BNCOLD

1984 CONFERENCE

University of Cardiff
September 1984

Technical Papers

CONTENTS

Published by The British National Committee on Large Dams, at The Institution of Civil Engineers, Great George Street, London, SW1P 3AA

Printed in England by The Chameleon Press Ltd, 5-25 Burr Road, Wandsworth, London SW18 4SG.

BNCOLD LECTURE

Insidious Threats to Dams and Reservoirs

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SYNOPSIS

Attention is particularly directed towards dams and reservoirs constructed before about 1950 in the United Kingdom. The basis of this watershed is that, before them, very few experienced engineers received any formal geological training. Additionally, the then new science of soil mechanics had been absorbed by only a few. Apart from a series in the Engineering News Record and some available papers from the Boston Society, Terzaghi's concepts could only be read in German. In this country the Building Research Station deserves great credit in that context.

Given the then rather pragmatic methods of estimating floods as well, it would be fair to say that several of these older dams contained features unsuspected by either their owners or inspectors.

More recent searches in some evidently anomalous cases have yielded very disconcerting mission historical material.

In parts of South Wales there is a near unique congregation of hazards.

Firstly, the coal basin, though deeply incised, is of relatively low relief but somehow sustains long average annual rainfalls of up to 2,600mm per year.

Secondly, almost all valley sides have been over-steepened by ice and are collapsing either rapidly or slowly, in addition, in some areas mineral workings have left no natural land surface at all.

Thirdly, underground aquifer:aqueclude conjunctions are both basically intractable and further modified by gross undermining and faulting.

In such an environment, the safe disposal of floods may be the least of the problems for a dam engineer and a discussion of definitions of risk follows.

The Estimation of Seasonal Probable Maximum Flood

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Northumbrian Water

Probable maximum flood (PMF) estimation described in the ICE Guide on Floods and Reservoir Safety incorporates a method of deriving maximum precipitation separately for winter and summer seasons. In this paper the seasonality of other components of the PMF and effects on design flood estimates are considered - antecedent wetness, percentage runoff, snowmelt rate, initial snow cover and initial reservoir condition.

Investigations suggest that present methods result in exceedence probabilities of each component which differ considerably between seasons and catchments. The consequences of equalising the probabilities have been examined and some suggestions are made on the use of local data to produce estimates of more consistent severity between catchments and seasons. The role of snow in winter PMF estimation has been reassessed.

INTRODUCTION

The hydrometeorological approach to the assessment of maximum flood potentials as described in the Flood Studies Report(1) and developed in the ICE Guide(2) has become accepted practice for reservoir spillway design and evaluation in Britain. The term used for the computed flood, the probable maximum flood (PMF) is defined (2) as 'the flood hydrograph resulting from probable maximum precipitation (PMP) and where applicable snowmelt, coupled with the worst flood producing catchment conditions that can realistically be expected in the prevailing meteorological conditions'.

The design storm rainfall is intended to be the theoretical maximum for given location, duration and season. However the definition of associated conditions as 'worst ..... realistically expected' leaves scope for conditions, which, though severe are not necessarily maximised.

In the case where the design discharge hydrograph is computed entirely from catchment and climatic characteristics, precise methods of defining the associated conditions are described. However except for the occurrence of wind waves, the probability of these conditions is not specifically defined. Investigations in northeast England suggest that the derived conditions may differ widely between catchments and seasons in the likelihood of their occurrence. Where data for the catchment or adjacent catchments are available, or where the engineer or hydrologist must consider the effect of season or reservoir operating rules, difficulty may arise in the interpretation of the expected associated conditions. Recommendations of the ICE Guide give scope for judgement by the engineer but in some instances provide insufficient criteria on which to base this judgement.

Apart from the primary need to achieve the required level of safety, it is believed the method of estimating flood discharges for reservoir spillway design and evaluation should have the following properties.

1 The controlling meteorological and catchment conditions should result in a design flood which is of broadly similar severity between catchments and between winter and summer events. It is important to provide equitable treatment especially to dam owners.

2 There should be objective means of applying local data to the design storm.

3 Methods should be sufficiently flexible to allow for the modification of design criteria in the light of improved understanding of components, without the necessity of a full reconsideration of feasible combinations with each advance in knowledge .

Newton[3] in reviewing the range of procedures available for estimation of spillway design floods in the United States suggests that although it is not possible to define the probability of a PMF, a broad assessment of individual probabilities can constructively be used to guide the selection of associated conditions. Consideration of component probabilities in PMF estimation may help to meet the criteria above.

The main components in PMF determination are the principal storm rainfall, amount and time distribution, antecedent rainfall, initial moisture conditions and percentage runoff, initial snow cover and rates of snowmelt and initial reservoir conditions.

PRINCIPAL AND ANTECEDENT STORM

Principal Storm Rainfall Amount

No analyses have been carried out here of the principal storm rainfall totals. However it is worth noting the divergent methods used to obtain seasonal rainfall totals (Summer is defined as May - October; winter, November - April).

The method used to determine the maximum precipitation[4] involved the combination of a national maximum storm efficiency with a regional maximum dewpoint and subsequent adjustment by reference to maps of precipitable water of 5 year return period and maps of estimated maxima obtained by statistical procedures. The storm thus obtained is essentially a summer event.

However subdivision into summer and winter rainfall amounts for given durations (ICE Guide Table 2), is on the basis that the expectation that the seasonal maxima are in the same ratio as the 100 year values. It is desirable that rainfall estimates for summer and winter events should be obtained by a common method or at least that the winter estimates obtained as a percentage of the all year value should be shown to be consistent with a value calculated by the physically based procedure.

Storm Profile

The probable maximum precipitation (PMP) storm profile is chosen such that the estimated maximum fall occurs in every duration centred at the peak of the storm profile. Determination of the seasonal PMP profile is performed from the storm centre outward so that the ratio of seasonal to all year rainfall is preserved for all durations.

The seasonal PMP profile thus produced may be compared with the seasonal percentage profiles illustrated in FSR II.6.3. (Fig 1). Seasonal PMP profiles were determined for two catchments, Harwood Beck, a wet moorland Pennine catchment and the River Skerne at Bradbury, a dry lowland catchment in south east Durham. The centre of both summer PMP profiles lies close to

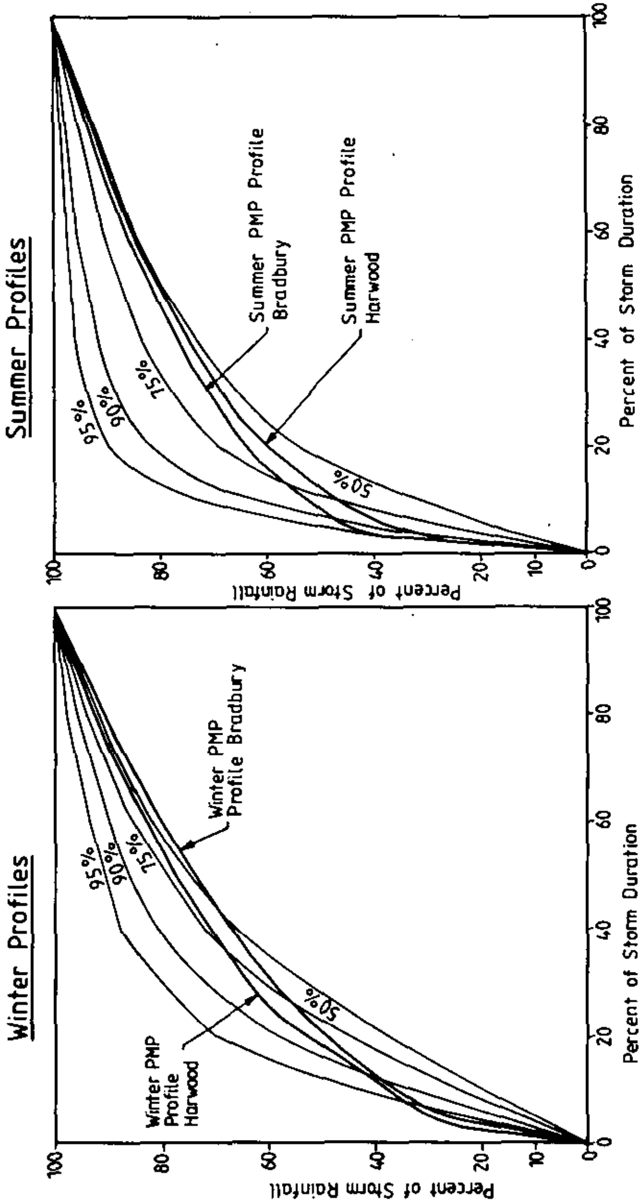

Fig 1 Comparison of seasonal probable maximum precipitation profile for Harwood Beck and the River Skerne at Bradbury, with seasonal percentile peakedness profiles

the 95 percentile of summer profile peakedness, but beyond the central 5% of duration, the severity of peakedness diminishes and approaches the 50% summer profile beyond 25% storm duration. The winter profiles have a similar pattern but with a comparatively more extreme central peakedness (well above the 95% winter profile). The upland catchment has a relatively more peaked winter profile and the lowland one is more peaked in summer.

However, there is sufficient similarity between the seasonal PMP profiles and their respective seasonal percentile profiles to give confidence that no undue seasonal or regional bias will arise from this source.

Antecedent Rainfall

In the calculation of antecedent rainfall the assumption is made that the D hour maximum storm occurs in the middle of the 5D hour maximum storm. Thus the prior 2D hour rainfall is equivalent to half the difference between the 5D and the D hour maxima. Snowmelt is added to the winter event.

Unlike several American PMF estimation systems, the antecedent storm is not compounded with the main storm for catchment or reservoir routing. It therefore plays no part in the initial inflow discharge to the reservoir nor to the initial reservoir level at the commencement of the principal storm. The latter is separately specified (ICE Guide Table 1). Antecedent precipitation is used solely as a component in the computation of percentage runoff in the main storm event.

PERCENTAGE RUNOFF AND INITIAL MOISTURE CONDITIONS

Flood estimates whether for maximum events or for events of specified return period, are extremely sensitive to the choice of percentage runoff

(PR). Percentage runoff is calculated as:

$$PR = SPR + 0.22\ (CWI - 125) + 0.1\ (P-10.0) \tag{1}$$

Where P is the principal storm rainfall, and

CWI is the catchment wetness index

SPR is the standard percentage runoff - a fixed value for the catchment calculated by:

$$SPR = 95.5\ SOIL + 0.12\ URB \tag{2}$$

Where SOIL is the mapped Flood Studies index of winter rainfall acceptance potential and

URB is the urban percentage of the catchment area

For the design PMF the CWI is assumed to be 125 (soil moisture deficit = 0) at the beginning of the antecedent storm for both summer and winter events, and at the beginning of the principal storm is obtained by:

$$CWI = 125 + Pa + 0.5^{D/24} \tag{3}$$

Where Pa is the antecedent precipitation.

Comparison of Design and Observed Percentage Runoff

It seems reasonable to assume that the relative frequency of the percentage runoff as calculated by this sequence of formulae should be similar on different catchments and in different seasons. The hypothesis was tested, again using the Harwood Beck and Skerne catchments.

For comparison, percentage runoff was determined from the gauged record for all rainfall events over a threshold for the entire period of runoff record. To simplify extraction, PR over two days of record was obtained from tabulated daily runoff volumes and daily rainfall. Comparison with the PR obtained by conventional hydrograph separation showed close similarity except on few occasions when rainfall was marginally distributed with the 2 days. On the Harwood Beck 147 events with 1-day rainfall > 30mm, in 14 years, and on the Skerne 79 events > 25mm, in 10 years, were examined; snowmelt events were excluded.

At Harwood the estimated PMF percentage runoff was a common occurrence with 9 of 65 summer events and 21 of 82 winter events exceeding the respective estimates. On the Skerne in contrast no measured percentage runoff approached the estimated PR in either season, the highest recorded value being 50%. Thus the relative frequency of the estimated PR where unadjusted for gauged data, may differ considerably between catchments, resulting in flood estimates of unequal severity.

Adjustment of Standard Percentage Runoff

A method of using gauged data to adjust PR, by refining the fixed standard percentage runoff term (SPR) is described in Flood Studies Supplementary Report No 13.[5] Equation 1 is reversed to give:

$$SPR_{OBS} = PR_{OBS} - 0.22\ (CWI - 125) - 0.1\ (P-10) \qquad (4)$$

and for the observed event

$$CWI = 125 + API5 - SMD \qquad (5)$$

where API5 is the antecedent precipitation index taken over a period 5 days before the storm, and,

SMD is the initial soil moisture deficit.

Application of this procedure to the observed events in the two catchments resulted in an annual average value of SPR of 62.0 at Harwood and 23.2 at Bradbury. These were then used to determine a new design value of PR from which a new PMF discharge was determined (Table 1.2).

There is a substantial range in the computed values of SPR and subdivision of the year gave significantly different seasonal values which were again used to compute a PMF discharge (Table 1.3). Comparison of these revised discharges with those determined strictly by the Guide procedures shows that at Harwood, the seasonal floods are reversed in order of magnitude but both are higher. At Bradbury the summer event is still the higher, but the flood discharges are much reduced.

Adjustment of Initial Wetness

In the original all year PMF outlined in the Flood Studies Report(1) it was assumed that on all catchments the soil was at field capacity (CWI = 125; SMD = 0) at the commencement of the antecedent rainfall. Although the frequency of occurrence of such a condition varies both with location and season, no seasonal alteration was suggested in the Guide(2).

The annual probability of zero SMD before storm events was calculated for the two catchments and was found to be 23% at Bradbury and 61% at Harwood. The equivalent SMD for the same percentile probabilities was determined on a seasonal basis. On both catchments this gave zero SMD for the winter event but values of 40 and 35 mm at Bradbury and Harwood respectively in the summer season.

TABLE 1 Seasonal PMF flood estimates for Harwood Beck and the River Skerne at Bradbury and the effects of adjustments to percentage runoff and snow parameters.

1 Strictly FSR and ICE Guide procedure from catchment characteristics

CATCHMENT	SEASON	SOIL	SPR %	PA mm	CWI	P mm	PR %	Q PEAK m^3/sec
Harwood Beck	S	.500	47.7	59.2	171.0	213.8	78.2	310.3
SAAR = 1720.5	W	.500	47.7	95.5	193.2	217.7	84.8	296.4
Skerne								
(Bradbury)	S	.450	43.3	37.6	148.0	238.7	70.9	263.8
SAAR 691.0	W	.450	43.3	145.6	214.1	171.4	78.7	186.0

2 Percentage runoff calculated from annual average SPR derived from gauged data

Harwood Beck	S	(.65)	62.0	59.2	171.0	213.8	92.5	366.6
	W	(.65)	62.0	95.5	199.2	217.7	99.1	345.9
Bradbury	S	(.24)	23.1	37.6	148.0	238.7	51.0	190.3
	W	(.24)	23.1	145.6	214.1	171.4	58.9	140.2

3 Percentage runoff from seasonal average SPR derived from gauged data

Harwood Beck	S	(.58)	55.5	59.2	171.0	213.8	86.0	340.9
	W	(.70)	66.8	95.5	199.2	217.7	103.9(100.)	348.7
Bradbury	S	(.21)	20.4	37.6	148.0	238.7	48.3	180.4
	W	(.28)	27.2	145.6	214.1	171.4	63.0	149.7

4 Percentage runoff with adjusted SPR and seasonally adjusted initial wetness

Harwood Beck	S	(.58)	55.5	59.2	136.0	213.8	78.3	310.7
	W	(.70)	66.8	95.5	199.2	217:7	103.9(100.)	348.7
Bradbury	S	(.21)	20.4	37.6	108.0	238.7	39.5	147.6
	W	(.28)	27.2	145.6	214.1	171.4	63.0	149.7

5 Snowmelt profile with 5.0mm per hour peak rate - otherwise ICE Guide procedures

Harwood Beck	W	.50	47.7	95.5	199.2	238.8	88.9	328.0
Bradbury	W	.45	43.3	145.6	214.1	212.0	86.4	246.4

6 Snowmelt profile with 5.0mm per hour peak rate - percentage runoff adjusted as in 4

Harwood Beck	W	(.70)	66.8	95.5	199.2	238.8	108.0(100.)	369.0
Bradbury	W	(.28)	27.2	145.6	214.1	212.0	70.7	201.6

The revised seasonal initial wetness conditions have been applied to the determination of percentage runoff in Equation 1 in conjunction with SPR adjusted as above. The resulting PMF estimates are shown on Table 1/4. No attempt at regional adjustment has been attempted here.

Conclusions and Recommendations on Percentage Runoff

Percentage runoff revised by using observed data on SPR and initial wetness still shows some variability in frequency of occurrence between catchments and seasons. At Harwood, 9 of 65 summer events and 2 of 82 winter events exceed the estimates whereas at Bradbury the new design values still exceed all observed values in both seasons. It seems that the generalised equations are unable to represent fully, the complex and varying relationships between initial wetness, storm rainfall and percentage runoff on a range of catchments.

For gauged catchments it would therefore be advantageous to express the design PR as a function of measured PR or its probability of occurrence rather than indirectly through a synthesised antecedent rainfall, assumed initial wetness and generalised rainfall-runoff relationship. This requires further investigation. However the following adjustments may be applied to improve consistency of estimates based on observed information.

1 Standard percentage runoff (SPR) may be determined from gauged data as described,[5] but separately for summer and winter events.

2 Separate summer and winter initial soil moisture capacity may be determined based on the annual frequency of occurrence of zero SMD and the associated seasonal SMD at this frequency.

SNOWMELT

Nowhere is the problem of maintaining consistency between seasons and locations seen more clearly than in estimation of the snowmelt contribution to PMF. Two divergent views are possible. On the one hand, snowmelt might be considered a separate contribution whose occurrence has a distribution independent of rainfall. In this case in order to maintain consistency with summer estimates, a median value of snowmelt might be assumed in conjunction with maximised rainfall. At most locations this value would be zero. Alternatively snowmelt might be considered an integral part of the precipitation, whose components are separately maximised and then combined.

In the Flood Studies Report(1) an intermediate approach was adopted. A value of 42mm/day (1.75mm/hr) was chosen as a suitably rare occurrence to be added as a constant rate to the maximised rainfall. In the first instance it was suggested (FSR I.6.8.3) that this contribution be combined with the all-year rainfall - an essentially summer event. However with the provision of separate seasonal maximum rainfall profiles in the ICE Guide,(2) it was recommended that the same constant snowmelt rate be compounded with the winter event only. The choice of this rate was based on the work of Jackson(6), who assigned to this snowmelt rate a return period of the order of 50 years in lowland Britain and rather less at higher altitudes.

Work in northeast England(7, 8) on the assessment of rates of snowmelt runoff and on modelling the snowmelt runoff process, shows that this snowmelt rate, either as a peak rate or as a daily average, is a comparatively common occurrence in some upland areas. It was exceeded in more than one event in a single winter (1979) on several catchments, including Harwood.

In addition the simulated peak rates of release at the base of the snow are substantially greater than peak melt rates, demonstrating the importance of liquid water retention in the snowpack. It is the rate of yield at the base of the pack which should be used in combination with the design rainfall. Flood experience in the United States,[9] has also shown the need to re-examine the snowmelt contribution and the augmentation of yield by snowpack storage.

Although insufficient work has been done to assess snowmelt rates of specified return period reliably, maximum values over a limited number of years suggest that a 50 year return period point yield of 5mm/hour would be appropriate in most parts of the country. However a constant input at this rate is unrealistic. In an extreme event with heavy rainfall on a deep melting snowpack, the first part of the storm is stored temporarily in the pack. Then over a progressively increasing area yield occurs at the base of the pack until the whole catchment is contributing; this continues until the pack is depleted.

To reflect this pattern of flood development, the following modification to winter PMF estimation is suggested. The antecedent snowmelt input is assigned a rate of 1.75 mm/hour as before. Then the rate of melt yield increases linearly from the beginning of the main storm to a maximum of 5mm/hour to coincide with the peak of the storm rainfall, melt continues at this higher rate for the remaining duration of the storm unless the snow cover is depleted. The effect of this modification on the resulting winter flood estimate, with and without associated adjustments to PR estimation is shown in Table 1.5 and 1.6.

SNOW COVER

At a design snowmelt rate of 42 mm/day, melt can be maintained throughout the design event except on a few large catchments. With a larger assumed melt rate, this is no longer the case, and the assessment of the initial snow cover assumes a greater significance in design.

The initial snow water equivalent of 100 year return period was recommended (FSR I.6.8.3). The estimate of water equivalent is based on the 100 year return period snow depth and an assumed density of 0.13 g/cm^3. Jackson[10] describes the method of estimating water equivalents of specified return period at a given locality and altitude. For example this procedure yields estimates as follows:

At 400 m in northeast England	130mm
At 800 m in northeast Scotland	285mm

Measurements in the Pennines suggest that these are probably underestimates. At one site in the Harwood catchment in 1979, undrifted snow exceeded 200mm on two separate occasions with virtually complete melt intervening[7]. The high snow cover volumes are confirmed by high runoff rates.

Investigations by Ferguson[11] of snowmelt runoff in the Feshie basin in the Cairngorms also yielded very high volumes as shown in Table 2.

Table 2 Snowmelt Runoff in the Feshie catchment

Year	Period	Rainfall (mm)	Runoff (mm)
1979	10.4. - 1.6	93	439
1980	25.3. - 16.5	24	236

Associated snow densities are commonly greater than 0.30 g/cm^3 before the main melt period commences[7] and the assumption of the lower rate of 0.13 g/cm^3 underestimates the water equivalent of a prolonged snow cover. In the light of this evidence it seems reasonable to increase by a factor of 2 the previous estimated value of initial snow water equivalent for design.

Snowmelt Flooding An Illustrative example

Severe flooding from snowmelt occurred on many rivers in northeast England in early March 1963, including the Yorkshire Ouse and the Wansbeck and Coquet in Northumberland. The flood on the River Wansbeck at Morpeth (catchment area 287 km^2) was the highest this century. The town centre was flooded and 460 houses plus industrial premises and shops were inundated. On the River Coquet the level at Rothbury was about 1 metre higher than any event recorded since that date. The flood also caused the complete failure by overtopping of an earth embankment and the draining of about 80 t.c.m.. of water impounded in Trewitt Lake, in a tributary of the Coquet. The dam which is approximately 5m high and 80m long now displays a breach 12m wide. This event occurred virtually without accompanying rainfall. The highest measured daily rainfall total on the Wansbeck catchment was 4mm and on the Coquet 9mm.

Initial Reservoir Conditions

Recommendations were made in the ICE Guide[2] on the initial reservoir levels to be adopted for routing floods through reservoirs in conjunction with design flood inflow. These levels are by no means extreme. For high risk dams an initial level giving spill equivalent to the long term average inflow was recommended but for most lower risk categories a just full condition was specified. These conditions apply to both summer and winter events with the reservation that 'where reservoir control procedure requires and discharge capacities permit, operation at or below specified levels defined throughout the year be adopted'.

The seasonal distribution of inflow and reservoir operating policy normally imply that maximum levels occur in later winter and levels decline through the summer. Seasonal probabilities of the design initial levels therefore differ. The probability of the reservoir level being at or above spillway level was determined for seven reservoirs in northeast England, for the full year and separately for summer and winter periods (Table 3).

Table 3 Annual and seasonal probability of reservoir fullness

Reservoir	% of time reservoir full			% of volume exceeded 20% of time	
	Annual	Winter	Summer	Winter	Summer
A	36.4	51.5	21.2	100	100
B	6.2	10.5	1.9	98	71
C	18.6	28.5	8.5	100	99
D	28.5	49.5	7.6	100	96
E	32.9	53.2	12.7	100	99
F	9.1	16.6	1.5	99	90
G	14.4	27.2	1.5	100	93
Average	20.9	33.9	7.8		

As expected the results show variation both between seasons and between reservoirs. If the average annual percentage of time the reservoir is full (c20%) is adopted, the equivalent seasonal level and volume for each reservoir may be determined and expressed as a percentage of the full volume (Table 3).

The effects of assuming typical levels of initial drawdown or overflow on computed peak outflow discharge was investigated on several reservoirs. The initial reservoir condition has a varying effect depending on reservoir outflow characteristics, and storage in relation to inflow volume. On some reservoirs only a small drawdown gave a substantial reduction in peak inflow discharge; for example in one case a drawdown of 5% volume reduced outflow peak by 20%. On the other hand there were no significant differences in peak between the condition of reservoir just full and spilling long term average inflow.

In some instances therefore the specification of a fixed exceedence probability of 20% might result in a much reduced outflow peak especially in summer. This is unacceptable in view of the comparatively common occurrence of this initial level; less extreme value of other components in combination with a full reservoir could result in a much higher discharge.

The choice of a less frequent initial reservoir level would overcome this difficulty but would in the majority of cases result in a design discharge little different from that given by the current conditions.[(1)] It is therefore considered appropriate that these simply derived initial conditions continue to be applied.

CONCLUSIONS

The frequency of occurrence of climatic and catchment components of Probable Maximum Flood estimates as derived by present methods, varies considerably between catchments and seasons, and may lead to flood estimates which vary in their relative severity. The specification of component probabilities should in most instances, permit a more consistent approach and should allow local data to be applied more objectively. Specific suggestions for the use of observed information on antecedent soil moisture conditions and percentage runoff have been made.

It was the intention of the Flood Studies Report to incorporate a snowmelt rate of some rarity. Analysis of events mainly in northeast England indicates that the recommended rate of 1.75mm/hour occurs quite frequently. It is suggested that, to preserve the intended severity the snowmelt contribution to winter events should be revised. A melt profile with a peak rate of 5mm/hour is suggested. In this instance the specification of a nominal return period of melt would enable future findings to be incorporated without the necessity of reconsidering all feasible combinations.

Seasonality has been considered here in terms of two fixed seasons. In reality the probabilities of each component vary continuously through the year. Future study of critical combinations through the year may lead to more realistic estimates.

ACKNOWLEDGEMENTS

This paper is published with the permission of the Director of Operations and Works, Northumbrian Water Authority.

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Floods and Spillways of the Mendip Supply Reservoirs of the Bristol Waterworks Company

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SYNOPSIS

Bristol Waterworks Company owns 14 reservoirs which come within the ambit of the Reservoirs (Safety Provisions) Act 1930, of these 8 are impounding reservoirs with some dating back to 1850. Towards the end of 1980 the Company embarked upon a programme for checking the capacities of the overflow weirs and spillway channels of its reservoirs in accordance with the Flood Studies Report[1] and the ICE Guide: Floods and Reservoir Safety[2]. This has revealed that seven of the reservoirs, in particular the older ones, are not capable of meeting the new criteria and as a result works have been undertaken or are envisaged for their enlargement even though they coped with the testing storms of 10th July 1968.

INTRODUCTION & HISTORY

The Company was incorporated by Act of Parliament in 1846 and soon afterwards its first major scheme was commissioned. This involved the construction of an aqueduct, known as the "Line of Works", see Fig. 1, to convey spring water from the northern side of the Mendips to the City of Bristol via a storage reservoir at Barrow. As part of this scheme the Company was required to supply compensation water in the valley of the River Chew by construction of reservoirs at Chew Magna and Litton.[3 & 4.]

Under the terms of the 1862 Act the Company was empowered to construct a second storage reservoir and a compensation reservoir at Barrow and these were completed in 1864. A third storage reservoir was authorised under the terms of the 1882 Act and this was constructed during the period 1887-1899.

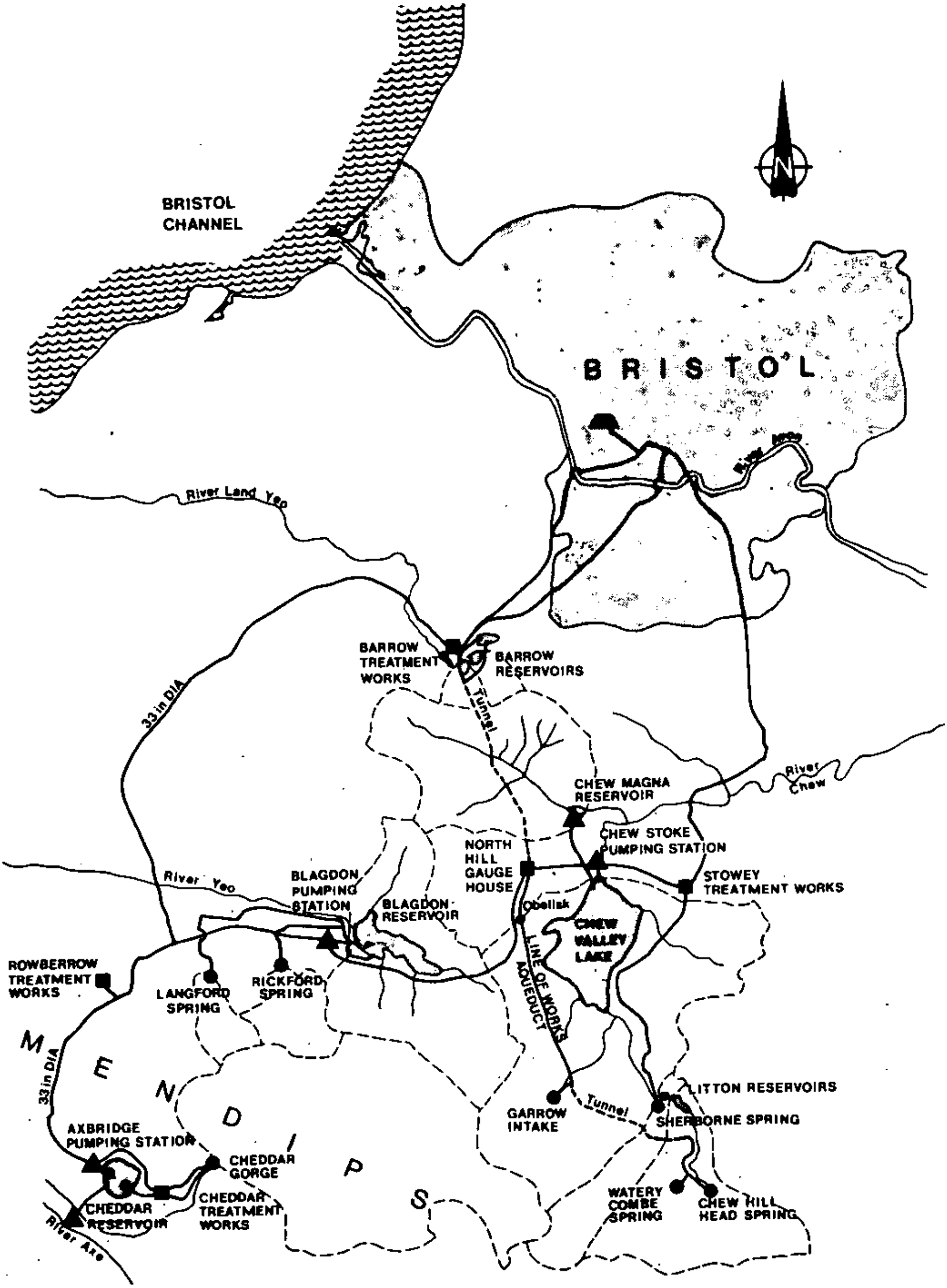

Fig.1 MENDIP SUPPLY RESERVOIRS OF BRISTOL WATERWORKS COMPANY

A further stage in the Company's Reservoir construction was reached when by the Acts of 1888 and 1889 powers were obtained for the construction of Blagdon Reservoir. This was completed during the period 1898-1901.

In 1920 powers were obtained for the abstraction of water from Cheddar Gorge[5] by the construction of an intake and pumping station with a 33 inch diameter rising main terminating in Blagdon Reservoir. By 1927 it had become necessary to obtain further legislative powers for the construction of a reservoir at Cheddar[6] to store the surplus flow from the Gorge. This reservoir was constructed during the period 1933-1937.

During the very dry winter of 1933-34 the idea of constructing a dam across the valley of the River Chew was re-examined in order to meet the ever increasing demand of the City of Bristol. The 1939 Act authorised the Company to construct Chew Stoke Reservoir[7] and various intakes on adjacent streams. However, the scheme was delayed due to the intervention of war and a temporary pumping station and intake were constructed on the River Chew as an interim measure in 1947. It was not until 1951, following a further delay while the scheme was considered on a regional basis, that construction of the dam commenced and this was completed in 1956.

THE STORMS AND FLOODS OF JULY 1968 ON MENDIP

It is now possible to look back at the storms and floods of July 1968 on Mendip as being of great national importance since their occurrence was in part responsible for the setting up of the NERC investigation that eventually resulted in 1975 in the publication of the Flood Studies Report.

A series of severe thunderstorms crossed the Mendips during the evening of Wednesday, 10th July 1968. The intensity of the precipitation, as well as the total amounts falling, gave rise to devastating floods throughout the region. The precipitation extended across the catchment areas of all the Company's Mendip reservoirs but the eye of the storm was centred on that of Chew Magna Reservoir.

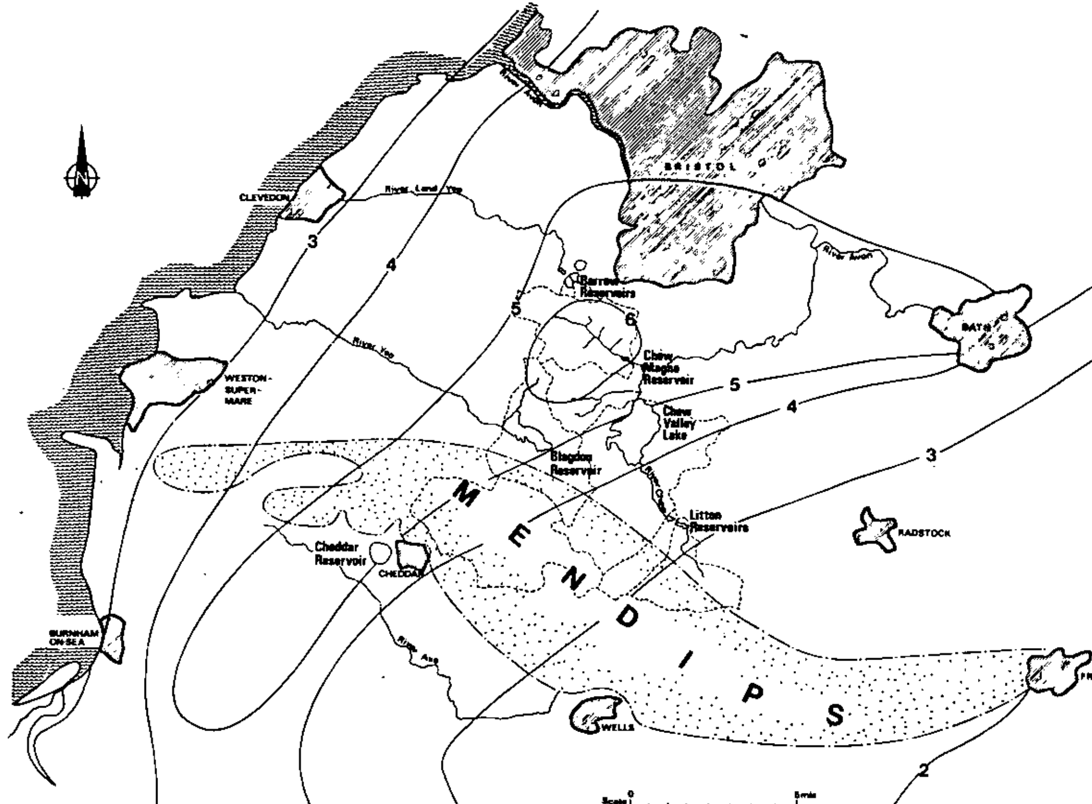

Fig.2 Distribution of rainfall in relation to reservoir catchment areas for the 24 hours preceeding 0900BST on 11th July 1968 (After Hanwell and Newson)

According to Hanwell and Newson [8] the processes leading to the violent storms could be attributed to the cells of humid air which rose rapidly by convection into the jetstream at 9000 metres. These thunderstorm cells became so concentrated that they formed localised cyclones which were unstable enough to be detonated by the apparently minor factor of the topography of the Mendips. As the deepening depression whirl approached the Mendips from the south-west various wind lanes were created up the gaps in the escarpment of the hills, particularly Cheddar Gorge and Shute Shelve. Such topographical funnelling assisted the lift and resulted in the heaviest rainfall occurring on the leeward side of the hills in the Chew Valley.

The Isohyetal map, Fig. 2, illustrates the distribution of rainfall and indicates that at the eye of the storm over 6½ inches of rain fell between 8 p.m. and 12 p.m. with intensities of up to 3 inches per hour. These parameters together with those in FSR have been evaluated to give an estimated return period of between 1 in 6000 and 1 in 7000 years, based on national growth curves.

CHEW MAGNA RESERVOIR

The reservoir is located on the Winford Brook, a tributary to the River Chew, immediately above the village of Chew Magna. The reserovir has therefore been placed in Category A, General Standard, and must be capable of passing the PMF.

During the storms on the night of 10th July 1968 the grass covered embankment was overtopped, evenly along its crest, to an estimated depth of 90mm. The discharge down the spillway has been calculated to be 40.5 cumecs whilst 2.8 cumecs discharged down the face of the embankment. The embankment was not breached but extensive erosion took place at the end of the spillway channel and a hole 4 metres deep developed in the floor of the stilling pond.

When the reservoir was constructed in 1846 the overflow weir cill was 19.43 metres long but this was subsequently extended to 22.86 metres in 1936 in response to an inspection under the 1930 Act. Following the floods of 1968 an auxiliary overflow, 30.48 metres long, was provided with a cill 300mm higher than the original cill and in addition the embankment was raised 1 metre in height.

Notwithstanding the previous modifications which have taken place, the flood routing procedure has shown the PMF peak inflow of 167 cumecs is only reduced to 163 cumecs and that the top standing water level will be just below emabnkment crest level. An additional allowance of 0.64 metres is therefore required for wave surcharge. The costs of these and other remedial works identified by the Inspecting Engineer, have been estimated to be in the order of £190,000. The option for demolition of the dam was also evaluated but since this appeared to be more costly it was rejected. Other factors which favoured retention of the reservoir were its partial flood attenuation role and its resource value, the reservoir acting as a header pond for pumping to Chew Valley Lake.

LITTON RESERVOIRS

The two reservoirs, which are in series, are located on the River Chew immediately above the hamlet of Coley and 3 kilometres upstream of Chew Valley Lake.

The cill length of the overflow weir on the Upper Reservoir was originally 7 metres long but this was increased to 10 metres when the embankment was raised in 1852. During 1936 the cill length was increased to a total of 29 metres following an inspection under the 1930 Act. Similarly, the Lower Reservoir cill length was increased, in 1936, from 8 metres to 31 metres.

As an aid to deciding in which categories the reservoirs should be placed, the Inspecting Engineer had a "dam break" study undertaken. This determined the degree of inundation that would take place to the three or four houses in Coley if the reservoirs were to fail. After due consideration he decided that both reservoirs should be placed in Category B, General Standard, and the spillways should be capable of passing a 0.5 PMF. The peak inflow during 0.5 PMF was calculated as 82 cumecs for the Upper Reservoir and 93 cumecs for the Lower Reservoir.

The configuration of the spillway channel at both reservoirs was considered likely to result in drowning of the overflow weirs and hydraulic analyses were performed at each reservoir to confirm the situtation. On the Upper Reservoir it was calculated that the spillway channel would be drowned out at 33 cumecs and the embankment overtopped at 50 cumecs whilst, on the Lower Reservoir, the figures would be 40 cumecs and 47 cumecs respectively.

Various options, including demolition and substantial reduction in volume as well as other options encompassing all the remedial works identified by the Inspecting Engineer, were investigated and costed for both reservoirs and it was decided that refurbishment of the reservoirs should proceed since it was the most financially attractive. The total project costs are likely to be in the order of £600,000 and it is anticipated the works will be completed in the Spring of 1984.

BARROW RESERVOIRS

The Barrow Nos. 1 and 2 Reservoirs and Barrow Compensation Reservoir are located on the headwaters of the River Land Yeo, see Fig. 3, immediately above the village of Barrow Gurney. Barrow No. 3 Reservoir however straddles the watershed between the River Land Yeo and Colliters Brook, the latter passing through the Bristol suburb of Ashton Gate before joining the River Avon.

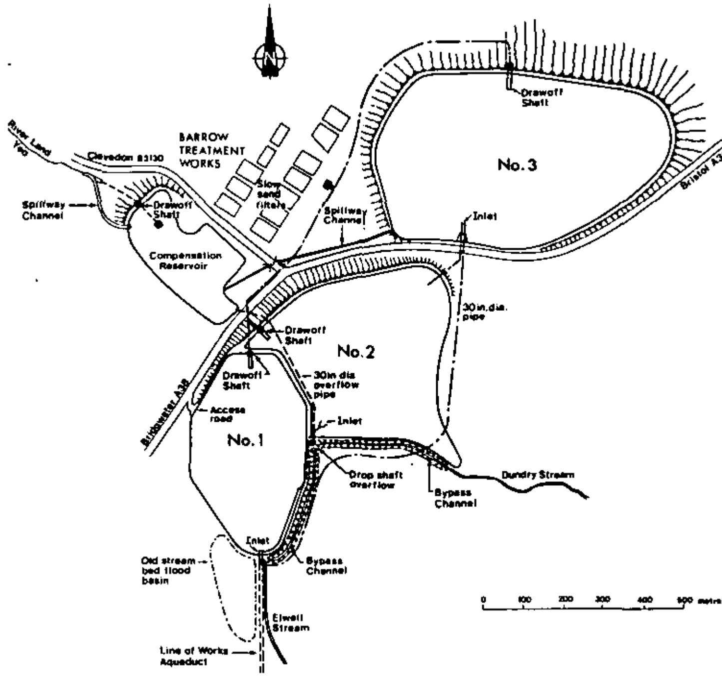

Fig. 3 BARROW RESERVOIRS

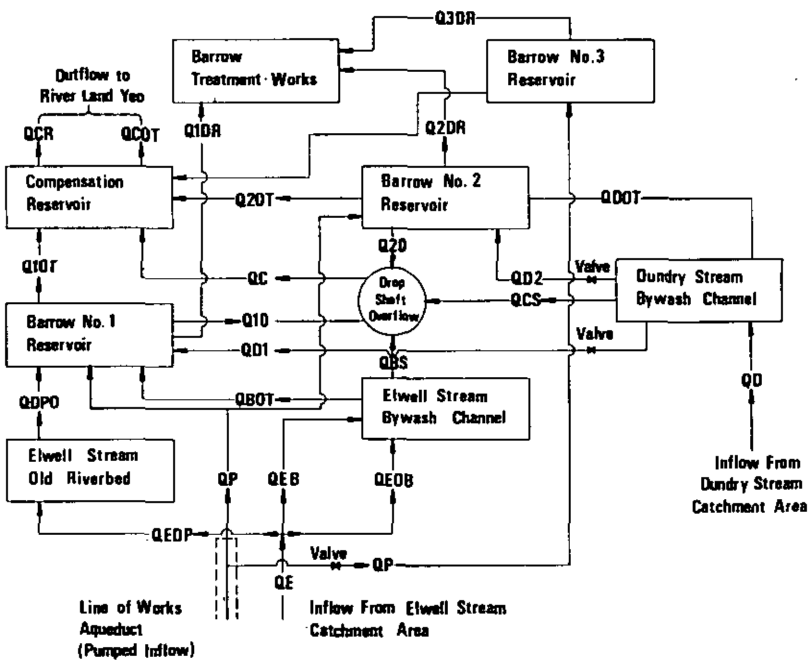

Fig. 4 DIAGRAMMATIC REPRESENTATION OF BARROW RESERVOIRS NETWORK

Barrow Nos. 1 & 2 Reservoirs

These two reservoirs, which are located alongside each other, are primarily used as terminal storage reservoirs to the Line of Works Aqueduct though each of them has its own small direct catchment area.

The reservoirs share a common embankment in which is located a dropshaft overflow. The bypass channels around the reservoirs connect with the base of the dropshaft and water is conveyed away through a 30 inch diameter pipe passing under the basin and embankment of No. 2 reservoir to discharge into the Compensation Reservoir.
During the storms on the night of 10th July 1968 the water level in No. 1 Reservoir rose to within inches of overtopping the embankment at its north-west corner where the access track from the A38 trunk road meets the embankment crest road.

The Inspecting Engineer determined that since the Barrow Compensation Reservoir is located immediately downstream, and this is kept empty, he would place the reservoirs in Category B, General Standard, and the dropshaft overflow be capable of passing a 0.5 PMF.

The complex arrangement of weirs, channels, pipes, valves and flood storage basins, shown diagrammatically in Fig. 4, resulted in the development of an equally complex mathematical model in order to evaluate inflows and outflows. This revealed that the 30 inch diameter overflow pipe under No. 2 Reservoir would not be able to discharge a 0.5 PMF and that the embankment would be overtopped.

The wave wall on No. 1 reservoir was therefore increased in height and the access track from the A38 trunk road upgraded so that, should the water ever reach this level, the end of the embankment would not be scoured away and the water would pass safely down the road surface. These works were carried out in 1982 at a cost in the order of £55,000.

Barrow No. 3 Reservoir

The reservoir is formed by an embankment on all sides. It does not have its own catchment area and the main inflow is derived from the Line of Works Aqueduct. The reservoir was not built with an overflow and during a weekend in March 1981 it filled to the extent that gale force winds and the natural shape of the reservoir, which focused the waves at the eastern end, caused large quantities of spray to be carried down the face of the embankment thoroughly saturating it. The Inspecting Engineer therefore recommended that the reservoir be provided with an overflow so that it could not inadvertently be overfilled. In order to overcome the problem of the waves creating spray it has been decided to lower the "top" water level by 300mm.

The total project costs, which also includes various toe drains and leakage measurement facilities, are in the order of £97,000 and it is anticipated the works will be completed in the autumn of 1984.

Barrow Compensation Reservoir

The reservoir was emptied and abandoned in 1882 following several attempts to seal leakage which was passing through the foundation of the dam. It subsequently became completely overgrown and the spillway channel fell into disrepair with much of the masonry being robbed for other building projects. A pumping main, from the adjacent treatment works to a local service reservoir, was laid along the crest of the embankment and the spillway channel was filled in, to provide support to the main, leaving the scour pipe as the only means of discharging any impounded water.

During the storms on the night of 10th July 1968 the reservoir filled to within several feet of the top of the embankment where it remained in a state of equilibrium for several hours before subsiding. During this time water was seen to leak profusely from the area of the downstream toe.

Since this reservoir had been abandoned it was for years considered to be outside the 1930 Act and only in 1979 was it realised that it should be subject to inspection. The Inspecting Engineer determined that because of its location above the village of Barrow Gurney the reservoir be placed in Category A, General Standard, and must be able to pass the PMF.

Various options, including demolition, were evaluated and found to offer less attractive engineering solutions than refurbishment as well as being more expensive. The spillway has therefore been reconstructed so that it can pass 6 cumecs but with the overflow cill reduced in level by 1 metre. These works, along with works to seal the leakage and refurbish the drawoff shaft, were carried out in 1982 at a cost in the order of £225,000.

BLAGDON RESERVOIR

The reservoir is located on the River Yeo some 7 kilometres upstream of the village of Congresbury. The reservoir has therefore been placed in Category A, General Standard, and must be able to pass the PMF. A particular feature of the overflow structure is the walkway located on the broad crested weir cill, the cast iron legs of the walkway providing the support for stoplogs which have raised the top water level initially by 600mm and subsequently by a further 400mm.

Whilst the overflow weir and spillway channel are capable of passing the PMF the Inspecting Engineer has observed that it could become restricted if debris is trapped against the superstructure of the walkway and has therefore recommended that this be removed and be replaced by an unobstructed concrete cill. The estimated cost of these works is in the order of £70,000.

CHEDDAR RESERVOIR

The reservoir is formed by an embankment on all sides and does not have a direct catchment area. The gravity inflow from the intake on the spring collecting pond in Cheddar Gorge is controlled by valves. Additional inflow can be obtained by pumping from the River Axe at Brinscombe Pumping Station.

The top water level of the reservoir has been raised by placing 12 inch deep stop logs on the overflow weir cill that discharges into a sump from which the water is conveyed away in a 21 inch diameter pipe. The amount of freeboard available is currently being checked to ensure that, under the same criteria that have been applied to Barrow No. 3 Reservoir, there is sufficient wave surcharge allowance.

CHEW VALLEY LAKE

The reservoir is located on the River Chew, 1½ kilometres upstream of the village of Chew Magna.

The Inspecting Engineer therefore determined that the reservoir be placed in Category A, General Standard, and that the spillway should be capable of passing a PMF. The peak inflow during PMF has been calculated as 375 cumecs.

The flood routing procedure has shown that the spillway structure will safely pass the PMF with an outflow of 142 cumecs. Since the PMF top standing water level will be 1.5 metres above weir cill level and the available freeboard is 1.8 metres, it is possible to consider raising the top water level without carrying out major works to the embankment, the wave wall on the crest of the embankment accounting for the wave surcharge allowance.

CONCLUSIONS

The cost of carrying out hydrological studies for these reservoirs has been in the order of £30,000 and they have each taken roughly 3 months to complete including surveys and site visits.

With the continuing process of refinement of the application of FSR it has been the Company's brief to the Consulting Engineers that each hydrological study should be fully written up and include all maps, diagrams and calculations so that, if in future re-assessment of any aspect of this work is required, there will be no difficulty in establishing again the original parameters.

The period required for applying the FSR and Guide at each reservoir has been small by comparison with the period for designing and executing the works. So far the Company has dealt with a reservoir or pair of reservoirs each year.

ACKNOWLEDGEMENTS

The Panel Engineer for the reservoirs is Mr A.J.H. Winder and the hydrological work was carried out for the Company by Messrs Watson Hawksley, Consulting Engineers.

The author is grateful to Mr. J.R.Browning, General Manager to Bristol Waterworks Company, for permission to publish this Paper.

TABLE 1: GENERAL INFORMATION ON MENDIP SUPPLY RESERVOIRS

PRIOR TO RECENT MODIFICATIONS

Name of Reservoir	Period of Construction	Capacity to Top Water Level (cubic metres)	Catchment Area (hectares)	Flood Category As indicated in Table 1 of ICE Guide	Flood Inflow (cumecs)	Flood Outflow (cumecs)	Flood Lift (metres)	Remarks
Chew Magna	1848-50	113,650	1763 (Impounding)	Category A: General Standard	167	163	1.95	An allowance of 0.64m for wave surcharge is required.
Litton Upper	1848-50	459,100	1601 (Impounding)	Category B: General Standard	164	-	Over-topped	The embankment will be overtopped, existing overflow capacity is 33 cumecs, the required overflow capacity is 82 cumecs.
Litton Lower	1848-50	109,104	1816[+] (Impounding)	Category B: General Standard	186	-	Over-topped	The embankment will be overtopped, existing overflow capacity is 40 cumecs, the required overflow capacity is 93 cumecs.
Barrow No.1	1848-50	654,300	165 (Impounding)	Category B: General Standard	31	-	Over-topped	The embankment will be overtopped.
Barrow No.2	1862-64	859,200	89 (Impounding)	Category B: General Standard	24	-	Over-topped	The embankment will be overtopped.
Barrow Compensation	1862-64 Abandoned 1882	115,000	273[x] (Impounding)	Category A: General Standard	20	Overflow blocked	Over-topped	Reservoir assumed empty at start of storm. New spillway required to discharge 6 cumecs. Scour pipe: discharge 1 cumec.
Barrow No.3	1887-99	2,363,900	Indirect[#] (Raw water storage)	Not applicable	2.34	2.34	0.59	An overflow structure capable of discharging 2.34 cumecs is required.
Blagdon	1898-1901	8,455,600	1990 (Impounding)	Category A: General Standard	208*	102	1.10	The overflow structure is capable of passing the appropriate flood but the walkway arrangement on the overflow cill is to be removed.
Cheddar	1933-37	6,137,100	Indirect[▲] (Raw water storage)	Not applicable	-	-	-	Investigation in progress.
Chew Valley	1951-56	20,457,000	5290 (Impounding)	Category A: General Standard	375*	142	1.51	No remedial works are required, t.w.l. could be raised by 300mm.

* Winter PMF + Snow + Frozen Ground
\+ Includes 1601 ha. of Upper Litton Reservoir
▲ 3626 ha. to Intake at Cheddar Gorge
x Includes 254 ha. of Barrow Nos. 1 and 2 Reservoirs
\# Filled from Line of Works Aqueduct

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Some Examples of Improvement Works at Earth Embankment Dams (Following publication of the FSR and Engineering Guide)

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Strathclyde Regional Council

SYNOPSIS

The publication of the Natural Environment Research Council's "Flood Studies Report"[1] and the Institution of Civil Engineers "Engineering Guide"[2] has led to major alterations being required to the overflows and spillway channels of a number of Strathclyde Regional Council's Reservoirs.

This paper uses six recent examples to describe some of the solutions adopted to bring these Reservoirs into line with the new standards. The Works described include providing auxiliary overflows and spillway channels, lowering overflow cill levels (with the consequent loss of storage and yield), improving spillway channel hydraulics, providing wave walls and the ultimate solution of demolition.

INTRODUCTION

Strathclyde Regional Council was formed in 1975, following the reorganisation of Local Government in Scotland. The Region's Water Department is an amalgamation of the former Argyll, Ayr and Bute, Lanarkshire and Lower Clyde Water Boards.

As recently as 1968, the Area now served by the Region was supplied by 45 separate Water Undertakings who depended, almost entirely, on upland impounding Reservoirs for their sources of supply.

The Region therefore inherited a large number of Reservoirs of various ages, types, and sizes. At present the Water Department has 142 Reservoirs with a capacity in excess of 5.0 Million Gallons (i.e. subject to the Reservoirs (Safety Provisions) Act 1930, together with a further 28 Reservoirs with a capacity of less than 5.0 Million Gallons.

AGE AND TYPE

All of the 142 Reservoirs within the ambit of the Act were constructed prior to 1975 and range in age from the earliest which is over 210 years old to the most recent which is 10 years old. Perhaps more significantly, 116 (or 80%) of the Reservoirs were constructed prior to the publication, in 1933, of the Institution of Civil Engineers Report "Floods in relation of Reservoir practice".(3)

As you would anticipate, the bulk of the Reservoirs have earth embankment Dams. In all, 88% are earth embankments, 7% are concrete gravity Dams and the remaining 5% is made up of masonry, brick and gated structures.

EFFECT OF 1933 REPORT

Of the 116 Reservoirs which were in existence when the 1933 Report was published, our Records indicate that 19 (17%) required to be upgraded.

The required upgradings were mainly achieved by widening overflows or by providing additional overflows; in a very few cases, the freeboard was increased by raising the embankment.

EFFECT OF F.S.R. AND I.C.E. GUIDE

Effectively, only Reservoirs which have been inspected since the beginning of 1979 have been subjected to both the Flood Studies Report and the I.C.E. Guide.

Of the 142 Reservoirs within the ambit of the Act, 91 (64%) have been inspected during the period since 1979. This can be sub-divided into 71 of the Reservoirs constructed prior to 1933 and 20 of those constructed since that date.

Overall, 43 Reservoirs (47% of those inspected) have been found to be deficient in terms of their overflow/freeboard relationship, spillway channel capacity, etc., requiring major upgrading Works. Again this figure can be sub-divided into 40 of the Reservoirs constructed prior to 1933 and 3 of the Reservoirs built since 1933. Figure No. 1 shows the distribution by age of those Reservoirs inspected and also shows the proportion requiring upgrading.

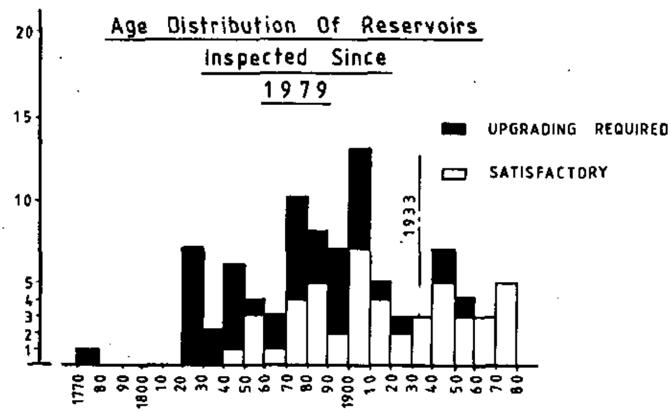

FIG. No.1

The examples which follow give some indication of the different types of solutions which have been adopted to deal with the deficiencies.

Daff Reservoir

Daff Reservoir was constructed for the Burgh of Gourock in 1921 and is situated above the village of Inverkip. The Dam is an earth embankment

Dam and was constructed with an overflow 12.50 m long and a freeboard of 1.15 m.

At the time of construction of the Reservoir, the run-off from the upper half of the catchment was already being intercepted by a substantial catchwater channel, which had been in existence since the mid 1800's. The catchwater conveyed the run-off into an adjacent catchment. In 1978, it was decided, for operational reasons, to discontinue the use of the catchwater and to allow the whole of the run-off to drain, uncontrolled, into Daff Reservoir.

The Reservoir was Inspected in 1979 and both the N.E.R.C. 'Flood Studies Report' and the I.C.E. 'Guide' were used to establish the design outflow. (The Reservoir is a Category 'A' Reservoir). With the design outflow established, it was clear that the 12.5 m overflow and 1.15 m freeboard were inadequate.

In the Statutory Inspection Report the Inspecting Engineer requested that the overflow structure be modified to allow an outflow of 46 m^3/S to be passed without the head on the existing weir exceeding 900 mm. In addition, he requested that the nominal freeboard of 1.15 m be re-established, where necessary, and that a wave-wall be provided to give a total freeboard of 1.70 m.

In view of the shape of the existing overflow and the nature of the existing spillway channel, a straightforward extension of the overflow was impractical. Discussions took place with the Inspecting Engineer and it was decided that a solution would be to provide an auxiliary overflow, with a crest level 300 mm above the existing weir. In order to satisfy the conditions specified in the Statutory Inspection Report, this required a weir 30.0 m in length.

As the auxiliary weir will only function infrequently, it was agreed that an unlined spillway channel would be adequate. In fact, it was possible to make use of a site, adjacent to the embankment, which has a rock invert over most of its length.

The Works were completed in 1982 and consists of the following:-

1) An additional concrete weir, 30.0 m in length, at a level 300 mm above the existing weir. The weir is curved in plan to make best use of the site conditions. The forebay is protected by the use of Reno Mattresses and a short concrete apron has been provided on the downstream side of the weir to prevent erosion,

2) A 15.0 m wide channel, with 2:1 side slopes was excavated through the natural ground adjacent to the Dam and discharges into the watercourse well downstream of the toe of the embankment,

3) A 650 mm high insitu concrete wave-wall was constructed along the upstream edge of the crest and was keyed into the puddle clay core of the Dam.

Cost of Works - £137,000.

Munnoch Reservoir

Munnoch Reservoir was constructed in 1877 for the Irvine and District Water Board. The Dam is a long earth embankment Dam and was constructed with an overflow 10.60 m in length together with a freeboard of 1.50 metres. The original embankment was raised by some 2.25 m in 1927 by increasing the slope of the upper section of the upstream face of the Dam and regrading the downstream slope.

The Reservoir is situated 9.5 km upstream of the town of Kilwinning in North Ayrshire and when it was inspected in 1979 it was placed in Category 'A' by the Inspecting Engineer. The hydrological assessment of the Reservoir and its catchment indicated that the existing overflow Works and freeboard were inadequate.

In his Statutory Inspection Report the Inspecting Engineer indicated that possible solutions might be to either provide an additional overflow, 30.0 m in length or, alternatively, provide a wave wall 900 mm high along the length of the crest together with lowering of the existing overflow crest by 750 mm.

Both alternatives were examined in detail and were found to be economically unattractive. In the first case, the only suitable site for an additional overflow, would have required the construction of a long additional spillway channel; in the second case, the Reservoir has a long embankment and the construction of a wave wall would have been expensive. (In addition the Inspecting Engineer had expressed the view that were a wave wall to be considered, then the stability of the embankment would require to be investigated in view of the method used to raise it in 1927).

The Operational use of the Reservoir was reviewed in the light of the Department's overall Resources in North Ayrshire and it was decided that a further option, which was financially attractive in the short term, would be the lowering of the overflow by 1.65 m to achieve the desired effect. In the present circumstances of depressed demand for water in North Ayrshire, the resulting 25% reduction in net reliable yield was considered to be acceptable.

Cost of Works - £33,000.

Kype Reservoir

Kype Reservoir was constructed for Hamilton Town Council in 1898 and consists of an earth embankment Dam, curved in plan, with a 31.0 m wide overflow located on natural ground at the east end of the Dam.

This is an example where, in addition to an inadequate overflow/freeboard relationship, the existing spillway channel was recently shown to have an inadequate capacity.

In October 1977, following a period of prolonged heavy rainfall, the walls of the spillway channel were overtopped and considerable erosion resulted. Fortunately, most of the severe damage occurred on the natural ground adjacent to the spillway channel, where some 60.0 m^3 of rock was washed out. There was, however, some minor damage to the mitre of the embankment.

As the incident occurred during the night and as there was no water level recording facilities at this Reservoir, we were unable to establish the exact depth of overflow which had caused the damage. The Inspecting Engineer was asked to investigate the incident and by calculation he came to the conclusion that the damage resulted from a flood with a return frequency of once in 200 - 400 years.

In 1979 the Statutory Inspection of the Reservoir was carried out and the Reservoir was placed in Category 'B' by the Inspecting Engineer. The design outflow, established using the 'Flood Studies Report' (factored in accordance with Table 1 of the I.C.E. 'Guide' for a Category B Reservoir) was 50 m^3/s. This was almost three times the flow which was considered to be responsible for the overtopping of the walls in 1977.

It was therefore decided to construct a hydraulic model of the overflow and spillway channel, to examine the hydraulics of the existing channel and to establish the best method of achieving the required upgrading.

The lower section of the existing spillway channel consisted of a steep cascade which had been totally reconstructed in the early 1970's. This section of the channel was considered to be adequate to deal with the design outflow, so it was decided that it should remain unaltered, if at all possible. The problem thus came down to improving the channel from the overflow weir to the beginning of the cascade.

After several attempts at altering the configuration of the model channel a solution evolved which consisted of retaining the upper section of the channel and providing additional cross-sectional area by constructing aprons, on both sides of the channel, at the level of the existing walls tops.

A study of the water velocities in the model established that under the design outflow conditions, low water velocities would occur over much of the length of the apron on the natural ground side of the channel. As a result it was decided to utilise 'Reno' mattresses to form the invert of that section and rely on the rise of the natural ground to retain the water. Elsewhere the apron invert and side walls were to be constructed in concrete.

The Works were carried out during 1982. Aprons 3.0 m wide, together with associated side walls, were constructed on both sides of the channel from the overflow weir to the beginning of the cascade. In addition, a pre-cast concrete wave-wall, 650 mm high was provided along the upstream edge of the crest of the embankment.

Cost of Works - £130,000.

Greenside Reservoir

Greenside Reservoir was built in 1897 for the Clydebank and District Water Trust. The Dam is an earth embankment Dam and was provided with a U-shaped overflow 17.2 m in length together with a freeboard of 1.60 m. The Reservoir is the lowest of three Reservoirs in series and is located some 3.0 km upstream of the village of Duntocher.

The valley downstream of the Reservoir is a steep sided valley, consequently there would be little attenuation of any release of water. It was therefore placed in Category A by the Inspecting Engineer when it was Inspected in 1982.

The hydraulic efficiency of the particular shape of overflow at the Reservoir, had already been questioned on a number of occasions and it was therefore decided that we would carry out a hydraulic model test of the overflow and spillway channel in conjunction with the Statutory Inspection. Testing of the original weir confirmed the reservations as to its efficiency.

The model was then altered to determine the nature and extent of the upgrading Works which would be required to allow the Reservoirs to safely discharge the Design Outflow of 38.5 m^3/s. It was agreed that the weir should be altered to allow the flood water to be discharged without the head over the weir exceeding the available freeboard; additional freeboard for wave surcharge would be achieved by the provision of a wave wall.

The spillway channel, as originally constructed, consisted of a natural rock invert with low mass concrete side walls. From the model flows, it was obvious that the walls would be overtopped at the higher flows. This was unacceptable as the spillway channel lies on the left-hand mitre of the Dam. A spillway channel, with an improved carrying capacity was therefore a further requirement.

Various lengths of side weir overflow were investigated together with some minor improvements to the spillway channel geometry. Eventually a solution was arrived at which required a 30.0 m long side weir overflow to be provided together with a 600 mm high wave wall. (Water profiles were plotted to determine the new spillway channel wall heights). In view of the high velocities which would be achieved in the steeply graded section of the new channel, the lower part of the channel was roofed over to contain the water at the point where the channel turns through 60° to flow away from the toe of the embankment.

The Works, which consisted of constructing a 30.0 metre long side weir overflow, a new concrete spillway channel and a wave wall along the crest of the Dam, were executed during 1983-84.

Cost of Works - £200,000.

Daer Reservoir

Daer Reservoir was constructed for the Daer Water Board and was completed in 1954. The Dam is an earth embankment dam with an articulated concrete core wall. The overflow is a side weir overflow, 67 metres in length, located at the north-east end of the embankment.

In this example, the Works which have been undertaken were necessary partly as a result of the new 'Standards' and partly as a result of the deterioration of the concrete structures.

Prior to completion of the Reservoir, in 1954, the various concrete structures had begun to show signs of deterioration. The deterioration was later identified as being mainly due to aggregate shrinkage; a phenomenon not fully understood at the time. Despite extensive routine maintenance over the years the upstream slope slab protection and the spillway channel walls and invert progressively deteriorated.

The Reservoir was Inspected in 1978 and was placed in Category 'A', as much for its strategic importance to the Lanarkshire Water Supply, as for the risk to lives downstream. In his Report, the Inspecting Engineer concluded that the overflow/freeboard was inadequate to pass the Design Flood of 320 m^3/s and in addition he requested that the hydraulics of the existing spillway channel be investigated under the conditions of this flow.

A Hydraulic Model Study of the overflow and spillway channel was carried out by Glasgow University and it quickly became apparent that the channel was not capable of discharging the required flood outflow.

Before any alterations were carried out on the model, certain constraints were applied:-

1) any alteration to the overflow and spillway channel should ensure that the head over the weir, whilst discharging the Design Flood, should be less than the available freeboard to Dam crest level,

2) due to the height of the existing spillway channel walls, which act as embankment retaining walls, any lowering of the channel invert (and subsequent underpinning) should be limited to 2.0 metres.

There were no further constraints on altering any other section of the spillway channel, as the deterioration of the existing structures had made their replacement inevitable.

From the model study, a revised channel cross-section and gradient was established which would allow the Design Flood to be passed safely.

The reconstruction of the channel is being carried out in three phases, starting at a point in the channel clear of the downstream toe and working upwards.

Phase I consisted of the reconstruction of the invert and walls of the existing channel, with a small addition to the cross-section at its upper level and was completed in 1982.

Phase II, involved lowering the channel invert by up to 2.0 metres, in order to regrade the channel, together with a widening of the channel. Again the invert and walls were totally reconstructed. This work was completed earlier this year.

Phase III, will consist of the reconstruction of the overflow together with the widening and deepening of the side channel and is due to commence later this year.

Cost of Works to date - £720,000.

Estimated cost of Phase III - £600,000.

Dunoon No. 2 Reservoir

This Reservoir, the middle Reservoir of three in series, was constructed for the Burgh of Dunoon in 1915. The Dam consisted of an earth embankment with a puddle clay core and mass concrete gravity section overflow, located at its south-end.

Throughout the life of the Dam there had been problems with excessive settlement and distortion of the embankment. Following a Statutory Inspection of the Reservoir in 1969, certain investigations and remedial Works were carried out in the early 1970's. These included the provision of a berm on the downstream face of the embankment and the lowering of the overflow crest by 750 mm.

In the late 1970's, the Water Supply to Dunoon was transferred to a new source and the three Reservoirs became redundant. Discussions were then held with the Local District Council to determine the future use of the Reservoirs. At the same time, the Reservoirs were again Inspected.

All three Reservoirs were deficient in terms of their ability to pass their Design Outflows so it was decided that Reservoirs No. 2 and 3 would be breached and that Reservoir No. 1, the lowest of the three (and the largest), would be brought up to the new 'Standard' and then be transferred to the District Council for recreational use.

The breaching of Reservoir No. 2 was carried out during the summer of 1982. The concrete overflow section was cut down in stages and the debris deposited within the solum of the Reservoir. The embankment was then partially dismantled and the material was used for some limited landscaping of the solum.

Although there was a significant accumulation of silt in the Reservoir, no attempt was made to remove it. Over a period of a few months, the silt dried out and self seeded, and the water course across the solum was re-established.

Cost of Works - £48,000.

Conclusion

The examples outlined in this paper are a selection of the different types of solutions which have been adopted by Strathclyde Regional Council, to bring those Reservoirs which have been found to be deficient, into line with the new Standards.

It was anticipated, when the new Standards were published, that upgrading Works would be required at a number of the Regions Reservoirs; what was not anticipated was that the numbers requiring upgrading would be so numerous.

Over the period since 1979, Strathclyde Regional Council has spent approximately £5.0 million upgrading Reservoirs. If the percentage of Reservoirs requiring upgrading remains at the level of 47% for the Reservoirs still to be Inspected, then the Region could well be faced with upgrading a total of 66 Reservoirs at a projected total cost of £10.0 million.

Acknowledgement

The author wishes to thank Mr. W.T. Devenay, Director of Water, Strathclyde Regional Council, for permission to present this paper.

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Embankment Dams and Reservoir Safety in Britain; Floods, Slides and Internal Erosion

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SYNOPSIS

The majority of British dams are old earth embankments and reservoir safety in Britain is therefore primarily concerned with the long term performance and safety of old embankment dams. Major classes of hazard to these earth structures are:

(i) floods - if the spillway is inadequate the embankment may be overtopped and destroyed by a flood,

(ii) slips or slides - shear failure may occur in one of the slopes of the embankment,

(iii) internal erosion - seepage and leakage of water through the embankment or its foundations may cause erosion and destroy or seriously damage the dam.

INTRODUCTION

The decision taken by the Government in March 1983 to implement the Reservoirs Act 1975[1] has again focussed attention on reservoir safety. The publication of two reports concerned with floods and reservoir safety in 1975[2] and in 1978[3] emphasised the necessity of ensuring adequate spillway capacity so that earth dams are not overtopped during floods. This has led to much effort being put into the estimation of floods and in some cases considerable expenditure on enlarging and re-building spillways. However, while overtopping is rightly considered a serious hazard to the safety of an earth dam, two other equally important hazards should not be overlooked. These are respectively, shear failure of the earth embankment (which may manifest itself as a slip or slide in one of the slopes of the embankment and also may involve the foundation) and internal erosion of the earth embankment or its foundation (which may occur due to seepage or leakage of reservoir water through or under the embankment, or associated with a leaking conduit within the embankment). There are of course other possible modes of failure but the majority of earth dam failures have been associated with one or more of the three types of hazard, namely overtopping, shear failure and internal erosion.

DAM CONSTRUCTION IN BRITAIN

By the middle of the nineteenth century earth dam construction was well developed in Britain. Usually a narrow central core of puddled clay formed the watertight element. Typical embankment slopes would be 1 in 3 upstream and 1 in 2.5 downstream. Very few of these embankments were more than 30 m high. This type of construction continued to be used until the middle of the twentieth century although other forms of construction (eg homogeneous embankments; concrete core walls) were adopted on occasions.

Statistical information about the 2000 reservoirs that are believed to come under the ambit of the Reservoirs (Safety Provisions) Act, 1930[4] (ie reservoir capacity greater than 5 million gallons) is not readily available. The Reservoirs Act 1975[1] when implemented will lead to the setting up of registers. In the meantime the World Register of Dams[5] also provides some information about over 500 large British dams (principally dams greater than 15 m in height). Figure 1(a) is derived from the British section of the World Register (although the section is headed Great Britain it also includes dams in Northern Ireland). The figure shows the number of large dams built in each decade in Britain during the last 200 years. Figure 1(b) shows the same data plotted as a cumulative total of dams built by the end of each decade. Prior to 1900 almost all British dams were earth embankments.

In 1964 at the 8th International Congress on Large Dams, Pippard(6) reviewed the dams completed in Britain since 1960 and those then currently under design or construction. He commented that there was a preponderance of rolled clay cores and those dams containing puddle cores had been designed some years prior to 1960. Pippard quoted the opinion of the engineers for Selset and Balderhead dams that the puddle clay core could be considered an obsolete and costly feature of British embankments. However Figure 1(b) shows that embankment dams built in the period up to 1950, which would mostly have puddle clay cores, form a substantial part (60%) of the existing total population of large dams in Britain. Many of the reservoir safety problems will be associated with these dams which generally were designed and built before the theories of modern soil mechanics were fully appreciated or modern heavy earth moving and compaction plant was available.

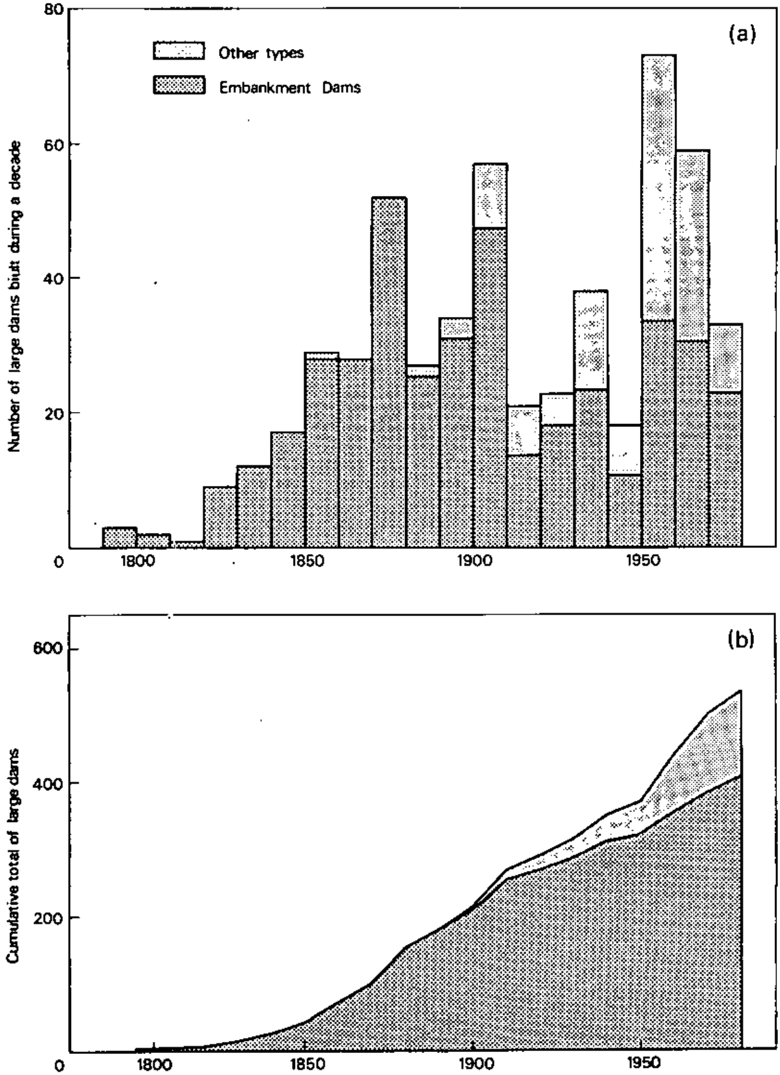

Fig 1 The construction of large dams in Britain (after World Register of Dams[5])

EARTH DAM FAILURES

There have been a number of attempts to collect statistics on dam failures. There are however difficulties in interpreting such data.

(i) The definition of failure. The widest definition of failure is simply the condition in which an earth dam does not fulfil its purpose. From the point of view of reservoir safety it is the in service failure that is the hazard and not only catastrophic failures (ultimate limit state) but also serious incidents (serviceability limit state) are of significance. Under the general definition of failure, construction failures would also be included although they are not directly related to safety in service. Excessive leakage through the foundations, even if it does not lead to internal erosion and affect the safety of the dam structure, could prevent the reservoir from filling and can therefore be classified as dam failure according to the above definition. Ideally the statistics should contain all these categories with a clear demarcation between them.

(ii) The cause of failure. This is not always known with any degree of certainty. Sometimes failure can be due to a combination of hazards.

In reviewing dam practice in the United States of America, Middlebrooks[7] listed unsatsifactory performance of about 200 earth dams. He classified the four main inadequacies which led to partial or complete failure as overtopping, seepage, slides and conduit leakage respectively. His analysis of the data is shown in Figure 2. 83% of the failures could be classified under one of these four headings. Overtopping accounted for 30% of the failures. Many of the failures classified by Middlebrooks as seepage or conduit leakage were associated with internal erosion and together these accounted for 38% of the cases. Slides accounted for only 15% of the incidents.

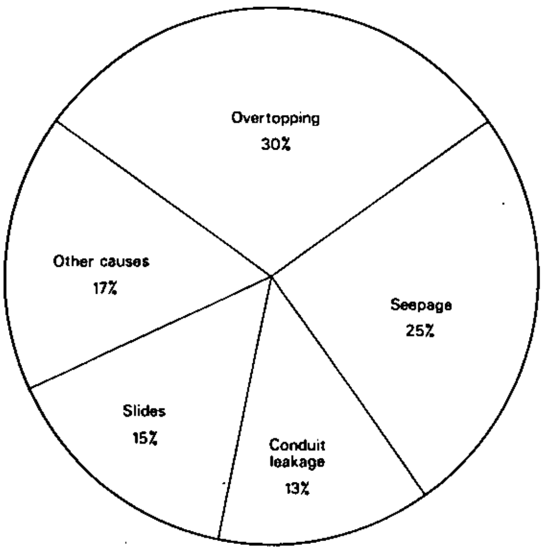

Fig 2 Causes of unsatisfactory performance of earth dams in United States of America as analysed by Middlebrooks(7)

It might be questioned whether such an analysis based mainly on experience in the USA would be relevant to British earth dams. An examination of in service failures and serious incidents (requiring emergency action) in Britain has been made and broad agreement with Middlebrooks analysis has been found in the following respects:

(i) The vast majority of failures, though not all, can be attributed to the three main hazards, ie overtopping, slips and various forms of internal erosion.

(ii) Overtopping accounts for about one quarter of the total number of failures.

(iii) Internal erosion is a much more common mode of in service failure than slips.

It should be noted that if failures during construction were included, then the proportion of failures due to slips would be much increased. However if less serious in service incidents were included, the proportion of the inadequate performances due to internal erosion would be increased.

FLOODS AND OVERTOPPING

Overtopping represents a serious hazard to earth dams and adequate spillway capacity is therefore essential for their safety. Procedures for the derivation of design floods and the estimation of wave surcharge and dam freeboard are given in the Institution of Civil Engineers (ICE) 1978 publication 'Floods and reservoir safety'[3]. In the last few years much attention has been given to this aspect of reservoir safety and there has been considerable expenditure on the spillways of many old earth dams.

Although overtopping has led to the failure of a number of British earth dams, man-made circumstances rather than extreme weather conditions have sometimes been the primary cause of the flood. Thus Coedty dam was overtopped and failed in 1925 due to a flood which resulted from the collapse of the small concrete Eigiau dam above it; Skelmorlie dam also overtopped and failed in 1925 - in this case a blocked culvert had led to a flooded quarry and it was the release of this water that destroyed the dam. Bilberry dam failed catastrophically in 1852 by overtopping after a long period of settlement, probably due to internal erosion, had reduced the freeboard(8). The rebuilt Bilberry dam was severely damaged during a storm in 1944 when part of the downstream slope was severely eroded by run-off from the valley sides onto the top of the embankment coupled with waves breaking over it.

The ICE report(3) suggested that earth embankments with a level crest and a surface roadway may well resist some overtopping successfully. A Construction Industry Research and Information Association (CIRIA) report(9) has quoted velocities that suitably chosen grass can withstand for various periods. The CIRIA report also points out that a badly maintained embankment with surface irregularities, shrubs and bare patches would have a much reduced resistance to erosion. Kennard(10) has given examples of low banks 5 m to 6 m high that have withstood overtopping. Penman(11) has drawn attention to the fact that when Warmwithens dam was breached in 1970 and the full reservoir emptied into Cocker Cobbs and Jackhouse reservoirs, the embankment dam at Cocker Cobbs was overtopped but did not fail. The use of synthetic matting to reinforce grassed slopes and control erosion could be examined as an alternative to building new or enlarged spillways. However Sowers[12] warned that although in a few instances earth dams of clay have withstood overtopping by a 0.3 m or 0.6 m depth of flow for several hours without failure, a large proportion of the few dams supposedly designed to withstand overtopping, and which have been protected against erosion, have been carried away.

Other mechanisms in addition to that of surface erosion may lead to the failure of an earth dam when overtopped. Marsland[13] has reported a full scale experiment in which a clay flood bank was breached. It was observed that a shallow slip occurred on the landward slope almost immediately a thin sheet of water commenced flowing over the bank. The time from the start of the slip to the formation of the breech was under 2 minutes. Marsland concluded that in flood banks constructed of shrinkable soils, a highly fissured outer zone develops due to drying and removal of water by vegetation and then under flood conditions slips develop in these fissured slopes just prior to or due to overtopping. The shallow slide in the downstream slope of Buckieburn dam occurred in 1970 during a period of heavy rain and high winds[14]. The slide appeared to have been initiated by surface water flowing down the slope together with water from wave action overtopping the parapet wall at the crest of the dam and the flow of surface water in the valley formed by the intersection of the embankment with the natural hillside. Water penetrating through the defective foundation of the wave wall may have been responsible for the downstream slide at Combs dam that occurred in 1976[15].

The high water level in a reservoir resulting from flood conditions can have other important effects on the performance and safety of an earth dam in addition to the risk of overtopping. Any weaknesses in the embankment are likely to be revealed in these circumstances. The high water level will result in high pore water pressures which will affect slope stability. Stability may also be affected by the pressure of water acting on the wave wall. An empty reservoir rapidly refilled during a flood could be particularly susceptible to the onset of hydraulic facture[16].

Kennard[10] has pointed out that some remedial works designed to prevent overtopping can cause problems. Extra fill may be placed on the crest or a more substantial wave wall built to increase freeboard. Howevèr this may adversely affect the stability of the top of the dam. A new wave wall could divert run-off which previously ran into the reservoir onto the downstream slope and cause erosion.

SLIPS AND STABILITY ANALYSIS

From the geotechnical engineer's point of view this is the most tractable type of hazard. Computer programs are available to analyse stability for both circular and non-circular slip surfaces. With information about the shear strength properties of fill and foundation and some knowledge of pore pressure behaviour, effective stress analyses can be carried out to examine the stability during construction, first filling of the reservoir, steady seepage condition and rapid drawdown. If the factor of safety against stability failure is found to be unacceptably close to unity, stability can be improved by weighting the toe, flattening slopes or adding berms, or improving the drainage to reduce high pore water pressures.

Many of the slips that have affected puddle core dams have occurred during construction (eg Chingford in 1937 and Muirhead in 1941). This is not surprising as the puddle clay is likely to have high pore water pressures during construction which will subsequently dissipate to steady seepage values. At Killylane, an 18 m high dam completed in Northern Ireland in 1960, pore water pressures of 90% overburden pressure were registered in the puddle clay during construction but fell subsequently to 30–35%[6]. End of construction or a rapid first filling of the reservoir are likely to be the critical periods of stability. Where slips have occurred much later in the life of a dam, they have generally been assocated with some exceptional circumstances rather than with a general 'ageing' process (eg a slip occurred at Tittlesworth[17] in the downstream slope when the toe was cut into for the construction of a new dam; the downstream slip at Blithfield[18] in 1962 was attributed to the action of waves overtopping the dam during a storm).

INTERNAL EROSION

Seepage, leakage, piping, hydraulic fracture, differential movements, and conduit leakage cause problems when they result in the erosion of fill or foundation material. The mechanisms involved are not as well understood as those connected with shear stability failure and no comparable analytical techniques are available to calculate a factor of safety against the hazard. This has meant that this type of problem sometimes receives relatively little attention yet it may be the most important hazard to the safety of puddle clay core dams.

The problem has long been recognised. Summarising British dam building experience in 1951 Steer and Binnie(19) wrote 'In some cases the puddled clay was placed on rock which, although sound in itself, had joints in it which were not impermeable. If at any point there was sufficient movement of water to produce the slightest erosion of clay above it, this movement almost invariably increased. The finer particles of clay in the line of the joint were washed away progressively in an upward direction, leaving behind the sandy particles, which nearly all natural clays contain, and ultimately assuming the form of a thin vertical sheet of almost clean sand traversing the puddle wall and having a thickness which has varied in different cases from a few inches to a couple of feet or more rising to a height of 30 or 40 feet.' They went on to describe leakage paths resulting from vertical steps into the bottom of puddle filled cut-off trenches and the settlement of clay around a flat topped culvert.

It was thought that open joints in the rock were responsible for the failure of Woodhead dam in 1850. The overtopping and failure of Bilberry dam in 1852 was due to settlement probably resulting from internal erosion reducing the freeboard. The Dale Dyke failure in 1864 has been attributed to hydraulic fracture. When Warmwithens failed in 1970, the breach occurred on the line of a tunnel recently driven through the embankment. The serious incident at Lluest Wen(17) in 1969/1970 was due to internal erosion of the puddle clay.

In considering internal erosion it is useful to distinguish three processes. Firstly the initiating mechanism should be considered. This could be hydraulic fracture within the puddle clay, hydraulic separation along some interface, differential movement or a fractured outlet pipe in the embankment. Secondly the resistance to erosion of the puddle clay should be examined. Thirdly the possibility of any process which will retard or halt erosion should be considered eg if cohesive selected fill was placed immediately downstream of the puddle core it might act as a filter and halt erosion. There is a great need for a better understanding of these processes and current research by the Building Research Establishment[16][20][21] into the safety of old dams is concentrated on this area. In contrast to overtopping and slides, internal erosion is essentially progressive in character and as a hazard to reservoir safety much depends on the speed of the process of erosion.

Grouting has commonly been used as a remedial measure. Where severe erosion has occurred, it may be necessary to construct a new core of plastic concrete using slurry trench techniques. If conduits placed in the embankment fill cause problems, new draw-off works may be needed.

CONCLUSIONS

1 Reservoir safety in Britain is largely concerned with the safety of old embankment dams, ie earth dams mostly designed and built before the theories of modern soil mechanics were understood or modern heavy earth moving and compaction plant was available.

2 Three major classes of hazard to these old earth dams can be identified:

(a) surface erosion of the embankment generally due to overtopping during floods,

(b) stability failure of the embankment generally in the form of slides or slips in the slopes of the dam and possibly its foundation,

(c) internal erosion of the embankment or its foundation due to a variety of possible mechanisms.

3 Internal erosion is the least understood of these types of hazard and may be the most important. An improved understanding of the processes involved and the efficacy of various remedial measures is needed.

ACKNOWLEDGEMENTS

Investigations into the safety and long term performance of old embankment dams form part of the research programme of the Building Research Establishment. The main customer for the work is the Water Directorate of the Department of the Environment. The paper is published by permission of the Director of the Building Research Establishment.

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Maintenance of Safety of Concrete Dams

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SYNOPSIS

The maintenance of safety of concrete dams requires attention to deterioration of concrete, seepage, cracks, drainage, and also to the re-assessment of stability in cases where uplift was not adequately allowed for in the original design and also where flood capacity is re-assessed leading to much higher peak flood levels.

In general, concrete gravity and buttress dams are usually stable without much concern, but attention to details and adequate maintenance is required, together with appropriate long term monitoring and surveillance to evaluate in service performance and define 'ageing' parameters.

INTRODUCTION

Concrete dams constructed of non-erodible materials are subject to different problems of deterioration and modes of failure than embankment dam. Recent experience of older concrete and masonry dams has highlighted several important aspects. These have included inadquate allowance for uplift in some designs; the problems of cracking and joints; and deterioration of concrete, and these matters are considered in the paper together with selected case histories. In particular increased principal stresses have to be planned for with a revised PMF food standard, often well in excess of the original design conditions.

The paper is based primarily on concrete and masonry gravity and buttress dams. Original figures and units have been used where applicable.

ASPECTS OF SAFETY

A conventional stability analysis is based on the weight of the cross-section; water load and uplift. Ice, silt, and wind loads may be considered in certain cases. Joints and cracks are not considered in a new design, but seepage from construction and contraction joints, and from later cracking cracks are considered as defects to be remedied. Many older gravity dams have proved to be very satisfactory when constructed without vertical contraction joints, for example Howden, Derwent, Vyrnwy and Elan Valley Dams, whilst other old dams have suffered from the lack of joints, for example Blackwater Dam, Scotland (1). Modern design methods, from the post World War 1 period have included contraction joints with water stops of copper, rubber, plastic and/or bitumen. Not all these joints have proved to be watertight, either due to lack of movement accommodation or degredation. Horizontal construction joints may also not be watertight and can allow water to enter the dam increasing pore and uplift pressures.

The seepage patterns that are known to occur and their control are important aspects of the safety evaluation matrix.

Dams designed as cyclopean masonry dams mainly within the period 1875 *to* 1920 without joints and often with no internal drainage measures, have often proved satisfactory although not meeting modern design and construction methodologies.

There are such examples in Iran and other overseas countries of gravity dams being of considerable ages having withstood the test of time.

The majority of early dams to 1900 had no instrumentation provision, but from 1900 onwards (for example Howden and Derwent Dams) structural analysis had advanced to the point whereby flexure could be related to water level and plumb bobs with verniers were installed for comparative purposes. Modern dams such as Clywedog have a full range of instruments for construction, immediate post construction and operational monitoring as follows; seepage underdrains, foundation and dam pore pressures, uplift and pressure relief piezometers, rosette strain/temperature gauges for principal stress and finite element analysis by computer, surface strain gauges over thermal cracks and construction joints together with buttresses; peripheral fault and stream gauging for water budget evaluation. Provision can also be made for such as geochemical analyses when assessing changes in underdrainage pattern.

The importance of optimal location of drainage galleries and uplift control were realised at an early stage, for example at Vyrnwy Dam [2].

STABILITY - GENERAL

Recent interest, especially when re-assessing the stability of existing dams, has been concerned with the water load due to flood conditions; uplift and earthquakes. When a dam was originally designed without uplift, it was usually on the basis that vector forces fell within the basal mid-third, and no tension was generated at lines of principal stress. A re-assessment with realistic uplift assumptions shows that tensile stresses can exist and a judgement has to be made on whether the dam can be considered safe. This has been done for Blackbrook, Elan Valley and Derwent Valley Dams.

STRESSES

Compressive stresses in a gravity dam do not become important until maximum compressive principal stresses approach the allowable concrete stress. With concrete having 4 MN/m^2 (2,000 p.s.i.) strength, this would not occur in dams under 150 m (500 ft.) in height.

Re-assessment shows that tensile stresses exist in many cases in gravity dams, although the basis of design was originally for "no tension". Concrete can accommodate tension and in arch dam practice, tensile stresses are usually permitted in areas where the occurrence of cracks would not adversely affect the structure. The permissible stress used by 4 leading authorities in the U.S.A. ranges from 120 to 180 p.s.i.(3). The authors consider such stresses would be allowable in an existing concrete dam, that does not show undue signs of distress. An example of this approach is at Upper Glendevon Dam (4), which showed signs of distress and where the stability needed to be re-assessed and tensile strengths measured on samples from cores gave values of 0.3 to 2 $MN/^2$ (43 to 275 p.s.i.).

Recent inspections at Elan and Derwent Valley Dams have suggested that tensile stresses on the upstream face of the order of 0.1 to 0.2 MN/m^2 can be generated under PMF overflow conditions. Uncertainty as to underdrainage performance and degree of foundation pressure relief afforded in the case of Elan Valley was judged to be marginal, and in certain cases additional foundation pressure relief was provided by drilling. In these cases an additional 1.5 m maximum additional surcharge was involved; and after drilling and coring the design parameters of bulk density and permeability were obtained. Crack treatment was carried out using resin injection after a period of strain gauge monitoring to determine the crack width response cycles.

WATER LOAD

Standard methods of design of gravity dams, in text books, such as U.S.B.R. "Design of Small Dams" have often been based on the water load due to top water level rather than on maximum flood level, except in cases where the flood rise is a large proportion of the head. The authors consider that in addition to design for top water level, and extreme conditions such as ice, where considered applicable, the design of the cross-section should also be considered for maximum reservoir flood level, with the corresponding tailwater level. This has not always been considered in the past. (In addition Deacon made an allowance of 40 p.s.i. for wind loading with Vyrnwy dam empty). Although extreme flood levels are normally of short duration, the effect on water load and uplift pressures still arises during the period of the flood rise with a considerable period in some cases for their attention. At such extreme loading conditions, a reduction in factor of safety, or increase of risk, can be acceptable. A recent study in the U.S.A.(5) considered the assumption that when tension exists at the heel of a dam, full headwater pressure extends for the entire tension zone. This reference shows that foundation drains can continue to function and result in significantly lower uplift pressures. Hence, cracks and tension are not necessarily a sign that remedial measures are essential, but that effective maintenance is critical.

In the design brief for Vyrnwy Dam Deacon in 1885 draws on (the authors believe) published experience of French engineers [6] in choosing an optimum location for the drainage gallery, and, significantly, considered a design case for full foundation uplift. In advocating the use of underdrains to the gallery he showed the maximum uplift would be 12.5 T/ft^2 if undrained, and as an additional margin the water cushion downstream is constructed to provide additional toe weight to counter-balance uplift pressures. Deacon's use of the term "drainage tunnel" is significant in showing the reason for an internal tunnel or gallery. With the passage of time, accretions have occurred and a tunnel was recently driven at Vyrnwy to access the drainage system from the valve shafts to enable maintenance to be carried out, and underdrainage records to be kept for correlation with rainfall and water levels. In the above cases investigative work was required to determine the in-situ material properties, supplemented by "down the hole" CCTV examinations for seepage planes or fissures.

Neverthless it has to be recognized that because of the analytical determinacy and historical knowledge of strength of materials in compression, that safety margins for concrete and masonry dams are often much slimmer than the higher factors of safety calculated for certain conditions on fill dams with their relative uncertainties.

Experience has shown that a 1.5 m (3 - 6%) rise in operating head due to flood conditions can fundamentally change the entire safety concept, and the assessment thereafter is based on significance relating to orders of magnitude and duration. The need for remedial work may not be clear-cut but coupled with uncertain foundation drainage or increased uplift, investigations would normally be recommended at least.

SEISMIC CONSIDERATIONS

For gravity dams of modest height in low seismic areas the static equivalent method is suitable. This method has been used for dams in Hong Kong, and the only known example in Great Britain is Lednock Dam, Scotland[7].

For future dam designs, it would not be prudent to ignore seismic forces in the design and detailing of a dam, by selecting an appropriate "g" factor as a basic consideration, and a horizontal acceleration of 0.1g is suggested by the authors.

Blackbrook dam near Loughborough (Leicestershire) was affected by earth tremors on 11th February 1957 with a severity of 8 on the then seismological scale rated to 10 maximum [8]. The dam is of gravity concrete very similar to Vyrnwy Dam being completed in 1906 to a height of 95 ft. and crest length 482 ft. The tremor was of two pulses with an interval of 3 seconds on some 10 seconds overall estimated duration. The reservoir was at TWL and the vibrations hit on the longitudinal axis, causing displacement of 0.75 ton copings and cracks to appear to the drainage gallery, and upstream/downstream faces. Crest gully grids and manhole covers were sheared and laterally displaced up to 20mm. Level and drainage monitoring established that the dam settled back to its original foundation but disturbance is noticeable at abutments to the pre-Cambrian rocks to the extent of 30mm, two dimensional displacement, with the drainage gulley being significantly cracked.

Had the line of the tremor been 90° displaced the result would have been catastrophic as the dam is located only 4 miles north of the epicentre. No remedial works were undertaken beyond rebedding copings and it is a source of subsequent regret that no definitive crack monitoring or detailed survey record work was carried out at the time, particularly as a recent inspection has indicated that the cracks are reactivating having previously been recorded as self annealed with calcite.

A tremor was also noticed at Newtown Powis on 15th April 1984 and lasting 15 seconds to the severity of 3.5 on the Richter scale. Both Clywedog dam (c. 1968, gravity buttress) and Nant y Geiffr (c. 1880, earth clay core) are within 6 km of the epicentre and an instrument check at 0900 the following day revealed no untoward circumstances although structural damage had been reported in Newtown.

Both areas above were previously regarded as aseismic, and the aspect of design for seismic activity in Great Britain therefore requires serious consideration.

ALKALI AGGREGATE OR SILICATE REACTION

AAR (or ASR) was a relatively unknown phenomena in the U.K. hitherto, but awareness is growing of its manifestation in structures constructed post 1956. Recent advice by the Cement and Concrete Association (9) suggested an upper limit of 0.6% Na as alkali, or use of sulphate resisting cement as the best specification for avoidance, but recent reservoir inspections have revealed its presence at Blithfield and Llandegfedd Dams and at the time of writing the overflow of Tittesworth Dam is under investigation and this is a particularly interesting case as the dam has a concrete core with sandfill gravity sections. Dr. Alan Poole at Queen Mary College has in the former case identified ASR at the 0.2% level, indicating the difficulties involved.

Severn-Trent have, for a number of years, as a matter of policy, been using up to 50% PFA or blast furnace slag for cement replcement at specified locations but current results with this technique requires additional quality control and mix specification details. A 70/30 mix has proved little different to standard concrete. ASR and cement economics apart, shrinkage steel can also be reduced.

ASR can have a typical crack formation, appearing with "pinkish-tinge" but remedial works are unlikely to be successful as the Val-de-la-Mare Dam, Jersey has shown (10) and in a 10 year period 300mm square columns at Hewletts Service Reservoir, Cheltenham, have developed maximum crack widths of up to 15mm, an indication of the speed and extent of ASR propagation.

The reaction has been slowed in some instances by grouting, crack sealing and water proofing techniques, but difficulties have been experienced with rock anchorages and post tensioning techniques.

Structural tests have indicated the fall off in strength at ASR affected structures may not be as great as intuitively felt so coring and proof loading is recommended, but remedial techniques would hinge around excluding water/seepage and in the case of a dam structural reinforcement through post tensioning, buttresses, rock beams, etc.

There is also the concept that at a given time the reaction may exhaust itself but prediction is a tenuous process and the utmost care is required as safety criteria and economic life span need to be carefully related.

IMPROVEMENT IN STABILITY

The principal methods of improving the stability of a gravity dam include the following:

- lowering top water level
- replacing spillweir by siphons (or other means) to reduce or restrict flood rise
- improvement of underdrainage, seepage control, etc. to reduce uplift pressures; installation of relief drains
- post stressing
- sealing joints and cracks; grouting; and treatment of upstream face
- adding rock fill toe weight

In some cases, a combination of two or more of these methods may be appropriate. Some of these methods are discussed by examples in the following section.

In Hong Kong, a number of old masonry faced gravity dams, had been inspected in 1975 and found to be in various conditions of distress. Cracking, seepage and high uplift pressures were all present, and the cross-section did not allow for uplift in the design. Re-assessment including allowances for uplift, PMF, and seismic forces, led to the TWL of reservoirs being reduced by 2.5 m by removing most of the spillweir crest sections. In another case, where the dam was in a better condition, and the reservoir was large, but where the line of thrust was close to the downstream face, the spillweir was replaced by siphons, with a slightly lower top water level. The resulting flood level had been experienced for over 60 years without apparent distress.

The lowering of water levels generally pre-supposes that seepage levels are too high and to arrive at an objective decision investigations are required to determine relationships between matrix density, permeability, etc. to enable economic and remedial considerations. Before expedients such as sealing cracks or joints, grouting, etc. can be considered, drilling and coring, installation and monitoring of piezometers is required to identify such as phreatic or seepage planes and uplift pressures. Treatment of downstream faces should be approached with the utmost care in avoiding raising pore pressures and, incorporation of drainage is frequently desirable. Elan Valley and Vyrnwy Dams have recently undergone investigations of this type, costing typically £75,000 - £100,000 per structure for up to 12 holes at varying depths of dam, foundation or abutments.

The above needs to be allied to geochemical investigations to assess the rate of fissure dissolution by aggressive water in determining the future pattern of seepage potential. Tritium analyses can also be very useful for seepage assessment.

Grouting of upstream areas and in the vicinity of drainage galleries is a time favoured method of seepage control, the latter based on access availability. Modern chemical grout mixes have much to offer compared to past inflexible, high viscosity, short travel, cementitous, treatments, now achieving strengths higher than the original materials. Typically cracks 0.2 to 40mm are dealt with by drilling 25mm dia. holes into the gallery or face, followed by carefully controlled pressure injection of low viscosity epoxy grout to search out, seal, and "wet up" crack interfaces. This is followed by higher viscosity grout to seal fissure flow, and permeable zones.

Frequently grout is seen flowing out of adjacent areas where crack continuity is present, and drilling/coring is advisable to check the effect of treatment on permeability and grout location compared to measured injection "takes". Chemical grouts displace water, are of high mechanical strength, are non-shrink and can be applied in underwater locations. Glass fibre reinforcement can be applied within the surface area to assist in shrinkage and where the dam is an overflow section.

Design of grout mixes and application pressures are critical and it is essential to ensure the full range of crack movement is monitored and seasonal harmonics due to temperature, seismic or hillside movement, and dynamic flexure known before a decision be made. Certain well documented examples have noted exacerbated structural determinations by transferring seepage paths or stresses to alternative areas !

CONCLUSIONS

Whilst not attempting to present a comprehensive "state of the art" resume the authors have drawn attention to what are considered to be the most significant means affecting maintenance of safety of concrete and masonry gravity dams in a contemporary environment whereby ageing continues and financial availability is reducing, yet PMF's get larger !

The authors' aim has been primarily to promote discussion on these matters.

ACKNOWLEDGEMENTS

1. The authors are grateful to Severn-Trent Water Authority and Rofe, Kennard and Lapworth for permission to present this paper.

2. The help of Messrs. Binnie and Partners and Sir William Halcrow and Partners is gratefully acknowledged in referring to investigative work carried out at Vyrnwy and Elan Valley reservoirs.

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Concrete Dams: Long Term Deterioration and Remedial Works

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SYNOPSIS

Associated with the reservoirs operated by the North of Scotland Hydro-Electric Board are 49 concrete dams listed in the World Register of Dams. Gravity and Buttress types predominate and all were constructed between 1930 and 1965. This paper reviews the remedial work, with costs, undertaken on 35 of these structures during the last 5 years and describes in more detail the testing, maintenance and restoration work carried out on pressure relief systems and the refurbishment of gates for the control of water. It is hoped that this experience will enable some improvement to be made in the future design and construction of dams.

INTRODUCTION AND REVIEW

Of the 84 dams and weirs creating the 76 statutory reservoirs operated by the North of Scotland Hydro-Electric Board, there are 49 dams classified under the various concrete types listed in the World Register of Dams. Two of these dams are over 50 years old and the average age of the others is 27 years. This paper reviews the long term deterioration occurring at these dams and their appurtenances together with the remedial work undertaken. During the last 5 years remedial work has been carried out under some 60 contracts on 35 of these dams and the costs are tabulated below. Routine maintenance by local Board staff is not included.

TYPE OF WORK	CONTRACT COST: £ 1983 PRICE LEVEL
Pitching and Gabions	40,000
Steelwork and Painting	62,000
Gravel Clearance	89,000
Pressure relief systems	108,000
Concrete and Joint Repairs	127,000
Gate Refurbishment	449,000
TOTAL (equivalent to £5,000/dam/annum)	£875,000

Glascarnoch Dam where the reservoir has a long exposed fetch accounts for 75% of the cost of pitching repairs. Where subject to heavy wave action stone pitching grouted with cement mortar gives poor service. The fines and small rock fill on which the pitching is placed are washed out and settle allowing the grouted surface layer to cave in and be destroyed during a storm. Heavy rip rap provides very much better protection as it readily dissipates wave energy, settles without failure, and is easily replaced if damaged. Where the banks of rivers are occasionally damaged by high flows, especially where the natural flow has been increased by a diversion, satisfactory repair is achieved with stone filled gabions, generally of plastic coated wire, well staked to the underlying bed.

The repainting of screen cleaning gantries at 6 dams forms 66% of the painting costs. The problem of renewing the internal coating on steel pipes which form bottom draw-offs through dams is now being tackled but the provision of an adequately sized drain from just behind the gate would facilitate this process. However leakage problems are overcome, ventilation and dehumidification is provided, the steel is grit blasted to Swedish Standard Sa2½ using silica free abrasive and then protected by a coal tar/epoxy based coating of 375 micron minimum dry film thickness. Improved standards of safety with regard to access and working areas for the Board's and contractors' personnel and also the public, has led to improvements in handrailing and ladderways. One has to judge the type of person - children, grandparents, experienced hill walkers or employees - who are either invited or can easily gain access to the site, to judge the hazards in relation to these people and to protect accordingly. Unless steel handrails are treated against corrosion, such as by galvanising, before they are built into standards, handrails will fail by corrosion at the joints and involve what should be a needless expense in replacement.

Dredging to permit proper operation of the scour sluice at the 53 year old Ericht Dam was responsible for 66% of the cost of gravel clearance while the remainder was expended at Awe Barrage on the intake to the 7 m diameter tunnel. In both instances dredged channels are involved, in one case the problem being exacerbated by deposition of gravel and sand from an aqueduct and in the other by flood flows.

Pressure relief systems are reviewed in detail later in the Paper and suffice to say here that the cost tabled is for remedial work on 12 dams.

The figure for remedial work to concrete relates to 14 dams. The main cause of disintegration of concrete is the freeze/thaw action on thin sections, parapet walls, crest roadways, bridge seatings, piers and movement joints: especially where the concrete is saturated because a damp proof course has been omitted or because poor provision has been made for the drainage of surface water. In general upstream and downstream faces have required little repair but where this has been necessary it has been at lift joints, sharp corners and areas of poor concrete so proving the need for constant control of quality. However on two sites where some grouted pre-placed aggregate was employed surface deterioration to a depth of 50 to 75 mm has occurred sometimes over the whole area with deeper disintegration locally. For economic reasons the application of bitumen coatings to upstream faces is now restricted to structures which have thin or vital concrete sections, eg the prestressed Allt na Lairige Dam or occasionally, for aesthetic reasons, to a dam which is a tourist attraction. Spalled concrete on roadways is repaired with either hot bitumen and chips or asphalt and this also prevents surface water from saturating the underlying concrete. Failure of joint sealants in roadways and precast slabs forming spillway slopes on buttress dams is a problem. The successful application of modern sealants necessitates very careful detailing: a new seal can be easily ruined by the continual exudation of bitumen compounds originally applied prior to the placement of slabs, or by the failure to remove completely any contaminants from contact surfaces.

Refurbishment of gates is also described in depth later in the paper but 80% of the expenditure quoted was on 10 free roller gates whose average age is 30 years.

PRESSURE RELIEF SYSTEMS

Of the 49 dams, 33 have provision for the relief of uplift but only about 10 have been designed and constructed such that the pressure relief systems are readily accessible for maintenance. Typical examples of drainage systems are given in Figures 1-3. Evidence of slow deterioration has come to light during regular inspections within the last decade. Where it is suspected that drainage is less effective than it should be, a thorough testing procedure is undertaken to identify blockages and to determine whether they should be removed or bypasses established. The whole procedure for testing, which has been developed from experience, depends firstly upon a study or all known information about the system. The next stage requires observation of tell-tale standing water levels and leakages at the dam. In the third stage a flow of water, provided by a temporary flexible hose arrangement, is passed into each inlet or outlet point of the drainage system, one at a time, and observations made of the resulting outflows. Lastly observations are made of any improvements that have taken place. Finally the information from the field and the information from the drawings is combined to assess the overall picture. From this it is possible to determine where failures have occurred, to propose clearing methods and to specify any new holes that may be required. To date reports have been compiled on 15 dams.

Drain clearing contracts have been undertaken on 12 dams. Straight holes can be cleared by conventional drilling, even from small galleries, provided that there is sufficient clearance to the wall for a small drill rig. However tests and remedial works have revealed that drains, rather than being designed and constructed as well graded runs of straight pipes are often curved, have bends and are badly misaligned at joints. In addition holes have been found to be blocked with constructional materials such as timber, steel and grout as well as deposits of organic material, gravel and calcium carbonate. In order to attempt to clear such drains two contractors have imported flexible "down the hole" drilling equipment. This equipment can clear blockages down holes caused by deposits of calcium carbonate and, in conjunction with pressure water jetting, gravel and timber but it cannot deal with bends, badly misaligned joints and blockages by grout or steel. Therefore in some instances new holes have had to be drilled to intercept a rubble drain or replace a blocked hole.

Pressure relief systems require regular maintenance: the use of normal equipment such as drain rods is not always capable of keeping drains clear, but flushing the system with water can clear soft deposits even from otherwise inaccessible situations. The continual de-position of calcium carbonate is one of the problems of maintenance. However this can be prevented by neutralisation with reservoir water which is naturally acidic as it comes from peat covered upland catchments. Accordingly a system for passing a continuous flow of reservoir water through the drains has been installed at 3 dams.

After drain clearing has been completed the policy is to rod all inlet and outlet pipes every 1 or 2 years and to flush the system about every 10 years.

Thus where pressure relief systems are required, design and construction should incorporate accessibility for maintenance, drains should be comprehensively tested prior to impounding and their arrangements and details should be fully and accurately shown on record drawings. Drains through concrete should not be less than 150 mm in diameter, preferably larger, and go straight from the toe or low level gallery to the upstream rubble drain. Consideration should be given to locating an inspection gallery at rock foundation level. The possibility of avoiding the need for any system at buttress dams should also be fully considered.

REFURBISHMENT OF GATES

Associated with the larger dams there are over 150 gates of various types excluding single face sluices. For flood control purposes drum gates are up to 27 m long by 5 m high, radial gates range to 8 m square and vertical lift gates to 7.6 m square. Most of the gates were constructed when stainless steels were practically unobtainable, or prohibitively expensive and when long life coatings were in their infancy.

As a result of the Board's policy for testing, inspection and maintenance of gates it was realised that many of the gates were approaching a state when refurbishment was required. Typically this policy requires gates to be tested during commissioning and every 25 years, partially exercised every 3 months, fully exercised annually and refurbished as found necessary every 25 years. Refurbishment is required particularly in the case of the more complicated types such as Free Rolling Gates, a section through which is shown in Figure 4.

A pre-requisite in a refurbishment contract is the removal of the gate and this can be a major item when the gate is at the foot of a shaft, say 50 m deep, with no provision for lifting it out. The problem is compounded if a temporary gate has to be provided to avoid draining the reservoir. In general all wearing parts are renewed in stainless steel and skin plates, whether new or old, and other mild steel items are prepared and coated with a coal tar/epoxy resin based coating. The object is to eliminate the products of corrosion, because their build up can jamb rollers, to provide more durable wearing surfaces and to extend the periods between major maintenance. However experience has shown that it is not easy to renew fixed paths to suitable tolerances, due to the manner in which many gate frames were manufactured and the lengths over which very accurate measurements have to be taken in confined and damp site conditions.

On an operational reservoir where there is no secondary gate the renovation of a control gate or a bottom draw-off gate would be greatly simplified if suitable grooves and lifting point for a temporary gate were incorporated in the design. These provisions should be specifically detailed on record drawings.

Radial gates have required little maintenance apart from painting and renewal of steel wire ropes. Drum gates require comparatively much more painting but again require little maintenance apart from ensuring that all small bore pipework is clear and that the control mechanism and valve are operational. However if situated on a river, rather than a loch, cleaning of the screens on the control inlet can be a time consuming but vital necessity. Seals require renewal after about 25 years.

CONCLUSION

That 49 dams with an average age of 44 years should require so little maintenance is a credit to the 10 design organisations and 21 contractors involved and suggests that the dams will last very much longer than their amortization life of 80 years. Although it may not be strictly correct to extrapolate the £875,000 spent in the last 5 years to £175,000 per annum for concrete dams in the future, nevertheless this compares favourably with a total annual expenditure for the maintenance and improvement of all civil and building works associated with the Board's hydro schemes of about £1.6 Million. However maintenance could be reduced in future dams by improving design details and overall quality control: not only in respect of concrete work, pressure relief systems and selection of gates but also with regard to facilities for maintenance now that we live in an era with high cost of labour and improved standards of safety.

Finally the Authors wish to thank the North of Scotland Hydro-Board for permission to present this paper and to acknowledge that it could not have been written without the help and experience of their colleagues.

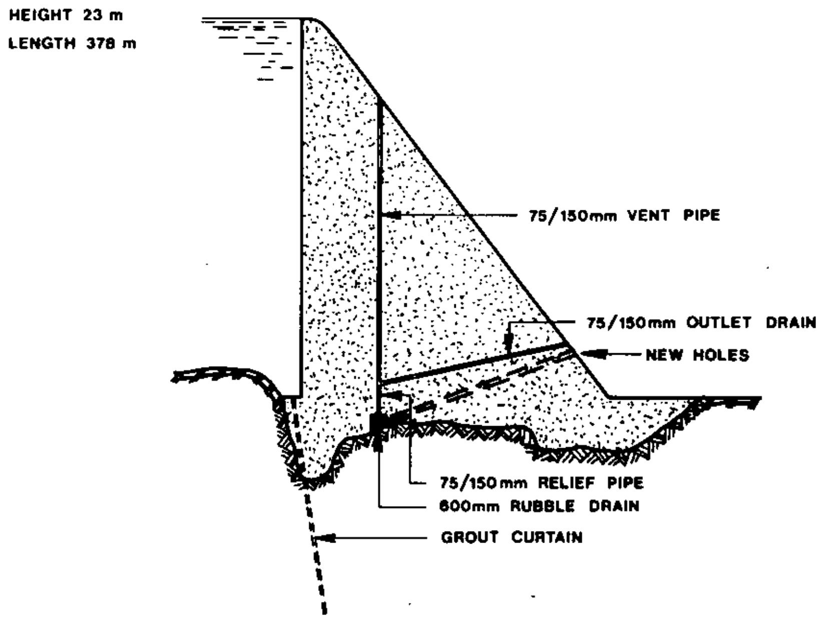

Figure 1. Glashan Dam - Drainage System

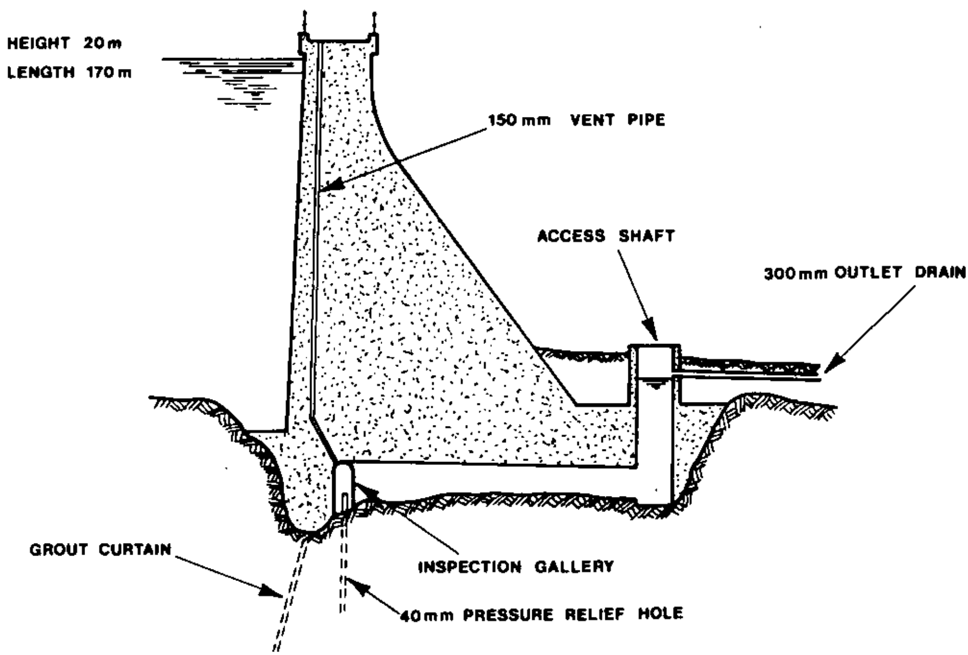

Figure 2. Loichel Dam - Drainage System

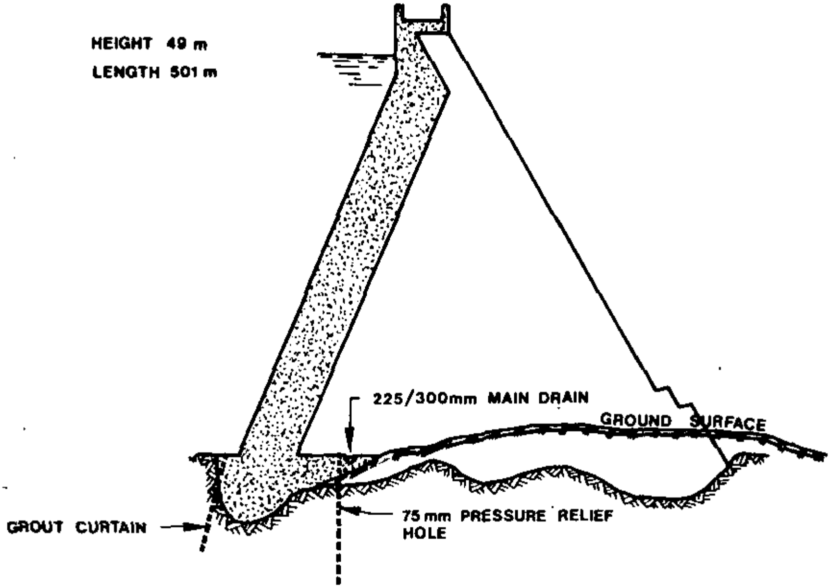

Figure 3. Errochty Dam - Drainage System

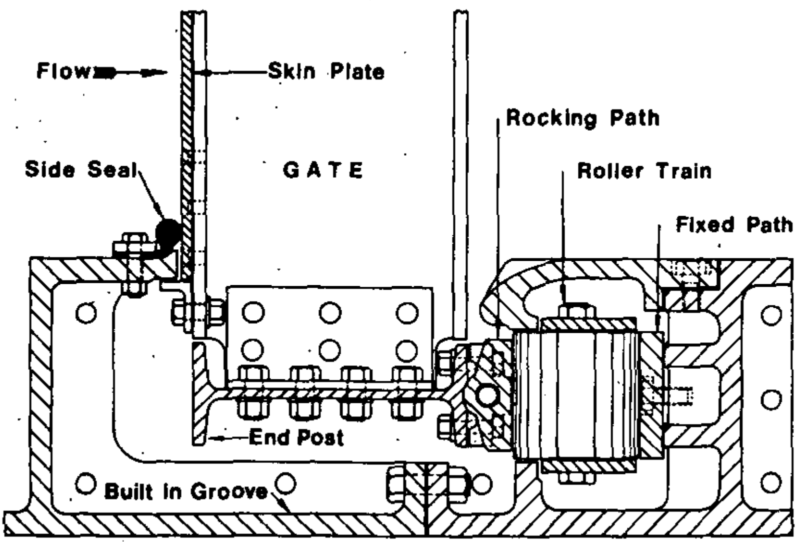

Figure 4. Free Roller Gate - Section through Groove