occurred on site during construction which have affected the durability of the concrete. It was assessed that the concrete was becoming critically saturated and deteriorating by the action of freeze-thaw and/or wetting-drying mechanisms.

DESIGN OF THE WORKS

Prior to designing the dam crest reconstruction works, a flood study and seismic assessment were carried out. These were part of SSE's rolling programme of flood and seismic studies, and were carried out for Lednock prior to the dam crest reconstruction to ensure that if any additional works or alterations to the dam structure were required, they could be incorporated.

The flood study determined the maximum water levels during 1 in 10,000 year return and probable maximum flood (PMF) events. The study concluded that the top water level for the PMF event would be 353.7m O.D. As this is below the 354.0m O.D. level of the top of the dam, it will not be overtopped during the PMF.

Lednock dam is notable for its proximity (five miles) to the Highland Boundary Fault, with which many earth tremors in the area have been associated (Fulton, 1952). The original design of the dam allowed for earthquake forces corresponding to an horizontal acceleration of 0.075g. A site specific seismic hazard analysis study commissioned by SSE for all of its sites determined an horizontal peak ground acceleration for the 10,000 year return seismic event at Lednock dam of 0.24g. Using this data and the output from the flood study, a 3-D linear-elastic static and seismic response analysis was carried out for the dam. The results of the analysis gave comfort not only that catastrophic collapse of the dam with uncontrolled release of the reservoir contents is unlikely, but that reconstruction of the dam crest to the original profile was appropriate.

The sectional profile of the dam crest is shown in figure 3. The as-built drawings indicated that the top 610mm of the dam was constructed using a different class of concrete to the bulk of the dam. It was this Class E concrete which was defective, and was designed to be demolished and replaced. During detailed survey of the dam crest at the start of the construction period, it was found that the line of interface between the Class E concrete and that below varied from 500mm to 800mm below the top level of the dam. The reconstruction work was thus varied in depth to suit the level of the poor concrete, thereby minimising demolition of good quality concrete and the volume of new concrete required.

A 40N/mm^2 designed concrete mix was specified for the reconstructed dam crest. The original drainage arrangement was adjusted to incorporate gullies, and falls to these were increased to encourage effective drainage. The new concrete was detailed dowelled to the original, and nominal reinforcement introduced to control thermal cracking. Resurfacing of the roadway was included in the works.

Other ancillary improvements incorporated into the works were refurbishment of the handrailing and increasing the size of the buttress access manhole openings to allow for safe inspection using modern confined space equipment. In addition the rock fill used in the original construction to form the cut-off between the west end of the crest of the concrete gravity section of the dam and the rock bank was excavated and replaced with mass concrete to form an impermeable seal.

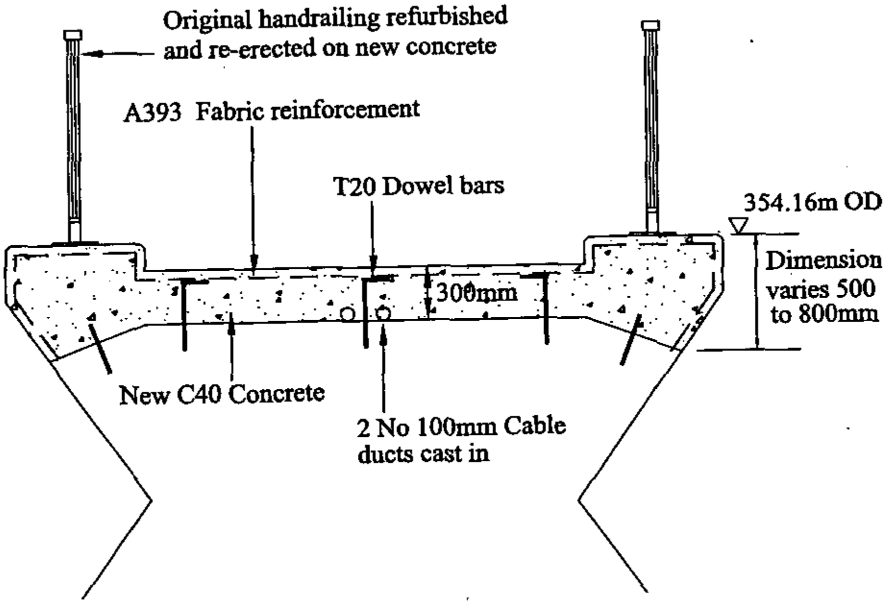

Fig. 3. Section through dam crest.

CONCRETE MIX DESIGN

Lednock dam was unusual for its day in the use of 20% cement replacement by fly ash (Allen, 1959). In the bulk of the dam, but excluding the crest, concrete containing fly ash has performed well over its life to date.

The new dam crest was procured with a performance specification for the concrete. This was a $40N/mm^2$ designed mix to BS 5328. The use of both air entrainment and polypropylene fibre reinforcement were considered, with a view to enhancing the durability of the concrete. Whilst these options were desirable they were not pursued in the end because of logistical difficulties with the long distance for transport to site of ready-mix concrete and its placement by pump.

The contractor's ready-mix concrete supplier, Bardon Concrete, proposed a mix designed for durability, as well as suitablility for the 1.5 hour travel time from their batching plant at Callander, and for transport up to 140m horizontal distance by concrete pump from the dam end to the furthest pours. The accepted mix contained a portland cement and ground granulated blastfurnace slag (GGBS) blend. The GGBS was sourced from Gartsherrie, not far from the site of the Braehead power station, the source of the fly ash used in the original concrete. The mix contained retarding and plasticising admixtures. A copy of the mix used is given in figure 4 below.

Class	Specification		Proportions Used by Weight				
	Min. 28 day strength N/mm^2	Max Agg. Size mm	Min. Cement Content kg/m^3	Water: Cement ratio	Aggregate Sizes		
					20mm %	10mm %	Sand %
C40	40	20	400	0.45	38	17	45
Cement:	60% Portland Cement blended with 40% GGBS						
Admixtures:	Sika Retard 50 (1.35 L/m^3) Grace Advaflow Superplasticiser (1.80 L/m^3) Sika Pumpaid (max 2 L $/m^3$).						

Figure 4 - Concrete Mix Design

CONSTRUCTION OF THE WORKS

The contractor for the works was John Mowlem and Company plc, who were engaged by SSE to carry out the works during 1998/99 under a NEC contract. The cost of the works was £375k. The work fell into two main categories: demolition and reconstruction. For both of these site location and access considerations were significant.

Site Location and Access Considerations

Lednock dam is situated at an elevation of 350m, and is reached from the nearby village of Comrie by an unclassified road featuring steep gradients, sharp bends and a hump-backed bridge. Supply of plant, materials and temporary works to the site were planned to recognise these access limitations. For example, the supply of ready mix concrete in trucks at less than capacity to prevent the concrete from spilling out on the steep hill approaching the dam.

There is no permanent access to the west side of the dam, except for pedestrian access across the spillway. A temporary access was required to allow plant and materials to reach this side of the dam. An early proposal from the contractor to construct a temporary bridge across the spillway was not pursued due to the size and weight of the structure which would have been needed to accommodate the plant and wind loadings. Instead a more cost effective temporary haul road was constructed up the downstream right hand bank to the west end of the dam crest. Demolition arisings from the east side of the dam were used in its construction, thereby reducing the environmental impact of the works by recycling materials and minimising haulage to the site. The haul road was fully reinstated after completion of the works.

Safe access for demolition and reconstruction of the dam crest, which overhangs a 35m drop to one side and water to the other, was taken from mobile gantries running on wheels in bolted down steel channels. The gantries were cantilevered out beyond the sides of the dam crest, and incorporated working platforms arranged to provide working access at the appropriate level. The gantries were supplemented where necessary by a fall arrest system connected to a wire rope or temporary handrailing. Plant operating on the dam during the demolition phase was protected by robust temporary scaffold tube barriers set into cored holes in the concrete.

Demolition

Demolition of the dam crest roadway was done by an hydraulic rotary drum cutter mounted on a 360 degree tracked excavator. This proved very effective in cutting out the concrete quickly and to an accurate profile, and with less impact to the concrete substrate than would have been caused by conventional rock breaking equipment. This method was not suitable at the overhung edges of the crest, where rock drills, plugs and feathers were utilised to break the concrete down in layers to a saw-cut line. Although this method was slower and more labour-intensive than the drum cutter, it enabled the demolition to be completed safely and without dropping arisings into the reservoir or down the downstream face of the dam.

Reconstruction

Reconstruction of the dam crest commenced from the spillway ends of each side of the dam using a travelling gantry incorporating formwork and safe access systems. The operation of the gantries was cumbersome due to their considerable weight, and difficulties were encountered aligning the forms to the eccentricities in the profile of the original construction. For these reasons, the gantries were replaced for the majority of the reconstruction work by a system of bolt on formwork accessed from lightweight access gantries. This allowed original tolerances to be accommodated, and allowed faster progress.

Ready mixed concrete batched at Callander was placed by a concrete pump located at the bank end of each side of the dam. This entailed a maximum horizontal pumping distance of 140m which was within the capacity of the plant used.

CONCLUSIONS

Reconstruction of the crest of Lednock dam was carried out in 1998/99. The work will ensure that the dam will continue to serve its purpose into the next century without the need for costly ongoing maintenance to the dam crest concrete.

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The future of Barcombe Reservoir

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SYNOPSIS A recommendation made in the interests of safety following an inspection under the Reservoirs Act resulted in a proposal to strengthen the retaining bank of a non-impounding storage reservoir. The paper examines the difficulties in balancing the cost of repairs to an apparently healthy 35-year old embankment against the asset value of a reservoir already experiencing operational and water quality problems. A number of issues pertinent to dam safety and the implementation of the Reservoirs Act are discussed.

INTRODUCTION

Issues arising from recommendations made in the interests of safety under the Reservoirs Act at relatively small reservoirs may sometimes be difficult to resolve, especially if influenced by operational matters and by the Owner's business strategy and perception. Proposals to strengthen the retaining embankment of Barcombe Reservoir, following the discovery of unexpectedly high groundwater levels at the downstream toe at one location, posed such a problem for South East Water. How could significant expenditure on stability improvement works be justified when the Company was already facing strategic and operational difficulties that were affecting the asset value of the reservoir?

The 545Ml capacity Barcombe Reservoir was designed as an integral part of the 1965 Ouse Supply Scheme (Fig. 1). Abstraction from the River Ouse, 5km north of Lewes and immediately upstream of the tidal limit, into the off-line bankside storage reservoir is followed by water treatment and distribution of the treated water within the Ouse catchment upstream, and eventually return to the river after use. Water abstracted from the river can be pumped either into the reservoir or directly to the treatment works.

It was originally envisaged that the reservoir would be drawn down to augment the river flow yield. In 1978, completion of Ardingly Reservoir upstream enabled increased abstraction at Barcombe. The primary role of Barcombe Reservoir has been that of operational storage, offering both a balancing and contingency supply facility. However, adverse river water quality and shallow water depth have contributed towards significant algal growth in the reservoir

Dams 2000, Thomas Telford, London, 2000.

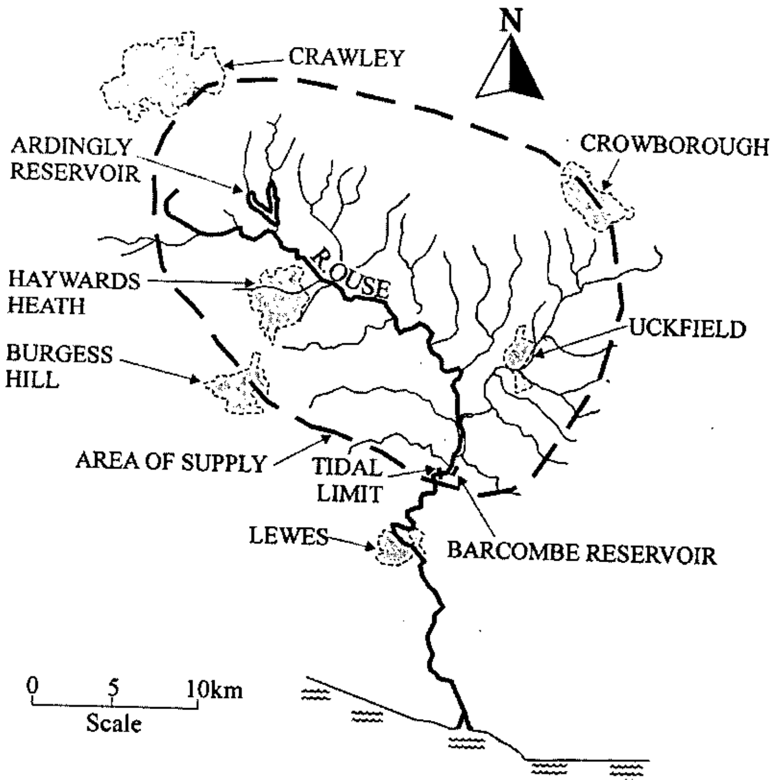

Fig. 1. River Ouse Supply Scheme area

during hot, dry periods. As a consequence, the reservoir is frequently bypassed, limiting its operational use.

MANAGEMENT OF THE RESERVOIR

General

A number of matters influence the Owner's perception of the asset value of Barcombe Reservoir. Firstly, the income and expenditure limits of the privatised water companies are strictly regulated by OFWAT. There is also the role of the Environment Agency. The Agency's interpretation of the existing abstraction licence at Barcombe and their yield assessment methodology has resulted in the quoted output of the Scheme being significantly less than the original estimated yield. This has caused the new Owner, SAUR, to review the value of the asset. On the other hand, the Agency's strategic planning requires the reservoir to provide a minimum emergency storage of 10 days. They also stipulate that any revision of the abstraction licence would be conditional upon the reservoir being restored to its full operational capacity. This has obvious implications for planning improved resources in a region where supply and demand are closely matched.

Water Quality

The river flow contains a high percentage of wastewater effluent and at times of low flow, the quality of the abstracted water encourages intensive algal growth in the reservoir. High levels of nitrates and phosphates in the river water, as well as a relatively high concentration of organic matter, led to the introduction of ferric sulphate dosing prior to storage in an attempt to prevent the formation of blue-green algae. This proved unsuccessful. The problem became sufficiently serious for chemical and biological fieldwork studies to be implemented. The conclusions of the studies were:

- The primary factors responsible for the algal growth are the shallow depth of the reservoir, the long average retention time and the poor quality of the abstracted river water.

- During winter periods the quality of the water drawn from the reservoir was satisfactory whereas during the summer reservoir storage could have a detrimental effect on the water quality due to algal growth.

- During the summer, the quality of the river water was better than that of the stored water and direct abstraction was normally employed. The resulting increase in retention time in the reservoir exacerbated the problem.

Given that a reserve of raw water is regarded as necessary in order to cope with a shutdown of the intake for reasons of pollution or river maintenance as well as providing balancing storage between river abstraction flow and the process plant operation, two solutions were proposed. One was biological pre-treatment of the river water prior to storage. The other was a reduction in retention time in the reservoir, by major remodelling, and improved circulation. In the meantime, ozonation and GAC filtration facilities were provided at the process plant.

The Ouse Supply Scheme, with the reservoir located at the downstream limit of the catchment area, is an example of indirect wastewater reuse. However the quality of the river water available for abstraction is outside the Company's control. The question of who should pay to treat the effluent discharges into the river in order to achieve a defined river water quality objective needs to be answered.

EMBANKMENT PERFORMANCE

The reservoir is situated on the Ouse flood plain between the existing river course and the natural valley side and is retained by an earth embankment 950m long and up to 4.5m in height (Fig. 2). The embankment is founded on very soft/firm Alluvial organic silty clay, between 3-6m deep, overlying Weald Clay, which also forms the valley side. Between the Alluvial Clay and the Weald Clay there is a narrow band of clayey silty sandy gravel, varying in thickness across the site from 0.1 to 1.8m. The embankment overlies a subsidiary river channel near the north-west corner of the reservoir and at two positions along the southern half of the south-western embankment. The embankment was constructed of Weald Clay fill won from within the reservoir area and was founded directly on the Alluvial Clay with slopes of 1 on 3 without a ground drainage blanket. It was completed in 1965. The upper half of the inner face of the reservoir is protected from wave action by concrete slabbing capped by a low wave wall. The design freeboard was 1.07m.

At the end of construction, the downstream shoulder failed over its full height in the vicinity of the old river channel at the north-west corner of the reservoir by sliding through the Alluvial Clay foundation. The slip was stabilised by the addition of a 60m long by 1.5m high berm. Contemporary photographs suggest that some instability developed locally in the upstream slope in this area and also along an estimated length of around 100m towards the southern end of the south-western embankment. In both cases, the movement of the slipped mass appears to have been limited, the vertical displacement at the back scarp being estimated at around 0.5-0.8m. The completed embankment profile includes a 1.5m wide Alluvial Clay fill berm below the toe of the upstream slabbing with an outer slope of 1 on 6. The fill for the full length berm was won from the reservoir bed just upstream of the toe.

There has been a history of settlement of the embankment crest in the two areas where upstream instability is suspected to have occurred at the end of construction. In the north-west corner, levels taken in 1973 indicated a

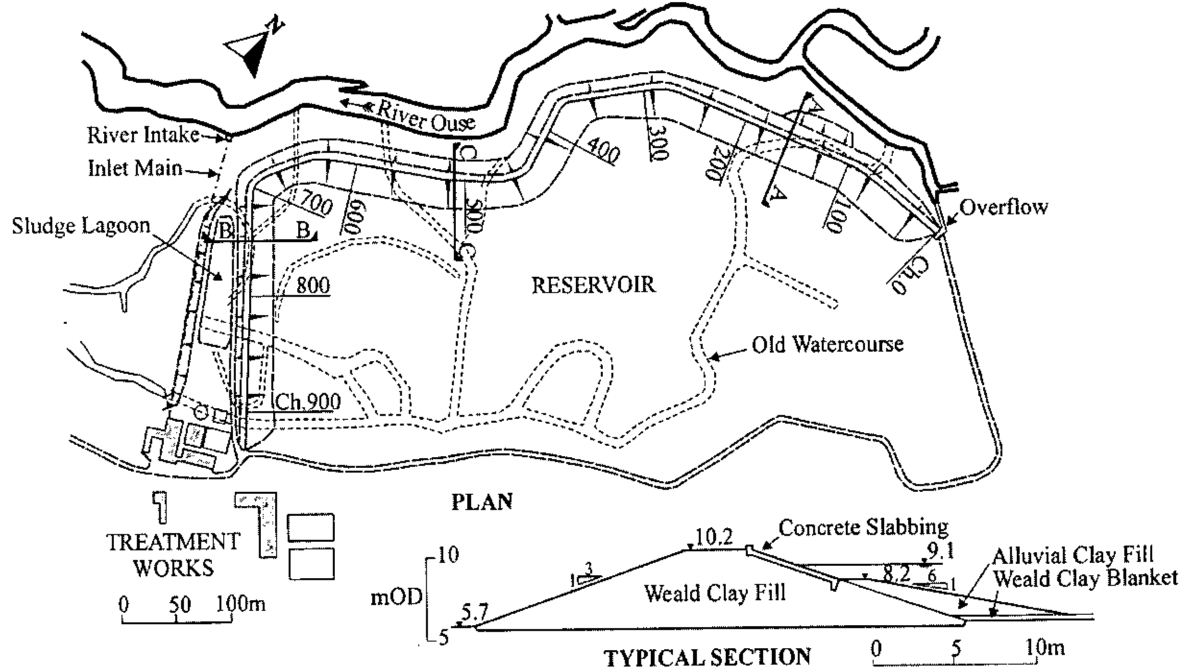

Fig. 2. Barcombe Reservoir - Layout and typical embankment section

maximum settlement of the wave wall relative to the design level of 440mm at a position close to where the old river channel passes obliquely beneath the centre of the embankment. By 1985, this settlement had increased to 570mm. The crest freeboard was made up in 1986 and the wave wall rebuilt. Further settlement of around 55mm was indicated between 1987 and 1995, increasing to around 85mm by 1999. Although less than previously, the indicated rate of settlement since 1987 has remained much the same. Elsewhere along the western embankment the indicated wave wall settlement has generally been around 60-80mm.

The whole length of the south-western embankment has exhibited settlement in excess of 120mm. Levels taken in 1973 indicated a wave wall settlement of between 320-360mm in the vicinity of an old watercourse (Chs. 745-765). By 1995, the maximum value had increased to 470mm and by 1999, to around 500mm, consistent with continuing settlement since 1973 at a roughly unchanged rate. Between Chs. 790-890, overall settlement of between 280-330mm has been indicated.

The embankment deformation in these areas is clearly indicated by incursions of the water line against the upstream slabbing and by slight upstream displacement of the wave wall. At the north-west corner the settlement of the upstream slabbing is locally more pronounced, the pattern of deformation appearing consistent with the development of a shallow surface slip.

STABILITY ASSESSMENT

Following a statutory inspection of the reservoir in December 1994, it was recommended in the interests of safety that the subsidence and stability of the embankment be investigated and, if necessary, remedial works undertaken to ensure that it has adequate factors of safety against failure by sliding. The investigations were to be carried out at at least three different cross sections to include the areas of maximum and minimum settlement. South East Water then appointed a specialist geotechnical contractor to carry out a ground investigation and stability assessment at three embankment cross-sections under the supervision of an independent AR Panel Engineer. As a precaution, the operational top water level was lowered by 0.5m.

The fieldwork consisted of sinking a number of boreholes through the downstream shoulder to the Weald Clay, recovering undisturbed samples for laboratory testing and installing pneumatic piezometers. The sections selected were at the positions of maximum settlement in the north-west corner (A) and along the south-western embankment (B), and at a location along the western embankment (C) where the crest settlement had been minimal (Fig. 2). Some water inflows from the Alluvial Gravel layer were noted during sinking of the boreholes. Unexpectedly, artesian head was met in the Alluvial Gravel at Section C in a borehole just beyond the embankment toe, the water level rising to 1.1m above ground level in 15 minutes and reaching a height of 2.1m overnight. Similar groundwater elevations were encountered at the same section in the foundation beneath the downstream shoulder, suggesting that there might be a connection between the reservoir and the Alluvial Gravel near this location. Whereas a pneumatic piezometer placed within the Alluvial Gravel in the artesian borehole subsequently registered a maximum water level of only 1.1m below ground level, a shallow standpipe in the Alluvial Clay was slightly artesian.

Effective stress analysis of the stability of the downstream shoulder at Section C using a phreatic surface based on the piezometer and borehole water levels gave a minimum factor of safety of 1.12. Analysis based on potential worst case steady seepage conditions gave a factor of safety of 1.05. On the basis that the minimum acceptable factor of safety should be 1.35, the standard normally adopted for the design of a new dam, it was concluded that measures to improve the stability of the downstream shoulder were essential at this section. Since it was unclear whether the high phreatic surface was typical of the conditions elsewhere, further investigation was recommended to define the areas to be strengthened more precisely but this was not pursued at that time. The preferred remedial measure was the addition of a berm. Strengthening of the downstream toe of the south-western embankment opposite the sludge lagoon (Section B) was also advocated. The stability at Section A, which had been strengthened by the berm at the end of construction, was satisfactory. Using an inferred failure slip surface and residual values for the angle of shearing resistance, a factor of safety of 1.55 was obtained for the observed piezometric levels.

Analysis of the stability of the upstream shoulder during rapid drawdown indicated a factor of safety of 1.5. In areas where previous instability is believed to have occurred, a value of around 1.0 was indicated when taking the angle of shearing resistance along an inferred slip surface as the residual value. Whilst displacement of the upstream slope during drawdown could result in operational difficulties, it would be very unlikely to result in escape of water that would endanger the safety of the reservoir.

RISK ASSESSMENT

Since reservoir safety is concerned with preventing the uncontrolled escape of water, a distinction needs to be drawn between a failure mechanism that affects the structure of the dam, without compromising its ability to retain water in the short term, and one which results in a breach leading to inundation of areas downstream. In the case of a dam located upstream of a heavily populated area, the potential casualties and damage resulting from a failure would be great.

Impounding and non-impounding reservoirs present different hazard and risk characteristics. Under normal circumstances, the non-impounding reservoir will be the less hazardous, having virtually no direct catchment. The inflow is pumped and therefore controlled and the maximum volume of the flood that would escape in the event of a failure is unlikely to exceed the storage capacity of the reservoir. In contrast, the volume escaping in the event of the failure of an impounding dam could be much greater than the capacity of the reservoir because of the additional inflow due to run-off from the catchment. For a bankside storage reservoir on a flood plain, the water escaping from any breach would disperse over a wide area. The impact of a failure would be further reduced if it were to occur at the same time as a flood in the river.

Barcombe Reservoir has a negligible direct catchment and is filled almost entirely by controlled pumping from the River Ouse. The most likely mechanism that could initiate failure of the embankment is not clear. Overtopping is not seen as a realistic threat and the risk of internal erosion of a homogenous clay embankment, without any structures passing through it, is judged very slight. The reservoir can therefore be regarded as presenting a relatively low hazard. Nevertheless, a dam break analysis was undertaken to identify downstream areas that would be affected and to put the risk of a potential failure into context. The inundation resulting from a failure of the reservoir was compared with the situation that would arise under various return period floods in the River Ouse.

Contributory factors to flooding in the Ouse valley include a very flat river valley, rapid run-off from the clay soil catchment and the effect of the tides on discharges over the complex system of weirs, gates and fish passes at Barcombe Mills just downstream of the reservoir. The current level of protection is such that flooding of properties at Barcombe Mills occurs above a 1 in 5 year event; the conversion to housing of previously uninhabited buildings has increased the hazard potential.

Since the mid-1950s, there have been four notable flood events. For the period 1956-1985, the largest recorded flood was 129 cumecs in 1974. Since then, peak flows of 122 cumecs (in October 1987), 144 cumecs (in January 1990) and 153 cumecs (in December 1993) have been recorded, with estimated return periods of 7, 15 and 20 years respectively.

Despite the uncertainty surrounding the exact mode of failure, the dam break analysis was based on the assumption that a breach would form through the embankment leading to an escape of water from the reservoir. The worst-case scenario of the failure occurring when there was no flooding in the main river was assumed; the effect of superimposing the flow from a breach of the embankment on larger floods in the river was not considered as there was no evidence that any past flooding had damaged the reservoir embankment.

The sensitivity of the analysis to the rate of formation of the breach was investigated. If the breach were to form rapidly, say in under 10 minutes, the resulting flood would likely be less than that which would occur in a natural 50-year flood event in the River Ouse. If the breach were to take 0.5 hour to form, the severity of the flooding would be reduced to that of a 20-year flood event.

The "bankfull" capacity of the River Ouse channel at Barcombe is 85 cumecs. For all cases considered in the risk analysis, much of the flood flow would be contained within the normal width of the river channel. The analysis also indicated a rapid attenuation of the peak flow downstream of Barcombe, such that a breach of the reservoir embankment would result in peak flows at Lewes equivalent to less than a 5-year return period flood.

The overall conclusions of the risk assessment were:

- the risk of catastrophic failure of the Barcombe Reservoir due to breaching is low

- the effects of a failure of the reservoir in respect of loss of life and flooding of properties downstream would be no worse than would occur in a 20-year flood event in the River Ouse.

OPTIONS AND SOLUTIONS

Improving Embankment Stability

The need for strengthening measures placed the reservoir Owner in a dilemma since the embankment was a proven structure having stood for 35 years with no visible outward signs of undue distress. It was difficult for the Owner to accept that it had become less safe. The implications of the recommendations were significant in the context of the operational difficulties already being experienced and the Owner therefore undertook an appraisal of the beneficial value of the reservoir to the Supply Scheme and to the overall business.

The options considered were:

- Abandoning the reservoir

- Reducing the reservoir capacity to less than 25,000 m^3

- Strengthening the complete reservoir embankment

- Delaying the implementation of the strengthening measures.

The lowest cost option, that of withdrawing the reservoir from operational use, was financially attractive in terms of capital expenditure, but introduced a severe risk by having to rely completely on direct river abstraction for an 80 Ml/d plant throughput. It was also politically unacceptable since the reservoir features in the Environment Agency's strategic resource planning. Removing the reservoir from the scope of the Reservoirs Act by reducing its capacity to less than 25,000 m^3 was considered unrealistic because the depth of water remaining would be minimal. Strengthening the full length of the reservoir embankment was financially unattractive on cost grounds.

Delaying carrying out the strengthening works is not unreasonable for a low embankment because reservoir safety is concerned with preventing the escape of water from the reservoir rather than with the reserve of stability of the dam itself. Failure of the downstream shoulder might not inevitably result in breaching of the crest and escape of water. Since the shearing behaviour of the Alluvial Clay is plastic, rather than brittle, the amount of displacement needed to restore equilibrium of the slip mass is expected to be limited. The photographs of the 1964 slips suggest that the heights of the back scarps of the slip masses were less than the design freeboard. Thus, any vertical displacement of the crest would not necessarily lead to breaching, provided that the design freeboard had been maintained. The upstream slabbing might also briefly delay the formation of a breach. However, the undesirability of adverse publicity in the event of a failure of the embankment might deter the Owner from adopting this approach. It was not considered further.

Subsequently, South East Water arranged for the next statutory inspection, due at the end of 1999, to be brought forward. The report of this inspection recommended that the downstream stability of the western embankment should be improved. If the analysis at Section C was accepted as typical, the previously proposed toe berm should be constructed along a 500m length. Otherwise, a series of standpipe piezometers and small diameter relief wells should be installed at the toe to investigate whether the groundwater conditions at Section C are representative and if so to define the required berm length. Alternatively the effectiveness of relief wells in lowering the phreatic surface to improve the stability should be examined. In addition, the sludge lagoon at the toe of the south-western embankment was to be partially infilled and the existing monitoring arrangements continued.

Improving Water Quality

The two options for dealing with the water quality problems retained the reservoir as an essential component of the Supply Scheme, as well as providing solutions to both the water quality and embankment stability problems.

Pre-treatment of abstracted water would utilise bioremediation to remove algal blooms in the reservoir and inhibit any algal growth hydraulically upstream of it. New ponds, at least 10 metres deep, would need to be constructed between the river and the reservoir. Extensive use of barley straw, together with hydraulic mixing, would be employed within the ponds. This option was rejected due to the lack of suitable sites near to the Works.

Reservoir remodelling would allow for the reservoir to be partitioned to provide two smaller reservoirs, one utilising the maximum depth of water available. The other could become an environmental lagoon. The basic layout proposed would reduce the percentage volume of shallow water and improve the operational flexibility of the bankside storage. The costs of strengthening works would be reduced because the measures would be required over a shorter length of embankment. Although this solution was included in the Company's plans, the high capital cost (of the order of £3.5m) coupled with OFWAT's 1999 price review have resulted in this option being deferred.

DISCUSSION

Implementation of Reservoirs Act

The fact that recommendations made in the interests of safety are mandatory does not preclude the Owner from considering further the implications to his business of implementing them. The risk to his overall business may be as important to the Owner as the Inspecting Engineer's evaluation of reservoir safety. Safety recommendations under the Act cannot be viewed in isolation.

At Barcombe there were already doubts over the future role and value of the reservoir because of recurring operational difficulties. When faced with the requirement of committing significant expenditure on an asset of questionable value, the Owner faced a dilemma: should he proceed with this expenditure for safety reasons when he was already doubtful of the long-term future of the reservoir, or should he seek an alternative approach that would prove to be acceptable in respect of both safety and operational benefit, even if that resulted in the withdrawal of that asset?

How enforceable is the Reservoirs Act? Does it take precedence over other legislation affecting dam owners such as for example, the Environmental Protection Act (1990) or the Water Resources Act (1991)? What if there is a conflict between the need to comply with a safety recommendation and the need to maintain supply to consumers? Whilst most objective opinion would acknowledge that dam safety must be given a high priority, the situation is not always clear-cut.

Experience has shown that for small and/or privately-owned reservoirs, an Inspecting Engineer's recommendations in the interests of safety involving extra expenditure may not be acted upon if they lack credibility. An Owner, who does not understand the reason behind a recommendation or is not convinced that it is needed, may prevaricate, explore alternatives or simply ignore the recommendation, despite it being mandatory. The task of evaluating the stability of existing embankment dams is ultimately one of engineering judgement; faced with a recommendation that his embankment dam needs to be strengthened to satisfy theoretical safety standards, an Owner may need to be convinced that a real safety risk exists and that the work is essential.

In many areas, society is nowadays more reluctant to accept the authority of the individual without question. As a consequence, the concept embodied in the Reservoirs Act of an independent Inspecting Engineer with individual responsibility is increasingly being undermined. This concept also does not seem to fit well with subsequent legislation affecting dam owners.

At Barcombe, the Owner's response to the proposed strengthening measures arising from a recommendation in the interests of safety was influenced by the Environment Agency's close interest in operational matters which contrasted with the Enforcement Authority's apparently purely administrative role in relation to the application of the Act.

Stability of Existing Embankment Dams

There is currently no consensus on what is an acceptable factor of safety against sliding for an existing embankment dam.

For a new, untried, structure, minimum factors of safety of 1.3 for the end-of-construction case and 1.35 for the steady seepage case are commonly adopted. The reserve of safety caters for differences between the actual performance of the structure and that predicted at the design stage from laboratory determinations of the strength parameters for the fill and foundation materials and of the rates of pore water pressure generation during fill placement and their subsequent dissipation. The most critical stage is normally the end of construction, with stability improving thereafter.

For an existing dam, the stability of which has been proved over a substantial period of years, a smaller reserve of stability may be considered appropriate. Major slope stability failures of an embankment dam in service have been very few, most problems occurring either during construction or during first impounding (Charles et al.,1998). The main factors likely to affect the stability of an existing dam, excluding drawdown failure of the upstream slope, are a deterioration in the strength of the constituent materials; a rise in the phreatic surface in the downstream shoulder; or an overall steepening of the embankment slopes, caused, for example, by modifications for additional works. In most situations, a deterioration in the effective shear strength parameters, unless caused by internal shear displacement, seems unlikely as evidenced by several dams with a proven performance record of 150 years or more. A rise in the phreatic surface is usually manifested by the appearance of reed growth, wet spots or seepages in the downstream face. The provision of a level of surveillance, combined with deformation and seepage monitoring, sufficient to detect the occurrence of these or any other warning signs is the best defence against some untoward incident developing; with such an arrangement it should be possible to counter the development of instability at an early stage.

An Inspecting Engineer assessing the stability of an existing embankment dam will exercise his professional judgement, taking account of the visual appearance of the embankment, its performance record and any underdrain flow measurements, crest settlement records and instrumentation observations available. Stability analyses are not usually undertaken unless some particular matter of concern is identified and even when they are, the assessment may not rely wholly on the numbers derived from the calculations. The stability at one section may not necessarily be applicable to the whole embankment. Nevertheless, an item of unwelcome evidence, whether observed or calculated, can be very difficult to ignore.

Any concerns about the state of the embankment can be addressed in the first instance by introducing monitoring arrangements and increasing the level of surveillance. Exploratory boreholes sunk through the embankment to recover samples for laboratory testing or to install instruments need to be undertaken very carefully to prevent either the soil properties or the dam's integrity being adversely affected by the investigation techniques themselves.

If the stability assessment of an embankment dam depended on having to show that the calculated factor of safety exceeded a minimum value, the situation could arise where strengthening measures were advocated for a dam which had hitherto exhibited a seemingly satisfactory performance history. This could lead to a credibility gap between the Owner and the engineer and a reluctance to undertake costly repairs. The Owner could question the viability or representativeness of the soil parameters determined from a ground investigation, query the assumptions made in the calculations and perhaps seek a second opinion. With each dam site being different, there are dangers in a generalised approach. Any factor of safety criterion would also need to be related to the hazard posed by the reservoir, particularly to life and property downstream.

CONCLUDING REMARKS

The implementation of a mandatory recommendation in the interests of safety under the Reservoirs Act may not be straightforward, particularly at small reservoirs if the asset value of the reservoir is declining and there is no other benefit to the Owner. Dam safety cannot be considered in isolation from strategic issues and operational constraints and other legislation affecting dam owners.

Establishing that an existing embankment dam has a sufficient reserve of safety against sliding may be problematical. The need for remedial measures merely to satisfy a minimum number criterion could meet with resistance by the Owner, particularly for small reservoirs and if the dam has an apparently unchequered performance history. Intrusive investigations to determine soil parameters in establishing the reserve of safety need to be undertaken and interpreted with care.

At Barcombe, further investigation is proceeding to establish the extent of the high groundwater level conditions at the downstream toe of the western embankment and to determine how effective relief wells would be in lowering the groundwater pressures.

ACKNOWLEDGEMENTS

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Geotechnical investigations at Abberton Dam, Essex

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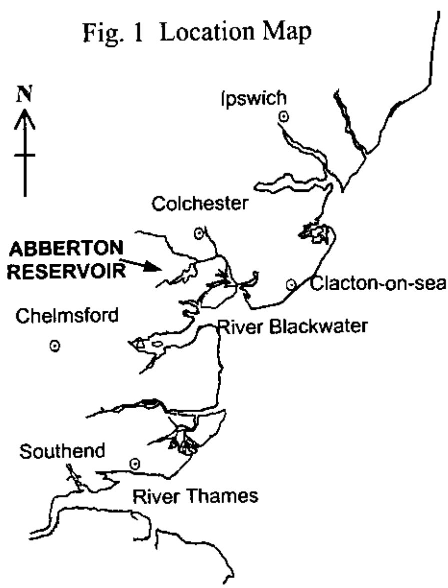
SYNOPSIS. Geotechnical investigations have been undertaken to assess the present condition of Abberton Dam in Essex, prior to consideration of options for raising the reservoir. The 16m high embankment dam suffered a failure during construction in July 1937, nine days before a similar and famous failure at Chingford Reservoir in Essex. The paper describes the dam and a little of its history, and reports the principal findings of the fieldwork and the piezometer monitoring programme.

INTRODUCTION

Abberton Reservoir is situated on the Layer Brook, near and to the south of Colchester in Essex, refer to Fig. 1. The embankment dam is 700m long and at its highest point the crest is 16m above original ground level. The dam is 60 years old; construction started in March 1936 and was completed in August 1938. It was built to a conventional design for the time with a central core and cut-off of puddle clay. During construction, in July 1937, there was a major slip over the central section of the upstream slope and the failed area was rebuilt with very flat slopes.

Fig. 1 Location Map



Construction of the dam was completed in 1938 but, as a precaution against bomb damage, the reservoir level was kept below TWL throughout the Second World War. The reservoir level was then allowed to rise and TWL was reached for the first time in March 1947.

The dam has performed well during its life and appears to be in good condition. Leakage through the dam and foundation are low. Records suggest that total settlement since the end of construction is in the order of 600mm at the highest section of

Dams 2000, Thomas Telford, London, 2000.

the dam, which is considered normal for a puddle clay dam (Watson Hawksley, 1990).

Geotechnical investigations were undertaken between 1995 and 1997, including a line of boreholes to investigate a principal cross-section through the dam. A system of piezometers has been installed and is being monitored.

GROUND CONDITIONS

The dam site is underlain by London Clay to a depth of about 30m, which in turn is underlain by Lambeth Group deposits. Across the valley floor the London Clay is covered by various superficial deposits. On the valley sides, head deposits are present to a depth of around 2 to 4m. At the west abutment old river terrace deposits including sands and gravels are found beneath the head deposits. Surface gravel deposits are present at the east abutment. Recent alluvium fills the centre 250m width of the valley, to a depth of around 6m. This consists of a variety of soils including clay, sands and gravels, and a thin layer of peat.

The vicinity of the dam is distinctive for a number of geological features. The site was probably at the margin of the gravel valley trains of the proto-River Thames (Whiteman, 1992) before later being just beyond the limit of the Anglian ice front (less than 500,000 years ago) and so covered also by outwash gravels (Allsop and Smith, 1988). The valley which the dam crosses was mostly eroded after the ice retreated by downcutting to the deeply incised River Blackwater valley a short distance downstream. That river later suffered capture leaving the underfit Roman River to occupy the valley (Bristow 1985). However, the locally undercut right bank of that valley formed in the London Clay was unstable and has left a Late Pleistocene active landslip system close to, but not directly affecting, the dam site.

There may have been some valley bulging at the site during the Late Pleistocene but the limited evidence is complicated by possible tectonic displacements along a fault line, possibly the Galleywood Fault which lies at depth beneath the site. The epicentre of the 1884 Colchester earthquake and its smaller precursor (1883), probably at Peldon, had their epicentres within 1km of the dam site (Skipp et al, 1985).

DESCRIPTION OF THE DAM AND RESERVOIR

A plan of the dam is given as Fig. 2 and a typical cross-section as Fig. 3.

The main water retaining element is a central core of puddle clay, which extends below ground level to form a cut-off into the underlying London Clay. The outer shoulders of the dam are formed of locally won sands and gravels. The downstream shoulder incorporates a vertical coarse gravel wall drain and a horizontal drainage blanket. The upstream slope is protected by concrete block pitching 300mm deep laid on a 150mm gravel underlayer with a concrete wave wall along the crest. The crest is about 3.5m wide with a track constructed of hoggin.

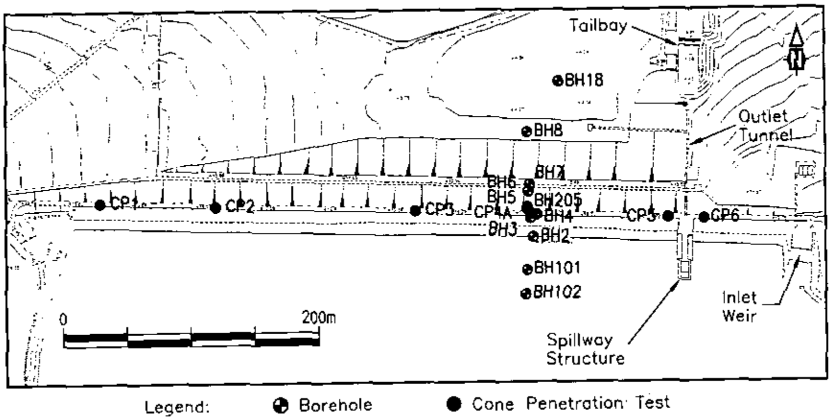


Fig. 2 Plan of Dam

The upstream slope was originally constructed at 1v:4h. Following the failure during construction, it was reconstructed to a revised design with very flat slopes of gravel, see Fig. 3.

Pumped water enters Abberton reservoir over an inlet weir located at the right abutment of the dam. Water levels in the reservoir vary during the year within the approximate range +14m to +18mOD. Towards the eastern end of the dam there is located a combined scour pipe outlet shaft and 'swallow hole' spillway. A tunnel culvert leads from this through the dam embankment to a tailbay.

The reservoir formed by the dam covers a large area of relatively flat Essex countryside. The usable volume of the existing reservoir is some 22000Ml and the surface area at TWL is some 485ha. Total volume at TWL, based on bathymetric survey, is approximately 25000Ml. The reservoir is crossed by two road causeways which split it into three pools. The main eastern pool is the bulk of the reservoir; it extends from the dam wall in the east to the B1026 causeway at its western edge. The second pool is much smaller and lies between the B1026 causeway to the east and the Layer Breton causeway to the west. The westernmost pool is retained behind the Layer Breton causeway to a TWL some 0.5m higher than for the other two pools.

The two western pools are connected hydraulically to the main reservoir through weirs at the road crossings. A small inflow to the reservoir arrives from the Layer Brook via this route. Both of these western pools have become important refuges for birdlife as a result of the shallow water, the seasonally exposed mudflats and the lack of human habitation and access to the shoreline.

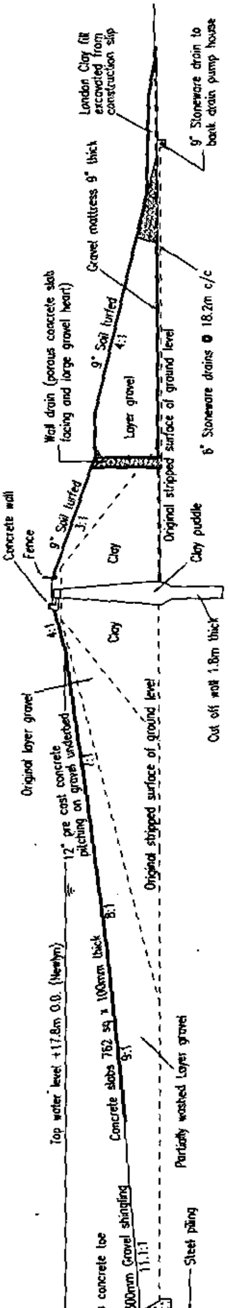


Fig. 3 Typical cross-section of dam

The water company raised a Parliamentary Act to construct the reservoir and also to purchase a swathe of land around the site. The land which is not occupied by the reservoir is rented out to mainly arable farming. Ownership of this land allows ESW to have perhaps more management control over the catchment than would otherwise be the case.

Water is abstracted from the reservoir at a pumping station located on the northern shore of the lake and is transferred to the Layer Treatment Works about 1km further north.

THE FAILURE DURING CONSTRUCTION

The slip took place on July 20 1937. The embankment construction was within about 2m of the intended top level. Archive photographs show the overall scale and nature of the slip, see Plates 1 and 2, and it appears to have occurred over a period of about a day. The dam crest dropped by 3 to 3.5m and the upstream toe moved outward by about 15m. The top part of the puddle clay core was displaced. A survey of the slip was made and is reproduced as Fig. 4 (Watson Hawksley, 1990). The extended S-shape of the slip is consistent with a non-circular failure surface running through the foundation below original ground level.

An archive long-section which charts the reconstruction of the puddle clay core indicates that the slip zone was about 175m long, extending from 60 to 235m west of the outlet tunnel. The slip zone coincided with the highest part of the dam and also the area of alluvium in the valley bottom.

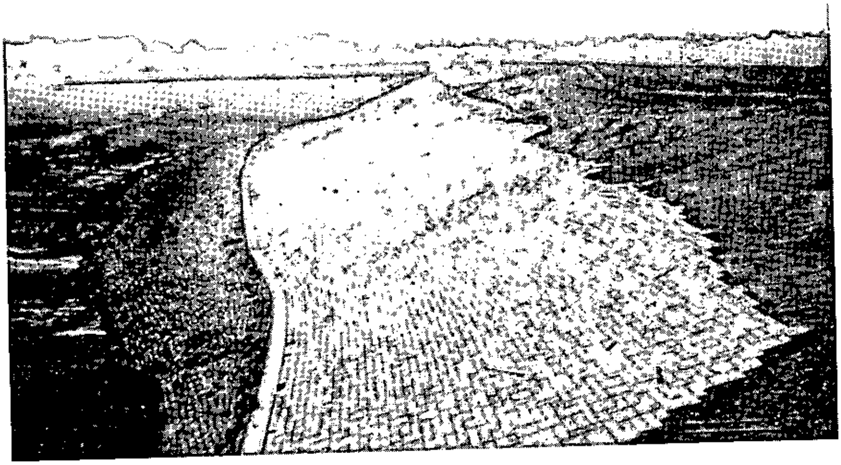


Plate 1 General view of failure on 21 July 1937

Plate 2 View of displaced puddle core on 20 July 1937

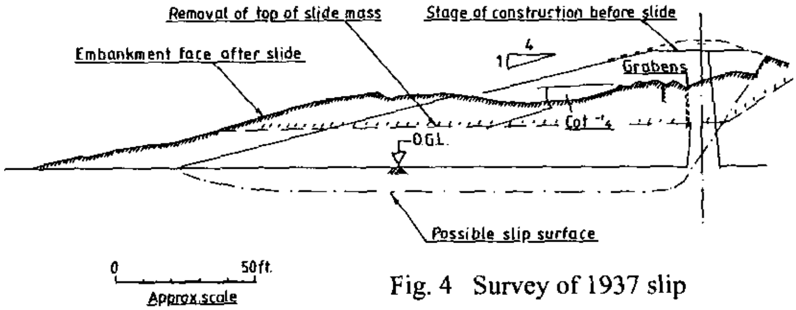


Fig. 4 Survey of 1937 slip

As a matter of historical interest, the slip occurred nine days before a similar failure during the construction of Chingford Reservoir in Essex. The failure at Chingford is recognised as a milestone in the early development of soil mechanics in the UK, and the investigation into the failure marked Karl Terzarghi's first professional assignment in this country (Penman, 1986; Cooling & Golder, 1942). However, Terzaghi did not become involved at Abberton and there is no record of formal soil mechanics principles being applied in either the original design of the dam or in its reconstruction.

Disturbed slip material was removed prior to reconstruction of the upstream slope, but the extent to which this was done is not clear from the archive information. The formal description of the works dated 1947 states that all disturbed material was removed whereas a sketch dated 1939 suggests that the majority of the slip mass was left in place and trimmed off to about +9mOD.

GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations were undertaken in two phases between 1995 and 1997 and were designed to provide geotechnical data for assessing the engineering feasibility of raising the reservoir level by up to 3m.

The Phase 1 investigation took place in the vicinity of the main dam embankment in May and June 1995. This investigation included boreholes, trial pits and cone penetration tests (CPT) put down to confirm the sequence and properties of the construction and foundation materials, seepage conditions through the dam and the condition of the puddle clay core. The Phase 2 investigation was carried out between December 1996 and March 1997. It comprised a supplementary borehole investigation of the main dam embankment. This included boreholes put down from pontoons through the upstream slope and investigation by boreholes and trial pits of locations around the reservoir that would be affected by any raised reservoir scheme, such as existing causeways, possible col dam sites and potential borrow areas.

The location of the main exploratory holes across the dam are shown on Fig.

2 and the results of the fieldwork are summarised below.

Trial pitting in top of core

Several hand-dug trial pits were dug into the crest of the dam to investigate the top of the puddle clay core. The construction arrangement at the dam crest was found to be generally as shown on the archive records. The top of the core slopes downwards towards the upstream, by about 0.1m to 0.3m in level across its width of 1.5m. The top of the puddle clay is protected by hardwood planking which is in excellent condition.

The puddle clay was found to be in excellent condition, being both moist and malleable. In situ tests in the top metre or so of the core using a hand-held shear vane indicated undrained shear strengths of between 23kPa and 33kPa with an average of 28kPa. On completion of testing the excavated puddle clay was 'heeled' back into place and short lengths of drainage pipe were installed for CPT probes to pass through subsequently. The hardwood planking was then cut to fit and replaced and the crest road reinstated.

It is interesting to note that during the war the reservoir level was kept 4 feet below top water level as a precaution against the effects of bombing. There were concerns at the time that the top of the puddle clay core might deteriorate under these conditions but trial pits dug in 1945 revealed it to be in excellent condition. The reservoir level was then allowed to rise and top water level was reached for the first time on 14 March 1947.

CPT probing through core

CPTs were undertaken at six locations along the dam crest vertically through the puddle clay core and cut-off trench, and penetrating into the underlying London Clay by between 2 and 5 metres. The overall depth of penetration was between 13m and 26m depending on location and height of the dam. A $10cm^2$ 'piezocone' was employed, with a porous element to measure pore pressures located at mid-height on the face of the cone. The equipment was operated from a standard 20 tonne truck operating from the crest road.

The piezocone probing was performed successfully, with saturation of the porous element being maintained throughout all of the tests. The depth to London Clay at all locations matches closely an archive long-section of the cut-off trench. Provision was made under the contract to grout up any hole created in the puddle clay core as the piezocone was withdrawn. However, grouting trials on site indicated that the hole created by the piezocone was closing up immediately of its own accord and as a result only the top 5m of hole was grouted.

Profiles of cone resistance against depth measured in a selection of the tests are plotted together on Fig. 5 and it can be seen that cone resistance is fairly consistent from location to location and generally increases with depth before increasing suddenly when the London Clay is reached. The results indicate undrained shear strength values of around 25kPa to 30kPa at the top of the core with usually a small but steady increase in strength with depth to around 35 to 40kPa. No marked changes in strength at the supposed interface at about +9mOD between the pre-slip and post-slip puddle clay were apparent. Also, no reduction in cone resistance is apparent where the core narrows into the vertically sided cut-off trench. A reduction in cone resistance at this point would be consistent with problems of arching action and low total stresses within the core.

Consecutive sampling through core

Sampling of the puddle clay core was undertaken at two positions on the crest using the Goudsche Machinefabriek (GMF) system. The objective was to obtain 'consecutive' high quality samples through the core for detailed description and examination in the laboratory. The GMF sampler is a proprietary device designed to recover high quality samples in soft to firm cohesive soils. The samples are 66mm diameter by 750mm long. The sampling device is pushed into the ground using standard CPT equipment and samples are taken in thin wall PVC liners, with each sample following on immediately from the previous sample. The sampler is operated by wireline within a casing which supports the borehole at all stages.

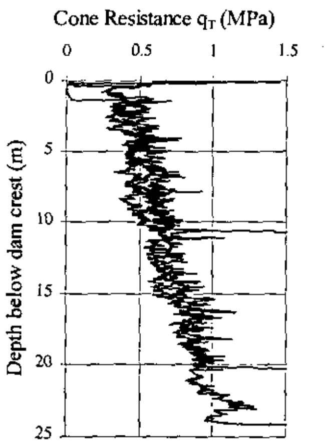


Fig. 5 CPT probings in core

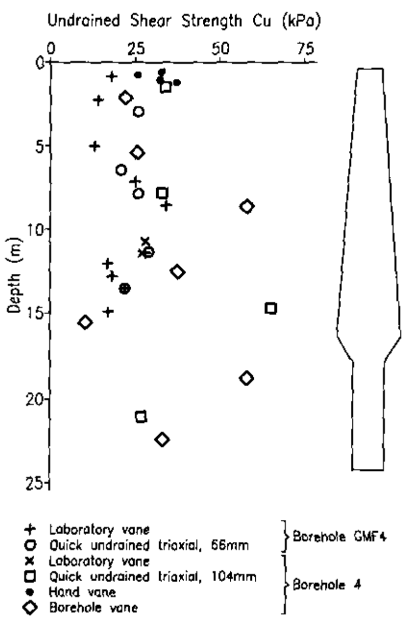


Fig. 6 Undrained shear strength tests in core

The GMF sampling was performed successfully to depths of penetration of 18.3m (the base of the cut-off trench) in the first hole and 15.2m in the second. The GMF boreholes were grouted up using a 1/1/12 bentonite/cement/water mix upon completion of sampling.

The consecutive samples were split and described in the laboratory with particular attention being paid to a possible 'construction joint' in the core due to reconstruction of the dam following the 1937 slip. The puddle clay was found generally to comprise reworked soft greyish brown slightly sandy silty clay with isolated pieces of fine to medium siliceous gravel. The material was generally recovered as soft to firm stiff clay relicts (<80mm in size) set in a matrix of very soft to soft silty clay. Organic debris was occasionally found. Local variations in colour, gravel content and fabric were found throughout the depth of the boreholes, but in general there appeared to be no difference between the puddle clay above and the puddle clay below the level of reconstruction indicated on the archive long sections. Local changes in colouration, gravel/organic content and fabric suggested a possible core reconstruction level in good agreement with the archive long-section.

The results of undrained shear strength determinations made on samples of the puddle clay obtained from Borehole 4 and GMF4 are shown on Fig. 6. There is a much greater scatter of results than for the CPT probings.

<u>Investigation of cross-section through dam</u>

A line of eleven cable percussion boreholes was put down to investigate a cross-section through the dam at chainage 120m west of the outlet tunnel. This chainage corresponds approximately to the highest part of the dam and is within the old slip area. A cross-section showing the location and depths of the boreholes is shown as Fig. 7.

Scaffolding was required for those boreholes put down through the dam slopes (Boreholes 2, 3, 5 and 6). Boreholes 101 and 102 through the upstream slope were undertaken from overwater positions using a pontoon. All boreholes were fully cased throughout their length. For Boreholes 3, 4 and 5 special checks with an inclinometer were made to ensure verticality. Anti-pollution measures were enforced for boreholes on the upstream side to prevent any leakage of hydrocarbons or other deleterious materials into the reservoir.

The dam cross-section was found to be somewhat different from the formal contemporary records in that slipped material in the upstream slope appears to have been left in place and trimmed to about +9.5mOD. The construction of the downstream slope conforms with the records except that the drainage blanket was not encountered as a distinct gravel band.

The overall geology and foundation conditions beneath the embankment dam have been confirmed to be similar to those as shown on an archive long-section

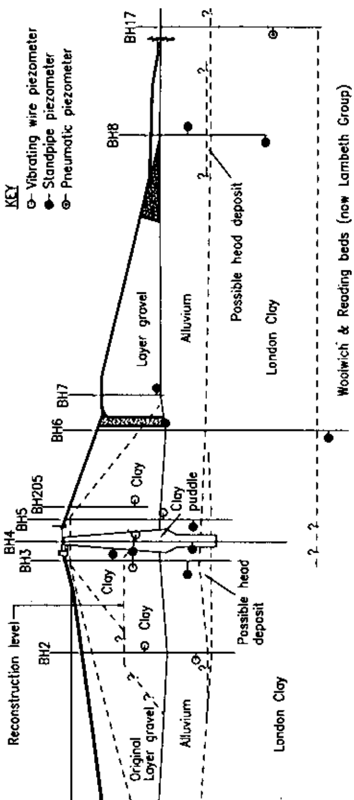


Fig. 7 Cross-section showing location of boreholes and piezometers

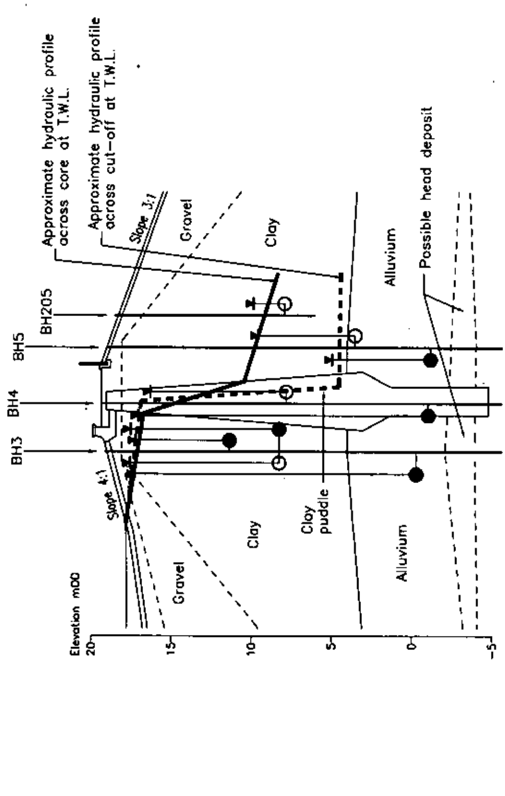


Fig. 8 Cross-section through core and cut-off

through the core trench, consisting of London Clay covered by alluvial deposits. Slightly weathered London Clay is encountered at around -4mOD. In Boreholes 2, 3, 4 and 8 the London Clay is covered by a 0.6 to 1.8m thick layer of weak, sheared material described variously as colluvium, head or frost shattered London Clay.

BACK-ANALYSIS OF CONSTRUCTION SLIP

Preliminary back-analyses of the 1937 slip have been undertaken and two possible failure mechanisms have been considered.

- a non-circular failure mechanism running through the Alluvium (see Fig. 4)

- a rather deeper failure mechanism running through the top of the London Clay, through the layer of weak, sheared material described above.

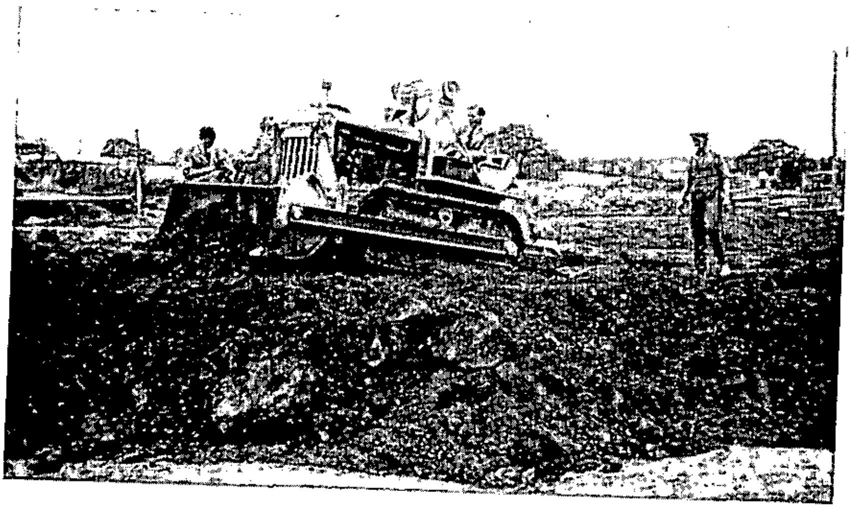
The analyses indicate that the failure was caused by excess pore pressures in the foundation. The value of the pore pressure ratio r_u required to bring about failure is in the order of 0.5 to 0.6, depending on the exact failure mechanism considered. This range for r_u is entirely plausible given the rapid construction period and the absence of any drainage installed in the foundation. Due to the presence of natural sand and gravel layers, r_u values during construction are likely to have been lower in the Alluvium than in the London Clay and the analyses suggest that a failure surface through the top of the London Clay is the most likely failure mechanism.

It is interesting to draw parallels between the failure at Abberton with the failure at Chingford. Both slips exhibited the classic 'S' shaped profile associated with a spreading failure of fill placed above a weak layer. At Chingford the weak layer comprised a soft yellow alluvial clay, whereas at Abberton the weak layer was most probably the upper, soliflucted surface of the London Clay.

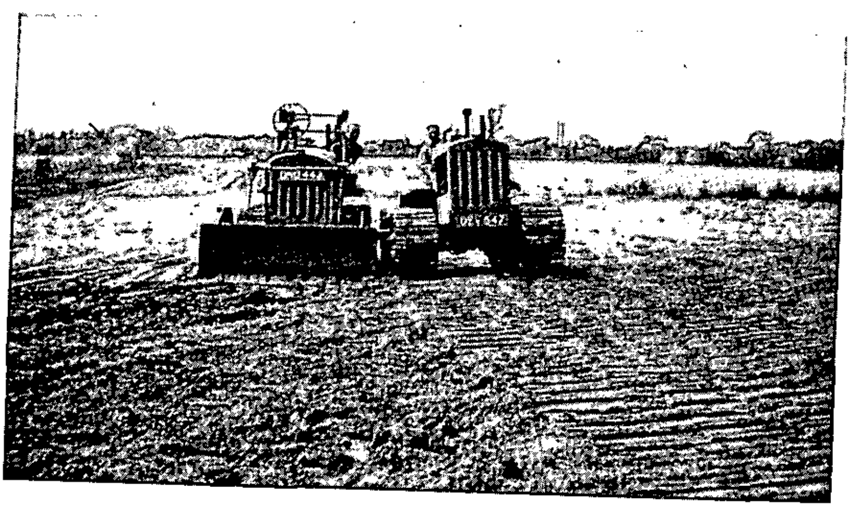
Both dams were built to traditional designs that had previously been constructed successfully. However, they were amongst the first dams in the UK built using 'modern' earth moving equipment (see Plates 3 and 4) and construction was quicker and more efficient than had previously been common practice. The rate of construction was certainly fast at Abberton, fill having been raised to near full height in 11 months from August 1936 to July 1937. It would seem likely that a contributory factor to the failures at both dams was that the more efficient, rapid construction techniques than had hitherto been the case resulted in less time for pore water pressures in the foundation to dissipate during construction.

PIEZOMETER MONITORING

Various types of piezometers were installed in the boreholes on completion of boring, as shown on Fig. 7. Rapid response piezometers were installed in the upstream slope to enable monitoring of the soil pore pressure response to changes in reservoir water level.



Plates 3 and 4 Earth moving plant in action at Abberton



Monitoring of the piezometers was carried out intermittently between May 1995 and April 1999. Since then a regular monthly programme of monitoring has been adopted. The earlier readings are often confusing due to their intermittent nature and the difficulty of separating effects such as piezometer de-airing from any underlying trend of readings. However, clear patterns are now emerging from the regular readings.

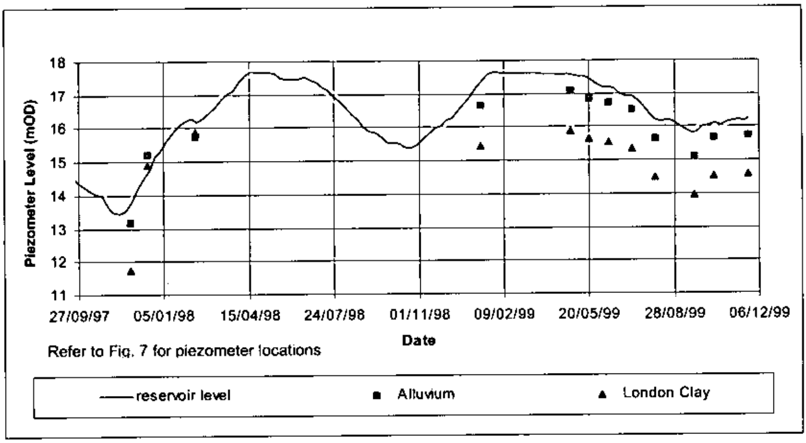


Fig. 9 Piezometer monitoring – Borehole 102

Piezometric levels through the dam are summarised on Figs. 7 and 8. The monitoring programme is at an early stage but the following conclusions have been tentatively drawn.

- Readings at Boreholes 101 and 102 through the upstream slope indicate a general tendency for piezometric levels to move with the fluctuation in reservoir level, for example see Fig. 9 which presents results for Borehole 102.

- The piezometric level in the London Clay beneath the upstream slope is a metre or two below the changing reservoir level, refer to Fig. 9.

- The piezometric level in the Alluvium upstream of the cut-off stays within about 0.5m of reservoir level, refer to Fig. 9. The piezometric level in the Alluvium downstream of the cut-off is close to original ground level at about +4mOD and appears to be controlled by the natural drainage layers in the Alluvium rather than by any drainage measures in the downstream slope. There is thus a steep hydraulic gradient across the cut-off trench, refer to Fig. 8

- Across the core and the downstream clay shoulder, the hydraulic gradient is shallower. Piezometer readings for Borehole 3 (just upstream of the core) and Borehole 4 (within the core) appear to follow reservoir level closely, whereas the piezometric level measured towards the base of the clay shoulder fill in Borehole 205 just downstream of the core is at about +9mOD. The seepage regime in this zone of the dam appears complicated, with 'perched' pore pressures in the downstream clay shoulder and down-drainage into the Alluvium, refer to Fig. 8.

- Downstream of the clay shoulder the piezometric level is generally at about original ground level. However, an artesian pore pressure equivalent to 2m head above ground level was encountered in the London Clay at 17m depth in Borehole 17, suggesting a hydraulic link somewhere under the reservoir catchment area within this horizon.

CONCLUSIONS

The failure of Abberton Dam during construction in 1937 is of interest historically because of similarities to the famous Chingford failure nine days later.

Preliminary analyses suggest that a slip surface through the top of the London Clay is the most likely failure mechanism. However, further consideration and analysis is required to identify the slip mechanism with more certainty.

The recent investigations confirm that the dam was reconstructed satisfactorily following the failure and that after 60 years of service it is generally in good condition. A system of piezometers has been installed and is being monitored. Results to date indicate that the piezometric regime through the dam is complicated and influenced by a number of factors including fluctuations in reservoir level and the high relative permeability in the Alluvium underlying the dam.

ACKNOWLEDGEMENTS

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Ladybower Dam: analysis and prediction of settlement due to long term operation

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R W CHALMERS, Babtie Group, UK
M MACKAY, Babtie Group, UK

SYNOPSIS. Ladybower Dam is located in the Peak District National Park in Derbyshire. It was completed in 1944 and is currently operated by Severn Trent Water. Originally constructed 43 metres high, it is the highest embankment dam with a puddle clay core in the UK. Since completion Ladybower Dam has experienced continual settlement and despite an initial settlement allowance of 0.9 metre, the crest has required to be raised on a number of occasions. Study of the settlement records up to 1985, when the crest was reconstructed, indicates a near linear settlement/log time relationship. However over the subsequent 10 years the settlements observed are more than twice that which would have been predicted from settlement/log time plot. These later settlement records suggest that another mechanism is involved and the investigation into the possible mechanism is presented. This is further demonstrated by the more active recent operation of the reservoir, particularly during the drought experienced in 1995. The investigation has established that the settlement is a function of the cumulative drawdown and re-impounding. The investigation has shown that settlement is also proportional to dam height. Accordingly a revised settlement drawdown ratio has been defined. Finite Element analysis showed that the section of dam analysed to be stable and confirmed the agreement with predicted settlements during drawdowns to be reasonable. A contract to raise the embankment by 3.5 metres was let in April 1999 to provide the appropriate settlement allowance. The contract is now complete.

Dams 2000, Thomas Telford, London, 2000.

INTRODUCTION

Ladybower Dam is located on the River Derwent in Derbyshire. It is the lowest of a cascade of three dams providing water supply to the Midlands and south Yorkshire. It is an embankment dam with a puddle clay core and shoulders of granular fill placed with minimal compaction. Placing of the core started in 1939 and the embankment was substantially complete in 1944. Originally constructed at 369m long and 43m high, it is the highest dam with a puddle clay core in Britain. There is a central concrete-filled cut-off trench below the core with a maximum depth of 77m. The height of the dam was increased from the original design during construction by 1.5m without increasing the base width. The downstream slope was increased near the crest to 1:1.5, unusually steep for this construction. The upstream slope was kept constant and the top of the dam was retained by a 4m high rubble masonry wall topped by a wave wall. The core had a width of 2.9m at its top, with side batters of 1:12 giving a maximum width of the order of 10m. A photograph of the dam is shown on Fig. 1 and a section at the highest point is shown on Fig. 2.

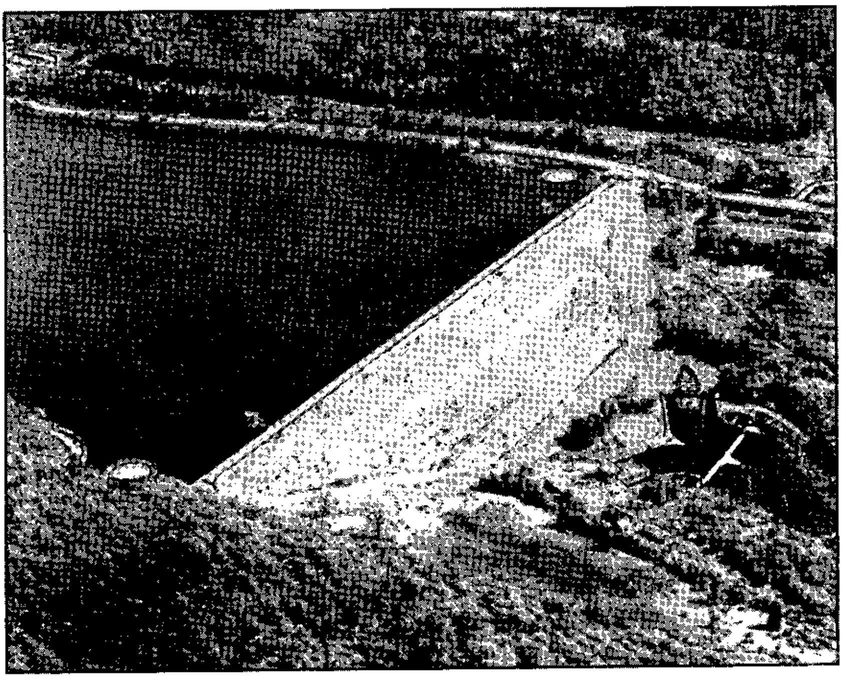


Fig. 1. Photograph of Ladybower

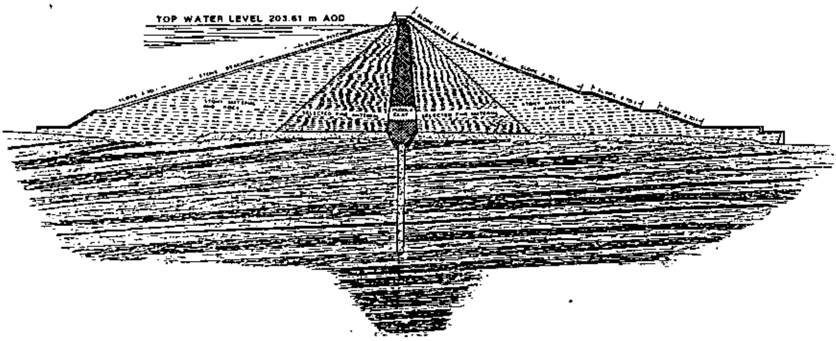


Fig. 2. Cross-section of Ladybower dam as constructed

Since construction the dam has continued to suffer substantial settlement. The crest has been raised on a number of occasions throughout its life culminating in the recent contract.

GEOLOGY

The geology of the underlying bedrock comprises sandstones and siltstones of the Namurian series. Below the dam the strata are intensively folded, probably as a result of valley bulging in the Devensian glaciation. The Devensian ice came to within a few kilometres of the site. There is gravel associated with the river channel.

The sides of the valley are covered with a sandy gravelly colluvium with a clayey matrix derived from the underlying rocks; and is typically about 2m thick. Below this the rock is broken for about another 2m. The dam was founded on the colluvium.

CONSTRUCTION AND FILL MATERIALS.

Shoulder fill

During the original construction the fill was placed using a 0.9m gauge railway system, along with some small dump trucks late on in construction. There was no supplementary compaction other than that from the placing plant. Construction photographs show that layer thicknesses were variable up to 3m. Shoulder fill was excavated from shallow borrow pits along both sides of the reservoir, level with fill placing. The excavation consisted of about 2m of silty sandy head with sandstone gravel and 2m of highly broken rock, and was carried out by small steam shovel. As was common at the time, the construction drawings show selection of fill with the finer material near the core and the coarser material towards the slope. The investigations and photographs show that two zones were placed on the downstream side but the upstream fill to have been placed without selection.

The fill is sandy and various laboratory tests indicate a drained shear strength (ϕ') of the order of 35°. The estimated Coefficient of Volume Compressibility (m_v) of the upstream fill and downstream transition fill is $0.4m^2/Mn$ from 0-100kPa and $0.2m^2/Mn$ from 100-500kPa. The downstream transition fill and the upstream fill are well graded from about 100mm particle size to a little less than 10% smaller than 2μm. The downstream shoulder fill is generally more coarser but with its grading filled with silty sandy fines. Permeability measured in-situ and in the laboratory is in the range of 1×10^{-8}m/s to 2×10^{-5}m/s. There are no excess pore pressures in the downstream fill. Pore pressures close to the core on the upstream side change with reservoir level. Open drive samples have given an average dry density of $1.8t/m^3$. The fines present retain water and the average water content on the downstream side is giving a degree of saturation of about 90%.

Puddle Clay Core

The puddle clay was obtained from a colluvium deposit 3km downstream of the dam. This deposit was derived from more clay rich rock than that available adjacent to the dam. The clay had the following properties: Liquid Limit, w_L, 63 - 42%, av. 53%; Plastic Limit, w_P, 37 - 21%, av. 28%; %<2 μm 40 - 1%, av. 20%. The clay was locally heterogeneous. Water contents indicated that the clay may have been placed a little wetter than was usual for puddle clay. The measured compressibility of the core was quite high, the average Coefficient of Volume Compressibility, m_v, being $0.8m^2/Mn$ from 0-100kPa and $0.2m^2/Mn$ from 100-500kPa.

SETTLEMENTS AND CREST RAISING

Significant settlements of the core were noticed during construction, possibly due in part to outward yielding of the steep downstream slope. On completion, the dam crest was raised by 0.9m at the highest point as a settlement allowance. This was 2% of height, which is rather more than the usual allowance of 1% typical for puddle core dams. Settlement has continued since completion and the records indicate that the crest and core have been raised on at least four occasions. Settlement records lack continuity, but a best interpretation indicates that the settlement allowance was exceeded after about 20 years. Total settlement recorded up to 1999 amounts to a maximum of about 1.5m.

CREST WORKS IN 1986/7

Following publication in 1978 of the ICE Guide to Floods and Reservoir Safety the design flood was re-assessed. A 2m high masonry faced reinforced cantilever wall was added to the crest to provide the head necessary to pass the Probable Maximum Flood (PMF) through the original twin bellmouth spillways. This work was carried out in 1986/7 and is described by Mackey (1988). As part of this study future settlements were assessed. It was recognised that settlements due to primary consolidation would have occurred quite rapidly. The ongoing settlement was attributed to creep. The settlement data was assembled and plotted against logarithm of elapsed time. These plots showed a near straight line relationship between settlement and logarithm of time for the period 1945 –1985 giving a settlement index value of S_I = 0.02 Charles (1986). A typical record is shown on Fig. 3. Note that settlement records are continuous from 1974. Gaps in the data prior to this were bridged by interpolation at the mean gradient of the plot on either side of the gap. Extrapolated, these relationships predicted a maximum settlement of 180mm over the next 25 years. A settlement allowance of 250mm was added to the new crest works. When the works were completed, a more intensive system of crest settlement observation was introduced.

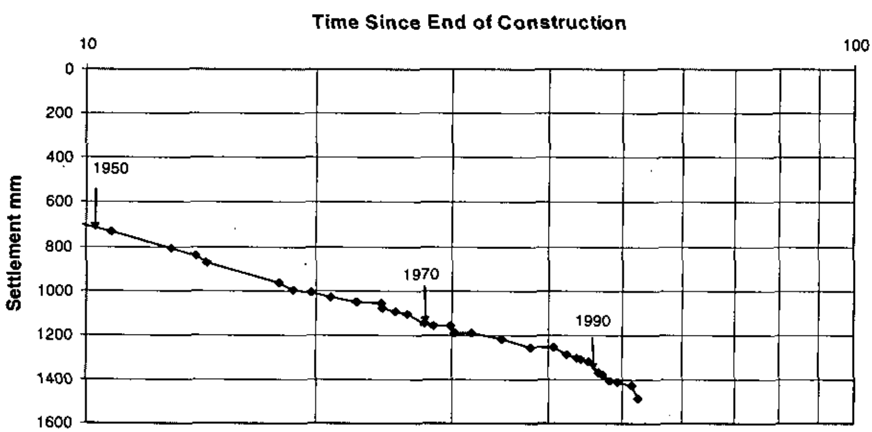


Fig. 3. Crest Settlement since Construction

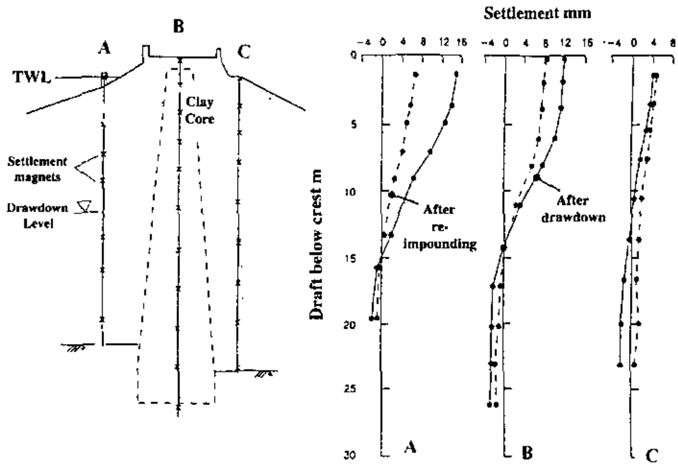
SUBSEQUENT SETTLEMENT

A statutory report under the Reservoir Act was made in 1993 and the new settlement records were interpreted. It was noted that the current rate of settlement exceeded that predicted in 1986 and that it was not decelerating according to a linear relationship between settlement and log. time. More frequent inspections were recommended, together with a further review of future settlement and freeboard requirements. Severn Trent Water retain a Review Panel to advise on matters of dam and reservoir safety. They reviewed the situation and options for maintaining safety while passing the PMF through the spillway and while maintaining the full reservoir capacity. They recommended that additional fill should be placed on the downstream slope to enable a new raised crest to be built to maintain freeboard without relying on a crest wall. They further recommended that these works should provide for a further 50 years settlement. Babtie Group were appointed by Severn Trent Water in April 1997 to undertake a feasibility study and subsequently to design the strengthening works.

INTERPRETATION AND PREDICTION OF SETTLEMENT

It can be seen that the gradient of the plot of settlement versus logarithm of time of Fig. 3 increases by more than twice over the period from 1986 to 1998. Such an increase in rate in a creep phenomenon can be an indicator of eventual instability.

An extensive series of observations of old puddle core dams had been conducted by the Building Research Establishment (BRE) in UK which involved observation of deformation during substantial drawdown cycles (Tedd *et al.*, 1997). The possibility that the settlements at Ladybower could be due to a similar cause was investigated. The BRE research identified that there is clear evidence of settlement during drawdown without full recovery during re-impounding. Also when a reservoir is full and the core in equilibrium there is very little settlement, indicating that creep was very small and the cyclic movements are not elastic. These movements were analysed at Imperial College (Kovacevic *et al*, 1997) using the finite element analysis ICFEP. They were found to be consistent with results from a cyclic oedometer test (Tedd *et al*, 1997).



Ogden Dam: cross-section showing location of instruments and settlement profiles in the upstream fill, the core and the downstream fill at the end of stages 4 and 5 relative to the beginning of stage 4

Fig. 4. Deformation of Ogden Dam after Tedd et al (1997)

Settlement measurements at Ladybower were not sufficiently frequent to identify settlements for every drawdown cycle. The permanent settlements where this could be done are plotted against depth of drawdown on Fig. 5. There is a reasonable correlation between settlement and size of drawdown. If it is supposed that permanent settlement per drawdown cycle is linearly proportional to the magnitude of the drawdown (which assumes that stiffness in unloading and reloading are constant but different), then settlement would be a function of cumulative drawdown. The settlement records plotted in this way are shown on Fig. 6. Again the gaps in the record are bridged by interpolation. The plot shows that initially the settlement rate as a ratio to cumulative drawdown is high but it reduces to a constant value after a cumulative drawdown of about 150m after 20 years. Similar plots were obtained for various settlement markers along the crest of the dam. The apparent increase in settlement as a creep rate after 1987 of Fig. 3 is due to more aggressive operation of the reservoir. The higher initial ratio of settlement to cumulative drawdown is consistent with the effects of first loading and shakedown. It is also shown in the cyclic oedometer test (Tedd *et al* , 1997). The ratio has remained constant for the last 30 years.

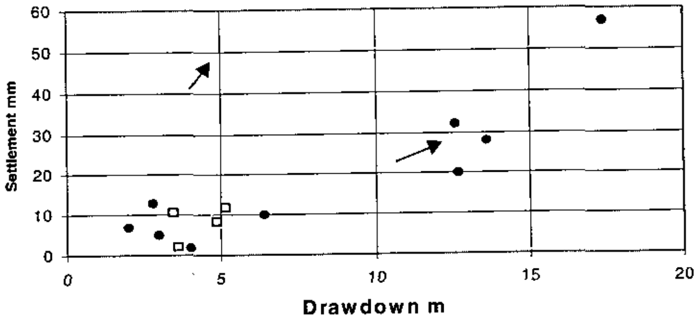


Fig. 5. Settlement during Drawdown v Drawdown Depth

It might be expected that larger drawdowns with larger stress changes would produce larger ratios of settlement to drawdown. Trial plots of the Ladybower data showed that this was not so. It might also be expected from the laboratory data that settlements would be larger during a first drawdown or during a period when the reservoir was drawn down further than previously. Fig. 4 shows some greater settlement early on. Unfortunately Ladybower was subject to its largest drawdown of 20m in 1959, when there was a gap in the settlement record due to reconstruction of the crest. It is assumed that settlement might be greater if the dam was subject to a drawdown greater than 20m.

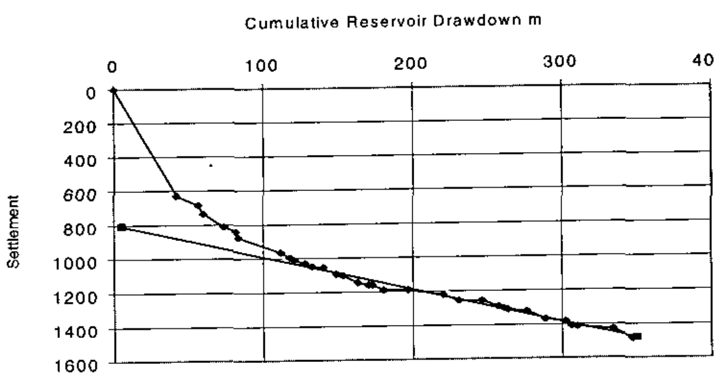


Fig. 6. Settlement v Cumulative Reservoir Drawdown at Station 199

It is convenient to normalise the ongoing settlement in terms of the height of the dam as well as the cumulative drawdown. For this reason a Settlement Drawdown Ratio, R_{SD}, has been defined as [Settlement increment (mm)]/[Cumulative drawdown increment (m) x Dam height (m)]. The derivations of the Settlement Ratio for the settlement records at four sections of different heights are shown on Table 1. These figures are derived using the height of the dam above foundation level and data taken from the plots of settlement *versus* cumulative drawdown. Results are reasonably consistent for the different embankment heights. As an independent check the Settlement Ratios were derived for four periods prior to the 1986-7 rebuild where settlement records were continuous. The results are shown on Table 2. They agree reasonably with the values shown on Table 1.

Table 1. Settlement Drawdown Ratios from settlement *v.* cumulative drawdown plots

Chainage m	Embankment height m	Settlement Drawdown Ratio, R_{SD}, mm/m^2		
		1950 - 1959	1960 - 1975	1975 - 1990
49	18.8	0.215	0.116	0.0565
138	32.8	0.174	0.095	0.0584
199	40.1	0.156	0.087	0.0491
260	31.3	0.134	0.074	0.0496
Averages		0.170	0.093	0.0534

Table 2. Settlement Drawdown Ratios from periods with continuous settlement records for the highest section

Period	1945 - 50	1950 - 54	1968 - 73	1978 - 86
Cumulative drawdown m	53 or 71*	22.2	29.9	36.9
Settlement mm	640	122	66	85
Settlement Drawdown Ratio, R_{SD}, mm/m^2	0.30 or 0.22	0.137	0.056	0.058
* to top of gravel or base of colluvium		R_{SD} (average) = 0.0579mm/m^2		

* Initial water level history uncertain

Table 3. Variation of settlement drawdown ratio across the valley based on surveys from 1987 to 1998

Chainage m	86	136	211	236	286
Embankment height m	31.1	40.6	41.2	41.2	26.9
Settlement Drawdown Ratio RSD mm/m^2	0.0558	0.0576	0.0604	0.0608	0.0544
Average settlement drawdown ratio, RSD = $0.0579 mm/m^2$ Height of dam includes depth of colluvium on valley sides					

A final evaluation of settlement drawdown ratio was made using the more accurate settlement data from 1987 to 1998. This is shown in Table 3. Initially this evaluation showed higher relative settlements over the west abutment (low chainages). The foundation rock here is more fractured and folded and it was thought that deep foundation settlement might have occurred. The relatively compressible colluvium is also thicker here (up to 7m). In the re-assessment shown in Table 3 the height of the dam is taken to the base of the colluvium. The route of the River Derwent was west of the dam centre line. The difference between the two abutments is eliminated.

FINITE ELEMENT ANALYSIS

As a final check on the settlement analysis a finite element analysis was commissioned from the Geotechnical Consulting Group using the same programme and techniques as used by Kovacevic *et al* (1997). The soil model was changed, however , as Kovacevic *et al* (1994) had shown that an elastic-plastic soil model gave displacement vectors during the construction of an embankment which disagreed with those observed in the field. This was corrected if a model due to Lade was used. This model incorporates plasticity before failure. The problem mainly arises because the direction of plastic strains coincides with the direction of the principal stresses whereas elastic strains coincide with the direction of the principal stress increments. The unload/reload stress strain characteristics are modelled by non-linear elasticity. The real time history of reservoir operation was not reproduced but a total of 7 drawdown cycles of different magnitudes was analysed. The analysis showed the existing section to be stable, although with continuing small plastic displacements of the crest wall.

A second programme of analysis was commissioned when the design of the new fill was complete to investigate the stability of the modified dam, particularly of the steep upstream crest wall which was now loaded from downstream, to investigate how future settlement would be modified by the new fill and to predict the deformation patterns as the new fill was added.

Some of the predicted settlements during drawdowns are summarised on Table 4. Considering that the unload/reload properties had to be estimated from a limited precedent rather than measured, the agreement is reasonable, the predicted values for R_{SD} being in the range 0.04 - 0.07mm/m^2, compared with the observed value of 0.06mm/m^2. The analyses show a shake down effect with R_{SD} reducing by a factor of two. They also predict a bigger settlement when the reservoir is draw down further than before.

Table 4. Settlement Drawdown Ratios predicted by finite element analysis compared with those observed

Predictions from finite element analysis.						
No of drawdown cycle	1	2	3	4 overall	4 (to 10.8m)	4 (10.8 - 19.6m)
Depth of cycle m	10.8*	10.8	10.8	19.6	10.8	8.8*
R_{SD} Run 3 mm/m^2	0.118*	0.076	0.069	0.080	0.069	0.094*
R_{SD} Run 6 mm/m^2	0.091*	0.046	0.038	0.0625	0.042	0.088*
Field observations (max height dam)						
Period	1945 – 50	1950 - 54	1968 - 73	1978 - 86	1987 - 98	
R_{SD} mm/m^2	0.26	0.137	0.056	0.058	0.060	
* Equivalent to 'first loading'. Soil data in Run 3 is best estimate. Unload/reload properties from precedent. The parameters in Run 6 were modified slightly to get vectors of displacement during drawdown cycles with more realistic directions.						

A preliminary estimate of future settlement based on this study using R_{SD} = 0.05mm/m^2 for the next 50 years gave 900mm for the current average annual drawdown of 9m and 1200mm if the average annual drawdown were increased to 12m. An additional settlement of 200mm was estimated for the effect of rebuilding, for drawing down to a lower level than previously and for emptying, followed by consolidation. The design of the new fill and crest was based on a settlement allowance of 1m.

RECONSTRUCTION

Fig. 7 shows the section as now rebuilt. The section was re-analysed using the same finite element analysis with the same assumptions for soil properties. The new analysis showed the embankment to be stable, although the plastic displacements of the old crest wall were slightly increased.

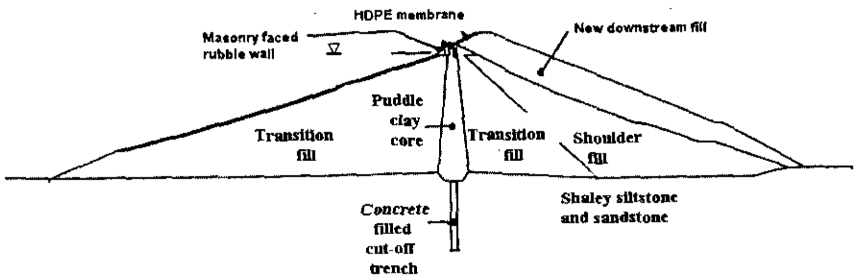


Fig. 7. Section of Reinforced Dam

The settlement prediction made for the original embankment is modified by the new construction. Stress levels in the embankment are raised and initially, settlement due to drawdown involves more first loading effects. The analysis predicted an initial increase of Settlement Drawdown Ratio to 0.23 for the old crest and 0.28 for the new one, to be compared with the prediction from the previous analysis of 0.04. After a number of drawdown cycles the Ratios decreased to 0.03 for the old crest and 0.02 for the new one. The new crest is further downstream than the new one and less influenced by the core or by stress changes due to drawdown cycles.

It was thought more accurate to base the revised settlement prediction on the Settlement Drawdown Ratio for the old dam, but modified to allow for the effects of the new construction. From the analysis the long term constant ratio for the new crest was estimated to be two thirds of the current ratio for the old crest, which was 0.06mm/m^2. From precedent and analysis the initial ratio after reconstruction was assumed to be 0.2mm/m^2, decreasing linearly with cumulative drawdown to the long term stable value of 0.04 mm/m^2 after a cumulative drawdown of 100m. An extra 130mm of settlement is predicted if the reservoir was fully drawn down. The predicted settlements are as shown on Fig. 8. The predicted settlement after 50 years is 1.09m if average drawdown is 9m/year and 1.33m if it is 12m/year. The elapsed times for 1m settlement are 44 years for a drawdown rate of 9m/year and 33 years for 12m/year. It is of interest that the new works are predicted to increase settlement for about 50 years, after which the settlement drops below what which would have occurred had the dam not been raised. In view of the uncertainties in the calculations and the relative ease with which the new crest could be raised at a future date, it was decided to apply the preliminary settlement allowance estimate of 1m.

Monitoring of the settlement of the dam has continued throughout reconstruction. The settlement trend observed to date reflects that predicted however the magnitude has currently not been realised.

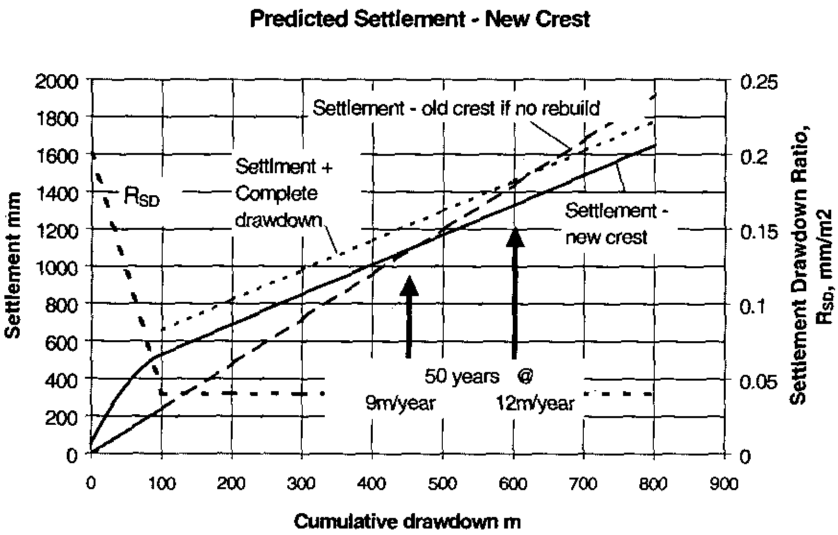


Fig. 8. Settlement Prediction for Rebuilt Section

SETTLEMENTS OF OTHER DAMS

It is of interest to compare the observations and predictions made for Ladybower Dam with observations of similar other puddle clay cored dams. Tedd *et al* (1997) give measurements of permanent settlement due to individual drawdowns. A summary of these observations is given on Table 5. The ratios for the other lower dams are generally lower than that for Ladybower, except where drawdowns are very large. For instance Ogden dam gave a ratio of 0.26mm/m^2 for a complete drawdown and 0.032 for a subsequent drawdown to mid height. A complete drawdown might not have occurred before. It seems that ratios vary considerably between dams and should be established individually for each dam.

Table 5. Settlement Drawdown Ratios derived from individual observations (Tedd *et al*, 1997 a Dams & Reservoir Paper) compared with Ladybower.

Dam (dam ht m) (Age years)	Reservoir drawdown				
	Max dept h (m)	Total depth (m)	Duration (months)	Permanent settlement (mm)	Settlement Drawdown Ratio (mm/m^2)
Ramsden (25) (100)	17 6	22* 6	9 9	52 8	0.122 (0.095)* 0.053
Walshaw Dean (22) (85)	10 13 12 17	14* 13 13* 17	8 8 5 23	7 8 8 16	0.032 (0.023)* 0.030 0.030 (0.028)* 0.043
Yateholme (17) (120)	7 6 4	7 6 4	9 6 2	6 2 2	0.050 0.020 0.029
Ogden (25) (135)	20 10	31* 12*	24 10	130 8	0.260 (0.168)* 0.032 (0.027)*
Widop (20) (110)	17	17	10	52	0.153
Ladybower (40) (53)	-	-	-	-	0.060

^ Depth from max to min water level. * Total cumulative drawdown when the level oscillates during the cycle.

CONCLUSIONS

Investigation into the settlement records for Ladybower Dam indicates that the settlement was significantly greater than that predicted from conventional consolidation and creep theory. The data studied indicates that crest settlement appears to correlate well with the cumulative amount by which the reservoir is drawn down. Embankment settlement can be expressed as a Settlement Drawdown Ratio, the total settlement over a period divided by the product of cumulative drawdown and embankment height, as proposed by Tedd *et al.*, 1997. A constant ratio has been observed for more than 30 years at Ladybower dam. Settlement then becomes a function of reservoir operation rather than time and its rate can increase if there is an increase in water level fluctuation. This does not imply instability. There are first loading effects and settlements increase if the reservoir is drawn down further than it has been before. This mechanism has been recovered by finite element analysis.

This approach may be found to interpret long-term settlement records better than the conventional approach often adopted in which long-term settlement is assumed to be a creep phenomenon occurring as a function of the logarithm of elapsed time. The assumption of a creep mechanism is likely to lead to an underestimate of future settlement. This was so at Ladybower Dam.

Although settlement measurement throughout drawdown cycles is desirable, Settlement Drawdown Ratio can be established by plotting occasional settlement records against cumulative drawdown. Records do not need to be continuous.

Provided that settlement records are available for a reasonable period of time and the Settlement Drawdown Ratio has become constant, settlement predictions should be reasonably accurate. However, Settlement Drawdown Ratio may be expected to vary from one dam to another. Reconstruction of the embankment crest will temporarily increase the ratio because of first loading effects. The long-term constant ratio may also be modified. Settlement prediction then becomes more uncertain.

Finite element analysis is a useful method for interpreting field records and estimating the effect of crest reconstruction. It is unlikely to be accurate if used without calibration against field settlement data.

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Monkswood Reservoir – the leaking Bath water

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SYNOPSIS. Monkswood reservoir is retained by a 15.5m high embankment dam with a central puddle clay core and was completed in 1896. A leak was found in 1931 and attempts were made to cure it by grouting and sheet piles. In 1945 the Building Research Station was approached for advice. The amount of leakage was influenced by reservoir level when it was within 3m of top water level and greatly increased when within 20cm. Investigations indicated the leakage could be occurring across the top of the core, through the core at depth associated with deformations during construction, by hydraulic fracture or due to high ground water levels in the hillside. The leak has been cured following construction of a cement-bentonite slurry trench cut-off wall in 1998. Monkswood is probably typical of many dams that have leaked for many years without significant continuing erosion taking place.

THE DAM AND ITS CONSTRUCTION

The reservoir was formed in the upper reaches of St Catherine's valley, where there was a natural lake, in a region called Monk Wood, to provide a water supply to the City of Bath approximately 7 km to the south. The embankment dam is 160m long with a maximum height of 15.5m, as shown in Fig. 1. It is of traditional design with slopes of 1:3 upstream and 1:2 downstream, a central puddled clay core, 5m wide at ground level tapering to 3m, built by the Paddington contractor Neave and Son under the supervision of William Fox, Consulting Engineer, between 1893 and 1896. The valley has been incised to the Lower Lias Clay. The valley sides and floor are covered by superficial deposits with alluvium, peat and solifluction material covering the valley floor. On the line of the puddle clay core, a trench was taken down to key the puddle clay into the strong blue Lias Clay, but under the dam shoulders the oolitic sand, peat and alluvial deposits were left in place except under the outer parts of both slopes, where wide key trenches were excavated to replace the weak material with rubble as shown in Fig. 2. A brick lined outlet tunnel passes through the clay core.

A brief description of the construction of the embankment is given by Fox (1898). The available material for embankment construction was described as excellent for puddle but difficult to deal with in the formation of the shoulders on each side of the core. In accordance with the practice at the

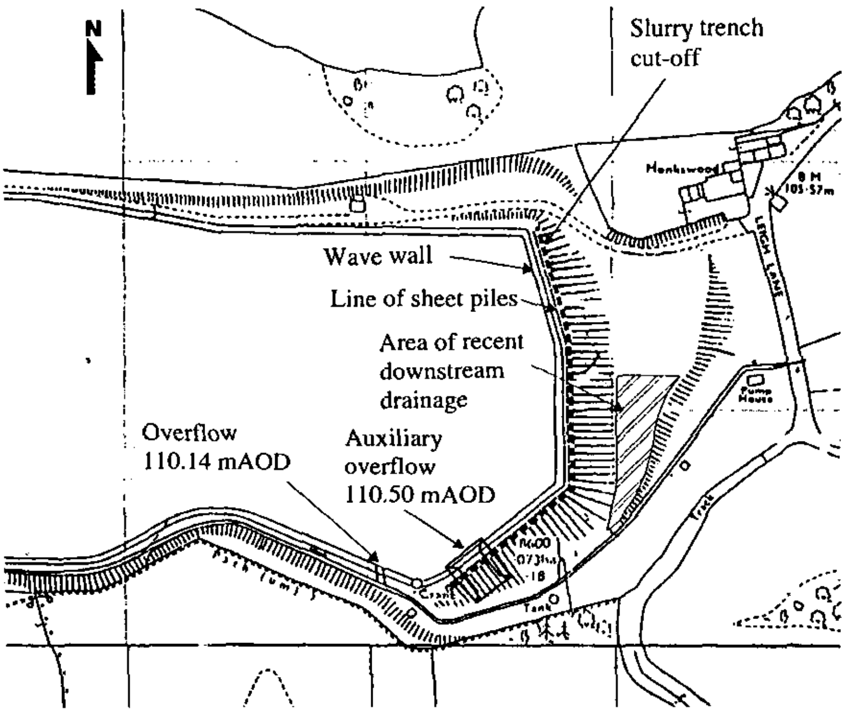

Fig. 1. Plan of Monkswood Reservoir showing extent of sheet piling and slurry trench cut-off wall

time, the most clayey material was placed next to the core and was tipped in 0.6m thick layers.

Very bad weather during the first year of construction made the fill very difficult to handle. The clay became exceedingly slippery and the oolitic sand became almost a running sand in wet weather. To guard against outward movement of the shoulders, the toe of the upstream slope over a width (u/s–d/s direction) of 17m was formed of rubble stone to a height of 5.5m which together with the underlying key trench made a total thickness of nearly 12m of stone with a width of 23m. On the downstream side the excavated oolitic sand, peat and alluvial deposits were formed into a large spoil bank that acted as toe weighting. Nevertheless, as the embankment neared completion, there were signs of instability, and stony material from the adjacent quarry was placed on the downstream slope, effectively flattening it to from 1:2 to 1:2.75. Movement of the upstream slope was resisted by loading it with stoney material, and building toe weighting above the rubble stone as shown in Fig. 2.

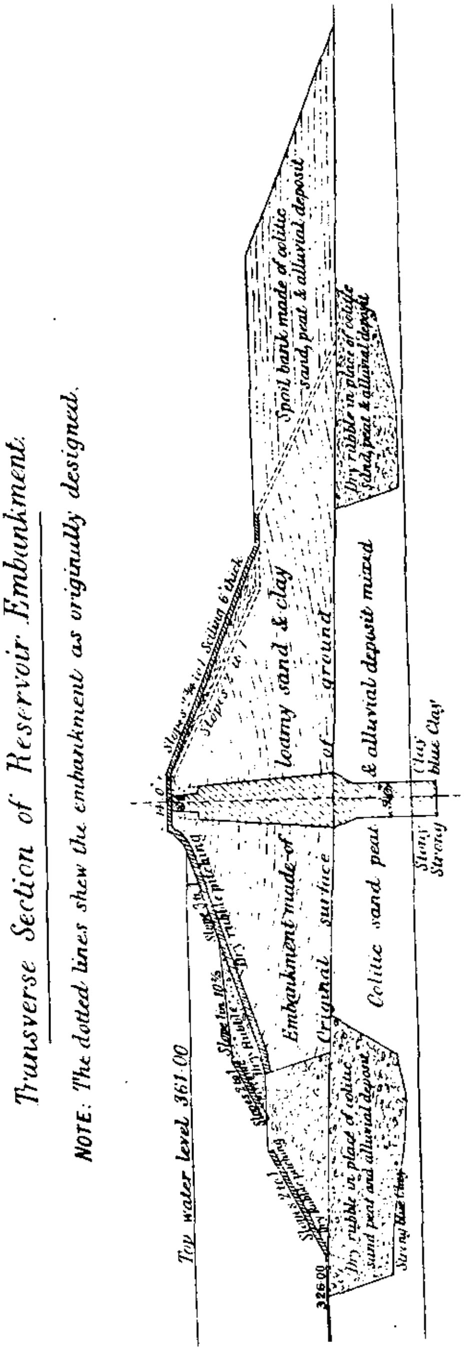

Fig. 2. Transverse section of main embankment as constructed after (Fox, 1898)

HISTORY OF EVENTS AFFECTING THE EMBANKMENT

Southern hillside and outlet tunnel

During construction, the slopes of both hillsides were steepened over their lower levels, below TWL, possibly to win fill. The cut slopes were protected against waves with concrete slabs and stone pitching lower down. When the reservoir was lowered in 1928, a slip was found in the southern hillside near the dam. Further movement were noted in the 1930s and a deep cut-off was constructed above the slipped area to intercept ground water flowing down from the hillside. Additional intermittent movements occurred during the early 1950s and an attempt was made to arrest the movement by installing two lines of sheet piles and improvement to the drainage by construction of french drains. Some movement occurred again in the 1960s. A small slip occurred during December 1959 at the top of the downstream shoulder of the dam involving a length of about 16m over a down slope distance of 4.5m

Deformation of the downstream part of the brick lined tunnel was accompanied by horizontal cracking at shoulder level and steel struts were installed in 1928 to limit movement. Grouting work late in 1990 showed no evidence of voids behind the lining with only minimum quantities of grout take.

Leakage

Under the Reservoirs (Safety Provisions) Act of 1930 Mr A P I Cotterell was appointed as inspecting engineer on the 25th July 1931. On the 4th September 1931, a local subsidence developed in the downstream slope. An exploratory excavation within the depression revealed water flowing through the fill. Flows measured at various locations showed that the major flow was coming from a southerly direction and flowing along downstream side of the puddle clay core, and disappearing into the fill where the subsidence had occurred. It was likely that the free flowing water channel had led to internal erosion and the subsidence. A concrete gauging chamber was built within the excavation and a length of 5cm diameter galvanised pipe was pushed into the fill to collect some of the flowing water. The maximum flow rate was approximately 68 litres/min.

The investigations failed to establish the source of the leakage. There was no conclusive evidence to prove that the leak was due to a local weakness in the core or that there was general seepage through the upper portion of the core. It was established that the amount of leakage was influence by the water level when it was within 3m of TWL and that it greatly increased when the reservoir level rose within 0.18m of TWL.

Remedial works to control leakage were undertaken between October 1931 and February 1932 and consisted of injecting a grout mixture of cement and sand through tubes driven to depths of 3 to 24m, at various position along the upstream side of the core. Approximately 280 tons of cement were used in 47 boreholes with injection taking place at depths of 1.5m to 24m. The remedial works had the effect of reducing the leakage into the gauging chamber from 68 to 7 litres/min. Initially, leakage, reservoir levels and rainfall monitoring revealed no pattern in behaviour, but after 1935 the rate of leakage steadily increased when the reservoir level was near TWL to the rate when the leak was first discovered. It has not continued to increase, indicating that the clay was non-erodible under the existing hydraulic conditions.

Early in 1945, a light section of interlocking sheet piling was driven into the middle of the puddle clay core over the central part of the dam to a depth of approximately 6m. The extent of this is shown in Fig. 1. This was driven in 2 lengths with a horizontal staggered butt joint. This work had no effect on the discharge of the leak.

BRS investigations

Mr Cotterell approached the Building Research Station (BRS) about the leak at Monkswood. He was aware that the Soil Mechanics Section at BRS had analysed the failures of two dams since 1937 and had recently investigated leakage through the puddle clay core of the King George V in the Lea valley. At Monkswood, ten shallow trial pits were excavated along the crest to reveal the level of the top of the puddle core which was found to vary from 1.06m above to 21 millimetres below overflow cill level. Nine hand bored 13 cm diameter holes was made in the core. Those on the downstream side were taken to about 4.5m depth to see if water was getting through the upper part of the core. Deeper holes in the area of the gauging chamber were taken of between 9 and 13m depth to observe the general nature of the core and to see if there were any free water channels through the core.

A profile of strength was obtained for each borehole using an unconfined compression apparatus. Except for the upper 2 to 3m, the puddle clay was described as a very fat plastic clay with a fairly uniform unconfined compressive strength of 30 kPa (c_u = 15kPa) at a water content of about 40%. The upper 2 to 3m showed sign of oxidation and drying, increasing compressive strength to 165kPa. Numerous thin seams of hardened grout were encountered with a general vertical trend, but with a wide mixture of angles of orientation. They gave the impression that the core had been full of thin fractures, but it could be argued that the grout had formed hydraulic fractures as it had been injected.

A draft report suggested that there may have been a rotational slip in the dam as it neared full height during construction, with the slip passing through the lower part of the core as indicated by Fig. 3. Using the measured strength of the puddle clay core, an assumed strength for the peat and soft blue clay underlying the downstream shoulder and $\phi' = 40^{\circ}$ for the rubble rock, the calculated factor of safety was only 1.1. If such a slip surface had ruptured the lower part of the core, allowing water to flow into the select fill, then water might have passed upwards against the downstream face of the core as shown in Fig. 4. This extra water could raise the phreatic surface in the downstream shoulder to the position shown by the dashed line, causing water to enter the galvanised pipe and appear as a leak in the gauging chamber. A small fall in the reservoir water level could lower the phreatic surface sufficiently to cut off the flow into the galvanised pipe so causing a rapid reduction in the flow of the leak.

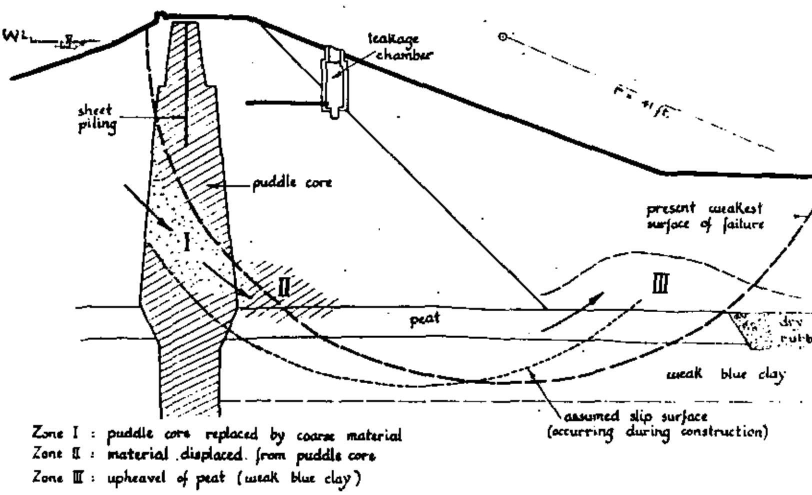

Fig. 3. Location of rotational slip during construction suggested in 1947 report to explain leakage through the core.

Four of the deeper boreholes, Nos. 1, 2 & 3, located in the area of the gauging chamber encountered water between 8.2 and 9.1m and the rapidity of the water rise in two of the boreholes indicated that there was a passage through the core at this level. The water rose to 0.6m below reservoir level, indicating that there was flow through the passage from the reservoir to the downstream side. Nowadays consideration would be given to the probability that this passage was caused by hydraulic fracture. This concept is supported by the fact that careful monitoring in January 1944 established that leakage reduced from 68 litres/min at TWL to 0.68 litres/min when the reservoir was approximately 0.5m below TWL. Also the water level in borehole No. 1 remained as much as 3.5m above reservoir level when it was lowered, and did not flow back into the reservoir. This could be due to the hydraulic fracture as the reservoir level reduced.

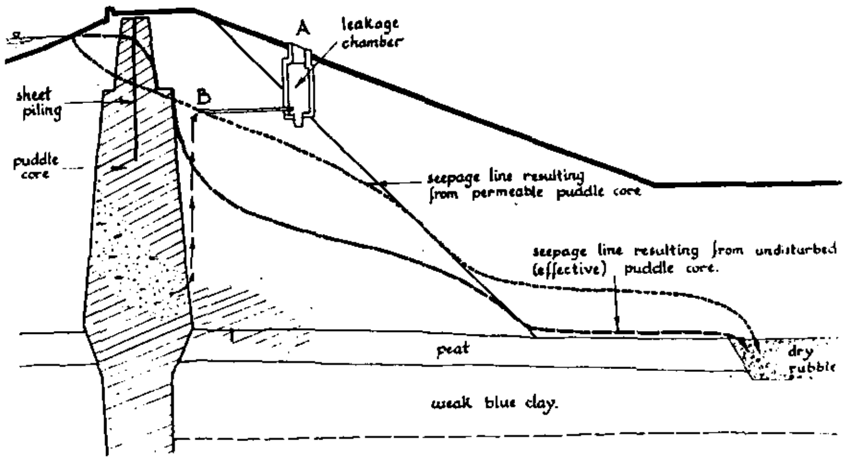

Fig. 4. Position of leakage chamber and possible seepage lines resulting from effective and defective puddle core (1947, report)

RECENT WORKS

Additional standpipe piezometers were installed in the 1970s following further tunnel deformation and the intermittent accumulation of standing water on the downstream berm. These showed high groundwater levels beneath the downstream shoulder at the southern end which were largely independent of reservoir level. The evidence of continuing seepage and loss of material into the tunnel was also of concern, and grouting through the tunnel lining was carried out in the early 1990s. Little evidence of voids was found.

Reappraisal of the reservoir hydrology and the overflow arrangements showed that the existing facilities were inadequate by modern standards. Improved flood discharge arrangements were provided by the creation of a 20m long auxiliary spillway on the southern limb of the dam. A reinforced grass system using interlocking blocks was provided down the slope and along the mitre to provide enhanced erosion resistance.

To deal with the contribution to the leakage coming from the southern hillside, a downstream drainage system was installed which would deal both with the localised problems of ponded surface runoff and the potential below surface contribution from the south. This comprised deep drains discharging into the spillway channel through a series of V-notches. A series of pressure relief holes were also drilled from the tunnel with provision for monitoring individual flows and replacement/cleaning out of the relief holes at a later date.

To solve the problem of the leakage it was decided to install a single phase slurry trench cut-off wall. The single phase technique involves excavation of a trench, typically 0.6m wide, under the support of a self hardening cement-bentonite slurry which is left to set to form a low permeability barrier. These techniques for repairing dams have been used at a number of dams in the UK (Tedd & Jefferis, 2000) and have also been used extensively on contaminated land to control the lateral migration of pollution. Specification for such cut-off walls has recently been published (ICE, 1999). At Monkswood a specially adapted backactor was used to excavate the trench to a depth of up to 15m, Fig 5. Slurry walls provide greater certainty of sealing any defects in the core than can be achieved by grouting.

Performance specifications are normally based on the properties of the slurry reaching specified values of strength and permeability. In an earth dam, there is also a perceived need for strain compatibility between the insitu fill and the hardened slurry. The agreed specification with the contractor Bachy Soletanche was for the set slurry at Monkswood to have the following properties:

- Permeability to be approximately 5×10^{-8} m/s at 28 days
- Unconfined compressive strength in the range of 100-200kPa
- Strain at failure in consolidated drained triaxial test to be greater than 2% with an effective confining pressure of 100kPa.

The mix used at Monkswood contained less solids than would be used in a containment slurry cut-off wall because the permeability criterion was much larger than the 1×10^{-9}m/s normally specified for pollution control barriers. Compliance testing at 28 days indicated that the material properties were as specified except that the measured strengths were typically towards and beyond the upper limit, with a mean of 194kPa.

The cut-off wall was constructed downstream of the centre of the core for the full depth of the dam over a length of 185 m. The reservoir level was held down by 3m during the installation of the cut-off to minimise the potential for pollution from the operations and to give some flood retention capability. This latter need was also enhanced by the requirement to lower the top of the dam by about 0.6m to give sufficient working width. The possibility of floods during the relatively short installation period was of concern as the spillway would be inoperable for part of the period whilst works in the tunnel and the adjacent length of dam were being carried out. The reservoir lowering allowed approximately 100,000m^3 of flood storage which would accommodate a 20,000 year event. The risk of flow over the auxiliary overflow during the few weeks of the work was assessed at about $5x10^{-6}$. If the reservoir was allowed to remain at TWL, the probability of flow over the auxiliary overflow was considered to approach 0.1. Whilst this value is an acceptable risk for many construction activities, it is not appropriate for dam remedial works

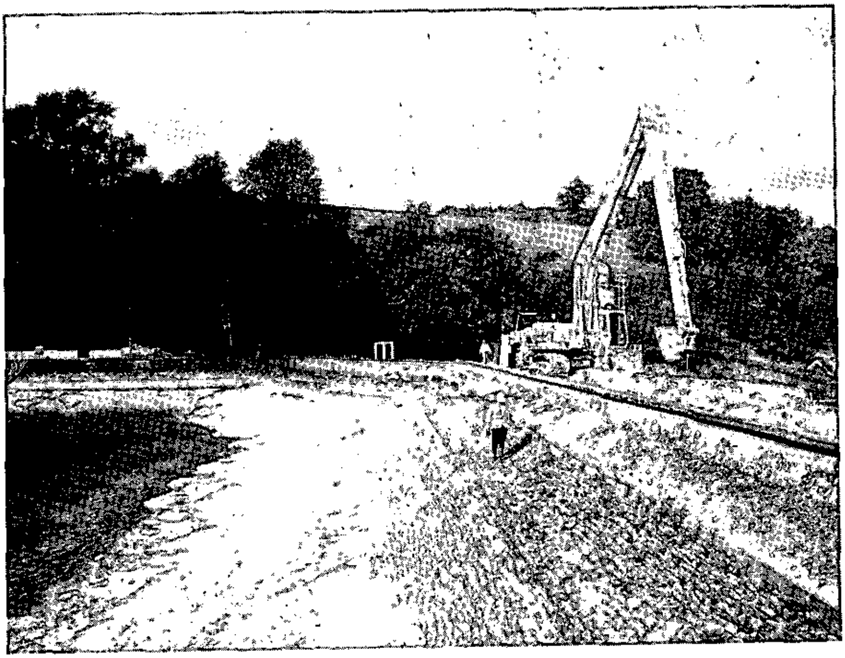

Fig. 5. Installation of slurry trench cut-off wall at Monkswood dam

A number of concerns exist over the use of cement-bentonite slurry for the repair of cores in dams:

- Incompatibility of the stiffness properties of the set slurry with the core
- Possibility of drying shrinkage
- Erodibility of the set slurry if leakage should occur

These concerns are dealt with in detail by Tedd & Jefferis (2000) and are only discussed briefly here. The set material is a low permeability, stiff, hard, brittle material that only exhibits ductile properties under drained high effective stress conditions. In the unlikely event that a cut-off wall does crack due to dam deformations the erosion resistance of the set slurry needs to be assessed. Samples of cement-bentonite slurry from Monkswood were cast in U100 tubes and some months later 100mm lengths of the sample were place in a modified pinhole erosion type apparatus developed at BRE. Water was passed through a 3mm diameter hole drilled longitudinal through the sample with a hydraulic gradient of 20. After 4 months no noticeable erosion of the sample had taken place.

The possibility of cracks due to drying also needs to be considered. As the set slurry typically contains approximately 80% water (it has a moisture content of 400%) it is prone to drying shrinkage and cracking. An isolated sample stored inside will quickly lose water, shrink, crack and eventually breakdown to a white crumble. It is therefore important to stop the top of the wall from drying out by protecting it with a clay cap as specified in ICE (1999). There is no evidence from the exhumation of a number of cut-off walls to indicate that they will dry out and crack in the British climate.

CONCLUSIONS

Monkswood is typical of many old embankment dams with central clay cores which leak or have leaked only when the reservoir approaches top water level. This can arise from a number of causes including defects in the core, hydraulic fracture at depth or drainage difficulties. Grouting in the 1930s and sheet piling in the 1940s did not stop the leak. Construction of a cement-bentonite slurry cut-off wall together with some drainage measures appears to have solved the leakage problem; ponding of water on the downstream berm has now disappeared, no flow has been observed in the measuring chamber but some flows have been reported from the tunnel drainage.

ACKNOWLEDGEMENTS

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Investigations into seepage at Rotton Park Reservoir using temperature distribution measurement

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SYNOPSIS. Investigations have been undertaken into the sources of seepage at Rotton Park Reservoir, Birmingham, initially utilising conventional ground investigation techniques, and relying on piezometer information to infer the source of groundwater. The need for greater confidence in the results has led to the use of temperature distribution measurement at the dam, using techniques patented by GTC Kappelmeyer GmbH. The results have supported the model produced by the conventional investigation.

BACKGROUND.

Rotton Park reservoir (also known locally as Edgbaston Reservoir) is operated by British Waterways. The dam is located within the City of Birmingham, some 2.5 km to the west of the city centre (Fig. 1). The dam comprises a 330 m long earth embankment structure, with a maximum height near the centre of 10 m. The dam is served by a single, brick lined spillway structure, which is situated to the southern end of the dam, and the upstream face is protected by a curved brick revetment and wave wall. A small bridge carries a single access track across the spillway at crest level.

The reservoir supplies water to the Birmingham Canal network, via a branch known as the Icknield Port Loop. Although the catchment is relatively small (147 ha), with part of the runoff being drained elsewhere via sewers, water is fed from the Titford Canal via the Titford Feeder, and the reservoir plays an important part in maintaining water levels on the lower canal system.

Seepage has been observed near the right abutment (i.e. the southern end of the dam) for a number of years, the seepage emerging on the face of a steep, 4 m high section of natural ground located close below the dam (Fig. 2). This seepage is associated with areas of instability, the face of the slope having undergone shallow failure in several places. The drainage system below the slope is constantly being cleared of fine sand and silt, and sandy deposits are present in the toe drain channel, indicating that the slope is suffering from active loss of material through internal erosion.

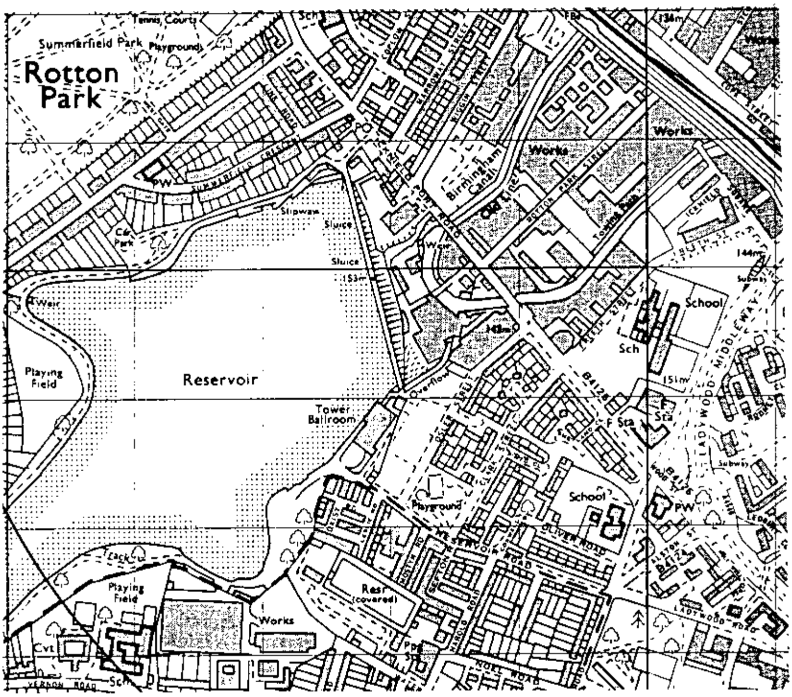

Fig. 1. Site plan - extract from 1:10 000 OS Sheet SP08NW, 1989 edition (not to scale). © Crown Copyright.

Seepage also intermittently affects the brick lining of the spillway channel, causing accelerated deterioration of the brickwork and mortar.

INVESTIGATIONS.

Following an Inspection of the reservoir in 1995 under the Reservoirs Act 1975, the Inspecting Engineer made recommendations in the interests of safety to investigate and determine the source of the observed seepage, and draw conclusions regarding the source of the water and the implications for the stability of the dam. The methods recommended by the Inspecting Engineer included the installation of a network of piezometers near the spillway, and monitoring of the instruments over a prolonged period.

Geotechnical Investigations.

In 1997, a ground investigation was designed and implemented by British Waterways Technical Services (BWTS), using Soil Mechanics as Investigation Contractor. Initially, a Desk Study was undertaken to determine the probable geology and history of the site; during this time, an assessment was made using piezometer results from a limited number of instruments which had already been installed at the site, as part of a pilot investigation scheme.

Following initiation of the Desk Study, during March '97 a total of 15 boreholes were formed using cable percussion rigs, and casagrande-type piezometers were installed to varying depths, in a layout intended to provide optimum information regarding the flow of groundwater in the areas of concern (Fig. 2).

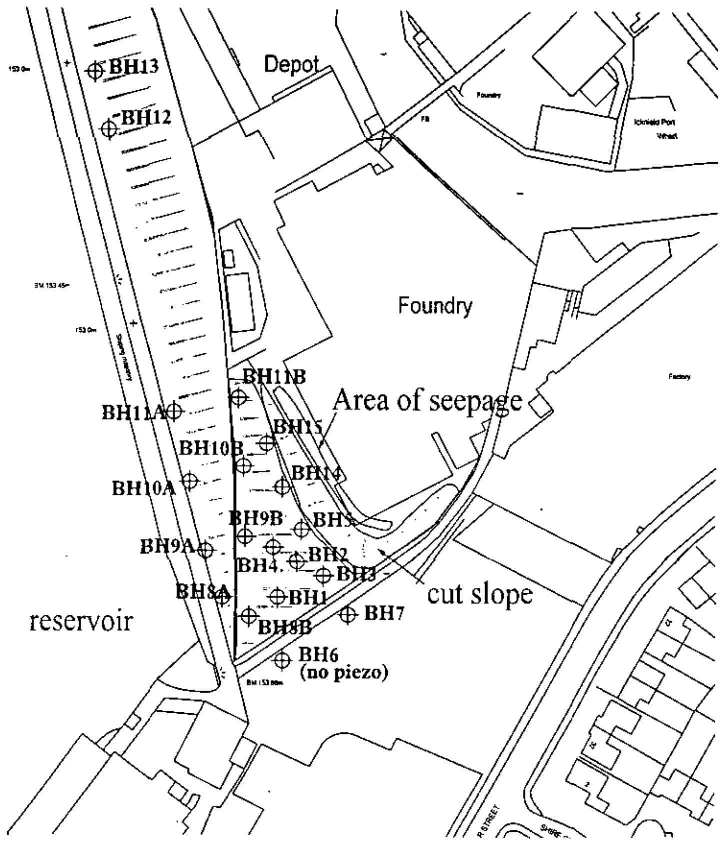

Fig. 2. Borehole location plan, showing locations of piezometers installed in 1997 (not to scale). © Crown Copyright.

In addition to the installation of piezometers, extensive sampling was undertaken within the boreholes, and a program of laboratory analysis scheduled to determine soil classification, shear strength and permeability characteristics.

Following completion of the piezometer installations, a programme of monitoring was undertaken for an extended period. Piezometers were generally read weekly as a minimum, with more frequent readings being obtained during periods when reservoir level was altered significantly.

Results.

The "Desk Study" phase (which in fact continued in parallel with the site works and monitoring, due to time constraints), allowed two main objectives to be fulfilled. Firstly, the underlying geology of the site was determined, largely from published geological information (British Geological Survey, 1991). Secondly, a significant amount of historic information was referenced, including several older editions of published Ordnance Survey maps at appropriate scales.

The geology of the site was found to comprise the following succession:

Quaternary: Glaciofluvial drift (mainly sand and gravel, with clay layers or lenses).

Glacial drift (till), mainly clayey or gravely clays

Triassic: Wildmoor Sandstone Formation – aeolian sandstones, locally pebbly.

Within the materials identified, records from nearby wells (known as the "Vivian boreholes") showed that the piezometric surface in the Wildmoor Sandstone was relatively deep, at approximately 45m below the level of the dam foundation (Milton, 1971). The clayey glacial drift appears therefore to be acting as an aquiclude, causing a perched phreatic surface to develop in the overlying glaciofluvial sands and gravels. The relationship between these materials and the perched water table is shown in Fig. 3.

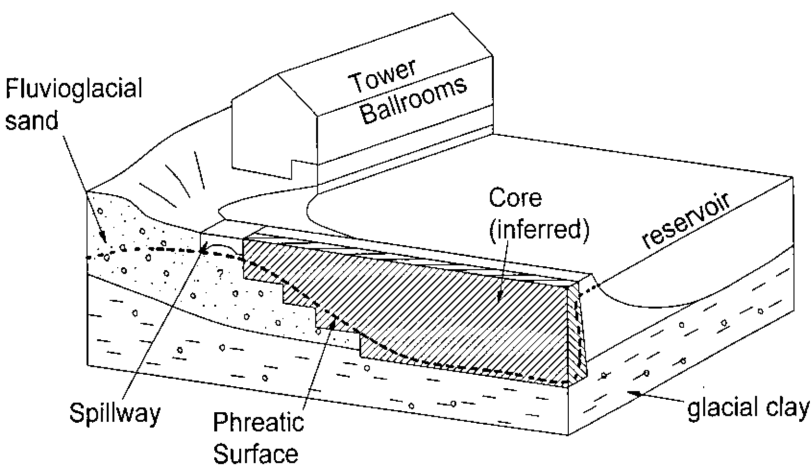

Fig. 3. Idealised block diagram showing the ground/ structure model developed from conventional investigations.

A review of the Ordnance Survey plans also revealed that the steep slope below the southern end of the dam from which the seepage emerges is a man-made feature, apparently cut into the hillside to allow construction of a Metal Works during the 1950's. Earlier editions of the map, such as the 1890 edition (Fig. 4) show that no steep slope is present, but the area is crossed by what appears to be a drainage channel, leading into the canal loop to the north. This is taken to indicate that a high phreatic surface has existed in this area for over 100 yrs.

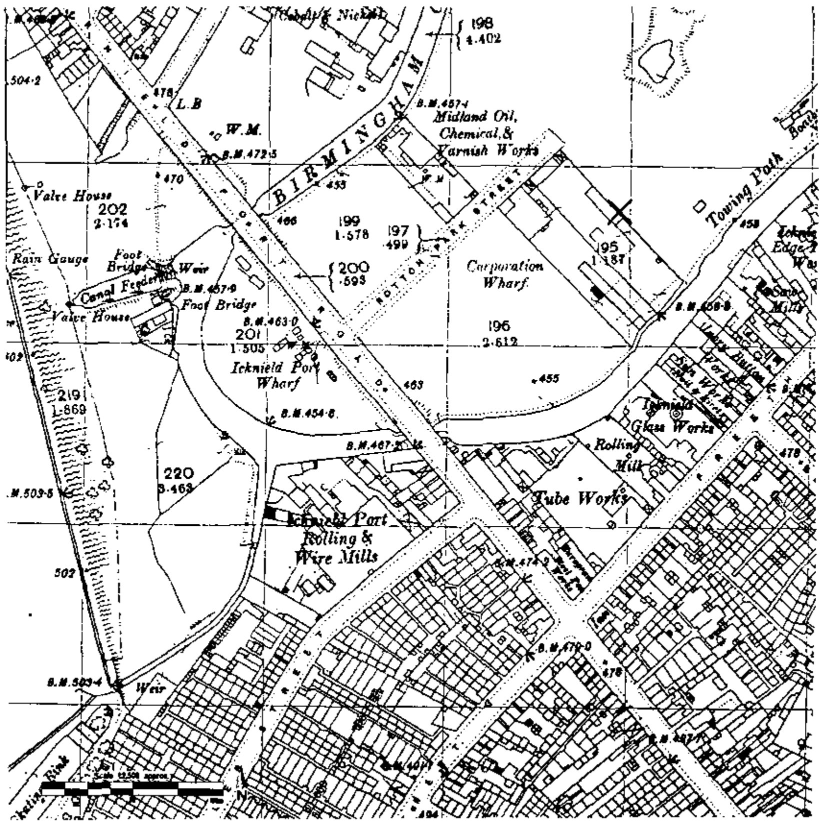

Fig. 4. Extract from 1890 edition of the 25" scale OS map (not to scale). © Crown Copyright.

The 1993 edition, however, clearly shows a cut slope existing in the angle between the dam and the spillway channel, around the perimeter of the metal works, a layout which remains relatively unaltered today (Fig. 2).

The results of the Ground Investigation phase have been analysed and interpreted in order to refine the Desk Study information, and to allow conclusions to be drawn regarding the origins of the seepage at the dam.

The piezometer results were collated by BWTS using a spreadsheet, and graphical results obtained. Sample results for a 6-month period during 1998 indicate a high degree of correlation between changes in reservoir water level, and corresponding changes in groundwater levels as measured by the piezometers (Fig. 5). The statistical significance of this correlation was tested using Pearson's Product Moment on sub-sets of the data. The results gave a confidence limit of >95% in the correlation. The conclusion was drawn from these results that reservoir water was, at least in part, contributing to the seepage observed below the dam, i.e. that a hydraulic connection existed.

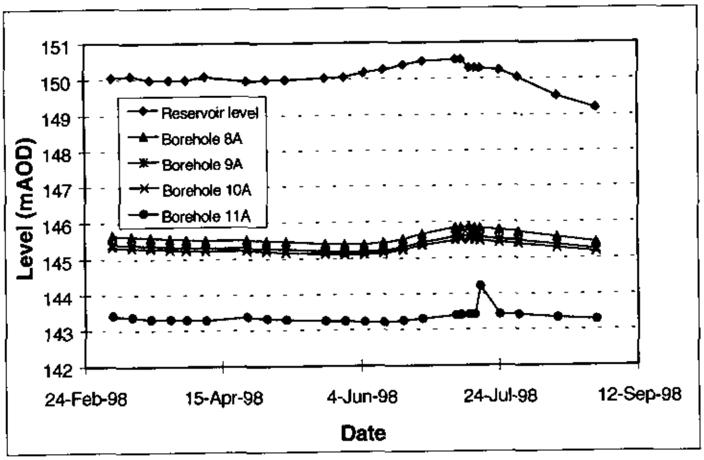

Fig. 5 - piezometer results (March - August 1998).

The existence of a hydraulic connection alone, however, does not give conclusive evidence that leakage is taking place through the dam; the ground model, evolved during the desk study and investigation phases, showed that the right abutment of the dam was formed by a low hillside comprising glaciofluvial sands and gravels, materials which are considered to be relatively permeable. The possibility that the seepage was occurring entirely through this body of material, i.e. passing through the natural ground to the right of the dam, was considered to be very strong.

In order to test this theory, the piezometer data were analysed more thoroughly. Values of the groundwater levels for all the piezometers and the reservoir water were processed using a statistical interpolation software package (SURFER) to give interpolated contours for the groundwater in the area around the spillway. These contours have been used to infer groundwater flow direction, a technique commonly used in hydrogeology (Cedergren, 1977).

The groundwater contours (Fig. 6) show a very steep hydraulic gradient exists across the dam itself, implying that material of relatively low permeability exists; one conclusion of this is that the dam possesses a core of low-permeability material, reinforcing the model derived from older cross-sections during the Desk Study.

In addition, the direction of groundwater flow, as inferred by the contours, strongly suggest that the dominant flow of groundwater is from the south of the spillway bridge. This supports the view that seepage is occurring through the more permeable glaciofluvial materials in the hillside, and not through the dam itself. However, due to the absence of data points towards the southern end of the dam, the contours are extrapolated into a critical area, leading to uncertainty regarding the accuracy of the results in this area.

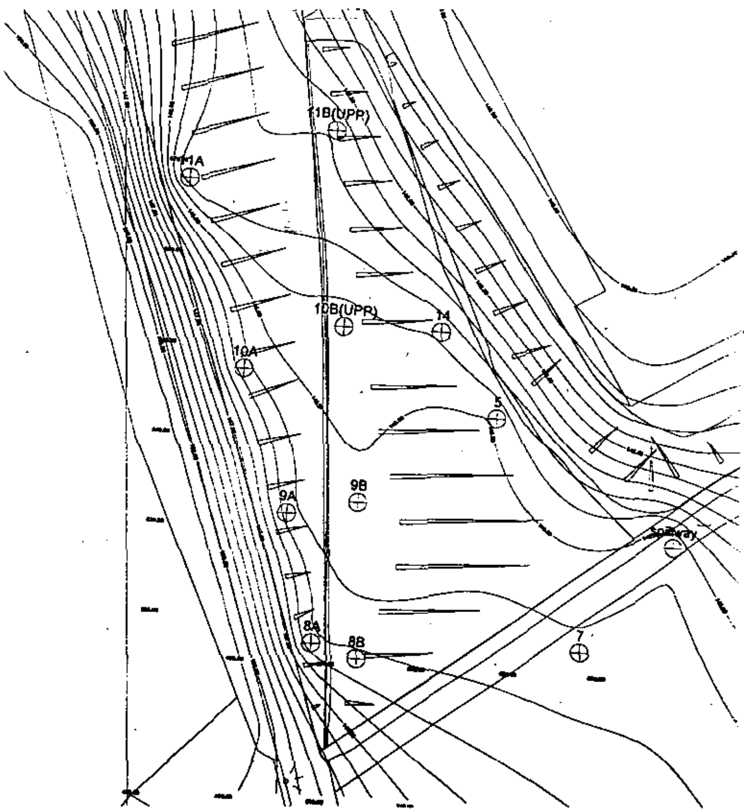

Fig. 6 - plan of groundwater contours in the spillway area. Contours are at 0.5 m intervals.

Geophysical/ Thermal Investigation.

It was concluded from the above results that since a clear correlation between the reservoir water level and the piezometer levels had been established, further information was required to allow conclusions to confidently be drawn regarding the source of the observed seepage at the dam.

A number of options were considered in order to achieve this objective; firstly, a proposal was developed to extend the existing piezometer network into the current area of extrapolation, as a development of the existing investigation. Alternatively, the use of geophysical methods was considered, as a complimentary technique to enhance the investigation data already gathered.

Of the geophysical methods considered, only two were thought likely to have a high enough chance of success in establishing the presence and distribution of groundwater. The first method is known as "Resistivity Profiling" (reference), and exploits differences in the electrical conductivity (and therefore resistivity) of geological materials between their saturated and unsaturated states. The interpretation of resistivity data, however, is often extremely difficult, as anomalous areas of resistivity can result from a number of factors, of which degree of saturation is only one. Anomalies are therefore often open to a range of interpretations, and are not by themselves conclusive.

The second group of methods considered to be applicable were thermal methods. Previous applications of thermal methods of leakage detection in the UK have largely concentrated on measurements of the surface temperature of the structure; infrared thermography and direct temperature measurement have both been attempted with some success (Tedd & Hart, 1988), although it was not possible to determine the source of the seepage at the sites studied.

An alternative approach has more recently been developed, which relies on measurements of temperature made below ground level within the fill of an embankment dam. The technique is referred to as Seepage Detection by Temperature sounding, and detects anomalies in the normal ground temperature distribution caused by percolating water with a significantly different temperature (Dornstädter, 1997). The method is patented by GTC Kappelmeyer GmbH of Germany, and prior to its use at Rotton Park, had not previously been attempted in the UK. It was considered that this method would give a higher chance of success than other geophysical techniques, and a trial was undertaken by GTC Kappelmeyer on behalf of British Waterways.

TEMPERATURE SOUNDING METHOD.

This technique exploits the seasonal variations in temperature which occur within both ground and surface water bodies. At or near the surface, the temperature of both the ground, and adjacent bodies of water will be similar throughout the year; in practice this similarity extends to around 2m below ground level. However, the low thermal conductivity of soils lead to a phase shift between the temperature of a surface water body and the temperature at greater depth within the ground. This difference becomes more marked with increasing depth, since the heat capacity of the soil reduces the magnitude of the seasonal temperature variations. Fig. 7 shows how an idealised temperature distribution varies with depth for winter conditions (solid curve); the temperature steadily increases between 2 – 5m depth, followed by a marked reduction in the temperature gradient with depth below this level.

The flow of fluid through the soil from a relatively cool body of water may lead to an anomaly in the temperature distribution with depth. The broken curve in Fig. 7 shows an idealised temperature distribution at a point where leakage is taking place from an adjacent body of water. The anomaly between this and the "normal" curve is interpreted as indicating the presence of leakage.

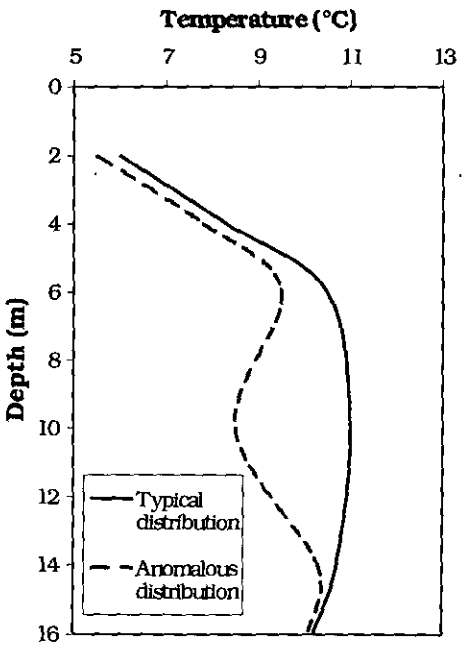

Fig. 7 - idealised temperature distribution with depth (winter).

The reverse situation can also be applied during summer months, when the ground surface and adjacent water body will be considerably warmer than the ground at depth; in this situation, the graphs shown in Fig. 7 are reversed. In spring and autumn however, the differences between temperatures within the ground and the temperature of surface water are relatively small. During these periods, anomalous low or high temperatures close to the temperature of the surface water within the seepage zone would indicate a leak at the location. In a 2D-graph this would appear as concentric isotherms around the leak, and in the temperature-depth graphs as a constant temperature over several metres.

In order to measure the distribution of temperature with depth, a series of small-diameter, thread-coupled hollow steel tubes are driven into the ground along a section line through the area of suspected leakage (Fig. 8). The tubes are driven by hand-portable equipment to the required depth; depending on ground conditions, depths of 20-25m can be achieved. Following installation, a series of temperature sensors are lowered into each tube on a cable.

The results are normally presented on two-dimensional sections, with temperature measurements contoured for ease of interpretation. Typically, the "raw" data results are presented as an isothermal distribution against depth and chainage. Interpretation is sometimes aided by normalising the temperature measurements against a "reference" thermal distribution, measured remote from the area of suspected leakage. The differences in temperature between the reference and measured profiles for each depth are then plotted as a temperature difference contour plot. However, the temperature-depth plots remain very important for the interpretation of the data for individual soundings. In this type of plot, the readings are not altered by interpolation and there is greater confidence in the interpretation of the results.

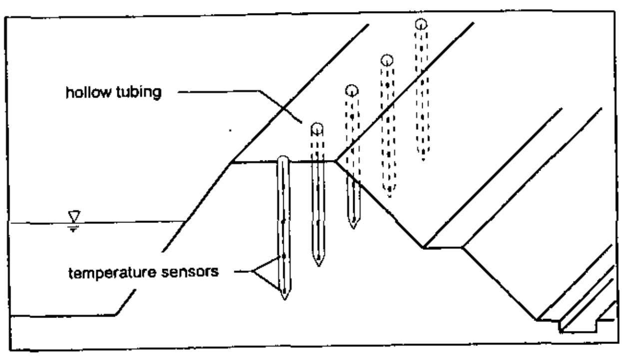

Fig. 8. Layout of probes for measurement of ground temperature distribution.

Results

The above technique was applied at Rotton Park during January 1999, to take advantage of the low seasonal temperatures. During the test period, reservoir water temperatures were measured at intervals, and fell within the range 4.8° - 5.4°C.

A total of 26 No. soundings were taken at nominal intervals of 20m across the dam. The probes were driven to depths of either 15 or 18m from the dam crest. The probe locations were situated on the downstream side of the crest, in order to try to avoid the clay core which is believed to be present.

Ground temperature measurements were made at depth intervals of 1m in each tube, referenced to the dam crest as zero depth. The readings were presented in graphical form, including depth/ temperature plots and as vertical isotherm sections, as described above. These results are shown in Figs 9 - 11.

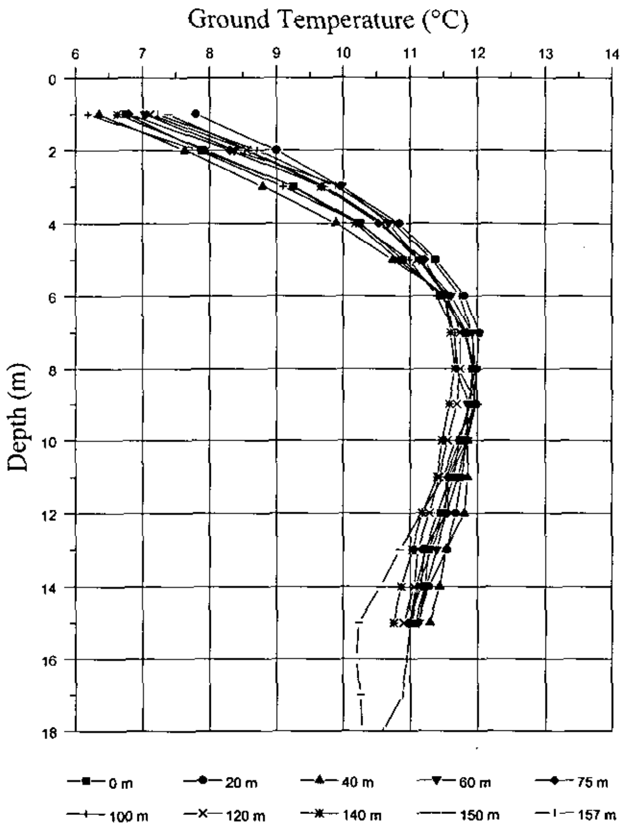

Fig. 9. Temperature distribution plots, chainage 160 – 360 m.

The depth/ temperature plots show that profiles in the vicinity of the spillway channel to the south are slightly anomalous in the upper 6m of ground (for example the profile at chainage 360m). This is thought to be due to the location of the probes within different material, and corresponds with the positioning of the probes within natural ground rather than the dam embankment. None of the probes measured temperature anomalies approaching the reservoir temperature.

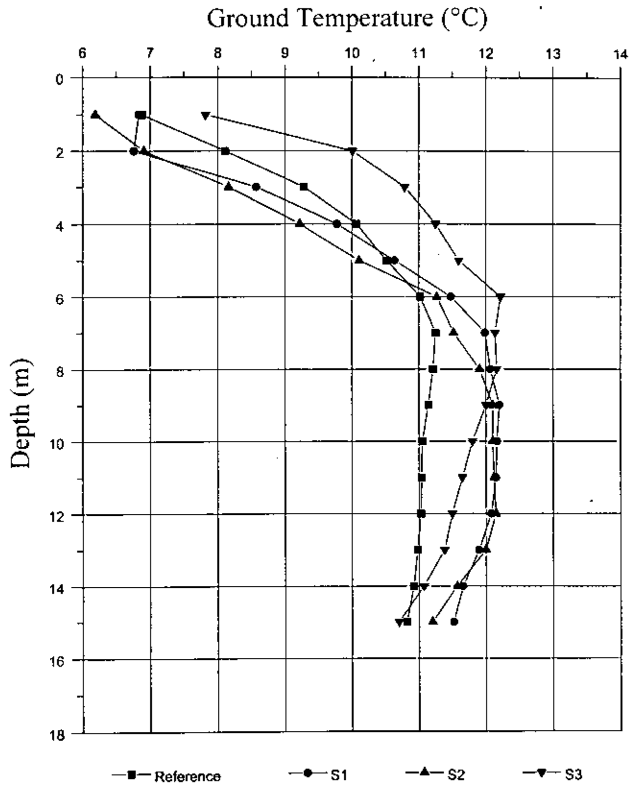

Fig. 10. Temperature distribution plots, spillway area (S1 – 3) and reference point.

A small number of profiles to either end of the dam suggest that groundwater is flowing through the natural ground to either side of the dam (S1 – S3, and the "reference" probe, Fig. 10). The depths at which these probes indicate that flow is taking place are 13-15 m to the south of the dam, and 6 – 15m on the north side.

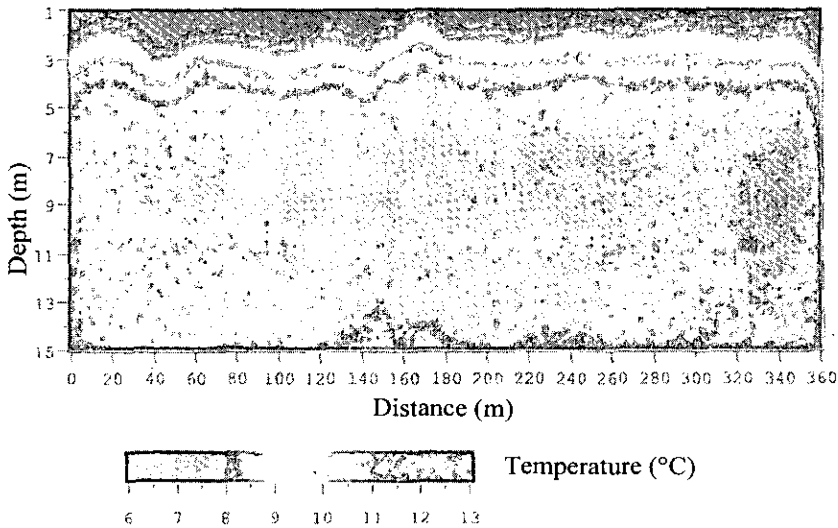

Fig. 11: 2-d isotherm section of Rotton Park reservoir.

DISCUSSION.

The temperature measurements made at Rotton Park indicate that no significant leakage is taking place through the dam, but rather that the observed seepage is due to groundwater. The clear correlation between the piezometers and reservoir level, however, indicate that this flow is also contributed to by the reservoir; this is not surprising, given the permeable nature of the glaciofluvial materials to either end of the dam. The slightly lower than expected temperature of the groundwater (8°C, as opposed to the 10°C expected), suggest that the reservoir water is mixing with the groundwater to a greater or lesser degree to alter it's temperature signature.

In addition to increasing confidence in the model which had been derived from the previous investigations, the temperature measuring exercise has also indicated that seepage of groundwater and reservoir water is probably taking place around the northern (left) abutment of the dam. This seepage does not emerge from the ground, probably due to the higher level of adjacent ground below the dam towards the northern end.

A number of site constraints affect the applicability of this technique, not least the need for marked (and consistent) seasonal temperature variations. However, given appropriate conditions, the use of temperature measurement within the ground clearly offers a viable tool for leakage detection within embankment dams.

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Leakage investigations at Guide Reservoir near Blackburn, Lancashire

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SYNOPSIS Guide reservoir is a J F Bateman design, completed in 1846 and raised in 1854 by approximately 900 mm. No written record of seepage exists, but piezometers were installed in 1986. In the summer of 1997 water was noted flowing across the toe area below the main embankment. A seepage path between the original bank and its raised crest was suspected. Resistivity survey does not confirm leakage at high level but locates an unexpected wetted zone at a depth of 7.5m. Further studies enable a hypothesis of hydraulic fracture and flow path to be advanced consistent with these results.

INTRODUCTION

Guide reservoir is located approximately 2 kilometres south east of Blackburn Town Hall at National Grid Reference SD 703 258. It was designed by J F Bateman, the first Engineer to Blackburn Waterworks. The contractor, Samuel Taylor, an experienced builder of reservoirs who designed and built the original Belmont Reservoir in 1827 and who worked on many occasions with J F Bateman, completed the work in 1846. The reservoir was subsequently raised in 1854 by approximately 900 mm. The records imply some doubts about the watertightness of the embankment in 1984, but during the summer of 1997 a water flow was observed beyond the toe. A seepage path between the original bank and its raised crest was suspected.

In an attempt to confirm the location of the seepage, and thereby reduce the cost of remedial works, a resistivity survey was carried out on the downstream crest.

GUIDE RESERVOIR EMBANKMENT

Construction

Information on the construction of this specific dam is limited to two contemporary engineering drawings, but guidance on his work can be inferred from the writings of J.F.Bateman (1884). The reservoir is formed by three earth embankments on the side of a hill and was probably built using mainly material excavated from within the reservoir basin. Only one drawing of the original embankment works and one of the later embankment raising remain in the archives. Watertightness is achieved by a puddle clay filled cut off trench in the Glacial Till with a puddle clay core above original ground levelBateman reassessed his design floods for his reservoirs following the disastrous floods at Bold Venture in Darwen during 1848 and the floods in the Longdendale valley in the following year, but it appears that Guide reservoir was raised simply to increase storage. The crest was raised by approximately 900 mm in 1854.

The slope of the upstream face of the embankment varies from 2:1 near the crest to 3:1 on the lower slope and is protected by stone pitching. In their raised condition the embankments reach a maximum height of 11 metres. The total length of embankments is 600 metres. The downstream face is at an average slope of 2:1, slightly steeper near the crest, and is covered with grass, as is the crest. The drawings show a tapered puddle filled cut off trench approximately 3.1 metres deep and core 3.1 metres wide at the base reducing to 2.5 metres wide at the crest. The raising of the embankment was not symmetrical, typical section shown in Fig.1, and resulted in the centre of the core coinciding with the downstream edge of the raised crest.

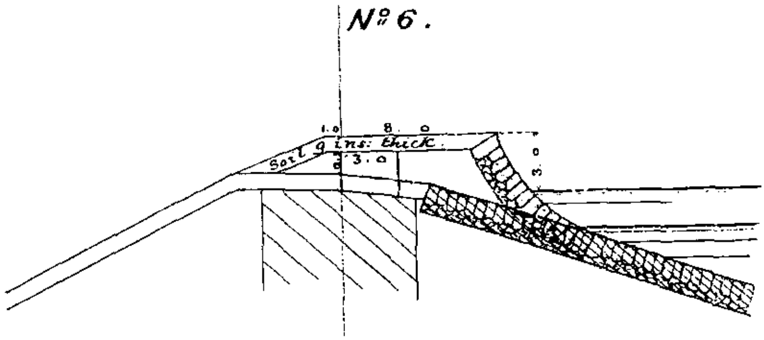

Fig.1 Raising of the embankment (from an original drawing)

The reservoir has virtually no natural catchment other than its surface area. Inflows can be piped to the reservoir from Pickup Bank, Fishmoor or Dunsop Bridge. It is classed as a Non Impounding Large Reservoir and has been inspected regularly since 1934. In the report of the original inspection the Inspecting Engineer Mr. Atkinson stated *"...trial holes showed that the embankments had been raised since the reservoir was first built, and 2ft 6ins to 3ft of new puddle placed on top of the old work without removing the surface soils from the top of the original puddle.... Even though the surface soil had not been removed from the top of the original puddle, no leakage had been observed through the layer of soil."* This is a clear indication there was no leakage.

Incidents

Following an inspection in 1984, P.S. Hallas, of Rofe Kennard and Lapworth, reported the embankments to be dry (the reservoir was empty for the inspection) but he required *"...a series of three standpipe piezometers be installed at survey points 11, 15, 18, and 20 to determine the phreatic* *surface within the embankment and the water levels be measured.."* Presumably there had been reports of seepage, but no written record of the problem exists. The piezometers were installed in 1986 and after the following inspection in 1994 B.H.Rofe commented that *".... the piezometers installed in 1987 give a useful guide to the seepage at foundation level but do not assist in describing the piezometric gradient through the embankment."*

In the summers of 1996 and 1997 there were influxes of gypsies, following which the area beyond the toe of the bank was bladed to remove rubbish. During their second visit water was noted standing on and later flowing across this area. The Supervising Engineer reported in October 1997 that *"...some toe piezometers show higher levels than those at the mid point...".* This has not been confirmed since and may well have been the result of a misreading of the levels. Investigations of the site in 1999 have revealed a culvert that apparently supplies a farmer's trough in the adjoining field. The culvert is fed from the overflow channel and is laid along the toe of the dam. It is suspected that the culvert may have been broken in the tidying up operations following the first influx of gypsies.

In practice leakages have only been noted at high water levels when water has been seen flowing across the level ground at the toe of the main or southwestern embankment.

Investigations

Guide reservoir is shown on sheet 76 of the British Geological Survey. Drift deposits at the site are Glacial Till (formerly known as Boulder Clay) overlying the Lower Carboniferous or Namurian series. The Solid map indicates that the Namurian series is represented here by Old Lawrence Rock sandstone. Geological faults lie beneath the eastern and western edges of the reservoir. The presence of faults is not unusual for Pennine dams and past Inspecting Engineers for this reservoir have commented that they are likely to be covered by sufficient Boulder Clay to prevent them becoming a source of leakage.

In 1986 and 1987 a series of borings were made through the embankments to investigate their stability and install piezometers (Geo Research Ltd, 1987).

Three piezometers 11A, 11B, 11C were installed respectively at the top middle and base of the downstream face in the centre of the southwest bank. Despite the fact that the embankment is approximately 11 metres high, only the top four metres of material from 11A, the borehole on the crest, is recorded as "Fill" strengthening the hypothesis that locally excavated materials were used in the embankment construction. There are no other recorded differences in the materials found in these boreholes indicating that the downstream shoulder is constructed of the same material as the core and that the fill material was not zoned. This conclusion is not unexpected, given that at this date zoning was not being practiced by Bateman on other dams.

What is unexpected, however, is the presence of pebbles and peat inclusions in the core material. This may well indicate that the borehole passed through or close to the core wall rather than the core being contaminated. It is also possible that the borehole wandered "off centre" with respect to the core and that grass sods have been described incorrectly as peat. That pebbles occur at low level would not be detrimental, indeed it was common practice to include pebbles in puddle cores or more generally in blankets to deter burrowing animals.

Bateman would probably have been well aware of the deleterious effects of peat, in the core, and Taylor, the Contractor, had at least 20 years experience on reservoir construction prior to Guide. On early canal reservoirs peat is frequently encountered in the shoulder fill, where the relatively slow rate of construction gave sufficient time for expulsion of its moisture and its compaction.

Piezometers

Existing records contain no indications of leakage through any bank but a series of piezometers were installed in 1987.

In the spring of 1993 the reservoir was emptied for remedial works, mainly to the draw off. Between April and May the reservoir was filled from 8.3 to 2.0 metres below top water level. This fairly rapid increase in water level appears to have had only a small effect on the piezometric levels, indicating that the banks were watertight between these levels.

B.H.Rofe, the Inspecting Engineer was, however, concerned in his 1994 report that "*...continuing settlement of such a long established structure would seem to indicate that some transfer of material was taking place through seepage and settlement could be expected to continue arising from the stresses induced by the continual filling and emptying of this reservoir.*"

Readings taken at piezometers 11A, 11B and 11C during the period from September 1998 to February 1999 are shown in Fig.2. Borehole 11A is located near the downstream edge of the crest, apparently unintentionally within the core.

In September 1998 reservoir water level was recorded as 195.08 m AOD. Piezometric level in borehole 11A is recorded as 191.35. Indicating that water flowing from the reservoir and through the core had lost 3.73 metres of head in the process. Boreholes 11B and 11C consistently record substantially the same water level, with a difference of only 3-400 mm. The pore water pressure levels are just below ground level at the toe.

Readings do not appear to vary with water depth. They are substantially constant.

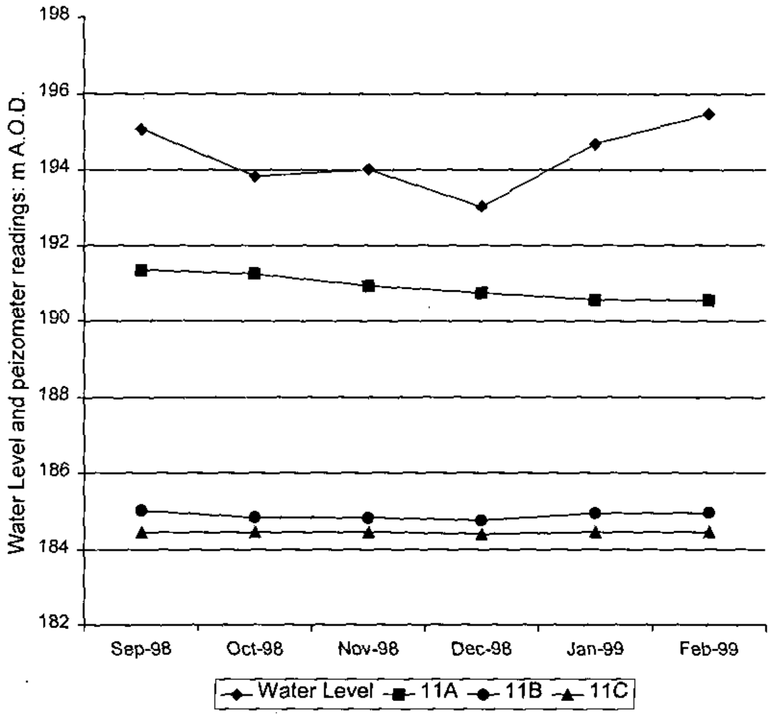

Fig.2 Piezometer readings (T.W.L. = 195.480m A.O.D.)

The readings shown in Fig.2 indicate approximate head losses of;

2 to 5 metres through the core

5 to 6 metres between peizometers 11A and 11B

<1 metre.between 11B and 11C.

The authors interpret the non-uniform hydraulic gradient as a result of a permeable topsoil layer remaining beneath the embankment.

RESISTIVITY SURVEY

Resistivity and earth materials

Resistivity is a fundamental material property that should be distinguished from the concept of "resistance". The resistance in ohms of, for example, a cylindrical body to the flow of electricity is a function of its shape and the electrical characteristics, i.e. the resistivity (units in ohm-m), of the material from which it is made. If the length of the cylinder is doubled then its resistance is also doubled. Resistivity, in Kearey and Brooks (1991), is defined as the resistance in ohms between the opposite faces of a unit cube of material. The earth has more complex boundary conditions and is invariably not homogeneous but layered with materials of differing resistivity.

These concepts lead to the question of how does electricity flow through earth materials? Rock and soil materials consist of aggregates of mineral, crystals or grains, between which there is void space. The majority of minerals are insulators and therefore do not conduct electricity. The void space, in general, is occupied by a mixture of air and water. below the water table level soil is fully saturated (degree of saturation, $S = 100\%$) and above the water table partially saturated materials ($0<S<100\%$) contain a mixture of air and water. Since air is an insulator electricity can only flow through interconnecting void water space. There is an implicit inverse relationship between the cross sectional area of interconnecting voids water and the resistivity value. One other contributor to the value of resistivity is the nature of the void water, itself an electrolyte, which will contain varying concentrations of dissolved salts. Saline water is more conductive than fresh water. In its guide to the physical properties of soils and rocks, ABEM (1971), does not quote a resistivity value for water but states that it is very sensitive to impurities.

The electrical resistivity of earth materials is an extremely variable property. ABEM (1972), provides a table showing values of 1 ohm-m for clay and marl increasing to over 10^6 ohm-m for crystalline, igneous, rocks. Natural clays and clayey soils are at the lower end of this range. Values quoted are in the range 6 - 600 ohm-m (Griffiths & King 1981), 1 - 200 ohm-m (ABEM, 1972) and 1 - 120 ohm-m (Parasnis, 1997). Clay minerals, by virtue of the presence of a thin layer of ions on their outer surface, conduct electricity. Thus the quartz /clay particle ratio of a soil (quartz being present as sand and silt grains) directly influences resistivity.

Measurements on laboratory compacted soils indicate a relationship between resistivity and moisture content and/or air voids ratio. McCarter (1984), presents a relationship for Cheshire clay (which the authors assume to be glacial till) as shown in Fig.3. The graph is a plot of resistivity against fractional volume of water (volume of water / total soil volume) rather than moisture content (mass of water / mass of solids). It is appropriate to use fractional volumes, since the cross sectional area of water is proportional to its volume and resistivity is an inverse function of the cross sectional area of water. It should be noted that other clay soils would produce differing plots.

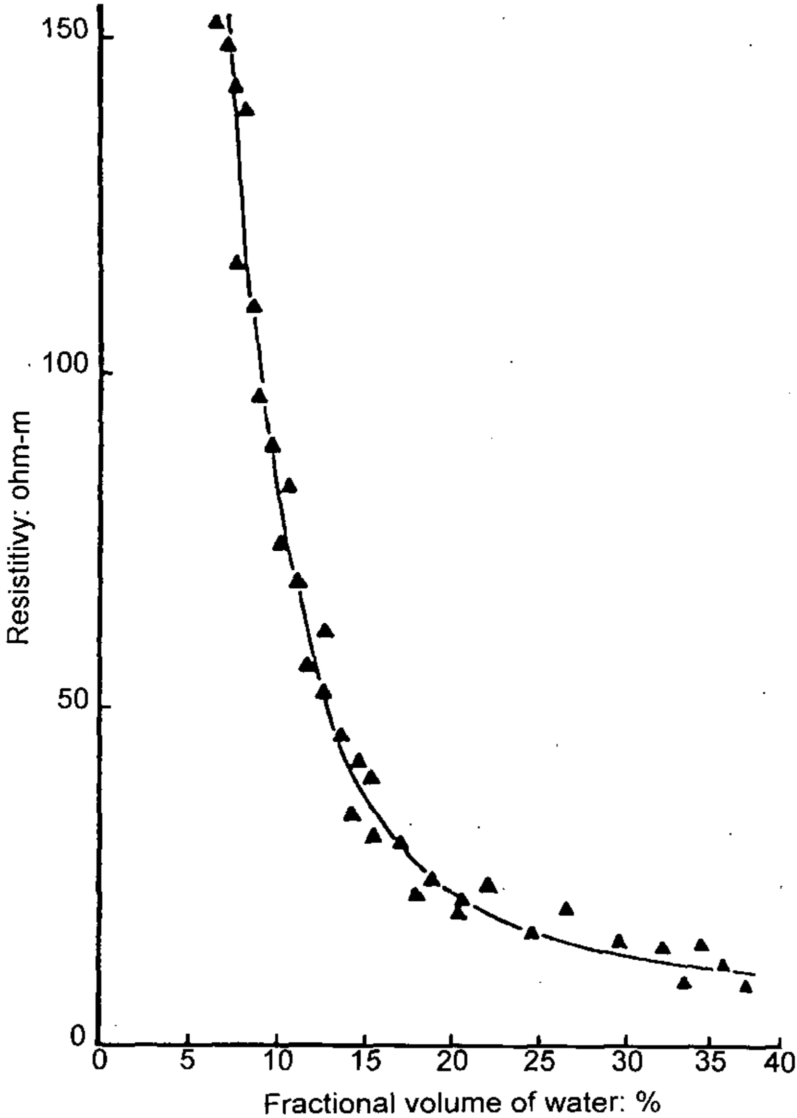

Fig.3 Resistivity characteristics of Cheshire clay

Resistivity method

In this application a low frequency, 20hz, alternating current is passed into the ground by two "current" electrodes applying a voltage of up to 400v. The resulting potential distribution at the earth's surface is measured by two "potential" electrodes. In the Wenner array used for this study four electrodes are set on a traverse line with equal spacing of 'a' metres. The resistance value (R ohms) measured, effectively at the centre line of the array, is a weighted average of the various strata in the ground and converts to an "apparent" resistivity, ρ_a ohm-m, by the standard equation;

$$\rho_a = 2\,\pi\,R\,a$$

The effect of increasing the distance between the "current" electrodes is to cause electrical flow to penetrate deeper into the ground, whereby the measured value of apparent resistivity is influenced by earth materials at greater depth.

The multi-electrode array has been a major technological development in the 1990's enabling the automated reading of many electrodes at variable increments of spacing. The Campus Imager equipment used for this application allows 25 electrodes to be connected from a cable with takeouts at 5m intervals. The spacing between electrodes can, therefore, be 1m through to a maximum of 5m. The electrodes are copper plated mild or stainless steel 8mm diameter rod about 250mm long. The resistivity meter is controlled by a field computer that also takes in the data.

The pattern of automated readings is such, that initially all adjacent electrodes are consecutively read in groups of four along the traverse line. This is repeated with firstly every other electrode missed out then every second until just one reading is taken using every eighth electrode. Thus, the 'a' spacing is incremented in steps of the spacing used to set out the array, e.g. an electrode spacing of 5m results in 'a' values of 5, 10, 15m etc. The automated reading process takes about 25mins to complete.

Advances in software design have enabled this wealth of data to be analysed by the process of inversion. Inversion is an iterative technique in geophysics whereby raw data is converted back to a theoretical "model" which could have produced the field data. In this instance a distribution of resistivity values in an assumed vertical plane beneath the traverse line, as shown in Fig.6. Inversion is ambiguous, in that several models can be devised from one set of field data. The inversion software used in this analysis does not allow alternative models to be produced.

Field procedure

As can be seen from Fig.4 the Campus "Imager 25" meter and Taurus "Rocky" field computer are a compact and portable field kit. The traverse line was placed on the downstream crest initially centred over the wet spot at the toe. Existing permanent survey stations set in concrete bases were used to set out the traverse line.

In order to test the theory, that water might be seeping through the interface between the old structure and the raised level, a traverse was performed on the downstream crest above the toe seepage with an electrode spacing of

1m. This spacing provides a useful profile to a depth of 3m, as shown in Fig.6.

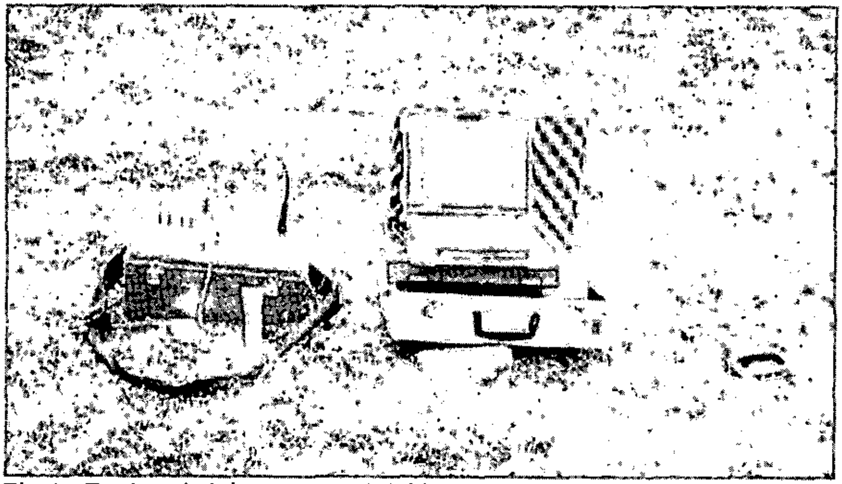

Fig.4 Earth resistivity meter and field computer

A full depth survey using an electrode spacing of 5m to give a profile down to 15m below crest level, revealed a resistivity "low" anomaly (see Fig.7A Guide T1 dated 30/07/98) at the northerly end of the profile. Subsequent traverse lines were moved 5m north to locate the anomaly more centrally.

It has been noted previously that the seepage flow at the toe is most marked at high reservoir levels. Since water level in this reservoir can be controlled a decision was made to adjust and maintain water level and monitor the resistivity profile over the period 30/07/98 to 21/01/99. The plots in Fig.7A and Fig.7B show the modeled resistivity profiles as dated, with reservoir water levels superimposed on the right hand scale.

Interpretation

The results of a relatively shallow survey to 3m depth, see Fig.6, indicate moderate resistivity (greater than 106 ohm-m) but show no signs of a low resistivity zone consistent with wet material at the base of the raised section.

Surveys to a depth of 15m below the crest, see Figs.7A and 7B, reveal an unexpected resistivity "low" (less than 50 ohm-m) at a depth of 7.5m below the crest visible as a dark grey/black zone.

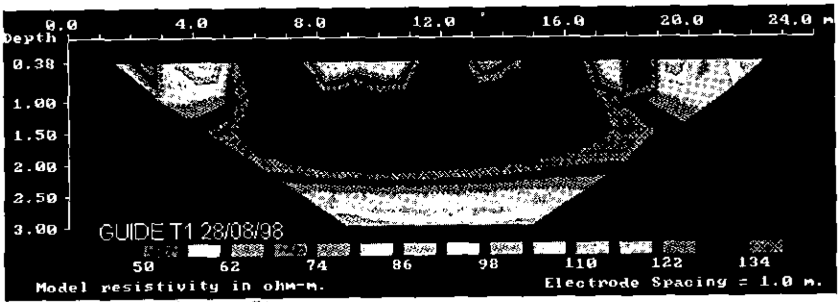

Fig.6 Resistivity profile on crest

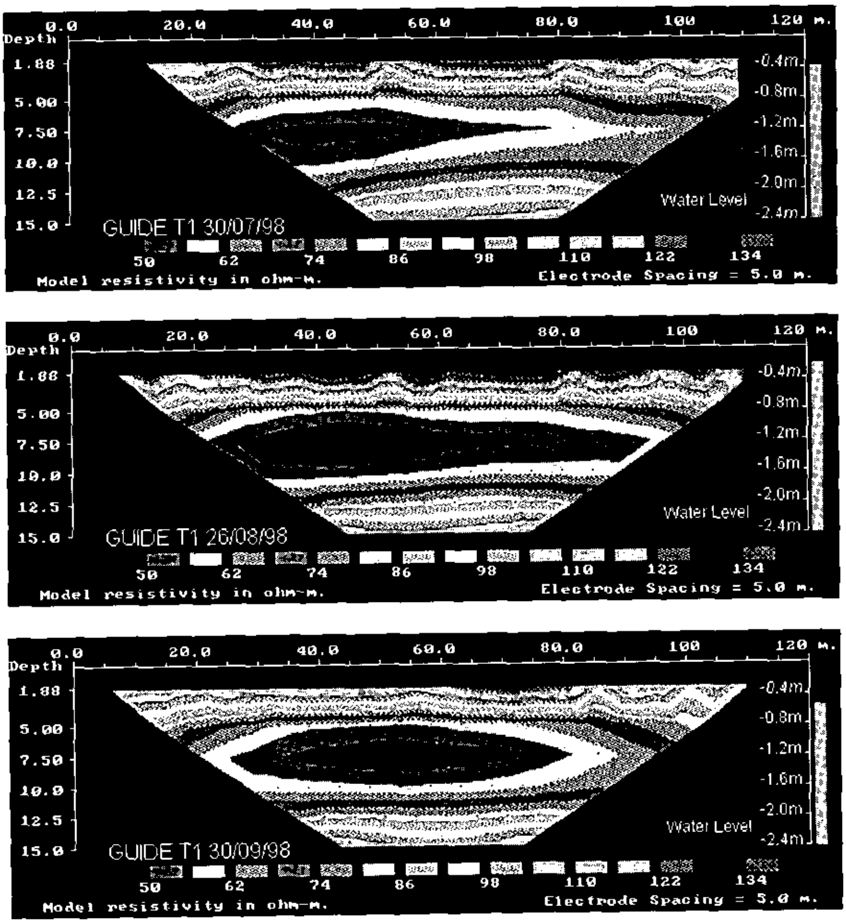

Fig.7A Resistivity profiles on crest

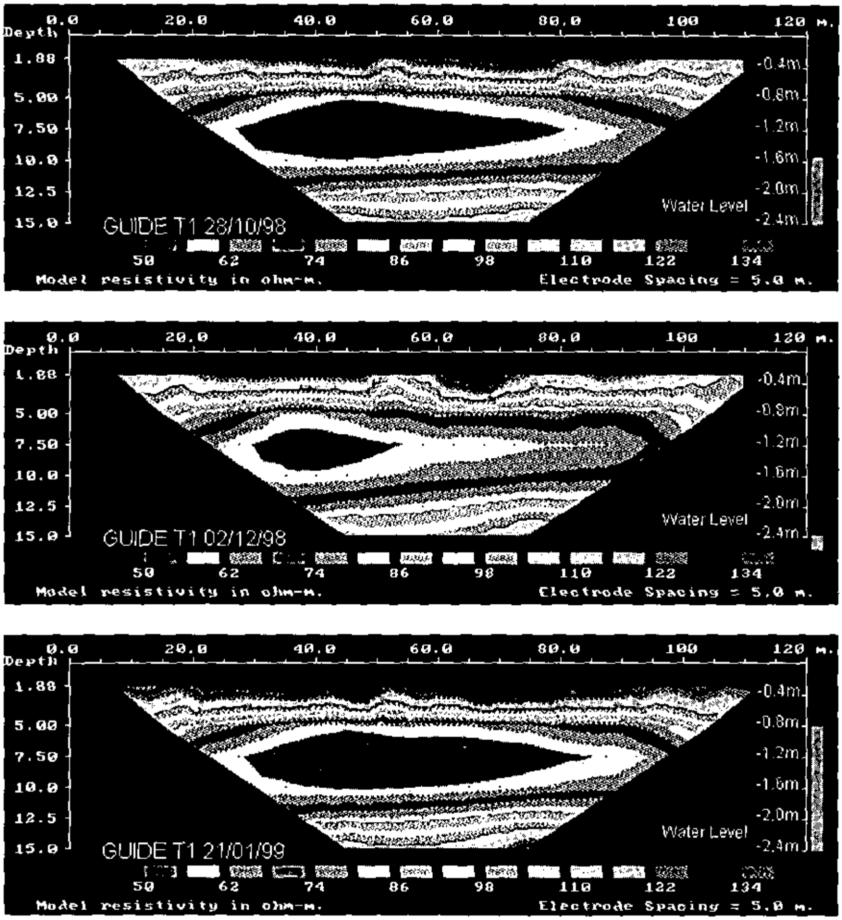

Fig.7B Resistivity profiles on crest

It is important to appreciate that the plots in Figs.6 and 7 are re-constructed or modeled on field data. In practice, even abrupt resistivity boundaries, such as clay against concrete, plot as graded contours over a distance either side of the boundary. The computer analysis has therefore a "fish eye" lens effect distorting the size of the original feature. It is highly unlikely that a feature with a width that varies up to 65m exists within the embankment. This does not, however, preclude an estimation of the location of the wet zone.

The anomaly is centred at a depth of 7.5m below crest level on all plots shown in Figs.7A and 7B. The constancy of depth is due in part to the regular grid of data points, seen as small dots and in part to the contouring algorithm. The actual depth is within the range 5 to 10m below crest level.

The position of the centre of the anomaly along the traverse line, measured at the point of maximum vertical plot thickness, varies from chainages 40 to 65m, averaging at 46 m.

A plot of drawdown against anomaly width, shown in Fig.8, demonstrates an approximate correlation between the amount of drawdown and anomaly width. Increased drawdown shrinks the anomaly and it is reasonable to assume that this reflects the extent of the wetted zone. Some of the drawdown was due to demand and maintenance so that no attempt could be made to ensure equilibrium conditions were attained prior to measurement, a feature that contributes to the scatter of values.

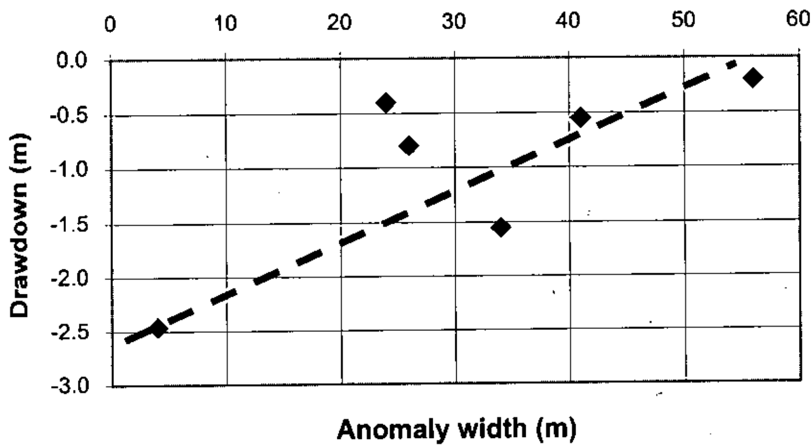

Fig.8 Drawdown versus anomaly width

In summary, the resistivity survey results do not show a low resistivity anomaly at shallow depth consistent with the hypothesis that water is leaking through the base of the raising. Deeper investigations reveal a resistivity low close to or just within the downstream face of the puddle core. Interpretation indicates a wetted zone at a depth between 5 and 10m located near chainage 46m.

INTERPRETATION OF INVESTIGATIONS

In 1989 North West Water commissioned Rofe Kennard and Lapworth to undertake a general review of hydraulic fracture and to give specific advice on a number of their reservoirs.

Skempton (1987) had earlier concluded that the approximate limiting condition for hydraulic fracture in a puddle clay core was a slenderness ratio, H/b of about 5, where H = height of dam from ground level and b = core width at ground level. Vaughan (1987) showed that for a core width of 2.4m and side batters of 12:1 or less, the risk of hydraulic fracture was essentially absent. For the horizon of lowest resistivity the slenderness ratio for the full core is 25/9, approximately 3. The effect of boring a trial hole through the centre of the core is to halve the effective width. The slenderness ratio increases to 6 or greater and hydraulic fracture becomes possible.

It is known from Bateman (1884) that at about the time that he was constructing Guide he was also building Rhodeswood Reservoir in Longdendale for Manchester Corporation Waterworks. In his section for Rhodeswood Reservoir, Bateman shows a wall of "Grass Sods" separating the core from the shoulders. Guide Reservoir was built in 1845 and raised in 1854 almost contemporary with Rhodeswood, which was completed in 1852.

Inspection of Bateman's Audenshaw No.3 reservoir embankment, built in 1875 and recently removed to build a new motorway in Manchester, has revealed the presence of topsoil under the shoulders. This pattern of construction would explain the pattern of water levels recorded by the piezometers at Guide.

If we assume that this section was used at Guide, combined with the possibility that the original grass cover was left in place as at Audenshaw then a possible mode of failure can be hypothesised. Prior to the site investigations there were small seepages through the embankment. A site investigation borehole was inadvertently drilled through the core instead of the downstream shoulder. There was no reason at the time to take special care over backfilling to ensure its watertightness. The effect is to substantially increase the effective slenderness ratio of the core. The reservoir was subsequently refilled rapidly which may have induced hydraulic fracture through the core. There are two possible routes for the leakage path. Either, when the reservoir approaches top water level water passes through the upstream shoulder and reaches the upstream grass sod wall and down the wall to the fractured section, approximately 7.5 metres below the crest. Leakage passes through the core and down the downstream sod wall until it reaches the original grassed ground surface and thence to the toe. Or alternatively, water passes down the borehole and then finds its way to the underlying original grassed ground surface emerging at the toe.

CONCLUSIONS

In the summer of 1997 water was observed flowing across the toe area of the main embankment. It was initially suspected that leakage was occurring at a high level beneath a raised crest section. The normal mode of investigation for a suspected high level leak would have been by trial pitting. Such a method would have failed to determine a leakage path.

Resistivity traverses reveal an unexpected resistivity "low" consistent with a wetted zone at a depth of 7.5m below the crest. The extent of the anomaly is dependent on reservoir depth.

Asymmetrical raising of the crest resulted in a borehole being inadvertently drilled though the core, thus locally halving its effective width. Rapid refilling of the reservoir following unrelated remedial works caused hydraulic fracture of the core. It is deduced that flow is occurring through the core at this horizon.

Investigations by the authors have revealed that it may have been normal practice for J F Bateman to leave any grass cover in-situ under the shoulders of an embankment, as seen at Audenshaw. In his contemporary work at Rhodeswood, Bateman incorporated walls of "Grass Sods" separating the core from the shoulders. This mode of construction explains the pattern of piezometric levels recorded at Guide and would account for a flow path down either the borehole or the downstream grass sod wall and along former topsoil to the toe.

Resistivity survey allows investigation of an embankment over its full height and at Guide has resulted in the discovery of an unexpected leakage path at depth without damage to the embankment.

ACKNOWLEDGEMENTS
The authors wish to express their thanks to D.B. Wickham and W. Aighton at North West Water and J. Parkin at Bolton Institute for their support during this research. They are also indebted to Martin Webster, geotechnics technician at Bolton Institute, for his cheerful and unflagging efforts in keeping the show on the road.

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Construction of a cement-bentonite slurry trench cut-off at Pebley Dam

R BROAD, British Waterways

SYNOPSIS. Pebley dam is a late eighteenth century embankment dam built to provide water for the Chesterfield Canal. This paper describes the construction of a cement-bentonite slurry trench cut-off wall with an impermeable membrane, which was undertaken in order to stop leakage that was occurring through the dam at high reservoir water levels.

INTRODUCTION

Pebley Dam was built for water supply to the Chesterfield Canal circa 1775 and forms an impounding reservoir with a catchment of 3 sq. km. Together with Harthill Reservoir lower down the valley, they provide the remaining water storage for the partly restored Chesterfield Canal and can provide sufficient water for six weeks running at 20 Mld. The feed enters the top pound at Kiveton. Two other reservoirs provided additional storage when the canal was built but they have been lost to mining operations and fishing. Pebley Dam is an earth fill structure without a visible clay core and is approximately 10m high and 160m long.

The dam is reported to have been constructed on the site of an earlier mill dam. There is an overflow channel at the east end of the dam and draw-off controlled by a 230mm diameter plug valve in a wet shaft midway along the dam. The draw-off inlet is well above the deepest part of the reservoir bed and there is no bottom outlet. There is a modern, curved section, concrete wave wall on the dam crest. Figures 1 and 2 show a plan of the dam and a section of the valve shaft.

LEAKAGE

The dam has suffered from mining subsidence despite the purchase of two protective pillars in two seams under the dam. For many years the dam has leaked in two areas:

1) In a gully near Pebley Brook some way from the dam.
2) In a morass area at an elevation of 109m AOD towards east of centre of the dam.

More recently the dam started to leak at an elevation of approximately 114m AOD just at the junction between the upper section of the dam and the top of the berm as shown in Fig 1.

Dams 2000, Thomas Telford, London, 2000.

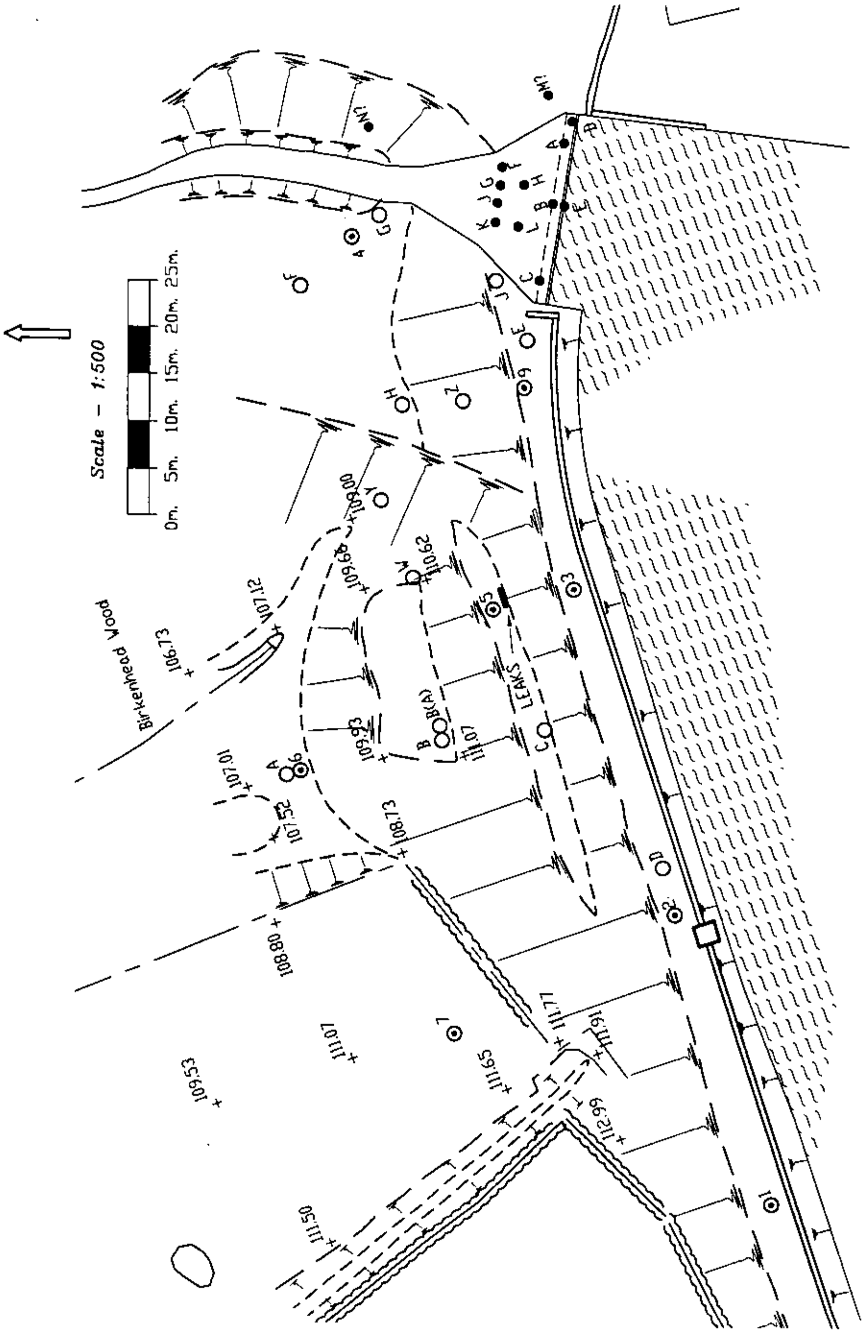

Fig. 1. Plan of Pebley dam showing location of boreholes and leaks

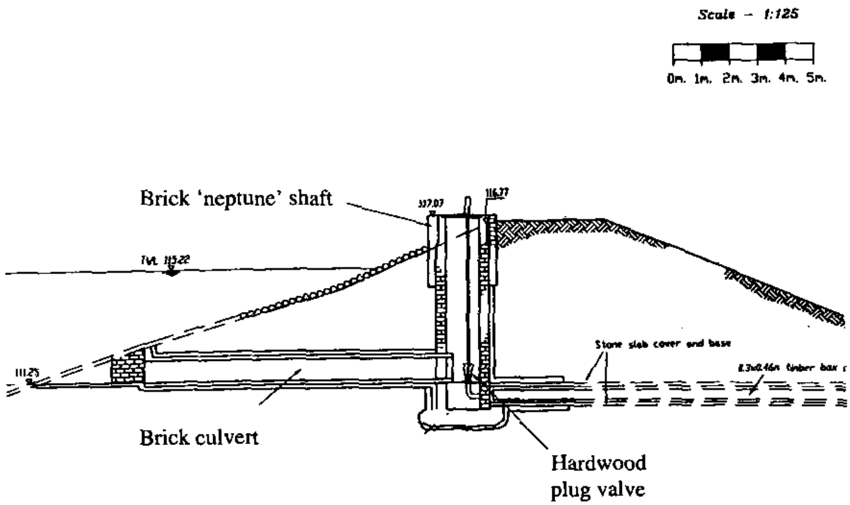

Fig. 2. Section through the valve shaft

There were 4 leakage areas discharging a total of about 3 litres per minute. The top of the berm became saturated and the stability of the dam was considered to be at risk. Leakage ceased when the reservoir level fell to 350mm below the overflow.

The reservoir level was lowered and the dam was inspected. A slumped area of dam was found under some concreted pitching connected to the wave wall foundation in a region approximately 350mm below top water level. The slumped area was at the same cross section as the leakage. The area was filled with concrete over a length of around 20m and the reservoir allowed to refill. The leaks recommenced 2 weeks later and instructions were issued to keep the reservoir down 500mm until further works could be carried out. This proved impossible as the draw off could not cope with storm inflows. Water level in the reservoir can rise by 1.0m in a few hours following a storm in the catchment area.

DAM STABILITY

A stability analysis was carried out at this stage by Robert H Cuthbertson and Ptns based on soil parameters from the report of the October 1986 inspection. Possible slip surfaces are indicated in Fig. 3 and soil parameters are summarised in Table 1. A plant load of 32.5 tonnes was included on the dam crest.

Table 1. Summary of soil parameters

	Unit weight, kN/m^3	ϕ'		c' (kN/m^2)	
		Mean	Lower	Mean	Lower
General fill	19	27	26	2	0
Clayey material	20	24	17	3	2.8

The analysis showed that stability was marginal with factors of safety as low as 1.1 for the upstream slope. The stability analysis was included with the tender documents. Subsequently the Contractor carried out his own stability analysis including his proposed plant loading on the dam crest. This showed that the crest profile required modification and a scheme to reduce crest level during construction was proposed.

The dam is constructed of local mixed fill materials, generally sandy silty clay with no apparent clay core. The dam foundation consists of a series of mudstones and shales, overlying sandstone at a varying level around. There are coal seams in the series and a lens of granular material at foundation level was identified in borehole D.

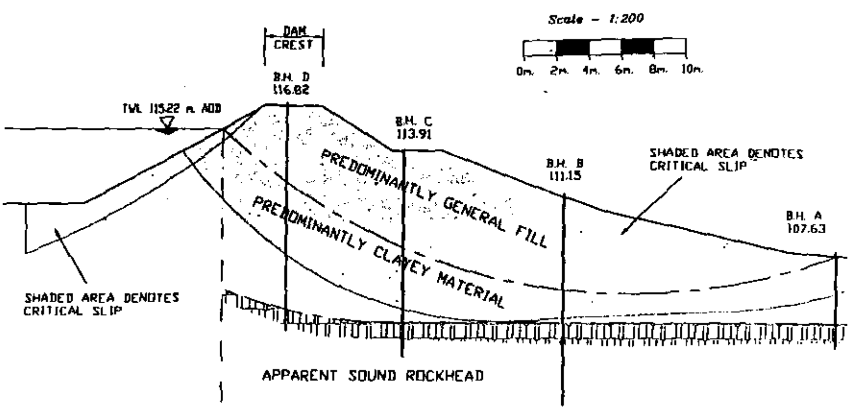

Fig. 3 Cross-section through deepest section showing potential slip surfaces

REMEDIAL WORKS

Options

Repair works were requested by the Reservoir Supervising Engineer due to the potential risk of the berm slipping and the inability to keep the reservoir down. A sheet pile cut off in the dam was proposed at an estimated cost of £160,000 and this together with 6 other options were considered by British Waterways Technical Services as follows:

a) Grout curtain
b) Secant pile wall
c) Concrete diaphragm wall
d) Cement-bentonite slurry wall
e) Vertical impermeable membrane
f) Upstream blanket

The favoured option was a cement-bentonite cut-off wall at an estimated cost of £100,000 for the following reasons:

a) more likely to seal leaks in a homogenous earth fill dam
b) construction plant loads less than other schemes
c) amenable to subsequent grouting work if not entirely successful
d) less costly than other options

Slurry wall works

Tenders were invited on an ICE Design and Build Contract in August 1998 to construct a cement-bentonite cut-off wall for the full depth of the dam, approximately 10m, from the valve shaft to the overflow weir. The extent of the wall was specified as there were no leaks in the dam outside these areas and the risk of crossing the draw-off culvert was avoided. The total length of the cut off wall was approximately 60m, 40% of the crest length.

The contract was awarded to Keller Ground Engineering in September 1998 and work commenced in October following a short period for the Contractors design and approval by the Qualified Civil Engineer. The Contractor proposed a temporary lowering of the dam crest by 1.3m as indicated in Figure 4 to achieve the objectives of widening the access and increasing stability of the embankment during the construction phase with a 32.5 tonne excavator working on the modified section. The location of the cut-off wall in relation to the crest and original profile is also shown in Figure 4. The trench width was specified to be not less than 600mm. The reservoir level was lowered to draw off level for the duration of the works.

An important detail was devised at the top of the dam to prevent leakage near top water level. Once the slurry had set, a slot was excavated into the wall and an HDPE membrane was secured into position (see Fig. 4) using a cement-bentonite slurry. The membrane was then extended using a welded connection and positioned between compacted existing fill as shown in Figs. 5.

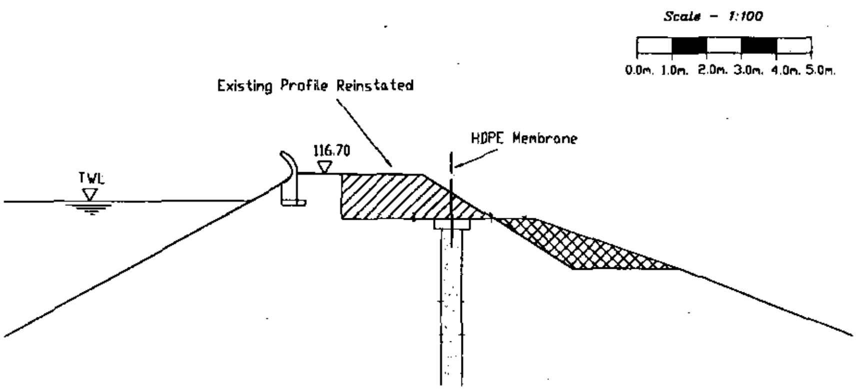

Fig. 4 Section showing installation of HDPE membrane at the top of the wall

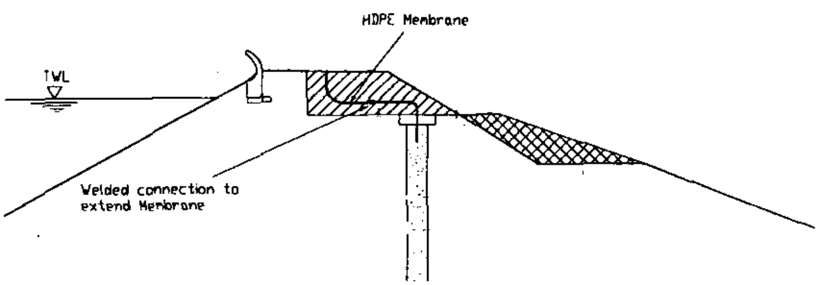

Fig. 5. Final location of extended membrane within compacted fill at the top of the dam

The construction proceeded without incident except when the excavator started to bring up some sandstone blocks in the middle area of the dam and from a level of about 110m AOD. At the same time an "upwelling" of slurry occurred. This lasted for about a minute or so and then subsided. No obvious explanation for this occurrence was apparent. The weather was very wet during the contract and the dam crest re instatement was delayed until Autumn 1999.

Slurry mix properties

The slurry mix design was not disclosed in detail but was a typical mix used by the Contractor for pollution control. It consisted of bentonite, cement, ground granulated blast furnace slag and water from the reservoir. The specification for the properties of the set slurry was performance based with the following being specified:

The minimum unconfined compressive strength (UCS) at 28 days to be 150 kPa

The permeability to be less than 1.0×10^{-8} m/s.

Results from the compliance testing are given in Table 2.

Table 2. Results of compliance testing of the set slurry.

Sample age days	UCS (kPa)	Sample age days	Permeability (m/s)
34	242	90	8.5×10^{-10}
31	153	90	1.5×10^{-10}
29	129	90	2.1×10^{-10}

POST CONSTRUCTION MONITORING

On refilling the reservoir the lower leak in the gully recommenced at its original rate. The leaks to the morass area recommenced at around 20% of their previous flow and measured 10mm on the V-notch. They achieved 22mm on the V-notch in April 1999 ie approximately their original flows with the reservoir at top water level.

It is thought that these leaks are due to the laminated and fractured nature of the siltstone and sandstone series below the level of the bentonite slurry cut off. These leaks show no evidence of eroded material. The risk to reservoir safety is thought to be negligible.

The upper level leaks on the berm are currently not visible. The reservoir achieved overflow level around the April 1999 and this level was maintained for several weeks and there was no evidence of the high level leaks. The reservoir level has generally been below spillweir level since that date.

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The use of slurry trench cut-off walls to repair embankment dams in the UK

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SYNOPSIS. The paper describes the development of slurry trench technologies and their application to embankment dams. Since the repair of Balderhead dam in 1965, both diaphragm wall techniques and the single phase method have been used to repair leaking embankment dams. More recently techniques and mixes developed for containment of contamination have been used both for new foundation cut-offs and to repair defective embankment dams. Specification for the required set properties of the single phase materials and concerns about their performance when used to repair defective clay cores are discussed.

INTRODUCTION

In most old British embankment dams, a narrow central clay core forms the watertight element. The safety of the dam is dependent on the satisfactory performance of the core and, should problems occur, on appropriate remedial works. Slurry trench cut-off wall techniques have been used to replace damaged cores in a number of dams.

The two phase method (diaphragm wall technique) and the single phase method have both been used for construction and for remedial works. The two phase method involves excavating a trench under the support of a bentonite slurry which is subsequently displaced by plastic concrete. The single phase method involves excavation of a trench, typically 0.6m wide, under a self-supporting cement-bentonite slurry which is left to set in the trench to form a watertight barrier. Since 1985, the single phase method has dominated remedial works since it is less costly and uses mixes that have been well developed for pollution containment. The technique has been used at a number of new dams to provide the cut-off in the foundation and abutments.

The cut-off wall forms a relatively narrow low permeability element and it is important that the properties of the wall are adequate for its purpose. Concerns have been expressed about the lack of compatibility between the relatively soft puddle clay and the very stiff cement-bentonite. Drying of the high moisture content material could lead to cracking if it is not protected with a clay cap. The susceptibility of the material to cracking at low confining pressures when subject to movements is also possible. The erodibility of the set material under high hydraulic gradient following formation of a crack needs to be assessed. The properties of the set cement-

Dams 2000, Thomas Telford, London, 2000.

bentonite slurry have been studied in connection with its long term performance as a barrier to control pollution migration.

HISTORICAL DEVELOPMENT

Jefferis (1997) provides an overview of the development of cement-bentonite cut-off walls in the UK and also a perspective on some of the earlier developments of the slurry trench system. The concept of excavation under bentonite to form a continuous structural wall was advanced by Christian Veder in 1938 (Xanthakos, 1979) although the use of supporting muds in well drilling is much older. Fauvelle, a French engineer, is credited with the first use of circulating fluid to remove drill cuttings in 1845 (Boyes, 1975).

Early applications of the diaphragm wall technique included impermeable cut-offs to depths of up to 40 m below earth dams in deposits of sands and gravels with large boulders (Veder, 1963). The cut-off walls were backfilled with concrete. Xanthakos (1979) describes concrete test panels that were inserted in linear trenches in the 1940s. Sherard et al (1963) report that except for some dikes below the McNay dam the slurry trench method was not used under a permanent dam structure until the Wanapum dam was built in 1958.

The first field trials of a slurry trench cut-off began in September 1945 probably independently of Veder's ideas in 1938. These trials were undertaken under the supervision of Major General M. C. Tyler, United States Army (Retired), the originator of the basic idea (Kramer, 1946). The initial concept was to use a vertical puddle clay cut-off wall to protect levees on the Mississippi river from erosion and sand boils. The trenches were excavated using a trench box for support and were then backfilled with puddle clay. However, the trench box concept was impracticable and as an alternative or auxiliary means of creating the restraining effect of a shield, the possibilities of applying clay slurry, in a manner similar to the utilisation in oil-well drilling operations were given consideration. Captain J. W. Black, Jr., US Army Corps of Engineers, who directed the field experimental work, subsequently developed and demonstrated the practicability of using clay slurry. Standard trenching machines with a maximum excavation depth of 7m were modified to obtain depths of up to 13m.

The trench was backfilled with puddle clay prepared using a box fitted with two rows of paddles to break down clay lumps to puddle consistency. Kramer (1946) quotes from *Engineering for Dams* by Creager et al (1945) *"there is nothing better for a cut-off than a puddle trench refilled with genuine puddle"*. The essentials of slurry filter cake formation, hydrostatic support and contamination were all identified. It was clear that there was a requirement for a flexible low permeability cut-off in the form of puddle clay that had been so widely used in British embankment dams. The extent to which the concepts developed in the trials were subsequently used in full-scale works on the levees is not known.

DEVELOPMENTS IN THE UK

Between 1967 and 1972 the clay cores of three UK embankment dams were repaired using two phase plastic concrete cut-off walls (Little, 1974). Some details of the dams are given in Table 1. The trenches were excavated under a bentonite slurry in panels 6m long and backfilled with a plastic concrete. In each case *tube-à-manchette* grouting had been used as part of the remedial work. Little states "*it was desired to have a more positive erosion-free membrane*" and therefore a plastic concrete cut-off was used. At Withens Clough consideration was given to producing a concrete that would resist the highly aggressive reservoir water having a pH of 3.8. A concrete mix with a significant proportion of flyash (pfa) gave the required permeability and resisted disintegration. For each of these three dams the plastic concrete mix had a water to solids ratio of approximately 0.26. Details of the various mixes and measured material properties are given by Little.

Another development of the slurry trench method was undertaken at Alton Water to provide an "*impermeable and comparatively cheap cut-off*" in the wing cut-off trenches in sands and gravels (Hetherington et al, 1976). At this site, the use of conventional diaphragm wall techniques would have been several times more expensive than a single phase process on account of the additional material, construction of concrete guide walls and problems associated with removing the bentonite slurry from the site. It was therefore decided that conventional diaphragm wall techniques using plastic concrete only should be used where leakage could endanger the dam or any other structure, but not where the result would be merely loss of water. The single phase method adopted for the works consisted of excavating a trench 600mm wide and 11m deep through sands and gravels into London Clay, using a cement bentonite grout to support the trench. The excavated material was then mixed with the grout and placed back in the trench. The set material was described as having the consistency of soft cheese. After overcoming some problems and experimenting with mixes a permeability of 1×10^{-9} m/s was achieved using a cement content of 125kg per cubic metre of bentonite slurry. The bentonite content was not specified.

Table 1 lists some of the dams where both two phase walls and single phase have been used to repair dams. It can be seen that since 1981 only the single phase method has been used but the depth of the repair has been restricted to 15m.

Table 1. Repair of dam using the slurry trench method

Dam name	Date built	Repair date	Height of dam	Depth of wall,	Reference
Two phase method			m	m	
Balderhead	1964	1968	48	46	Vaughan et al, 1970
Earlswood Com.			5	5	
Lluest-wen	1898	1973	20	35	Twort, 1977
Withens Clough	1894	1972	22	27	Arah,1975
Blenheim Lake	1760	1981	7	5	
Single phase method					
Doffcocker Lodge	1870	1985	6.5	9.5	Connery, 1985
Cadney Carrs	1974	1985	6	5.4	
Cod Beck	1953	1987	23	24	McLeish et al, 1987
Luxhay	1905	1995	19	4	Millmore et al, 1998
Lower Ormsgill	pre1851	1996	4	6	
		1998	4	6	
Monkswood	1893	1998	15	15	Penman et al, 2000
Pebley	1776	1998	12	12	Broad, 2000

DEVELOPMENT OF THE SINGLE PHASE MIX

The developments are described in detail by Jefferis (1997). In the early 1970s, the literature on bentonite excavation slurries generally regarded cement as an undesirable and damaging contaminant that could get into bentonite slurries during the concreting of structural diaphragm walls. The use of high water cement ratios (typically 1:2 to 1:10) in cement-bentonite materials bordered on the perverse when considered in the light of conventional concrete practice. In 1974, Guner (1978) started research on cement bentonite systems that had a major impact on UK slurry cut-off design. Following French practice, he investigated the effects on the properties of cement bentonite mixes of cement replacement with ground granulated blastfurnace slag. Significant benefits were imparted to the slurry, particularly at high replacement levels, substantially reducing bleed though having more limited effect on fluid loss. High replacement levels of cement with slag also markedly increased the strength of the material and reduced its permeability as shown in Figs. 1 and 2.

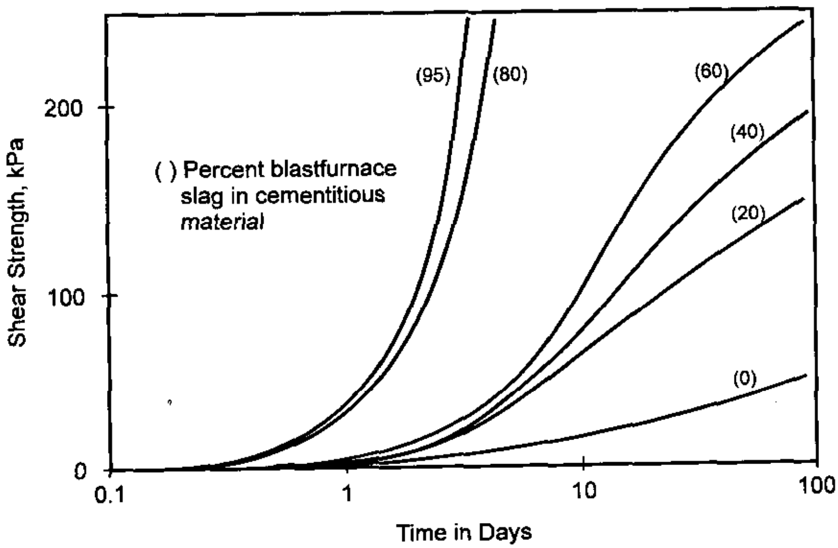

Fig. 1. Effect of slag on the strength of cement-bentonite mixes (Guner, 1978)

Table 2 shows some dams where slag cement-bentonite slurry cut-offs have been used. The first slag-cement-bentonite wall in the UK was used to form a cut-off in gravels beneath and through the diversion dam at Kielder in 1975. The specification required a permeability of less than 1 x 10^{-8} m/s after 500 hours permeation under a hydraulic gradient of 450 and a minimum deformation of 5% under a deviator stress of 125 kPa when tested undrained with a cell pressure of 500 kPa at a sample age of 90 days (Coats & Rocke, 1982). The low specified strength and high strain at failure caused considerable concern to those developing a mix design as there was a very limited research base on such materials. A mix was developed with 100kg of cementitious material per cubic metre of slurry (70% slag) and 60kg bentonite per cubic metre of water. With this type of mix, the permeability and strength conditions were not a problem but the strain criterion was difficult to achieve.

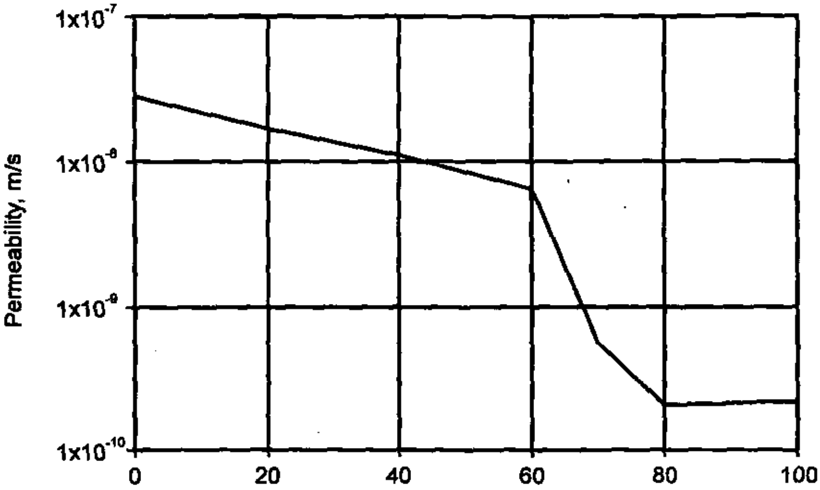

Fig. 2. Effect of slag on the permeability of cement-bentonite mixes

Table 2. Slag-cement-bentonite foundation cut-offs used during construction of new dams

Dam	Date built	Dam height (m)	Max. cut-off depth (m)	Reference
Kielder	1975	55	20	Coats & Rocke, 1982
Elvington	1994	6	6	Appleby, 1995
Blashford Lakes	1989	12	12	
Broadwood Loch	1993	~6	10	Barr et al, 1996
Lt. Testwood Lakes	1996	2	17	

Mixes for the single phase method typically consist of 35-50kg of bentonite, 35-40kg of cement and 90-120kg of slag per 1000 litres of water. The bentonite used for slurry cut-off walls in the UK is nearly always calcium bentonite converted to a sodium form with sodium carbonate, referred to as sodium activated bentonite. Pulverised fuel ash (pfa) cannot be used in the same way as slag. Direct replacement of the slag with the same quantity of pfa in a mix produces a mix that hardly sets.

The first use of an HDPE membrane in a slurry trench cut-off wall was for the 14.5 km of cut-off wall of the Jordan Arab Potash Project in the Jordanian sector of the Dead Sea. For this wall it was decided to use a Portland cement-attapulgite-Dead Sea water mix. The design of the 87,000 m^2 cut-off, completed in 1980, is described in Brice & Woodward (1984) and some of the undertaken in 1979 and as no jointing system was available for the HDPE membrane, adjacent panels were overlapped by 1 m. The first use of an HDPE membrane in a cement-bentonite cut-off in the UK was in 1990, on a project for gas and leachate control. As far as is known, HDPE has only been used in dam construction at one dam, Broadwood Loch, a flood storage reservoir near Cumbernauld, (Barr et al, 1998). An HDPE membrane has also been used at the top of the cut-off wall in the repair of Pebley dam (Broad, 2000).

SPECIFICATION AND PROPERTIES OF CEMENT-BENTONITE SLURRY FOR REPAIR OF DAMS

Currently there is no accepted specification for the properties of the hardened cement-bentonite slurry used for the repair of embankment dams. Also, there appears to be a lack of understanding of what is required of the material, its long term behaviour and what variables affect its properties. Recently, a national *Specification for the construction of slurry trench cut-off walls as barriers to pollution migration* (ICE, 1999) has been published. Although not written for repair of dams, this specification contains much relevant information both for new construction and for the repair of leaking embankment dams using the single phase method and the typical slag cement-bentonite mix used for pollution control. The specification makes recommendations for frequency of sampling and testing and the types of compliance tests required for frequency of quality control.

For an embankment dam, a cut-off permeability of 1×10^{-8}m/s may be acceptable whereas for pollution control walls a permeability of 1×10^{-9}m/s is generally required. However, a dam cut-off may be subject to larger strains, for example due to reservoir fluctuations, and higher hydraulic gradients than for a pollution control cut-off wall. The objective of the specification and compliance testing for dam cut-off walls therefore must be to produce a barrier that has the required low permeability at the relevant hydraulic gradient, that will not crack and in the event of leakage will not erode.

The use of cement in a material can provide good resistance to erosion but fundamentally cemented materials tend to be stiff, hard and brittle only exhibiting ductile properties under drained high effective stress conditions. The strength and stiffness of typical cement-bentonite slag mixes will be significantly greater than that of puddle clay which will generally have an unconfined compressive strength of no greater than 100kPa. In contrast the final cured unconfined compressive strength of cement bentonite cut-off mixes is typically in the range 200-800kPa with a corresponding stiffness in the range of 200-300MPa. A principal concern is that movements of the dam associated with reservoir fluctuations will lead to cracking of the cut-off wall and subsequent erosion.

Typically, a performance specification for the set properties of a slurry used to repair a dam will include controls on permeability, strength and possibly strain at failure. However, like other cement based materials, the properties of cement-bentonite cut-off materials develop with time. It is therefore important to specify the age at which the compliance testing should be carried out. Table 3 provides examples of specifications that have been used for repair of dams.

Table 3. Range of parameters specified for set slurry

	Permeability		Strength		Strain at failure		
	m/s	Age days	UCS kPa	Age days	Strain %	Age days	σ_3' kPa
Site 1	$<1 \times 10^{-8}$	28	80 to 220	NS	>3	90	150
Site 2	$<1 \times 10^{-8}$	NS	150	28	NS		
Site 3	$<5 \times 10^{-8}$	28	100 to 200	NS	2	28	100

σ_3', effective confining pressure
NS, not specified

In addition to the above, durability and erodibility also need to be considered.

Permeability

A laboratory measured permeability of less than 1×10^{-9} m/s can be achieved after 90 days with cement-bentonite mixes with high levels of slag replacement in which the bentonite is hydrated prior to use. The most significant reduction in permeability due to ageing of the sample generally occurs in the first 90 days as seen in Fig. 3. However, duration of the permeability test also has a significant effect. Although 48 hours is specified for the permeation time in ICE (1999), a significant reduction in permeability can occur after 48 hours of permeation

Increasing the effective confining pressure only causes a significant decrease in permeability when the sample is relatively plastic as in a high bentonite, low cement mix or when the sample is tested at a relatively young age, eg at 14 days. The recommended effective confining pressure of 100kPa (ICE, 1999) provides a standard but does not generally replicate the in-situ confining pressure. Increasing hydraulic gradient from 10 to 100 increases permeability by a small amount and could increase the risk of leakage between the sample and the membrane.

Techniques for measuring insitu permeability of slurry walls have been reviewed by Tedd et al (1995). Falling head permeability tests using the BRE packer system placed inside a small hole drilled vertically in the middle of the cut-off wall have provided satisfactory results at a number of sites. Ensuring that the hole is central within the wall becomes increasingly difficult with depth.

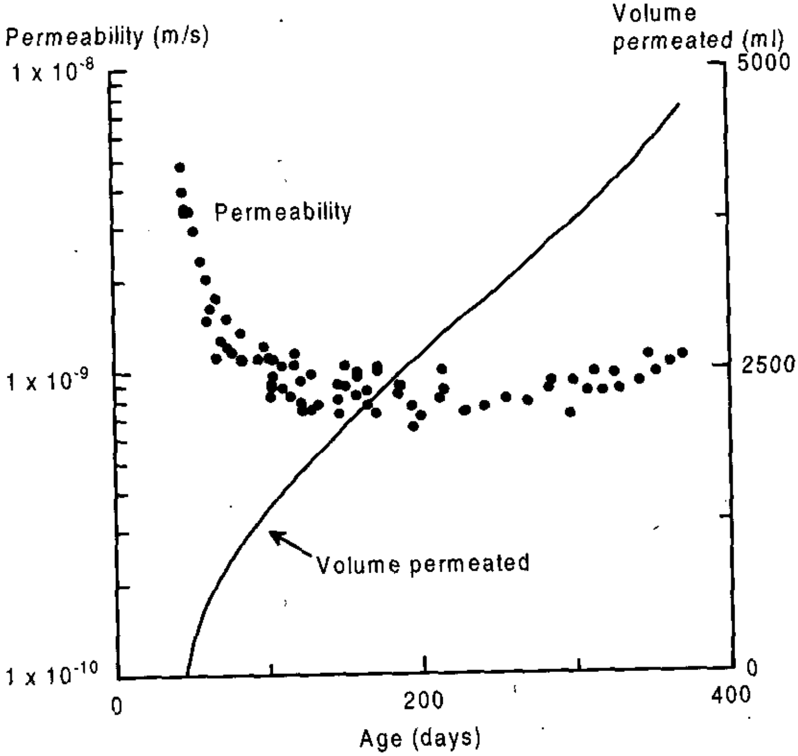

Fig. 3. Variation in laboratory measured permeability with sample age and permeation time

Strength

Unconfined compression tests are commonly specified to provide a relatively quick and cheap method of measuring the consistency of the strength of the material at specified ages. Large variations in UCS generally occur irrespective of sample age although there is trend of increase in strength with age. Analysis of data from a number of sites shows that UCS values at 90 days vary between 200 and 800kPa. Despite the large variation in measured UCS values, there is little variation in the axial strain at failure with values being typically between 0.5% and 1.5%

Deformation properties - strain at failure

It had been the practice in the UK to specify that the hardened cement-bentonite should have an axial strain at failure greater than 5% in a consolidated drained triaxial compression test because of a perceived need for a deformable plastic cut-off wall that will not crack and leak if subjected to movement. The validity of the strain at failure criterion can be questioned and ICE (1999) has deliberately omitted it for pollution containment.

The stress-strain behaviour of cement-bentonite depends upon mix design, sample age, type of test, effective confining pressures, rate of strain and drainage conditions. In a triaxial test the effective confining pressure has a major influence.

Figure 4 shows the typical effect of a range of effective confining pressures from 50kPa to 200kPa. The strain rate for these tests was 0.15% per hour. The strain at failure (defined by maximum deviator stress) increased with increasing effective confining pressure. The initial part of the stress-strain curve is characterised by a reasonably linear section to approximately 0.5 to 1% with modulus values in the region of 50 to 70 MPa at 28 days and 200 to 300 MPa at 90 days. This is followed by plastic strain hardening deformation. At low effective confining pressures, 50 kPa, brittle behaviour soon develops with failure occurring at less than 5% strain and the sample failing along a single shear. At higher effective confining pressures, 200kPa, plastic or "ductile" type behaviour occurs. With increasing age, the material becomes stiffer over the initial part of the loading from 28 to 90 days, the stress at which plastic strain hardening commences increases and the strain at failure tends to become lower. Increasing the cementitious content and the ratio of slag to cement make the set slurry more brittle such that it will be stronger but will fail at a lower strain.

The loading conditions in the triaxial test do not represent the loading that is likely to occur in-situ. If a strain at failure criterion is included in the performance specification, it is important to specify the effective confining pressure and drainage conditions and to be aware of what can realistically be obtained if the mix has been primarily designed for low permeability (less than 1×10^{-9} m/s at 90 days). Generally, a low permeability slurry will be more brittle. If the wall is likely to undergo deformation when its stiffness has been fully developed, tests at 28 days will not provide the relevant data

28 days

90 days

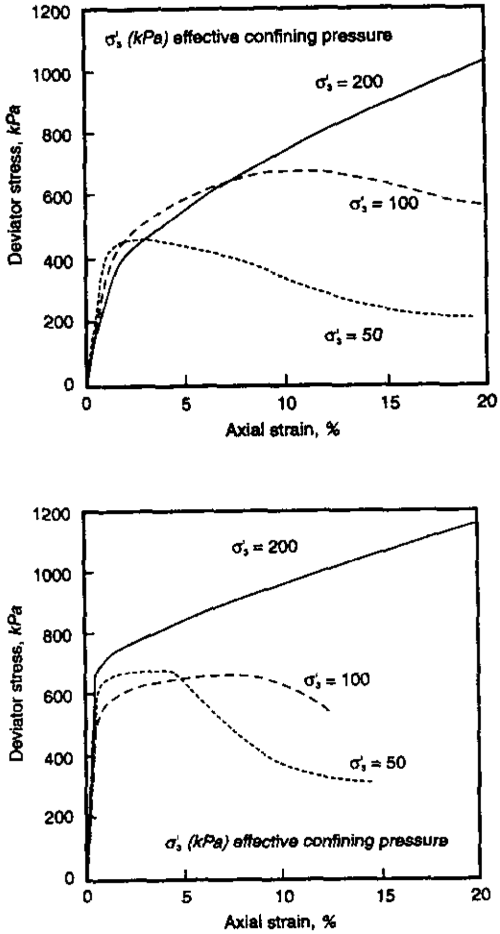

Fig. 4. The effect of effective confining pressure on the stress-strain behaviour and strain at failure in a drained triaxial test at 28 days and 90 days.

Durability:

Durability needs to be considered in relation to the reservoir water. At Withens Clough the plastic concrete was designed to resist the low pH water. In the long term, water is potentially one of the most aggressive agents for any cement based material as it can leach the more soluble species, particularly free lime, causing degradation of the calcium silicate hydrates. However, under confined conditions this leaching is not damaging to permeability which may reduce by two orders of magnitude.

Drying

As the set slurry typically contains approximately 80% water (it has a moisture content of the order of 400%) it is prone to drying shrinkage and cracking. A sample left on the laboratory bench will lose water, shrink and after a few days crack and eventually break down to a white crumble. It is therefore important to stop the top of the wall from drying out by protecting it with a clay cap as specified in ICE (1999). There is no evidence from BRE studies and others that cut-off walls that have been properly protected will dry out and crack in the British climate. Exhumation of a number of walls down to 5m depth has shown no signs of deterioration.

Erodibility

The set slurry should resist erosion if a crack is formed across the wall. As far as is known no consideration has been given to the erosion resistance of either plastic concrete or cement-bentonite when specifying the material for use in the construction or repair of a dam.

Erosion resistance of the set slurry needs to be assessed for the unlikely event that a cut-off wall does crack. Drilling into set slurry re-slurrifies the material, giving the impression that it may be susceptible to erosion by water. Beier & Strobl (1989) describe some long term erosion tests using various mixes. Samples of cement-bentonite slurry from Monkswood dam (Penman et al, 2000) were cast in U100 tubes and some months later 100mm lengths of the sample were place in a modified pinhole erosion type apparatus developed at BRE and water with a hydraulic gradient of 20 was passed through a 3mm diameter hole drilled longitudinally through the sample. After 3 months no noticeable erosion of the sample had taken place.

CONCLUSIONS

Slurry trench walls provide an effective means of forming a cut-off in the foundations of dams and for repairing defects in the cores of embankment dams. Slurry walls have the advantage over grouting that there is greater certainty that all defects in the core will be treated. Despite this, their use in remedial works has been much more restricted than the use of grouting.

When developing specifications for cement-bentonite material it is important to recognise that their properties develop with time, at least up to 90 days and that early age testing may not reliably predict later age performance.

The durability of the set slurry needs to be considered where there is the possibility of aggressive ground water. The in-situ effects of long term leaching of the calcium ions from a cement-bentonite are unknown although the time scale for such leaching can be predicted. A set slurry using a typical slag, cement, bentonite mix is less erodible than clay.

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Erratum

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Prediction of downstream destruction following dam failure: no quick solution

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In the processing of this book, the following text was omitted in error from the end of this paper. The publishers apologise to the readers and authors.

downstream of a dam failure in a steep and narrow valley will be greater than that from a similar dam in a wide and flat valley. The parameter of valley shape needs to be considered for each individual reservoir when estimating potential extent of damage downstream.

Another important consideration in predicting the extent of downstream damage is the failure mechanism and resulting breach characteristics. Dimensions such as breach width, depth, and time to maximum size vary depending on the mode of failure, whether overtopping or piping or a different mechanism. The breach dimensions affect the peak outflow from the breach and therefore the velocities and depths of flow downstream of the dam which influence the type and extent of damage.

CONCLUSIONS

The results show that, whichever of the three parameters is used, there is only a tenuous relationship between extent of inundation or damage and any of the parameters. The most consistent results are for dam height. There is some logic in this as dam height is indicative of both the total volume released and the size of breach which dictates the peak flow.

Use of a relationship relating extent of damage to a single parameter such as dam height may be adequate for assisting in determination of the necessary downstream limit of a dam break model. However, the examples given in this paper demonstrate that the skill and experience of a dam break modeller is equally if not more important for this determination. There may be a complex numerical solution relating extent of damage to three or more parameters, but even this would not be suitable for hazard management or contingency planning.

The results of the research confirm that the extent of damage and inundation in valleys downstream of dams can only be made by a full dam break assessment. The use of simple formulae or set distances in determining a hazard classification for a major dam is not appropriate.

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