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CONTENTS

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Spillways and Flood Estimation

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Rofe, Kennard & Lapworth

INTRODUCTION

Since 1930 many old dams have been inspected and the spillway capacities checked by different engineers using various methods for computing the design floods and so far as can be assessed at the present time adopting somewhat different philosophies in regard to the safe passage of the flood water over the dam and back to the river channel.

Every Engineer seems to have had his own method for assessing his design flood. Although numerous formulae have been published most allowed for some factor or coefficient to be subject to the engineer's judgement. Thus different estimates were obtained by different engineers who followed the same procedure. Hence no standards developed until the "Flood Studies Report" (FSR) and the "Floods and reservoir safety - an engineering guide" (F and EG) were published [(1),(2)].

Despite the individuality of the design flood assessment some methods tended to become popular. Before the 1933 Interim Report of the Committee on floods in relation to reservoir practice 500 cusecs per 1000 acres of catchment was something of a guide for upland catchments. The Interim Report itself introduced the Normal Maximum Flood (NMF) and recommended a 'Catastrophic' Flood of twice the NMF as the ultimate. This appears to have been accepted generally subject to factors being applied to adjust for lowland or mountainous catchments as the case may be. Subsequently the factor of two was brought into question when the sub-committee on Rainfall and Run-off proposed an appendix to the Interim Report in 1959. This included a replot of Figs. 3 and 4 of the Report on a log/log basis to which data gathered since 1932 was added. The figures obtained from the Lynmouth floods[(3)] featured prominently and an enveloping line, the slope of which appears to have been fixed by two points, suggested that for small catchments a 'catastrophic' flood could exceed 4 times the NMF. This appendix does not appear to have been accepted in the same way as the Interim Report but the possibility of a flood greater than 2 x NMF was more widely realised.

BASS: SPILLWAYS AND FLOOD ESTIMATION

My Partner, P.S. Hallas, compared pre 1975 and post 1975 estimates for design floods [4] and concluded that for small catchments, less than 2,500ha (approx. 6,000 acres) the estimates based on the FSR were likely to be less than previous estimates (i.e. those made between 1960 and 1975). Conversely the FSR estimates are likely to exceed previous estimates for larger catchments but records are fewer in number. Although over the years the later estimates have generally shown an increase the latest method often shows a decrease yet in many cases reservoir owners are now faced with large expenditures to improve the capacity of the overflows.

PRESENT SITUATION

The Flood Studies Report and the engineering guide of the Institution of Civil Engineers have both been produced following very careful investigation and discussion by experts and are undoubtedly the best recipe for the safety of dams that exist. Both claim to allow scope for engineering judgement, local knowledge etc. but is it possible to really take advantage of this? The two documents were published at a time when society had little respect for professional opinion and it is therefore extremely unlikely that an engineer could recommend anything that differed from the views expressed in these documents. Also I have no knowledge of reluctance being shown by a Water Authority when two or three houses have been regarded as a community by an engineer and Category A awarded to the reservoir upstream in accordance with Table 1 of F and EG. Although it was not intended we now have a standard which is as rigid as any could be.

The older reservoirs which are now presenting problems were probably designed to pass the estimated flood under a head of about 3ft. with a similar margin of freeboard up to crest level. The purpose of the margin is not clear for in a number of reservoirs the spillway channel was restricted to the estimated flood discharge. Where the spillway channel is not so restricted the margin can be used to give an extra head for free discharge over the weir and about 2.8 times the estimated flood could be discharged. In cases where the estimated flood was equal to the NMF the margin would allow the EMF based on the FSR to be safely catered for except possibly for the larger catchments.

The wave surcharge is an additional requirement but where this can be covered by the improvement or provision of a wave wall the cost is not exorbitant. Thus the new standards have not, of themselves, introduced drastically increased design floods requiring expenditure on improved overflows but their apparent rigidity precludes approval of overflows which have been judged satisfactory in the past.

EXAMPLES

The following examples are reservoirs whose spillway channels either restrict or have a capacity less than the maximum free discharge of the overflow weir. Whilst it is accepted that some means of overcoming the problem is necessary the present 'standards' preclude any solution except provision to deal with the EMF.

Communities exist below the dams, albeit some distance down the valley, and the reservoirs therefore fall within Category A of Table 1. Computations of the EMF give very little difference between summer and winter conditions and for practical purposes they can be regarded as the same.

Details of the reservoirs are set out in Table 1. In each case reservoir lag is small and has been ignored.

Table 1

	Beacons	Cantref	Fernilee	Llwyn On
Total Catchment area km^2	7.7	17.5	20.9	43.0
Freeboard above overflow cill m	1.83	2.13	1.6	1.91
Discharge using half the freeboard. m^3/sec	73	91	93	146
As above $m^3/sec/km^2$	9.5	5,2	4.4	3.4
NMF $m^3/sec/km^2$	7.0	5.1	4.8	3.4
Max. from 1959 approx. $m^3/sec/km^2$	29.0	18.2	16.5	11.0
EMF $m^3/sec/km^2$	19.6	16.3	14.9	12.6
EMF m^3/sec	151	286	264	642

Records of overflows exist in the Forms F for nearly 50 years but in the case of Fernilee the overflows are rare and small and do not warrant analysis. For the three Welsh reservoirs the records are shown in Fig.1 in which the flood of 26th December 1979 show as outliers. The value of the plots has to be discounted by virtue of the following:-

1) Floods may occur when the reservoir is drawn down so that the overflow is no measure of a flood.
2) The length of record is insufficient to extrapolate to extreme events.
3) the lag effect of the reservoir on spates is greater than on larger floods.

Nevertheless the discharge with a return period of 1000 years are worth comparing with values in

Beacons	3.1 $m^3/sec/km^2$
Cantref	2.5 $m^3/sec/km^2$
Llwyn On	2.5 $m^3/sec/km^2$

Moreover they are substantially less than 0.3 PMF to which it is compared for

Category B reservoirs in Table 1 of the Engineering Guide.

PARTICULAR RESTRICTION OF THE STANDARD

For all categories of reservoir it is required that the reservoir be assumed to be full or overflowing at the start of the design storm. Presumably this is because an EMF type storm can occur at any time of the year but is this really the case. In Vol. II of FSR ten extreme storms are quoted and they are summarised below:-

Table 2

Date	Location	Rainfall	Duration	Avg. Rainfall Intensity
		mm	hr	mm/hr
11 July 1932	Cranwell, Lincs.	126	2	63
8 June 1957	Camelford, Cornwall	138	2½	55
4 August 1938	Torquay	152	5	30
28 June 1917	Bruton, Somerset	200	8	25
18 July 1955	Weymouth	280	15	19
15 Sept. 1968	S.E. England	190	20	9.5
26 August 1912	Norfolk	210	24	8.8
25/26 Sept. 1915	Inverness	201	40	5.0
2/3 Nov. 1931	W. Britain	244	48	5.1
20/23 July 1930	N.Y. Moor	304	96	3.2

In addition the FSR quote other major floods for which rainfall records are not so reliable:-

Table 3

Date	Location	Peak Flow	
		$m^3/sec/km^2$	mm/hr
8 August 1967	Dunsop Br	10.9	39.5
15 August 1952	Lynmouth (W. lyn)	9.4	34
5 August 1973	Cefn Brwyn	6.1	22
29 May 1920	Louth	2.7	9.8

Of the 14 major storms over half occur in July and August and 12 out of 14 occur between June and September. The average intensities for Table 2 are plotted on Fig.2 where the first 5 appear to be more severe than the last 5 by a factor of up to two and the former are all summer storms!

The above suggests that if some regard could be taken of a reservoir being drawn down when an extreme storm occurs then some overflows which are about to be improved may be found to be adequate or the reservoirs could be operated so as to make them adequate.

Although it is understood research into the seasonal occurrences of extreme storms is in-hand the results may not be available until substanial and possibly unnecessary expenditure has been incurred.

PROPOSED REMEDIAL WORKS

The improvement works proposed at the four reservoirs used as examples all leave floods of the kind that have occurred in the past unchanged for the future. In some cases this is the result of a policy decision. Allowing a reasonable margin over the greatest flood to date the proposed works only come into operation if a 1 in 300 year or thereabouts event is exceeded. If an alternative to the expense of such improvement works, which may never be brought into use, could be found this would surely be in the national interest.

Recent floods in 1979 caused damage to property downstream of Llwyn On. The return period of this flood in the lower reaches has been estimated to be of the order of 200 years. At the dam the flow was less than one quarter of the EMF. Should the latter occur there would be a national disaster no matter what happened at the dam.

CONCLUSIONS

The FSR and F and EG have not introduced a substantially higher design flood but the standards set out having regard to the present attitude of our society have effectively precluded deviation from these standards based on engineering judgement.

BASS: SPILLWAYS AND FLOOD ESTIMATION

There appears to be a strong indication that extreme storms are likely to occur in the summer. Advantage could be taken of this as the reservoir would normally be drawn down at this time. If necessary it could be drawn down to provide the required flood storage possibly without reducing the yield although it may mean foregoing the cheaper source of water in wet years.

Under present circumstances there appears to be a risk of incurring heavy expenditure which may in future be proved to have been unnecessary. To this there is no solution other than the intervention of the state to carry the insurance for disasters following extreme events.

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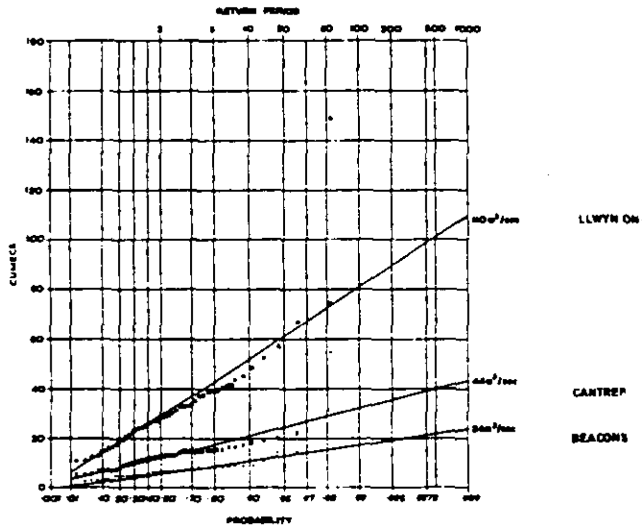

FIG 1 FREQUENCY CURVES

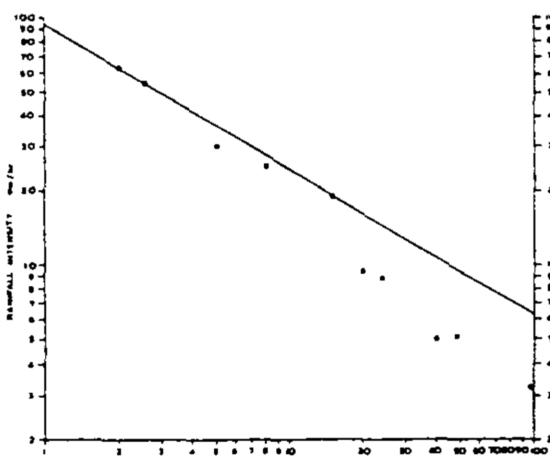

FIG 2 MAJOR STORMS FROM FLOOD STUDIES REPORT

Some Aspects of Modelling Slope Protection

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SYNOPSIS

The design of riprap slope protection depends upon empirical criteria obtained from small scale tests. The validity of model tests has often been questioned. The main criticism has been the inability to reproduce the correct Reynolds number. Recent evidence based on direct comparison of field results with model experiments reinforces the view that Reynolds number is unimportant. The work carried out has also quantified the variable nature of damage under apparently identical conditions, highlighting the probabilistic nature of the damage to such a structure, and indicating the magnitude of the scatter to be expected.

INTRODUCTION

Earth dams require adequate protection against wind generated wave attack. This protection is often in the form of riprap (graded quarry stone) laid on the upstream face of the dam. Where the impounded reservoir is large, high waves may be generated requiring a large stone size and consequently one or more sub layers or filter layers to prevent the leaching of the fine core material. The cost of slope protection is often a substantial percentage of the total cost of the structure and hence considerable efforts have been made to produce design data usually from hydraulic model studies. The bulk of such early experimental data came from regular wave tests at the Waterways Experimental Station, Vicksburg and the US Army Coastal Engineering Research Centre.

In 1962 the Civil Engineering Research Association (CERA) sponsored laboratory tests at the Hydraulics Research Station (HRS) which resulted in the publication of a report[1] giving design procedures for determining the riprap size required for a given design wave condition. This CERA work represents one of the first attempts to relate the results of tests using regular waves to those using irregular waves; in this case generated by wind.

Research on the subject continued at HRS, in collaboration with the Construction Industry Research and Information Association (CIRIA; the successor to CERA), using the then newly developed procedure of paddle generated irregular waves. The work culminated in a publication[2] which reviewed current practice under the headings of wave prediction, design procedure, design wave height, size, grade and shape of riprap, placing and thickness, filter design and run up. An extensive series of tests were made in a random wave flume in which a model riprap slope was subjected to wave attack and the resulting erosion damage measured at regular intervals. In all a combination of 4 slopes, 3 riprap sizes, 3 spectra and at least 4 waveheights were tested. A typical set of damage histories are shown in Fig 1(a) for a given slope, spectrum and riprap size. From all such results, for a given slope, design curves were produced as shown in Fig 1(b). A number of design procedures were suggested. It should be noted that some factors were kept constant throughout the test programme, eg the riprap and filter gradings and the riprap thickness. There is no definitive work on the effect of varying these parameters.

Two of the factors highlighted in the report[2] have been the subject of subsequent research at HRS and are reported in this paper. They are the problem of Reynolds scale effects and the repeatability of identical tests.

REYNOLDS NUMBER SCALE EFFECTS

In open boundary hydraulics research, phenomena primarily dependent on gravitational and inertial effects are modelled according to the Froudian laws. Secondary forces arise when water is the fluid in both prototype and model, because, for example, surface tension and fluid viscosity are not correctly modelled. Scale effects arise when these secondary forces become significant. For the research under discussion lift and drag on individual stones could be dependent on Reynolds number, which is a measure of the relative importance of inertia and viscous forces. Thus at the low Reynolds number of a small scale laboratory study the lift and drag forces may be larger than is the case under strict dynamic similarity. If significant this would result in the model showing too much damage and hence lead to overdesigning.

However, the tests at HRS did not indicate any Reynolds scale effect and consequently no allowance for this was recommended in the design procedures when using the model data for full scale design(2). On the other hand, Reynolds scale effects were claimed by Thomsen et al(3). They reviewed small scale and large scale research in the USA and concluded that only in the largest scale tests were the results free from Reynolds scale effects. Their data suggests that in the HRS tests(2), which covered a sixfold variation in Reynolds number, there should be a 30% scale effect in the stability number, N_{ZD}, at near zero damage; an effect which should have been detectable but which was not found. Their work also suggests that the armour size at full scale designed for minimal damage on the basis of the HRS tests could be up to 60% oversize. Before this potential saving can be realized the conflict between the work of Thomsen et al(3) and HRS(2) had to be resolved. This has been attempted along two lines: first by retrospectively testing field measured riprap damage and secondly by extending the original model test programme of reference 2 to higher and lower Reynolds numbers.

Field Measured Riprap Damage

Field measurements of riprap damage and the associated wave and tidal conditions were made by Messrs Binnie and Partners on contract to CIRIA in conjunction with the trial construction of an offshore embankment in the Wash estuary[4]. The embankment is trapezoidal in section and circular in plan with a maximum crest elevation of 14 m OD; the seabed elevation being -0.75 m OD. The outer 1 on 4 slope is armoured up to the 7.5 m OD level with riprap having a D_{50} size of 0.66 m over the seaward facing sector and with smaller armour over the landward facing sector. Four test panels of smaller sized riprap, each 6.5 m wide and extending from 1 m OD to 7.5 m OD were laid on a prepared section of the main seaward facing armour. This underlying main armour was blinded with coarse filter and then covered with an impervious fabric on which the test filter and riprap panels were laid. The test panels (1-4) had D_{50} sizes of 0.23, 0.40, 0.50 and 0.56 m respectively. Two pressure transducers mounted seaward of the toe of the test panels and recording to paper chart were used to measure the waves. They were automatically timed to give four records over each high tide.

The test riprap panels were completed in November 1975 and panels 1 and 2 surveyed. A storm then occurred, with a maximum measured value of $\overline{H}_3$ of 1.03 m which destroyed panel 1 and slightly damaged panel 2. Surveys were then made of the four panels and of an area of the main embankment armour adjacent to the panels and designated panel 5. Thereafter regular site visits were made, to survey the panels and to collect other data, until March 1978 when panels 2, 3 and 4 were found to have been destroyed. This occurred during a severe storm in January 1978 when a maximum $\overline{H}_3$ of 2.14 m was measured. Between these two storms $\overline{H}_3$ exceeded 1 m on only two occasions. In one a value of 1.01 m was measured and in the other a value of 1.4 m was hindcast from wind data; both wave recorders being out of action at the time.

Comparison of the field measured damage against that predicted by reference 2 did not reveal any scale effects.

The Retrospective Model Tests

There were 103 wave/tide events measured on the field in which $\overline{H}_3$ exceeded 0.5 m. Clearly it was impractical to reproduce all on the model. An examination of the field data showed that the test programme could be limited to four wave/tide events; three covering the first storm and one covering the second storm[4].

The model was built to a scale of 1:17 giving stone sizes covering the range of the original HRS work[2] and ensuring a range of Reynolds numbers for which the work of Thomsen et al[3] predicted significant scale effects. The test panels were laid out as shown schematically in fig 2 in a 6 m wide random wave flume. Riprap panels 1-4 were tested in a 4 m wide section of the flume at an angle to the incident waves whereas panel 5, the main embankment armour, was tested separately in an 0.61 m wide sub-flume. The modelled riprap panels 1-4 prior to testing are shown in Fig 3. The field wave/tide events were simulated on the model by a stepped tide; each tidal step lasting 5 minutes (model) and having the appropriate wave-heights determined by interpolation from the field data. The field wave data was analysed spectrally and the model spectrum reproduced using the HRS wave spectrum synthesizer[6]. The model waves were measured at the surface by twin wire resistance probes and at the scaled depth by a pressure transducer.

The model riprap was surveyed prior to the test run and after each wave/tide event. The resulting damage parameters, N_D, are shown in table 1 below together with the corresponding values measured in the field.

	Storm 1				Storm 2	
	Model			Field	Model	Field
Wave/tide	1	2	3		4	
Panel	Measured damage parameter N_D					
1	90	177	1064	839		
2	14	14	27	83	333	
3	9	11	13		179	231
4	4	10	14		81	162
5			4		13	12

Table 1 Measured field and model damage

Discussion

During storm 1 erosion of panel 1 continued steadily and by the end of wave/tide event 2 the eroded area was well within the field measured scour hole of storm 1 and since the final wave/tide event had only slightly higher waveheights but a lower maximum tidal level a good reproduction of damage was expected. However, during the third wave/tide event the model erosion reached the smooth support floor and this allowed greater run up and rapid erosion of the riprap higher up slope so that the final erosion of panel 1 at the end of storm 1 exceeded the field measured values both in quantity and extent. In the field the impervious fabric layer was torn away so that the uprushing and downrushing water was influenced by the roughness and permeability of the underlying main embankment riprap; a factor not included in the model tests. As a result the major damage on the model to panels 2, 3 and 4 during storm 2 occurred from cross slope undermining of the riprap rather than by surface erosion. Unfortunately no corresponding field observations are available. The results from the test on panel 5, which was not affected by undermining, and from the first two wave/tide events on panel 1 do not indicate any Reynolds scale effect.

Retrospective model tests were made by Tørum et al [7] of the Bilboa breakwater for which simultaneous records of waves and damage were available. This breakwater is armoured with 65 T concrete blocks. They found the final damage to the model armour to be in fair agreement with the field measurements. In particular the model measured damage never exceeded that measured in the field; a finding which suggests that scale or model effects, if any, do not necessarily over estimate the damage. Limited information about the progress of the field damage precluded any conclusion being reached by them about the influence of scale and/or model effects on zero-damage stability. By model effects they mean a failure to correctly reproduce on the model any of the field phenomenon; for example stone grading, stone shape, wave climate, etc. In addition they reassessed the work of Thomsen et al [3] and others and concluded that the apparent Reynolds scale effect could possibly be explained in terms of model effects or a wave period effect. Thus there is some agreement between the work of Tørum et al and HRS.

Extension of Original HRS Tests

An extension of the previous HRS work [2] was made by taking a particular test from the original series and making two models of it; one as small as practicably possible and the other as large as possible within the limits of the random wave flumes at HRS. Following the procedures of the original work ensured that only the Reynolds number was changed between the three models. The results, plotted on Fig 4, show no obvious trend with Reynolds number at low damage values. At near zero-damage there was a 13fold range in Reynolds number between the smallest and largest models which on the basis of the work of Thomsen et al [3] should show an 80% Reynolds scale effect in the stability number. This means that if on Fig 4 the results from the smallest model tend to zero damage at $\overline{H}_3/D_{50} = 1.0$ then the results from the larger model should tend to zero damage at $\overline{H}_3/D_{50} = 1.8$. Manifestly this is not so and thus the conclusion of the earlier HRS work(2) that no allowance could be made for Reynolds scale effect when using the model data for full scale design is confirmed. There is some indication that at high damage rates the small model may underestimate the damage.

THE REPEATABILITY OF IDENTICAL TESTS

The earlier work at HRS(2) on riprap suggested that the scatter of the data in the proposed design curves was probably due to inherent randomness in the phenomenon but no significant testing was made. Subsequently a limited number of tests have been made at HRS both on riprap and on dolosse armoured slopes.

The repeat tests on riprap were made with the smallest riprap used for the extended Reynolds number tests described above. Five repeat tests were made at a value of $\overline{H}_3/D_{50} = 3$; a value giving significant damage. The resulting damage history curves, shown in Fig 5, show an approximate twofold spread in the results.

The repeat tests on dolosse armour units were made on a 1:1.5 slope at three different waveheights giving minimal, moderate and severe damage. For each waveheight ten repeat tests were made. The damage history curves for the highest waveheight are shown in Fig 6, and Fig 7 shows the mean and standard deviation for each waveheight plotted non-dimensionally.

The nonrepeatability of identical tests is an important factor in both model testing and the use of model data for full scale design. For example it is necessary to ensure that an effect being investigated is not masked by the inherent scatter of the data. This can be illustrated by a point recently under investigation at HRS: "does wave directionality affect the stability of dolosse armoured slopes?" Five repeat tests were made with both directional and nondirectional waves and the resulting damage histories are shown in Fig 8. The results are well scattered but cover approximately the same range for both cases suggesting that the effect of directionality is probably small. However, to establish the dependance of damage upon directionality with any degree of confidence would require a very much larger number of repeat tests for each condition. It can readily be seen that the not uncommon procedure of "two quick tests" could give very misleading conclusions. In the same way the practice of "one test of the final design at the design wave height" can also be misleading. It is the author's conviction that many design assumptions, based on limited data, need to be reassessed and that provision should be made in any model test programme for extensive repeat tests.[8]

CONCLUSIONS

There is no evidence to suggest that models do not give adequate design information. The most recent work at HRS and Norway has shown, by comparing model prediction with full scale observations, that the Reynolds number effect is insignificant.

The important question in the design of slope protection is the variable nature of damage under apparently identical conditions. This added dimension to the probability of damage is not accounted for in present design methods. However, it is inherently included along with other probabilistic factors in properly conducted model tests, and can be identified.

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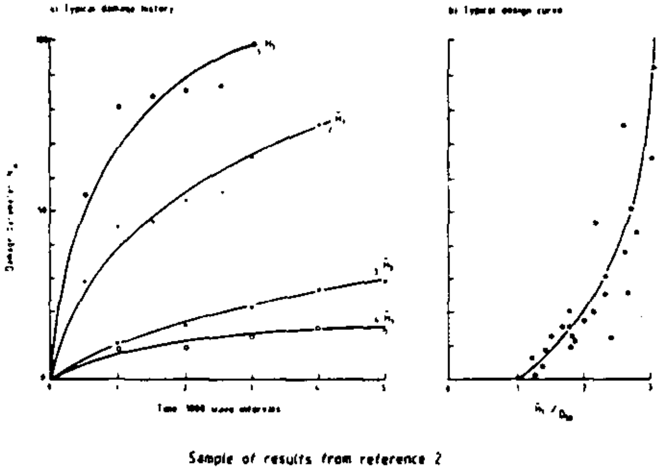


Fig 1 Sample of Results From Reference 2

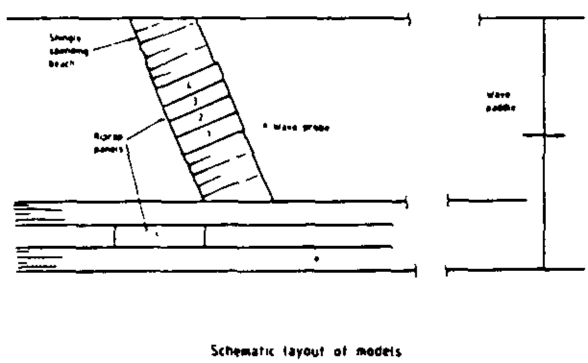


Fig 2 Schematic Layout of Models

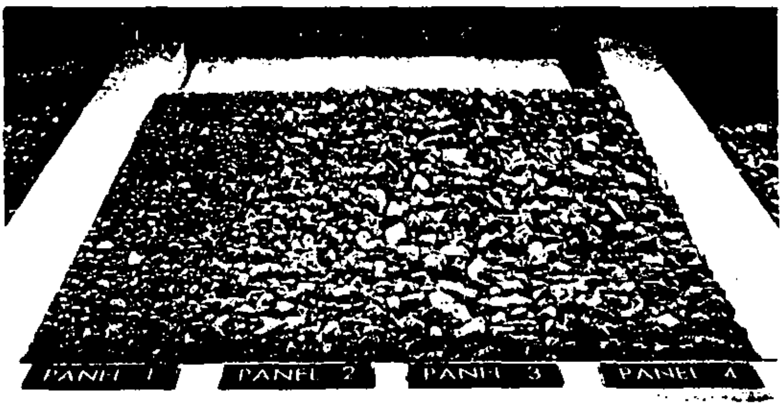


Fig 3 The Modelled Riprap Panels Prior to Testing

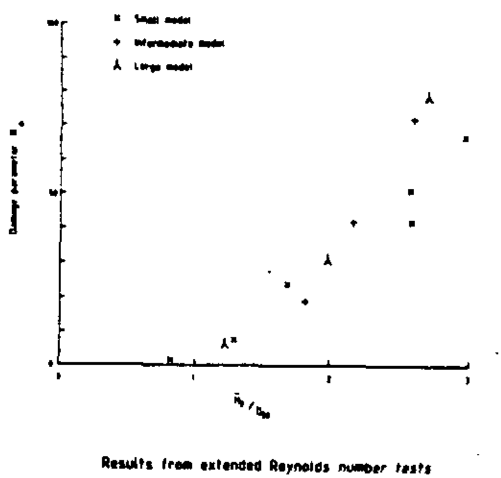
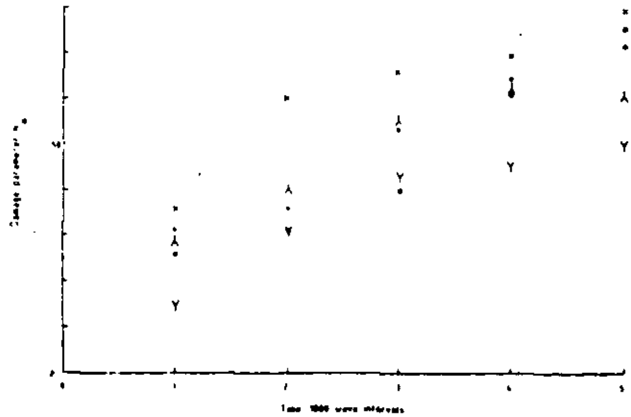
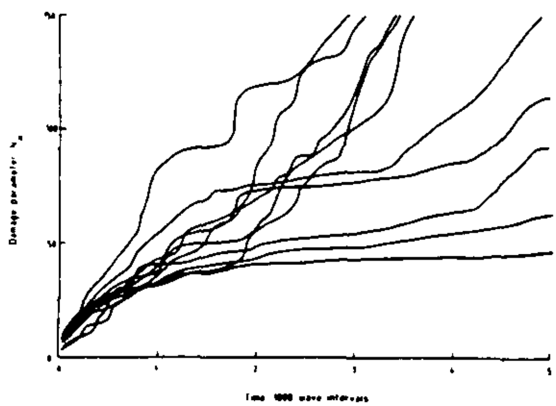


Fig 4 Results from Extended Reynolds Number Tests



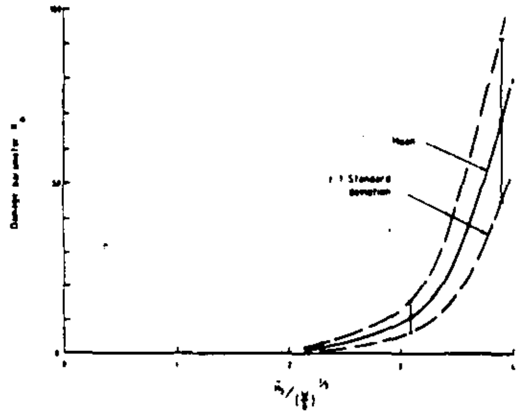
Results from 5 identical tests on riprap

Fig 5 Results From 5 Identical Tests on Riprap



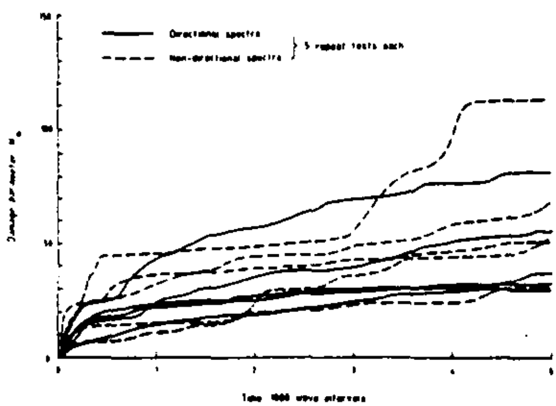
Results from 10 identical tests on dolosse armour

Fig 6 Results From 10 Identical Tests on Dolosse Armour



Dimensionless damage curve for dolosse armour showing inherent scatter

Fig 7 Dimensionless Damage Curve for Dolosse Armour Showing Inherent Scatter



Repeat directionality tests on dolosse armour

Fig 8 Repeat Directionality Tests on Dolosse Armour

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Remedial Works to Puddle Clay Cores

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SYNOPSIS

Several methods of dealing with leakage through the puddle clay cores of reservoirs have been considered and trials made on two reservoirs, namely Banbury (1972 and 1979) and Lockwood (1979 and 1980) in North London, see figure 1. This paper presents the alternative renovation options initially considered for Banbury reservoir and describes the method selected. The derivation of other options from this method are also discussed. An evaluation of each method is made and conclusions drawn.

INTRODUCTION

For many years, leakages have been observed from some of the older storage reservoirs of the Thames Water Authority located in the Lee Valley. These reservoirs are formed of an encircling embankment with a central puddle clay core which extends to the London clay stratum, see figure 2. The puddle clay is a silty alluvial clay which is known to contain fissures throughout its full depth. Staining on fissure surfaces has indicated a seepage of water through them. It is also known that, as a result of lowering the water level in the reservoirs during the period 1939-45, there has been some drying out and cracking of the clay core in its upper regions and for this reason it was suspected that the major part of the leakage was

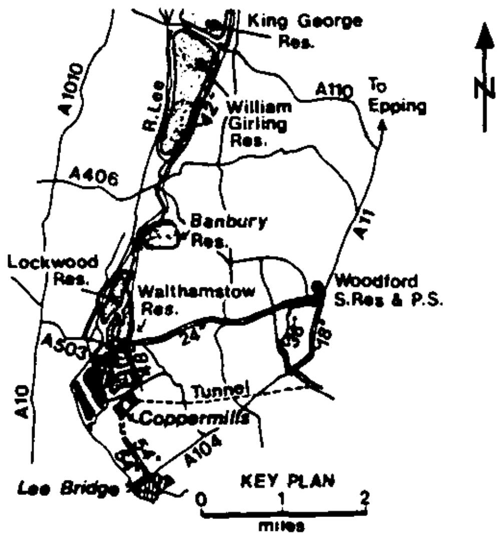


Fig. 1 KEY PLAN OF LEE VALLEY RESERVOIRS

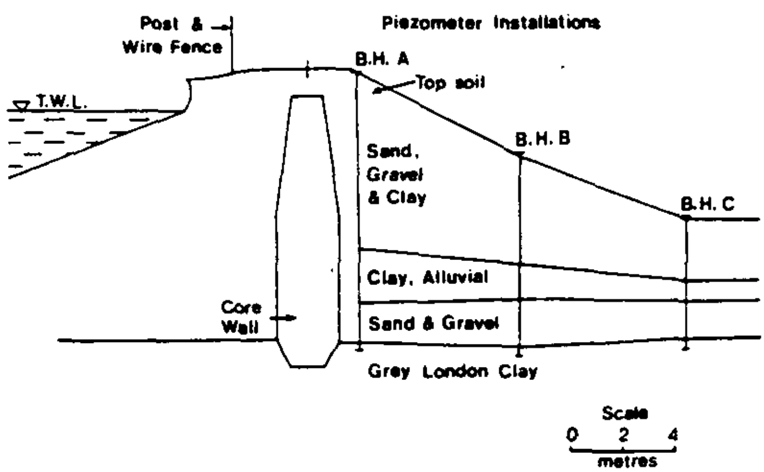


Fig. 2 TYPICAL CROSS SECTION

occurring in the upper regions of the clay core. However, boreholes in the core showed penetration at lower levels and in one case there was some indication that leakage could be occurring at the junction of the clay core and underlying London clay. For this reason, the trials were directed towards developing a technique which could be applied throughout the full depth of the core if necessary.

The leakage was evidenced by wet patches on the reservoir banks and often by boggy areas near the toe of the embankment where the ground is poorly drained. For many years it has been known that leakages are significantly reduced by relatively small reductions in reservoir level from top water. Consequently several reservoirs have been operating at restricted top water levels for several years in order to maintain reservoir safety and to reduce leakages to acceptable amounts.

RENOVATION OPTIONS

Banbury reservoir, capacity 2700 Ml and inaugurated in 1903, was the first to undergo substantial remedial works to improve the performance of the puddle clay core. Prior investigations into the leakage involved the excavation of trial pits to examine the condition of the puddle clay core and the construction of a number of boreholes vertically through the core. Continuous 'undisturbed' samples were obtained and leakages of water into the boreholes observed. Measurements of moisture content, shear strength and Atterburg limits were made, along with a very close visual examination of the samples recovered. In many instances, standpipe piezometers were installed into the boreholes to monitor leakage. There have also been attempts to monitor directly the quantity of leakage through the puddle clay core but the lack of toe drains around the reservoir and the problem of separating leakage flows from groundwater and rainfall presented difficulties.

In 1971, a number of methods were considered for dealing with the leakage at Banbury reservoir.

a) A bentonite slurry trench approximately 450mm wide might be excavated and filled with a bentonite/cement/gravel mixture. This method has the advantage that a substantial width of screen is provided of reasonably high strength but the disadvantage of handling large imported quantities of gravel and disposing of the spoil.

b) A conventional sheet pile wall could be constructed within the clay core. The main disadvantage of this was thought to be the leakage through the clutches which could be overcome but which would increase the already very high relative cost. On the other hand, it was also of concern that the introduction of a totally impervious barrier in the puddle clay could lead to further dehydration and deterioration of the core material downstream of the barrier.

c) The clay core and/or shoulder materials could be chemically grouted. The major disadvantage to this method lies in the reliance upon the grout to reach and positively fill the open fissures. This contrasts with other methods which involve physically cutting across the fissures.

d) Electrokinetic grouting employing the phenomenon of electro-osmosis was considered but this would have required substantial laboratory work to develop a suitable technique that could be used economically. This method, whilst of considerable interest, was felt not to offer sufficient certainty of producing a practical solution and the consequence of failure would have been unnacceptable delay in tackling the problems on operational reservoirs.

e) A thin grout screen could be formed in the core wall using a suitably formulated grout. The advantages of this method are that driving the pile(s) to place the grout would sever fissures and cause some remoulding of the clay. The grout itself would also present a physical barrier and some filling of fissures could be expected. The main disadvantage was the need to formulate the grout.

The latter alternative was selected as being the most suitable provided a grout could be designed which, ideally, possessed properties as close as practicable to those of the puddle clay.

GROUT SCREEN TECHNIQUE

Design

A grout screen installed as temporary works during the construction of Datchet intake as a means of dealing with ground water had proved successful. An inspection of the screen and consultation with the specialist contractor involved had led to the conclusion that a minimum practical thickness for the grout screen to be installed in puddle clay would be 50mm. In order to monitor the effects of the remedial works it was necessary to install piezometers in the outer embankment. In view of the uncertainty of the depth of screen required, it was decided to construct sections of different depths in the range 4 to 7 metres for comparison. There was also a constraint on depth whilst operating beneath overhead power lines.

In formulating the grout, the aims were that

a) the grout should develop an early shear strength not less than the puddle clay, say 35 KN/m^2 at 7 days

b) it should have a permeability comparable with that of the puddle clay of the order of 10^{-9} m/s and ensure that the puddle clay on the downstream side of the screen should not dehydrate

c) it should be durable and should not deteriorate due to gain or loss of moisture to an extent which would cause it to shrink or become more permeable

d) it should be flexible and able to withstand minor ground movements

e) the suspension should be pumpable and not subject to segregation when standing

f) in service, the grout must be resistant to chemical attack.

An extensive programme of tests was carried out at the Authority's engineering laboratories and the conclusion of these was the specification of a grout with the following constituents and proportions:-

Ordinary Portland cement	4.2% by weight
PFA	16.8
Bentonite	5.6
Kaolinite	16.8
Water	56.0
Sodium tripolyphosphate	0.3
Methyl acetate	0.3
	100.0

Construction

A 450 metre length of core wall, in two sections, where leakage had been identified was selected for the initial trial using the proprietary vibrated screen process. Construction began in November 1972 with the plant arrangement shown diagrammatically in figure 3(a). The mixing and pumping plant were stationary at the toe of the embankment while the power generator for the vibrator with integral control panel was towed by the crane as grouting progressed.

A high shear bentonite mixer was used to disperse the bentonite and kaolinite in the water containing sodium tripolyphosphate. The suspension was then transferred to a low speed paddle mixer where the remaining constituents were added. With minimum delay, the mix was pumped by an air driven double-acting ram pump through a 38mm diameter pipeline to the pile forming the screen. The maximum pumping distance attained was approximately 250 metres. The screen was formed by vibrating a single vertical I-section steel pile into the centre of the core wall. Grout was injected through a 25mm diameter tube welded to the pile and discharging at the bottom through a special shaped shoe into the void formed as the pile was withdrawn as shown in figure 3(b). This process was repeated with an overlap between successive insertions of the pile into the clay core.

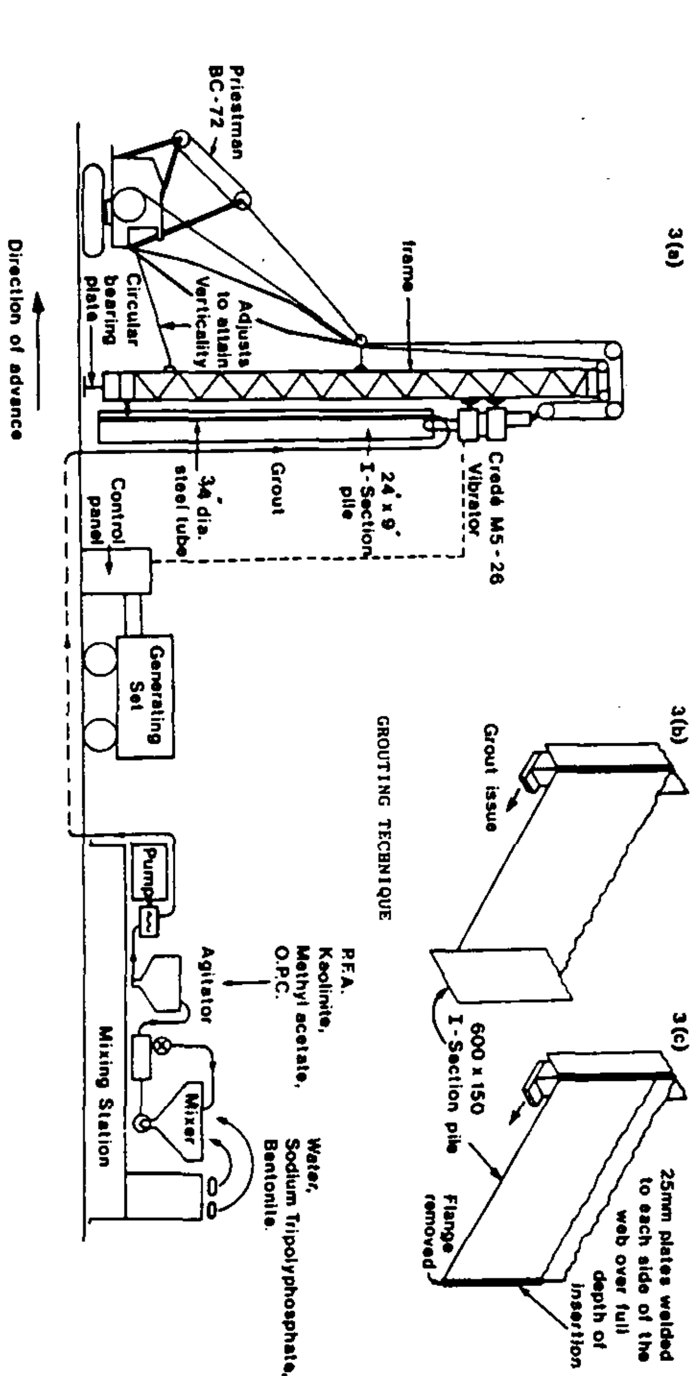


Fig. 3 BANBURY RESERVOIR TRIAL GROUT SCREEN DIAGRAMMATIC PLANT LAYOUT

It was important that there were no delays whilst delivering the grout to the pile because the grout tended to gel very rapidly. The pipeline always had to be cleared with compressed air when delays occurred to avoid a blockage. Weaknesses in flexible sections of the pipeline gave problems initially but once a good working routine was established it was found possible to achieve a satisfactory level of production.

Shortly after construction a programme of continuous sampling of the screen began, to examine the grout screen as placed. It was revealed that in driving the pile, the thickness of the grout in the preceding part of the screen was being reduced by clay being displaced by the pile and that the required thickness of screen was not therefore being achieved. Some of the displaced grout was being driven back to the surface. To remedy this, the trailing flange of the pile was removed and the web thickness of the pile increased by welding on plates, see figure 3(c).

Throughout the construction period, control of the grout mix was achieved by taking samples for site tests which included the determination of moisture content, immediate shear strength, viscosity and wet density. Samples were also sent to the Authority's engineering laboratories for immediate and long-term permeability tests.

Work on site was completed in February 1973. During the course of the Contract some 2400 square metres of grout screen were placed.

Performance and Evaluation

It was recognised that, apart from the formation of the grout screen, the performance of the puddle core would be improved through the severing of leakage paths and remoulding of the clay by the vibrated action of the

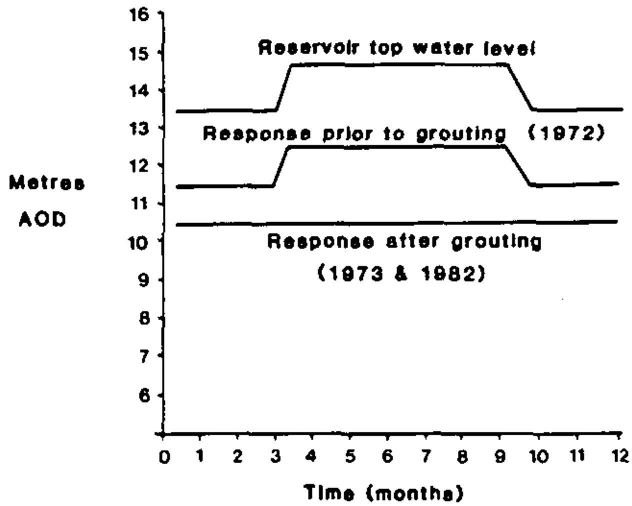


Fig. 4 DIAGRAM SHOWING EFFECT ON WATER LEVELS IN EMBANKMENT BEFORE AND AFTER REMEDIAL WORKS

pile. It was also observed that large voids and fissures away from the centre of the core wall had been penetrated by the grout. On many occasions, grout was seen issuing from the ground surface several metres from the point of injection.

Whilst the pumping and mixing plant was being relocated to treat a second section of the core wall, the reservoir level, which had been lowered 3.3 m during construction of the screen, was raised to test the effectiveness of the screen. It was apparent that piezometric levels in the embankment were being satisfactorily reduced. In the long-term this situation has been sustained and is represented diagrammatically in figure 4.

Samples of the grout have been recovered over an 8 year period to monitor its durability. There has been no significant change in any of the properties of the grout.

REMOULDING TECHNIQUE

Background

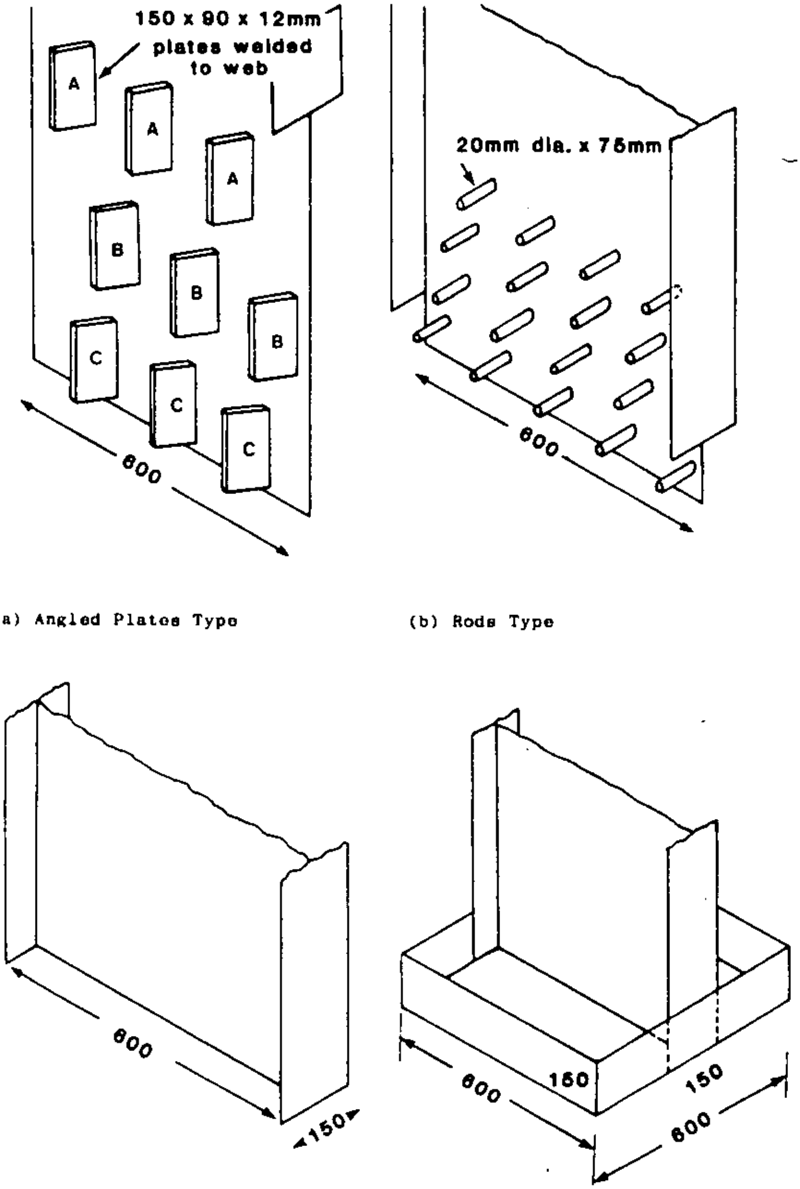
As mentioned previously, the borings made in the completed grout screen revealed that the specified thickness of grout screen was not being achieved with the original pile design. The full scale tests, however, demonstrated that leakages had effectively been reduced over the entire lengths treated. In places there was little more than a trace of grout following treatment using the unmodified pile section. It was thus concluded that the action of the pile in severing fissures and remoulding the clay might, in itself, without the introduction of a grout, be sufficient to reduce leakage through the clay core to an acceptable level.

In 1977, further leakages were detected at Banbury at a different part of the embankment to that treated in 1973. A number of piezometers were installed in the outer embankment over the length concerned and readings taken daily whilst the water level in the reservoir was varied. It was shown that piezometric levels in the embankment responded clearly to the changes in reservoir water level.

To reduce the leakages, it was decided to test the conclusion drawn from the results of the earlier remedial works at this reservoir and then to extend the works to deal with defective areas of the clay core at Lockwood reservoir which had been under observation for some considerable period.

Methods and Construction

Four designs of pile were considered for trial at Banbury reservoir and these are shown on figure 5. The method proposed was simply to drive and



(a) Angled Plates Type

(b) Rods Type

(c) Plain 'I' Section Type

(d) Box Head Type

Fig. 5 PILE TYPES - REMOULDING TECHNIQUE extract the pile, using a vibrating hammer, to an approximate depth of 7 m. along a line through the centre of the core wall. Successive drives, as with the grout screen, would allow a minimum overlap to ensure continuity of the treated areas.

In February 1979, a 220 metre length of the core wall was identified for treatment and a contract was let which provided for testing each of the four designs. The initial tests showed that the major problem with the 'angled plates', figure 5(a), and 'rods', figure 5(b) types was the large mass of clay which remained on the pile upon extraction. For this reason the 'box-head' type was preferred. Even though a certain amount of clay had remained on the pile during extraction, it was found that this could be reduced by lubricating the pile with water and work proceeded on this basis. However, as work progressed, appreciable ground settlements were observed to occur within the 'box' section as it was being extracted. It was thought that these settlements were the result of a mobilisation of the puddle clay, with a consolidation of the clay resulting in relatively large displacements at the ground surface. It was observed that the volume of the displacement approximated to the volume of the inserted pile.

Performance and Evaluation

As with the earlier works to install a grout screen, a number of boreholes were made into the clay core across the width treated and continuous 'undisturbed' samples were recovered. The line of the pile was not easily identified and consequently, the results of the visual examination of the samples could not be interpreted with any great confidence. There was an indication of a thin band of remoulded clay coincident with the pile web but no positive evidence that remoulding had occurred within the box sections or along the line of the upstream and downstream plates of the box section.

Water was located in the boreholes made on the upstream side of the pile which also seemed to indicate that the remoulding was achieved mainly by the continuous web of the pile and that the short plates of the box section were relatively ineffective. Based upon this observation, the work which followed in November 1979 at Lockwood reservoir to renovate a 250 metre length of core wall was carried out using the plain I-section pile, as figure 5(c), driven to a depth of 5 m.

At both reservoirs, the effectiveness of the remedial works was tested by raising the water level in the reservoirs from the temporary restricted levels to the designed top water levels. In the case of Banbury reservoir there was no measurable response from the piezometers and this situation has been sustained. However, the test at Lockwood reservoir was delayed for approximately 12 months while a further method of renovation, using sheet piles, was tried at a different section of the embankment. When the test was eventually made, in November 1980, there was a slight response in two of the piezometers installed over the length of the remoulded works which indicated that the treatment was not quite so effective. However, there were no visible signs of leakage and in practical terms there was a considerable improvement.

SHEET PILING TECHNIQUE

On the west side of Lockwood reservoir, a particularly serious leakage area had been under observation for some years. During tests prior to the remoulding works, when the reservoir level was raised above its restricted top water level, a very large area of ground at the embankment toe rapidly flooded. Consequently, the reservoir level was kept lowered.

It was considered, that in this case, a method providing a more positive seal against leakage should be tried. It was decided to use 1 m. wide corrugated sheet piles made from asbestos cement to a 9.5 m. thickness driven into the clay core, see figure 6.

Work began in January 1980, after some delays in obtaining the specified asbestos sheets. Problems soon arose during driving when some of the asbestos sheets split and subsequently shattered before achieving the specified depth. Modifications to the piling method failed to provide a complete solution and eventually a reduced depth of penetration of 2.5 metres had to be accepted. Where this could not be achieved, steel trench sheets were substituted for the asbestos sheets. For all the difficulties encountered, the method was shown initially to be effective but due to the presence of other leakage areas, it has not been possible to sustain a higher reservoir water level to make a full evaluation. It had been thought that there might be an advantage in this method of being able to extend and redrive the asbestos sheet piles to a lower level should the need arise. With the difficulties encountered on the initial drive, it was apparent that this would not be feasible.

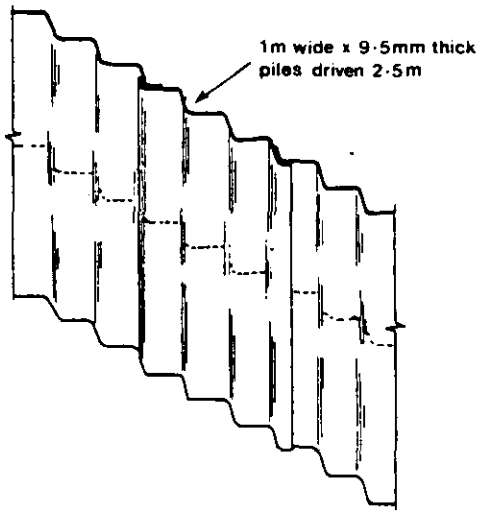


Fig. 6 ISOMETRIC VIEW SHOWING OVERLAPPING ASBESTOS CEMENT SHEETS

CONCLUSIONS

The relative costs (adjusted to 1982 price levels) of the various methods tried to improve the performance of puddle clay cores are as follows:-

Date	Reservoir	Method	Area treated (m^2)	Cost/m^2 (£)
1972/3	Banbury	Grout screen	2,300	40.00
1979	Banbury	Remoulding	1,725	18.00
1979	Lockwood	Remoulding	1,500	17.00
1980	Lockwood	Driven asbestos sheets	950	28.00

By far the most expensive method has been the installation of a grout screen. In assessing which method is appropriate for a particular case, consideration must be given to the reason for carrying out the works; whether as a matter of reservoir safety or as a means of recovering a lost volume of raw water storage.

A typical cost for providing raw water storage within Thames Water is in the order of £500/Ml at 1982 price levels . It is therefore a simple matter to assess the cost effectiveness of carrying out remedial works solely to recover lost storage, which of course varies with the area to be treated in relation to the proportions of the reservoir.

An objective of the trials has been to identify the most practical and most economic method of dealing with leakages at King George's reservoir inaugurated in 1913 and with a maximum capacity of 12,400 Ml. Since 1946, the reservoir has been maintained at a restricted top water level resulting in a lost volume of storage of approximately 1570 Ml. Areas where leakage is known to exist have been under surveillance and full scale tests carried out to provide information on the nature and extent of remedial works required.

In the short-term, all the methods tried have proved effective in renovating the puddle clay core. However, the sheet piling technique employing asbestos sheets encountered many practical problems and would not again be considered. The remoulding technique, employing the 'box-head' pile has not had such a long period for evaluation but has so far proved effective; the plain I-section pile appears to be marginally less effective. The effectiveness of the grout screen at Banbury reservoir may now, after 10 years, be considered proved in the medium term.

ACKNOWLEDGEMENT

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Instrumentation Developments

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SYNOPSIS

The continuing increase of computer application within the industry has highlighted the need for similar aids in the field of instrumentation data aquisition and processing. The first section of this paper describes how modern technology is being applied to satisfy this requirement.

A unique settlement profile measuring device developed for Tarbela Dam, Pakistan has since undergone several important transformations. The second section of this paper discusses developments from the original manual device through the fully automatic system which has largely replaced it in large dam applications to the recently-developed semi-automatic precision measuring system which has opened new fields of application.

SECTION 1. AUTOMATIC MONITORING OF SOIL INSTRUMENTS

The Problem

Production of soil instruments is a young industry and is evolving rapidly. These instruments are designed to measure soil conditions and movements so that predictions can be made of the soil stability and its likely shifts under various forms of load, for instance road embankments, high rise buildings, dams etc. These instruments have a relatively hard life. Most of the sensors are either buried or lowered down water filled boreholes in the earth. Portable reading instruments have to survive severe jolting in vehicles, dust and vibration at the reading site, mishandling by inexperienced personnel and must be able to operate in all weathers. Reading instruments have been dropped into the sea, over the sides of (empty) dams and run over by Land Rovers. Survival is marginal in these cases; it is perhaps surprising that delicate measurements can be made in these conditions.

Because of the importance of reliability early instruments used simple mechanical transducers in the sensors and mechanical reading methods which used such things as manometers and pressure gauges. Instruments based on these principles still form a large part of production. As the convenience of electrical devices was recognised and confidence grew in their reliability electrical transducers were developed to be read by electronic means. Because of the higher resolution of electrical units this development was followed by a demand for greater accuracy, with which the industry is coping at the present time. With wider understanding of the usefulness of soil instruments installations are now planned and being built which use not just a few sensor but (in some cases) hundreds. This brings new problems.

The normal method of taking measurements is to write down readings as they are displayed by the gauge or digital display. For analysis the readings are either worked out on a calculator and graphs drawn by hand or, increasingly, they may be typed into a computer. Each sensor requires anything between 2 and 1600 measurements each time it is read, dependant on the type. It will be appreciated that reading and analysing the results of even tens of instruments in this way is laborious and takes a long time. In addition many of the instruments need expert operation so that low grade labour cannot be used and useful engineers are occupied for long periods with repetitive tasks. There may also be an appreciable delay between taking and analysing the readings when hours may be important in fast moving conditions. There is a clear need for faster and simpler methods of sampling, in other words automation.

The Solution

As an illustration of what can be done the following describes the application of modern technology to an instrument frequently found on construction sites. One of the most tedious and time-consuming jobs on a site is the taking of inclinometer readings. For the reader who is not familiar with inclinometers this is an instrument where a probe is lowered down a borehole to measure its tilt. The borehole is generally anchored in a stable medium at its bottom and as the soil above moves the walls of the borehole move with it, pivoting around the base and the tilt alters. By taking readings at different times the speed of soil movements can be found. The probe is fitted with wheels which run in slots moulded into the borehole casing to fix the direction of measurement. The probe is lowered into the borehole manually using a cable marked with depths which also carries the electical signals from the probe to the readout unit at the top of the borehole. Tilt readings are normally taken at half metre intervals up the borehole in two planes at right angles. Two sets of readings are taken for each plane in opposite directions to eliminate offsets. Thus the probe is lowered to the bottom of each borehole four times. For a borehole with a depth of 200 metres there would be 1600 readings in a normal set. Despite the labour involved this is a favourite method of measuring soil movements as it is simple and sensitive; inclinometers can detect tilt changes of less than 20 seconds of arc (0.05 mm in 500). Semi-automatic methods are now available which ease the time and labour problems of taking readings. Attempts have also been made to automate the lowering and raising of the probe but no-one has found a safe, easily portable way of doing this. However microprocessors can be installed in the reading units to give the following facilities.

The operator no longer needs to write down readings. These are recorded automatically on magnetic tape cassettes. At each reading position a button is pressed. The recording unit takes 10 samples, computes the mean, converts the results into whichever units are desired then, together with the depth, displays the data and stores them ready for transfer to magnetic tape. All this takes less than a second. A simple keyboard is available for the operator to enter heading information. An alphanumeric display tells him how to use the keyboard step by step and during reading displays the depth. Recording in this way is quick and more than halves the reading time. In addition it is no longer necessary to lower the probe into the borehole four times. This is made possible by fitting two tilt transducers into the probe with their sensitive planes at right angles, careful construction of the probe and intelligent use of the microprocessor. This reduces reading time by a further factor 4. Back at the site hut the operator can connect a small printer directly to the recording unit so that he can quickly obtain a hard copy of the data for checking. Base data are stored permanently in part of the recorder memory and can be recalled to the printer for comparison. The printer could have been incorporated in the recorder but trade-offs indicated that it would increase the weight (the battery would have to be larger) and was not likely to be used much out in the field. Incorporation of full scale computing facilities was rejected for similar reasons and because of the extra complexity. The recording unit can do a modest amount of data reduction, however, without a weight penalty and can quickly supply the operator with means, differences, trends etc. There is increasing use of computers for final analysis of the data and the recording unit provides access to a computer in three ways without the need to type data in. First, if a computer is nearby, the recording unit can be connected directly to it and the cassette played back to place the readings directly into the computer memory. Secondly, if the computer is at a remote site the cassette or cartridge containing the data may be removed from the recording unit and sent to the computer by post or courier for replay in the computer's own mechanism. The third, and probably best, method is to tranfer the data to a remote computer over a telephone line. To do this the recording unit is attached to a telephone line socket and the data in serial form shifted down the line via the recorders own, post-office-approved modem. If the printer remains connected the data can be recalled from the computer and printed out to verify that it has been correctly received.

All these functions are performed by a portable box about the size of a small suitcase weighing about 16 Kg and using CMOS components with power-down facility for minimum battery weight. The box is hermetically sealed to prevent entry of dust or moisture into the electronic compartment. For reliability the internal electronics borrow spacecraft techniques in their construction, there are remarkable similarities between site conditions and the spacecraft launch and space operation environments.

The instrument described above approaches the ideal inclinometer recorder. Instruments presently available differ only in that two extra units are needed to send data over a telephone line, otherwise they have the same performance. Instruments incorporating the modem and terminal functions are feasible and are likely to be available soon.

Future Developments

The functions of the inclinometer recorder unit are determined totally by the programme placed in the memory, changing the programme alters the functions. With different software the identical electronic package can be used for a wide variety of jobs. For instance a data logging system has been designed where slave stations automatically control and monitor mixed groups of static soil instruments. These stations send the collected data to a central station which can interface printers, computers etc in a similar manner to the inclinometer recorder. Only the housings are different, the internal electronics of the slave stations, the central station and the inclinometer recorders are the same. The slave stations need only the addition of a switching board and scanivalve cascades if pneumatic and hydraulic sensors are to be scanned. Using a standard package in this way eases manufacture, promises to make instruments cheaper and more reliable and will allow units to be tailored to special requirements relatively quickly.

In the near future we are likely to see smaller, lighter instruments with wider applications. Magnetic tape recorders are likely to be replaced by the more robust magnetic bubble recorders. When prices are right the use of fibre optics on sites presents intriguing communication possibilities as well as immunity to lightning damage.

SECTION 2. FULL CIRCLE - THE DOUBLE FLUID SETTLEMENT PROFILE MEASURING TECHNIQUE

During the early stages of construction of Tarbela Dam, Pakistan, the project engineers developed and installed a unique settlement measuring system which was subsequently named the TAMS Double-Fluid Settlement device (DFSD). The system was based on the use of a continuous length of small bore nylon tubing installed horizontally in the embankment foundation to a maximum length in excess of 1000 metres, with both extremities of the loop terminated at a downstream monitoring point. The operating sequence essentially consisted of the following stages.

i. Filling the entire tubing length with water by circulation.

ii. Establishing a water/mercury interface at the terminal and, by means of a mercury head provided to a defined higher elevation, causing the interface and the following continuous mercury column to advance along the tubing bore to a pre-determined point.

iii. Halting the interface at the measuring point by isolating the previously opened distant end of the tubing loop and extending it onto a pressure gauge.

iv. Allowing hydraulic equilibrium to establish, at which time a gauge reading was taken.

v. Opening the water end of the system to atmosphere to re-instigate the interface advance, halting and reading cycle described above, thereby reaching another pre-selected reading point along the tubing loop.

The basic principle of operation of the system is shown in figure 1 as a 'U' tube arrangement, the horizontal limb of which represents the length of tubing installed in the structure.

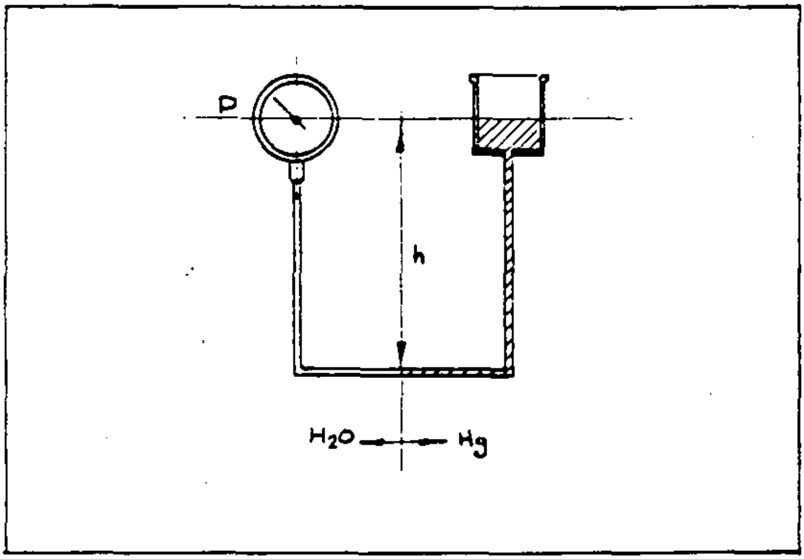

Figure 1. Operating Principle of the TAMS DFSD

In the equilibrium state shown the pressure indicated by the gauge is equivalent to the mercury-minus-water heads over the common height h from the mercury surface and gauge centre to the mercury/water interface level. This is equal to 1.26 Kg/cm^2 per metre of elevation between the two points. Thus if h is altered the gauge reading will change in accordance with this factor. The elevation of the interface may therefore be determined relative to the terminal equipment.

Soon after the system was commissioned it became apparent that the practical method, though capable of providing settlement data to an accuracy better than 15 mm of elevation at any measured point, suffered the disadvantage of requiring a high degree of operator expertise to achieve such results. Furthermore a single test required that such effort had to be continuously sustained over hours or even days of boring repetitious activity.

Although this inconvenience was tolerated in order to aquire such otherwise unobtainable and comprehensive data, the development of a fully automated system was regarded as a very attractive prospect. However those same features of the operating sequence which demanded expertise and judgement rendered the system almost impossible to automate. The stumbling blocks were as follows.

a. Mercury over-run as the attempt was made to halt the interface at a defined point caused by elasticity of tubing and content and volume change within the measuring gauge.

b. The long and varying time required to achieve hydraulic equilibrium before each reading was taken. This effect was magnified by increase in the line length, mercury /water content ratio and elevation change of the interface at successive reading points.

The above limitations were never completely eradicated during the construction phase of the Tarbela Dam project with the result that the manually operated system became firmly established. The device is still providing valuable data at the project some 12 years after the first installation was commissioned.

Development undertaken by Soil Instruments Limited in the period 1976 to 1978 resulted in the production of the Automatic Settlement Plotter, a system which provides a continuous profile of tubing elevations over distances up to 1 Km loop length to an accuracy of 10 mm without need of human assitance. Results are presented in the form of a recorder chart plot scaled in engineering units (X axis, distance; Y axis, elevation).

Figure 2 indicates how the operating principle was modified to overcome the limitations inherent in the original manual system. The horizontal portion of the 'U' tube system shown represents the installed tubing length within which an interface delineates continuous mercury and water columns in the respective vertical limbs. In preference to the start-stop operating mode employed in the manual system a continuous uniform rate of interface progression is applied. The mercury delivery pump shown at the top of the mercury-filled limb is connected to a pressure gauge which symbolises an electrical pressure transducer in the practical system. Since mercury head to the interface produces a greater pressure than the opposing water head it is necessary to apply a back pressure to the water to prevent uncontrolled mercury column advance. This restraining pressure (K) is accurately maintained irrespective of water through flow as the interface is advanced by the mercury pump. If dynamic losses are ignored

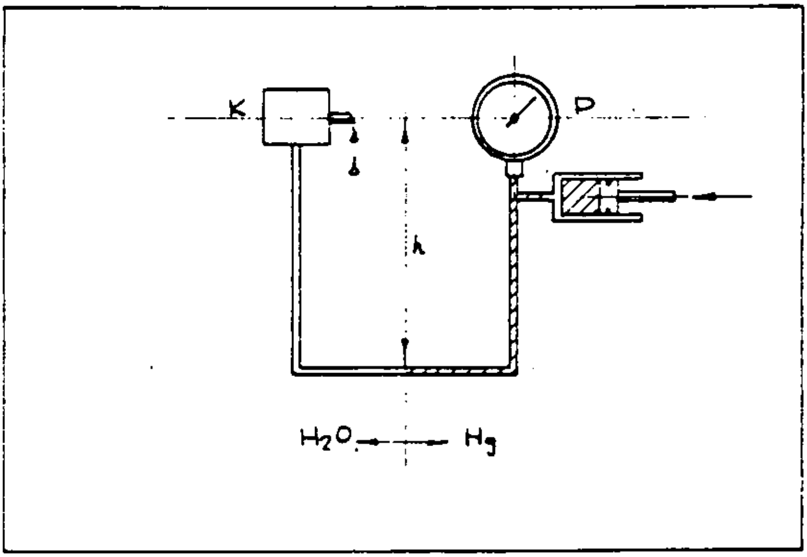

Figure 2. Operating Principle of the Automated DFSD

P, the pressure indicated by the gauge at any instant = K - 1.26h, where h is the elevation difference between the gauge and interface in metres and P & K are expressed in Kg/cm^2. In practice this pressure is monitored by an electrical transducer which sends a signal to the chart recorder enabling the elevation profile to be plotted. Clearly the accuracy of such a measurement depends upon how precisely the restraining pressure (K) is maintained. Pressure stability better than 1 part in 2000 is achieved by the arrangement shown in figure 3 which represents a typical element of the back pressure unit chain. In the neutral condition illustrated the mercury/water interface levels of both limbs equate. However as water flow is accepted into the small bore limb from the return line of the installed tubing mercury is forced before it into the large bore cylinder thus producing a back pressure proportional to the interface level difference of the two limbs. This process continues until the small bore limb is

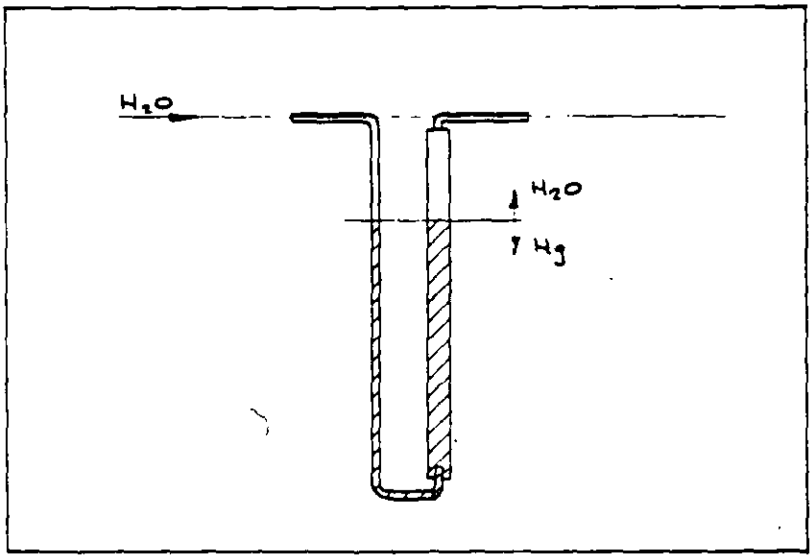

Figure 3. *A Typical Element of the Back Pressure Unit Chain*

entirely water filled. Further inflow results in the entry of water into the large bore cylinder in the form of droplets which float upwards through the mercury column without breaking it. Thus whilst inflow is maintained a constant pressure equivalent to mercury-minus-water head over the height of the mercury column is exerted on the water input point. In practice four such elements are connected in series to produce a back pressure of approximately 4 Kg/cm^2, sufficient to cater for measured elevation changes in excess of 3 metres.

The explanation of the operating principle outlined above ignores head loss effects due to fluid flow through the tubing, but these must be taken into account in the practical system. At the commencement of a test cycle the entire tubing length is water filled and overall head loss is a function of water flow only. At the end of the test cycle the tubing is mercury filled and overall head loss is a function of mercury flow at the constant rate. At intermediate points total head loss is the sum of mercury and water losses, each of which is a function of repective column lengths. In a common environment mercury head loss is greater than that of water, therefore in the constant flow, constant tubing bore practical system effective head loss increases linearly with time from test commencement. Correction is therefore straightforward and in practice is applied electronically.

Figure 4 shows the simplified arrangement of the latest device which employs the

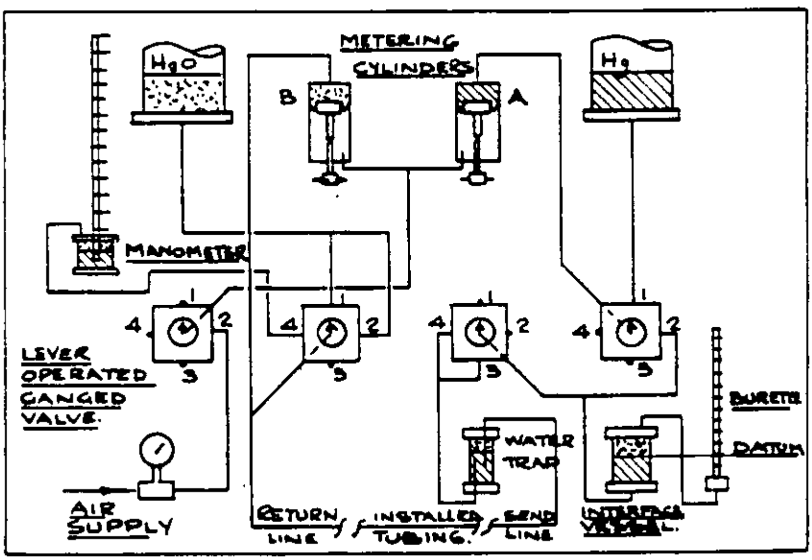

Figure 4: Simplified arrangement of the new DSFD

original start/stop interface advance and reading method but incorporates automatic fluid metering and valve sequencing techniques to simplify and speed up the measuring operation. This method enables the elevation of each measured point along the tubing length to be established within ± 2 mm. Distance increments between reading points are selectable from 0.25 to 2.0 m and the standard system accomodates tubing lengths up to 120 m. Portability, a high degree of measuring accuracy and the ability to operate from self contained pneumatic and hydraulic sources renders the system uniquely suitable for use in hazardous environments. This feature is of particular interest to the oil industry in monitoring settlement below large storage tanks and similar structures. Potential applications in dam construction include economic monitoring of small embankments and to determine differential settlement extent and distribution across interfaces between rigid headwork structures and contacting embankment soils.

Referring to figure 4 the operating sequence consists of 4 stages, the selection of each being controlled by a lever operated ganged valve assembly. Metering Cylinders A and B employ rolling diaphragms to minimise friction and may be adjusted to accept or discharge precisely defined fluid volumes in the range 0 to 12 cc (0 to 2m of tubing length). The sequence is as follows.

Position 1. Mercury is fed from the reservoir to Metering Cylinder A.

Position 2. Water is discharged from Metering Cylinder B to the reservoir. Mercury is discharged from Metering Cylinder A to the Interface Vessel causing the water level to rise in the burette.

Position 3. Mercury flows from the Interface Vessel into the line causing the burette water level to fall accordingly. Displaced water from the line is received by Metering Cylinder B until it is full, whereupon inflow ceases.

Position 4. The reading manometer is connected to Metering Cylinder B and the line. The reading is recorded by the operator.

The above process is repeated to the next reading point. In practice the arrangement minimises the degree of operating expertise necessary to ensure accurate data by automatic control of valve and fluid delivery functions. The overall effect is to enable readings to be obtained at a rate exceeding 60 per hour to a hitherto unobtainable millimetre accuracy.

Operation and Maintenance of Reservoirs in the Severn-Trent Water Authority Region: a Perspective

P.G.MACKEY, DLC, BSc, MSc, MICE, MIWES

Severn-Trent Water Authority

SYNOPSIS

The system of operational surveillance is explained for the Severn-Trent Water Authority Region together with policy and management objectives for its 76 large reservoirs.

Four recent case histories are examined whereby investigation utilizing advanced techniques of TV and materials examination are undertaken together with conventional methods, and leading to options for decision making.

INTRODUCTION

The Severn-Trent Water Authority (STWA) is the second largest Water Authority in the United Kingdom and has 76 large reservoirs falling within the ambit of the Reservoirs Legislation, together with Carsington Reservoir and Hallgates No. 5 Service Reservoir at construction and design stage respectively. The Authority's reservoirs range in size from 27 ml to 60,000 ml, of aggregate volume 300,000 ml and of up to 135 years known recorded age.

These reservoirs comprise 25 upland impounding reservoirs, 19 lowland impounding reservoirs, 9 pumped storage, 18 treated water service reservoirs and 5 miscellaneous impounding structures for land drainage or balancing purposes.

Based on the Institution of Civil Engineers Floods and Reservoir Safety: an Engineering Guide[1] the risk category assigned by Table 1 breaks down the reservoir stock as follows:

Category A	56%	Category B	23%
Category C	1%	Category D	20%

The following statistics refer:-

(i)	Earth gravity embankments with a variety of clay cores, sheet pile cut off walls or concrete cores	49 no.
(ii)	Masonry gravity structures	4 no.
(iii)	Mass concrete gravity structures	12 no.
(iv)	Reinforced concrete (RC) structures (Services Reservoirs)	7 no.
(v)	Brick gravity structures	4 no.
	Total	76 no.

A histogram depicting the dam population by age appears as Fig. 1 from which it is of note some 55% are over 65 years old.

Of this latter class, some two-thirds are represented by earth dams constructed in the Victorian or early Edwardian eras and indicates emphasis in maintenance priority.

OPERATION AND MANAGEMENT

Management seeks to emphasise multi objective principles, and an essential part of the operational philosophy is that of reconciling potentially conflicting requirements. Nowhere is this more apparent than in the maintenance and inspection programme. Storage drawdown imposes constraints on amenity and public access, these latter necessitating consultation with interested bodies in devising acceptable guidelines for undertaking maintenance.

RESERVOIR MONITORING AND SURVEILLANCE

The Water Industry has an exceptionally good record in meeting the requirements of the Reservoirs (Safety Provisions) Act 1930 and, in STWA there are well established procedures for discharging statutory obligations through the Director of Operations.

Supervisory provisions of the Reservoirs Act 1975 have been incorporated into job descriptions of operational staff, as in the majority of WAs. Overall benefits in terms of personal competence and management control have been manifest in the areas of monitoring and surveillance as well as management and operations.

The Authority utilises 15 trained and qualified designated "Supervising Engineers" to watch the 76 reservoirs. There are six monthly formal inspections with an annual report submitted to the Director of Operations. In the event circumstances dictate more frequent surveillance than normal operational visits, these are arranged by the Supervising Engineer.

RECORDS

Recently, the Authority has responded to circulars from the Secretary of State for Wales and the Department of Environment on the functioning of the 1930 Act, and submitted evidence to the House of Lords Sub Committee on Water, based on the Authority's policy and management of this function. Statutory records are microfilmed and there is available a central records system which assists in responding to statutory requests for information, in addition to expediting information for inspections.

FLOOD STANDARDS

Following publication of the Flood Studies Report in 1975[2] together with the complementary Floods and Reservoirs Safety Document of 1978, the issue of Flood Standards has been the subject of close appraisal bearing in mind that standards were set formerly by the ICE interim report 'Floods in relation to reservoir practice' - updated in 1960[3].

Each catchment and dam site is subjected to the fullest hydrological appraisal utilising the above together with local records where these are available, in order that staff responsible are fully aware of operating margins. Also, Inspecting Engineers have a framework whereby the respective flood standard can be included as a basic parameter in evaluating the overall equation of "How safe is this reservoir?".

The techniques of computer simulation enable the relatively theoretical concepts of PMF embodied in Table 1 to be translated into such tangible factors as design rainstorms, flood routing, diversion operation, etc. More importantly, there can be derived head over spillways, weirs, byewash channels etc., and freeboard in relation to wind/wave propagation for the dam geometry.

This technique is particularly helpful when there has been an embankment slip or crest slump, leakage or mechanical failure of spillway, drawoff or scour, as the safe operating level of the reservoir can be fixed to known criteria when assessing the component of risk associated with flood operation[4].

INSTRUMENTATION

There has been an increasing move towards installation of fixed location line and level survey points in dam cores and at other strategic locations and up to 90% of structures now have continuity of record in this respect.

Instrumentation is a valuable operational aid but, widespread installation of movement gauges, inclinometers, piezometers, etc. is a relatively new phenomenon. Recent philosophy has been based on providing instrumentation for monitoring construction or immediate post construction behaviour leaving the undertaker to make do with those which survive, for his operational records! Thus a situation is arrived at whereby frequently, inadequate instruments wrongly located are the only basis for assessing operational performance at a critical time.

MAINTENANCE OF BANKS

Some of the more complex problems involving Victorian earth dams require a protracted period of evaluation following installation of instruments entailing losses of time and operational capability, both important factors if the economic value is marginal. In addition to the continuing problem of earth embankments and the associated limitation of engineering standards there are incipient weaknesses of certain aspects of Victorian water engineering lucidly explained by Mr. G. M. Binnie[5] and which most Water Authorities have to some degree inherited. Factors initiating modifications or extended maintenance work are:

(i) Work arising from Statutory Inspection
(ii) Uprating to revised standards (overflows, spillways etc.)
(iii) Sudden or unexpected failures (severe weather, mechanical soils failure etc.)
(iv) Vandalism : a regrettable feature of modern life requiring special considerations of security.

INVESTIGATORY TECHNIQUES

Considerable time and money has been saved by use of advanced non destructive and surveillance techniques and the availability of good quality closed circuit (CCTV) and underwater TV survey has added a new dimension to reservoir engineering. No longer is it always necessary to drawdown reservoirs to inspect upstream faces or dismantle valves to examine (often inaccessible!) draft or scour tubes. Hardware up to 135 years of age has been radiographically mapped in situ or examined ultrasonically for evidence of graphitization, cavitation damage, corrosion or cracking.

The Authority has a Standing List of Contractors for a range of services relating to dam maintenance. The case histories presented later give an indication of the scope of these techniques as an aid to decision making.

CONCLUSIONS

In recent years, the question of risk and safety in all structures and technical developments has generated increasing interest, particularly with the legal obligations of such as Health and Safety at Work Act, Diving Regulations SI 399 etc.

The safety of dams is rarely very far from public attention and concern, as indeed are failures or incidents at any "high risk" installation.

In considering the operational performance of dams, it is essential to analyse past incidents and failures, as a more precise awareness of the mechanisms at work leads to increasing knowledge of dam behaviour, which in turn can be related to design, modification and operational criteria. Research into the behaviour of banks, concrete, and masonry, have often been initiated as a result of incidents or failures during construction or in service. By continuing development of the technical principles of design, construction, modification and maintenance of dams together with an ongoing programme of inspection and inservice monitoring, significant contributions can be made towards maintaining present standards or improving dam safety.

Four case histories follow presenting a range of operational and maintenance problems together with the solutions considered.

CASE HISTORY NO. 1

DEEP HAYES RESERVOIR (LEEK) (100 mg CAPACITY, 456 ml)

Constructed in 1848 by the former Staffordshire Potteries WCo as a compensation reservoir yielding 5 mld, the dam was an earth embankment 18.3 m high, 125 m long with a puddle clay core. There was no underdrainage and a small key trench was let into the natural bed rock to form a key (Fig. 2). The closure was at the narrowest part of the valley into the rock outcrop.

Investigations confirmed the geology as a synclinal basin of relatively impermeable shale overlying permeable sandstones with heavily weathered outcrops at various points in the reservoir basin. There were up to 3 metre transitions between the bedding planes of the sandstones and shales consisting of coal and seat earth, and located immediately under the dam foundations.

Although extensive reservoir records were available, there was no information on the dam construction. The spillway was reconstructed in 1966 and uprated to the 1960 revision of the 1933 ICE Standard and would have met Category B conditions. Maintenance of the embankment was not to a high standard, there being evidence of considerable activity from springs which subsequently complicated analyses during investigation.

Reporting between June 1972 and March 1976, Mr. T. G. Hammond indicated the embankment appeared sound, the spillway adequate, but that the drawoff works were potentially hazardous and in view of their age, direct access through the core and, there only being one downstream control valve, a new drawoff system was recommended in tunnel through the east abutment from a valve tower.

During the course of investigating piezometric data, it became apparent there was associated leakage, postulated as high level core rupture or weathering, and a limit was set on the maximum operating level in view of the possibility of piping or foundation failure. Despite extensive chemical tests, however, this theory was not positively confirmed. At this stage, a toe berm was advocated for additional stability.

In the event, economic criteria intervened when costs of remedial works were set at £650,000, whereas a scheme of lowering the dam and provision of alternative compensation water would be achieved for £367,000, and this decision was implemented (1980 prices).

As a carollary, it was only during the course of the scheme of lowering that, with the reservoir drained, the full extent of liability became visibly evident. The piezometers has only given a partial perception of the whole range of problems as the leakage appearing as a spring was due to core weathering in the top metre, but additional leakage was apparent through the east abutment on the line of the new proposed drawoff works. This completed the picture as to why the piezometer results, rationally though they could be explained, did not tie in with the chemical data of over 100 samples from springs, relief wells and piezometers.

CASE HISTORY NO. 2

SNITTERFIELD RESERVOIR (NR. STRATFORD ON AVON) (80 ml)

The reservoir is formed by an earth bank, constructed on the arc of a circle, between two promontories in the hills to the south west of the village of Snitterfield. It has virtually no catchment, water being led into the reservoir by aqueducts from streams in neighbouring valleys.

Constructed c 1884, to afford supplies to Stratford RDC, it is now disused. According to original drawings, the embankment is approximately

180 metres long, with a maximum height of 8.2 m. As designed, the overflow level was 88.4 m OD, but earlier this century the bank was raised by 0.75 m and the overflow lifted to 89.9 m OD.

The upstream face of the embankment lies at a slope of 3 to 1 and is protected by concrete slabbing. The crest has an average width of 4.5 m and is grassed, as is the downstream 2:1 slope.

The original discharge arrangements consisted of an 460 mm diameter cast iron pipe laid directly under the embankment taking water from a 610 mm diameter standpipe with entry of water controlled by 3 no. 460 mm diameter penstocks.

The original overflow was 1.5 m wide which led into a 305 mm diameter salt-glazed ware pipe discharging to a stream. The overflow was modified to 89.46 m OD with a 3.12 m weir leading to a 600 m diameter concrete pipe. The minimum height of the embankment was also increased in the 1930's.

Furthermore, the outlet pipe was lined with a 350 mm diameter PVC pipe and the annulus grouted for at least 1.5 m at either end of the length passing under the embankment.

The site can be classified for flood purposes under Category C.

Leakage from the north west sector of the bank at 1 - 2 metres above ground level was noted in November 1980 at a rate of 165 litres/hour.

Evaluation commenced by investigating the core and embankment and taking samples for chemical and soils testing. Piezometers and standpipes were installed and the reservoir drawn down and refilled over two cycles in the intervening 18 month period to observe piezometric and phreatic conditions (Fig. 3).

Conclusions drawn suggest the dam is basically sound but there is weathering of the top 1 - 2 metres of core, possibly associated with its former raising. This condition suggests the mechanism may continue at other localities as the process of ageing continues.

As the site is not used for operational duties, remedial schemes being considered have regard to the economies of either retaining the site as a reservoir or lowering the bank such that it no longer forms a large reservoir under the Act.

Remedial schemes considered:

(i) Cement/bentonite grout locally to leakage
(ii) Inverted filter drain at location of leakage
(iii) Jet stream grouting technique (GKN process)
(iv) The vibrating pile technique developed by Thames Water Authority and reported under Paper 3.

To date, £15,000 has been spent on additional monitoring and investigatory duties.

In addition to the above, the Acoustic Emission Technique was set up at the site of the leakage to ascertain whether sonic emission of water passing through the bank could assist in indicating the extent, scope, and source of leakage as, at that stage, it was by no means certain water was coming through or under the bank, and there were possibilities of spring activity. The results were, however, inconclusive.

CASE HISTORY NO. 3

NEW POOL RESERVOIR (CHURCH STRETTON)

This reservoir has a capacity of 44 ml with a dam 61 m long rising to 15.25 m above the valley floor, with diaphragm core and the embankment sloping at 2:1 being pitched on the upstream side and grassed on the downslope (Fig.5).

The dam has leaked significantly since construction in 1900 to the extent of up to 68,000 litres per day, as reported (Ref. 7) by the designer Edward Brough Taylor, but this was attributed to spring activity rather than direct leakage. Issues to the mitres and draw off tunnel appeared to confirm this diagnosis and the area of egress was provided with a rubble drain to prevent erosion of the embankment toe.

The operational history relates dealing with minor soil slips on the downstream face (put down to the steepness of slope and spring activity) together with making up "local consolidation" to the core on at least three recorded occasions separated by 15 yearly intervals.

In November 1977, there was a rapid increase in the recorded leakage and instructions were given for the reservoir to be drawn down. Leakage was measured in relation to reservoir head and a correlation obtained (Fig. 4) indicating the additional leakage commenced at 11.8 m to a peak of 25,000 l/h at the maximum permissible operating head.

The reservoir ceased operational duties in early 1978 and decisions were required. At this stage, evidence came to hand through routine monitoring of further core depressions and the reservoir was emptied in 1979 for inspection purposes whereupon new evidence came to light. It became clear there was a major depression on the crest and upstream face on a line of axis to the point of egress on the springs. The extent of the depression was masked by the arching of the higher level pitching on the upstream face (which had been hydraulically grouted) thus giving the impression of an undisturbed water line to even the most careful observer. It has taken some 82 years to emerge that the mechanism at work could well be that known as core erosion, and there is a strong probability that hydraulic fracture has taken place (Ref. 6) at the core base of the dam foundation with the high level leakage as a consequent process of tension cracking. Trial holes have indicated that slip planes are present.

The site can be classified as Category A but the present spillway (assessed against the 1933 Standard at its last Statutory Inspection) has capacity of 7.5 cumec as opposed to 11.6 required for the ICE 1978 Standard, indicating further expenditure is needed.

Remedial schemes considered (1982 costs):

(i)	Lowering dam with centre spillway	150,000
(ii)	Grouting and new spillway	210,000
(iii)	Lowering existing spillway to impound less than 5 mg	250,000
(iv)	Remove dam - discounted on amenity grounds	
(v)	Use of Thames technique plus new spillway	200,000

CASE HISTORY NO. 4

UPPER AND LOWER BLACKSHAWMOOR RESERVOIRS (LEEK)

Upper Blackshawmoor (20 ml c. 1760) and Lower Blackshawmoor (60 ml c. 1848) are in series located in the Upper Churnet valley. Although not strictly within the terms of the 1930 Act, the Upper Reservoir could contribute to the failure of the Lower hence they are considered jointly. The reservoirs were part of the Leek supply until the Leek UDC Act 1925 ceded these reservoirs for compensation purposes. They are Category D and flow directly to Tittesworth Reservoir some 4 Km distant.

The operational history of both structures is interesting and, although no drawings have been located, the provisions of the Act have worked well in providing engineering reports since 1931.

Following the ICE 1933 Report, the first inspection by R. C. S. Walters recommended raising of both dams by 2 m and reconstruction of spillways and byewash channels to improve free board and flood capacity.

In the intervening period to 1975, there were a succession of reports requiring attention to bank maintenance, drainage and armouring - but no serious problems encountered.

The Authority acquired these reservoirs in 1974 and Mr. P. S. Hallas inspected in 1975, commenting upon the need for maintenance and the requirement to bring up to standard the draw off works of both dams, there being downstream valves only on each draft tube.

Both dams were cleared of trees and vegetation by mid 1977 and it was noted that the Upper Reservoir had wet spots at the west mitre and a core depression over the supposed line of draw off works.

In order to prepare designs, both reservoirs were emptied and surveyed in autumn 1977 (Fig. 4).

There was a suspicion that the 150 mm dia. pipe through the Upper dam might have been a contributor to leakage at the west mitre, although water quality tests had proved surprisingly inconclusive.

With the Upper reservoir empty, there became visible the cast iron draw off pipe in the reservoir basin, and, emerging from the toe of the dam a cast iron 150 mm discharge pipe culminating in the control valve. The pipes visible were surveyed by ultrasonic and radiographical processes and conclusions reached that material was good, graphetization minimal and all pointed to satisfactory conditions for installing an upstream penstock or valve shaft. This left evaluation of potential leakage, and this was examined by means of a CCTV survey which revealed the section of conduit through the dam consisted of various sections of earthen ware and elm wood pipes, of some considerably advanced age!

Cement lining was clearly not feasible and it was confirmed lining with HPDE was the only option.

Survey of the lower dam revealed a very sound bank but tests on the 610 mm dia. cast iron draw off pipes revealed significant graphetization thus requiring structural support for the upstream valve arrangements.

A CCTV survey revealed some unusual gradients in the line of draw off pipe with its line passing close to original ground level through the core at key trench or foundation level. Despite its 140 years of age, the internal condition was good with jointing excellent and approximately 30 mm residual pipe wall thickness presenting no problems by way of scraping and lining at the proposed operating duties.

From 1974 to date, some £25,000 overall has been spent on maintenance and investigation, compared to an annual normal site maintenance cost of £7,500 (1982 prices).

Proposals to date stand as follows:

1)	Remedial works (a) *Upper Reservoir : Grout bank, reline draw off* works with upstream valve (b) *Lower Reservoir : Scrape/cement line draw off* works with upstream control	140,000
2)	Abandon site by breaching both dams and restoring valley	250,000
3)	Breach Upper Reservoir and lower the Lower Reservoir	100,000

ACKNOWLEDGEMENTS

1) The support of Mr R.Y. Bromell, Director of Operations, Severn-Trent Water Authority and many other colleagues in Headquarters and Divisions is gratefully acknowledged.

2) Help and advice over the years from the following Panel I engineers:

Deep Hayes Reservoir: R.M. Arah of Binnie and Partners
T.G. Hammond of Binnie and Partners

Snitterfield and Upper and Lower Blackshawmoor Reservoirs:
P.S. Hallas of Rofe, Kennard and Lapworth

Church Streeton Reservoir: J.T. Calvert of John Taylor and Sons.

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1 Institution of Civil Engineers (1978) "Floods and Reservoir Safety: an Engineering Guide"

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3 Institution of Civil Engineers (1933) "Interim Report on Floods in Relation to Reservoir Practice"

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Fig. 1

FIG. 2

Deep Hayes Reservoir

Plan and Section of Dam

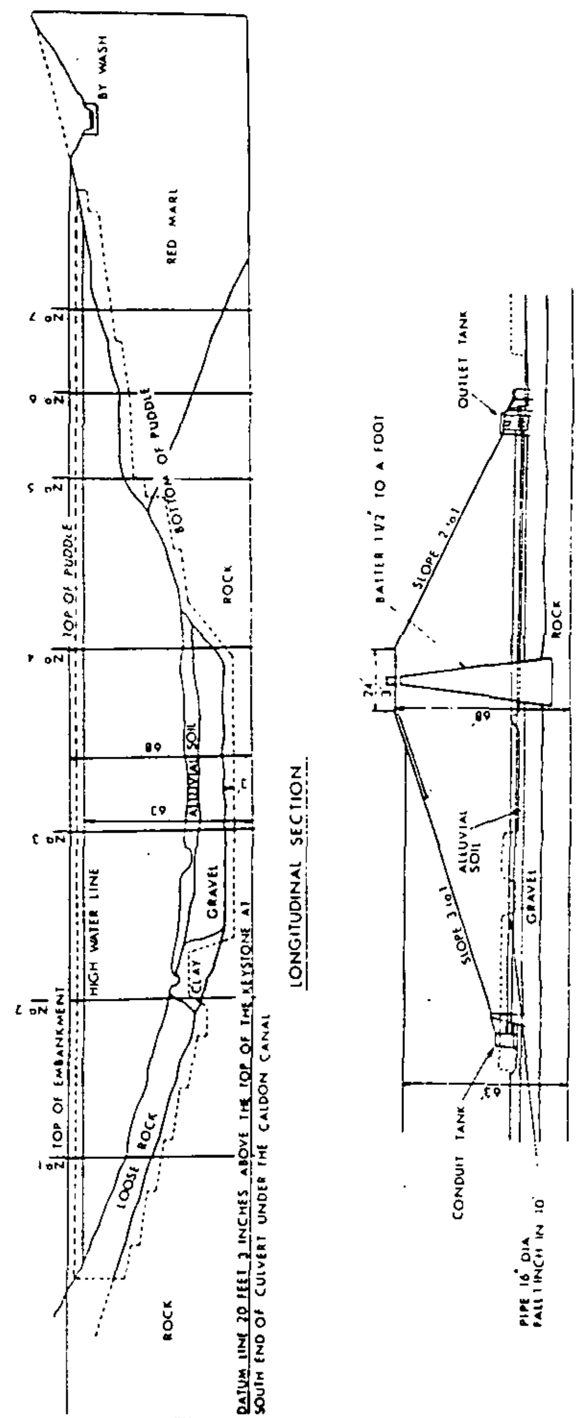
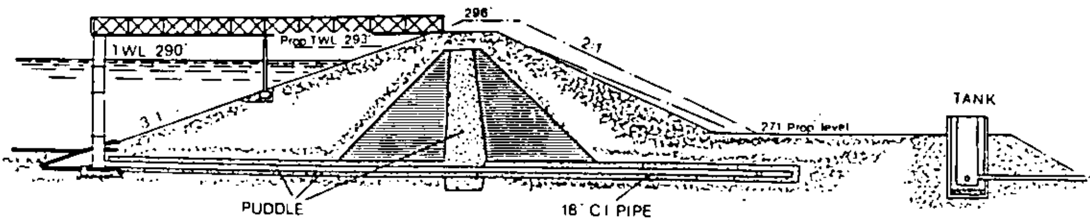


Fig. 3

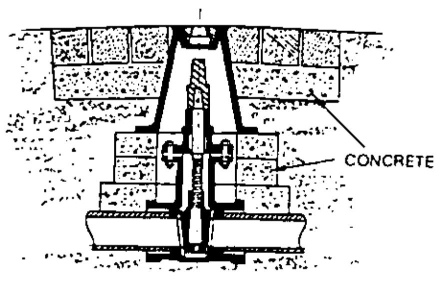
Snitterfield Reservoir

Extract from the plan produced by
E PRITCHARD CE
Westminster & Birmingham
1884



CROSS SECTION

TOE OF DAM

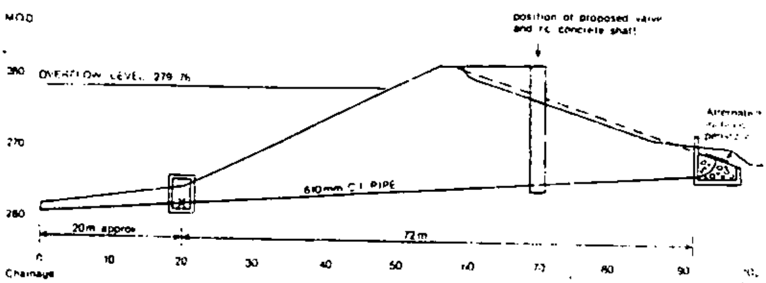


SLUICE VALVE

Lower Blackshawmoor Reservoir

Fig 4

Pipe material cast iron in sound condition
Levels taken from STWA Drawing UTLD/25/1



Upper Blackshawmoor Reservoir

Pipe material varies and includes cast iron earthenware and wood

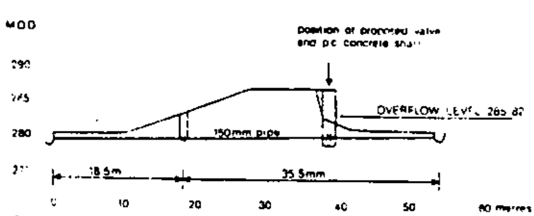
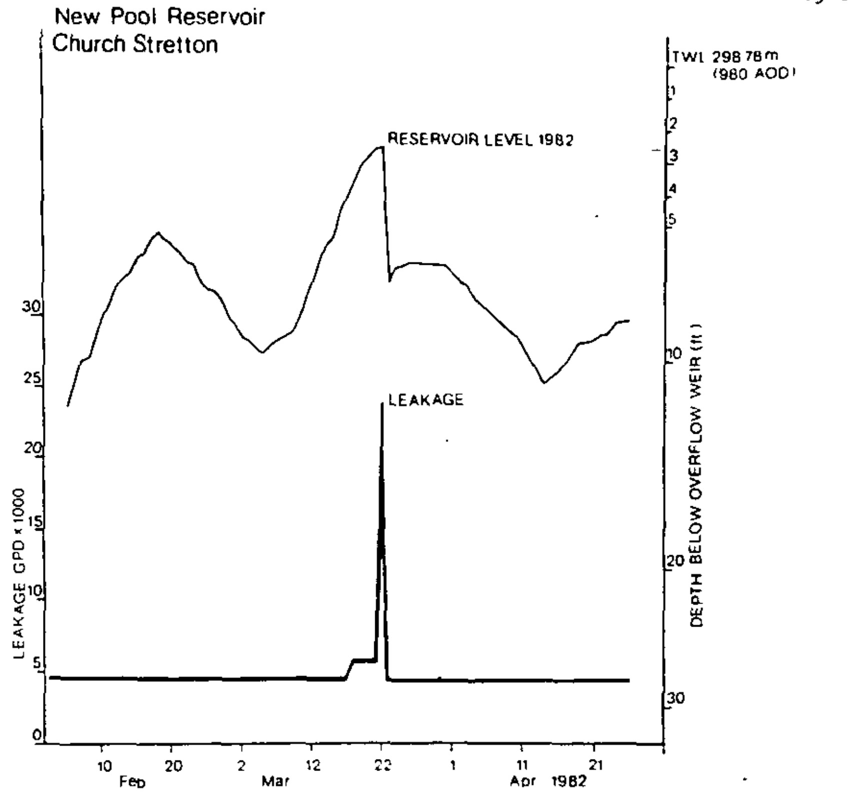
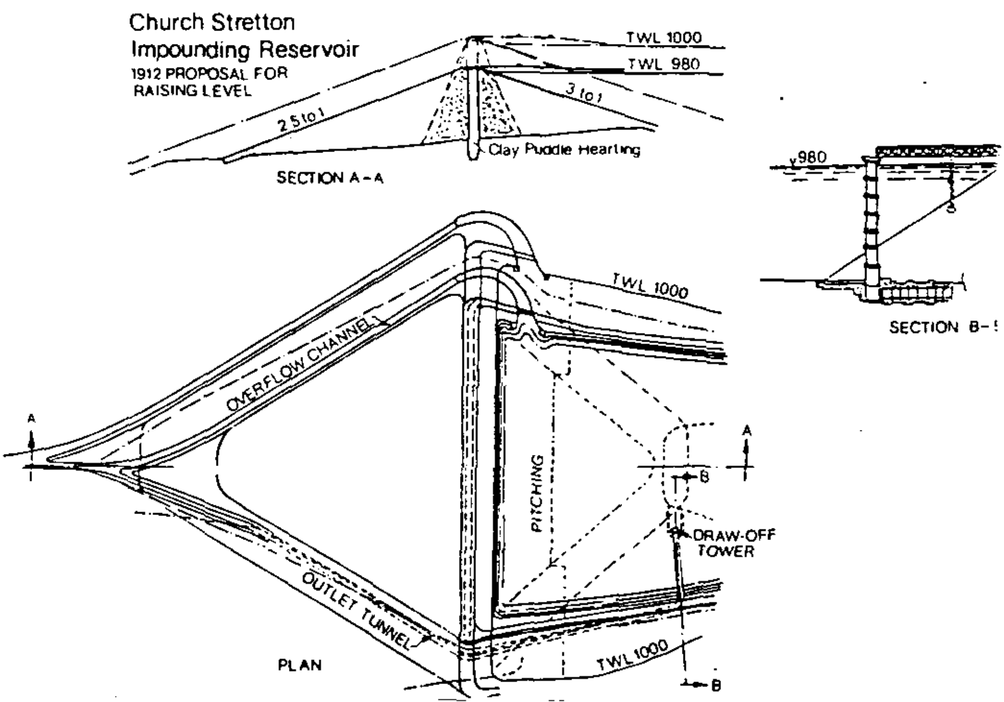


Fig 5


New Pool Reservoir
Church Stretton
TWL 298.78m (980 AOD)
RESERVOIR LEVEL 1982
LEAKAGE
LEAKAGE GPD x 1000
DEPTH BELOW OVERFLOW WEIR (ft)
0 5 10 15 20 25 30
1 2 3 4 5 10 20 30
10 20 2 12 22 1 11 21
Feb Mar Apr 1982


Church Stretton
Impounding Reservoir
1912 PROPOSAL FOR RAISING LEVEL
TWL 1000
TWL 980
3 to 1
2.5 to 1
Clay Puddle Hearting
SECTION A-A
980
SECTION B-B
TWL 1000
OVERFLOW CHANNEL
PITCHING
DRAW-OFF TOWER
OUTLET TUNNEL
TWL 1000
PLAN
A
B

Surveillance of an Authority's Reservoirs

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North of Scotland Hydro-Electric Board

SYNOPSIS

Management of the Board's many reservoirs is undertaken by Chartered Civil Engineers of the Chief Engineer's Division, with assistance by the local operational staff. Systematic surveillance commenced about 1960 and was recast in 1971. The objectives of this current policy are discussed under the headings of field inspections, leakage measurements, monitoring by instruments and flood studies. Field inspections are divided into statutory, supervisory, operational and special. Flood management in relation to safety is assessed by the Flood Study Group set up within the Board whose remit and objectives are reviewed. The FSR and ICE Guide have introduced new methods of assessing risks and improving reservoir safety. As a result of these policies and developments there is a trend towards more visual inspections and less instrumentation. It is considered that these changes have led to better surveillance without any overall increase in resources deployed.

INTRODUCTION

The North of Scotland Hydro-Electric Board owns and is responsible for 84 dams at 76 reservoirs which come within the requirements of the Reservoirs (Safety Provisions) Act 1930. There is a wide variety of types made up of 53 gravity, 9 buttress, 3 arch, 1 pre-stressed concrete, 6 earthfill, 6 rockfill and 6 combined fill and gravity dams. Sixteen dams have gates for the release of flood water and four have siphons. The predominance of concrete gravity and buttress dams is due to the good and tight rock and moderate depth of overburden generally found in the wide glacial valleys of the North of Scotland and the economics at the time of construction.

Nine of the Board's dams are more than 30 years old, the remainder having been completed as part of the programme of intensive hydro-electric development carried out in Scotland between 1945 and 1970. A large majority of dams can therefore be categorised as modern, incorporating the important advances in technology since 1945.

MANAGEMENT OF RESERVOIRS

Most of the day to day administration of the Board's reservoirs and dams is undertaken locally in four Generation Groups, each typically consisting of one or two major schemes comprising up to fifteen reservoirs and twelve power stations. Each Group is administered by a Generation Engineer with a staff of electrical and mechanical engineers stationed in a central location within its area. The Generation Engineer, in addition to his main duties is responsible for undertaking routine maintenance of civil and building works with advice from the Civil Division.

The Board's Head Office is located in Edinburgh where responsibility for the surveillance and maintenance of all dams and reservoirs comes under the Chief Civil Engineer and his Division which is staffed by qualified civil engineers. The Reservoir Safety Section is run by a Chartered Civil Engineer with a surveyor, clerical staff and seasonal assistance from vacation staff and supplemented by inspections by six Supervising Engineers drawn from all Sections of the Division. Six other engineers are currently being trained to carry out Supervisory Inspections.

DEVELOPMENT OF SURVEILLANCE POLICY

After the Vaiont and Malpasset disasters, the Board commissioned, in the early 1960s, Panel Engineers and geologists to undertake comprehensive inspections of all their reservoirs. As a result, facilities for taking settlement and alignment readings were installed in most dams with heights above 23 m.

By 1971 up to 10 years of readings had been obtained and the opportunity was taken to assess the value of inspections and instrumentation (Ref 1). This review, undertaken by the Civil Division, led to the formulation of a dam surveillance policy, the objectives of which were:

1 To provide a stricter and more frequent safety inspection of the Board's reservoirs by the Board's own civil engineers in anticipation of the requirements of the Reservoirs Act 1975.

2 To obtain, over the longer term, a better knowledge of the condition and behaviour of all the Board's dams and to identify and monitor any dams where there were defects, deficiencies, deteriorations or uncertainties with respect to safety.

3 To deploy more efficiently and effectively the limited staff available within the Board for dam safety work.

In formulating this policy, the following aspects were taken into account in defining the frequency and extent of surveillance:

a) Views on the height of a dam at which instrumentation becomes necessary ranged from 15 to 30 m.

b) Foundations, including abutments, were of primary importance and the type and condition of the rock and such features as faults and intrusions were carefully appraised.

c) The type of dam had a major influence on the surveillance considered necessary to maintain safety standards, with arch and embankment dams calling for special attention.

d) Significant variations occur in the safety factors adopted for design.

e) The design and construction techniques adopted, and materials used, are all characterised by the age of the dam and it was considered that there was some merit in categorising by age, viz up to 10 years old, 10-35 years old, 35-75 years old and over 75 years.

f) The function of a reservoir is important. Dams for pumped storage schemes are subject to much more onerous operating conditions than dams for water supply and hydro-electric generation.

g) The size and distribution of population downstream affect the potential risk presented by the dam, an aspect subsequently recognised in the Engineering Guide on Floods and Reservoir Safety.

CURRENT SURVEILLANCE POLICY

The existing surveillance policy which was recast by the Board in the early 1970s is made up of four principal components: field inspections, leakage measurements, monitoring by instruments and flood studies.

I FIELD INSPECTIONS

Four types of field inspections are carried out:

A Statutory Inspections are made by Panel Engineers appointed under the Reservoirs Act. It has been the Board's policy to appoint a Panel Engineer who has not been responsible for either the design or construction of the dam; one Engineer is normally appointed for all the reservoirs in a valley.

The recommendations made and advice given by this Engineer are classified by the Board in terms of priority and responsibility for action. It is normal practice for an Inspecting Engineer to be accompanied on his site visit by an engineer from the Civil Division who concurrently undertakes a Supervisory Inspection.

B Supervisory Inspections are made by Chartered Engineers of the Civil Division, at intervals of 2 to 5 years, the period varying according to the conditions and behaviour of the dams and their importance with respect to third party and Board interests. More frequent inspections are instituted for dams with abnormal behaviour or defects. The civil engineers employed on these inspections are all fully qualified with not less than 10 years' aggregated dam experience. At the same time as undertaking a Supervisory Inspection for safety purposes, the engineer assesses the condition of maintenance of the reservoir works, classifying work into priorities and defining the department to undertake the work. Standard report forms and checklists are used to ensure thorough inspection and progressing of subsequent actions.

As far as possible, maintenance work is carried out by the local Civil Squad of each Generation Group since this has been found to be the most effective and economic way. Each squad of six to eight men comprises typically foreman, mason or bricklayer, carpenter, painter(s) and usually three or four labourers/watermen. Where the tasks are too large or specialised to be tackled by these squads, the work is put out to tender to a number of selected contractors. The Civil Division is responsible for the overall direction of all this work and, in the case of the larger and more complicated projects, for the design, preparation of documents and drawings and supervision of the work.

A maintenance programme of work is drawn up in collaboration with the Groups in the autumn of each year, cost assessed, budget approved and the design and contract stage carried out in the winter in readiness for undertaking the work in the following season (spring through to autumn). The work in the Civil Division is serviced by a Section comprising a Manager, three Chartered Engineers, one Graduate Engineer, two Technicians and a travelling inspector, supplemented by effort from a general pool as work load demands.

C The Generation Engineers are responsible for many routine inspections and readings. Inspections at 3 to 12 month intervals are made by their staff who are normally mechanical or electrical engineers. These inspections are mainly connected with maintenance aspects but any important safety issues which arise are reported to the Civil Division. These engineers are very important eye witnesses and they can provide most useful reports on emergencies and other incidents.

The watermen are responsible for periodic readings, such as reservoir levels, weekly measurement of leakages and some instrumentation readings, including concrete strains, pendulum movements and temperatures. A general watch is kept for new leakages, flood damage, etc which is reported to the Group Maintenance Engineer for onward transmission to Civil Division if and as necessary.

In addition, Group staff are responsible for the exercising and inspection of all gates and valves at not more than yearly intervals. These inspections and operations are required to be recorded in writing.

D Visual inspections have shown that the underdrainage and pressure relief pipe systems at a number of concrete dams built in the 1950s have become less effective than intended, as a result of their original condition or various processes of deterioration. Testing of the working of these systems is therefore carried out under the direction of the Civil Division, whenever doubts arise from Supervisory Inspections. In all cases where their effectiveness is found to be below standard, flushing, scraping or re-drilling of the blocked vents and pressure relief holes is undertaken by contract.

2 LEAKAGE MEASUREMENTS

Leakage measurements at a dam and immediately downstream, along with an observation of the condition of the flow (ie clear, muddy, etc), are considered to be a key criterion for detecting short-term deterioration. These measurements are of first importance; it is of course a statutory requirement that they are observed and recorded weekly under the Reservoirs Act.

3 MONITORING BY INSTRUMENTS

The desirable objective is to install instrument stations on all important dams and to maintain them in first class order so that they provide a continuous record or can be brought into use immediately if and as the need arises. The reading of instruments, particularly where the dams are remote and situated in areas of inclement climate, can be heavy on surveyor's time, and a very strict scrutiny is carried out to select only those dams where instrumentation will be of real value. Full monitoring comprises the measurement of horizontal movement by electronic distance measuring equipment or by optical collimator, levelling, pendulums and joint gauges as appropriate, in parallel with the reading of related parameters (eg temperature and strain). Only instruments which are reliable, robust, durable and replaceable should be installed. The Board has 36 dams with one or more systems of instrumentation.

Regular readings are taken over a sufficiently long period of time (eg 5 years) to gain confidence in the dam's behaviour. Thereafter if the instrument readings are consistent and the behaviour predictable, the number of readings is reduced or the period between readings increased. Typically, readings would be reduced from an annual cycle to one cycle every five years or from reading all instruments to reading selected instruments. The types of instrument and frequency adopted by the Board are discussed in more detail elsewhere (Ref 2). As a dam becomes older or deteriorates, the frequency of the cycle may require to be increased. For a dam in uncertain condition or with abnormal behaviour, more extensive instrumentation may be installed and readings taken more often. For arch dams and large dams in pumped storage schemes subject to rapid variations in head, more frequent cycles are carried out. In addition, extra readings are undertaken after major floods, seismic activity or abnormal events.

4 FLOOD STUDIES

The publishing of the Flood Studies Report (FSR) by the Natural Environment Research Council in 1975 and the Engineering Guide to Floods and Reservoir Safety by the Institution of Civil Engineers in 1978 has had a significant impact on the standards to be applied to the safety of dams and reservoirs.

By the early 1970s some 20 years' operational experience was available for some of the Board's schemes and it was recognised that there were possibly deficiencies in a number of them. The opportunity was therefore taken to make an appraisal of the design of the schemes and to assess the flood management procedures in force. In 1972 a small Flood Study Group was set up under the Chief Civil Engineer comprising a meteorologist, hydraulic engineer, Reservoir Safety Engineer, and senior hydro engineer with the following remit:

1 To develop general methods of analysing floods using information becoming available from the impending Flood Studies Report.

2 To re-assess, using this data, the magnitude of floods in the catchment areas and river basins developed by the Board.

3 To establish safety criteria for each reservoir and river basin.

4 To review and revise, as necessary, normal operating and flood management procedures.

The criteria then adopted for safety have largely anticipated those now published in the ICE Guide. Additional aspects of particular concern to the Board in respect of both flood management and safety are:

1 Maximum floods in rivers below dams should be no greater than before the dams were built.

2 Some material damage to Board dams, plant and appurtenant structures may have to be accepted in passing maximum floods but the dam itself must remain structurally safe.

3 Flood management procedures should be compatible with normal operating regimes and where practicable should be refined to give a better utilisation of the catchment area and reservoir storage for hydro-electric output and flood control.

A feature of the Board's schemes is the very extensive diversion of water from outside catchments into the main reservoirs. A general computer program was developed by the Board for its schemes which generally follows the recommendations of the FSR and, although specifically designed for use in the Scottish Highlands, has been applied to other regions.

As a result of the work of the Flood Study Group the opportunity has arisen to study such varied problems as reservoirs in cascade, inadequate spillways, freeboard, the effects of floods following a dam breach and the stability of dams.

It is considered important to make a record for posterity - a record which would allow an experienced Engineer in the future with no knowledge of the Scheme, to fully comprehend the design and operating philosophies adopted. Flood Management Reports are therefore prepared on completion of all re-appraisals, and include: an outline of the original design parameters; the new design criteria; the reasoning behind the new flood handling procedures; the predicted results of their adoption; the consequences of plant and equipment failures; flood control and warning procedures; instructions for operational purposes; bibliography of related background information; data, including copies of relevant reports, correspondence, drawings and discharge curves.

Details of the results of the work by the Board's Flood Study Group are given separately (Ref 3).

RESULTS OF SURVEILLANCE POLICY

The employment of Panel Engineers not connected with the design or construction of the dam for statutory inspections has led to fairly radical appraisals and the bringing to light of aspects not suspected as being unsatisfactory.

During the past decade there has been a trend towards decreasing instrumentation and increasing comprehensive visual inspections. This has led to the curtailment of the use of surveyors, who only take instrument readings, and the expansion of the use of experienced engineers who, in addition to their normal duties, inspect all aspects of a reservoir and take into full account the previous statutory and supervisory inspections. In addition these engineering inspections have included a detailed maintenance inspection as well as a safety inspection. The result of this change of emphasis has been a much more thorough and rigorous surveillance of the Board's reservoirs with no overall increase in resources committed to reservoir safety.

The FSR has introduced a new facility into the assessment of dam and flood safety in Great Britain by placing analyses on a probability basis and allowing them to be quantified and thereby also permitting them to be related to equivalent safety analyses in other fields and industries.

The Board's analyses of storms have resulted in a number of refinements being made to the FSR specifically for application to Scottish Highland conditions.

To date the operational impact of flood studies has confirmed that the existing flood control procedures, frequently developed from operating experience, are generally satisfactory. In a few cases, particularly those in which long duration storms are important, the revised maximum reservoir levels were found to exceed design levels but generally the structures involved have been shown to be capable of accepting the higher levels.

The Board take the view that, provided the structural integrity of a dam is not in question for the appropriate design flood, major flood damage may be acceptable (eg the flooding of a power station). If the position with respect to loss of life is satisfactory it is primarily a matter of economics. The overall application of the FSR has given much increased confidence in these procedures and refined them to give improved operating conditions and flood control.

It is proposed to present a number of case studies (Ref 4) in the oral presentation of this Paper.

THE FUTURE

The Flood Studies Report has introduced a new facility into the assessment of dam safety in Great Britain by placing analyses on a probability basis. The method has already demonstrated defects in previous predictions of maximum design floods from the larger catchments.

The recommendations of the Guide categorise the risks presented by dams and indicate new standards to be adopted for reservoirs. The standards are not inconsistent with those adopted for other industrial projects, particularly in respect of reservoirs which pose a direct threat to communities. There are likely to result in requirements for the discharge of greater floods and more extensive protective works at dams, particularly older ones.

If and when the Reservoirs Act 1975 is introduced it will require more frequent inspections (supervisory) and will place additional responsibilities on the owner, his staff and the engineers. The introduction of the new Act should lead to improvements in reservoir safety and the earlier detection of defects and deterioration but at the cost of appreciable increases in surveillance and in requirements for improvements in reservoir works.

The use of instruments to detect deterioration and incipient failures has expanded with the years and is likely to increase, particularly with the growing trend towards earthen embankments and rockfill dams as their performance and behaviour are much more difficult to predict than those of concrete dams. This increased use is to be commended with respect to these types of dams but one must balance the added capital and running costs of monitoring by instruments with the increase in safety achieved to avoid a state being reached where the former leads to a false sense of security. For uncertain or defective dams, one can foresee the increasing use of automatic instrumentation with alarms to give the maximum period of warning of deteriorating conditions, especially in the light of ever increasing labour costs.

The development of EDM equipment has superseded many optical instruments over the last decade and its use seems likely to increase. It has led to notable advances in speed by which dam movements and behaviour can be measured. As it is less affected by climatic conditions it has resulted in appreciable savings in surveyor time on site.

It is the Board's experience over the past ten years that a well balanced combination of visual inspections, leakage measurements and monitoring by instruments, coupled with flood studies and stability analyses, is the most effective means of detecting deterioration and ensuring dam safety. From a review of these activities it has been found advantageous to increase field inspections and to reduce the effort spent on monitoring by instruments. The Board's experience also demonstrates the need for reappraisal to be a continuing procedure in reservoir surveillance.

Overall the above major developments in surveillance will lead to significant improvements in dam safety in the coming years but these will not be achieved without appreciable costs.

ACKNOWLEDGEMENT

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Dam Practice - Good and Bad

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SYNOPSIS

Despite considerable developments in the design and practice of dams and associated works, examples of bad practice exist, even in recent cases, and the probability of failure or trouble is increased. The paper gives examples of good and bad practice in dams and embankments with the intention of stimulating discussion and the airing of further examples.

INTRODUCTION

The continuing saga of dam failures, incidents and defects, despite considerable developments in theory, practice and expertise in recent decades is frightening. The lessons from the past are available but are not always applied. There are still too many examples of bad practice, and this paper attempts to present some views on this subject in order to lead to discussion of these matters of detail. The examples given have all occurred in recent years and generally relate to dams, but with reference to some highway embankments. The dams referred to are embankment dams, where despite the tremendous advances made in the teaching, research, development and application of geotechnical aspects of civil engineering especially in the last 30 years, examples of bad or incorrect application, or even ignorance, of geotechnical matters are apparent. Many errors, defects, delays, additional costs, claims, incidents and failures would not have occurred if better geotechnical design and application occurred. The fragmentisation of design and construction, with many "specialists" involved can lead to bad practice.

BACKGROUND

In a lecture by Professor Ralph B. Peck[1], in 1980, entitled "Where has all the judgement gone ?" he stated, amongst many other pertinent remarks:

> "whether we like it or note, there remain some aspects of geotechnical engineering in general, and of dam design in particular, that are not yet and may never be amenable to theoretical analysis. This does not mean that we are powerless to lead effectively with these aspects. It does imply, however, that we should not neglect the aspects for which we have no theory while we overemphasize the significance of those for which we do. There is considerable evidence that most failures of modern earth dams, except those due to overtopping have been the result of exactly these misplaced emphases."

Peck also referred to the fact that several reviews of the history of failures of earth dams have led to the conclusion that the probability of catastrophic failure of a dam during any one year is about one chance in 10,000 (probability of 10^{-4} per dam year).

He stated:

> "Would it be possible to reduce the incidence of dam failure by a factor of say 10 ? Is there reason to hope that the historical probability of failure need not be taken as the base level probability ? I think there is. I would venture that nine out of 10 recent failures occurred not because of inadequacies in the state of the art, but because of lack of communication among parties to the design and construction of the dams, or because of overoptimistic interpretations of geological conditions. The necessary knowledge existed; it was not used."

My examples are generally cases where bad practice occurred for a variety of reasons and the probability of failure or an incident was certainly not reduced.

Serafim[2] disputes the probability figure of 10^{-4} per dam year quoted by Peck and others. Serafim points out that if the useful life of a dam is considered to be 100 years, the probability of failure of a dam would be 10^{-2}, i.e. there would be one failure in every 100 dams of any kind built. He considered this basis to be erroneous and misleading as a deeper study of the data indicates that modern dams are much safer than old ones, but also, if appropriate site investigations and criteria are used and if acceptable construction and supervision standards are adopted, dams will become even more reliable structures than they are at present. The author agrees with this view, but unfortunately examples continue to occur when these high standards are not achieved.

EXAMPLES

A few examples are given to illustrate cases of bad practice, under the following headings:

Remedial and improvement works to existing embankments

Drainage

Slope stability

Instrumentation

Testing

Other examples

REMEDIAL AND IMPROVEMENT WORKS TO EXISTING EMBANKMENTS

Re-assessment of spillway capacity and the need to prevent overtopping for the design flood conditions, has often led to the construction of a wave wall, and increasing the height of the embankment. It is unlikely that the stability of the upper part of the embankment has always been considered.

The upper part of the upstream face on many dams constructed originally without adequate slope protection have been eroded to steeper slopes. In one case, concrete slabbing was added to the steepened slope (at about 1 in 1.7) in the 1930's and a wave wall added. Where the wall settled, and the embankment fill as well, additional fill was added to reinstate the top of bank level and additional concrete added to the wave wall. Eventually local slipping occurred.

Using typical soil parameters and for a factor of safety of 1.2 for a 1 in 2 slope, *it can be shown that for an undrained analysis, the addition of a* 0.9m (3 ft.) high concrete wave wall could reduce the F.O.S. to 0.9. For a *drained analysis the F.O.S. would be 1.05 with no pore pressures and lower* with pore pressure. The slip surface would be in the upper 1.5 to 2.0m from the crest.

An additional wave wall should be as light as possible, and the author has used lightweight aggregate in the concrete. Additional loading to a crest of a distressed embankment should only be carried out after a full stability analysis has been undertaken even if the fill strength is believed to be adequate.

Bad practice in remedial work could lead to adverse drainage effects, reducing the shear strength of the fill. Similarly, removal of trees and bushes could reduce the removal of water from the fill, and therefore allow the increase of pore pressure in the fill, and reduce the factor of safety.

Poor detailing of the end of a new or extended wave wall, can also be detrimental. A new wall diverted water from a track on the hillside to the unprotected downstream mitre from its original route to the protected upstream mitre. Heavy run-off caused erosion of fill adjacent to the downstream mitre.

Adding a downstream toe berm, or weight block, to an area where displacement has occurred, needs to be of adequate extent laterally so that the adjacent section has an adequate factor of safety. There are cases where neither cross-sections have been drawn or analyses carried out.

In one case, where a tension crack was observed adjacent to an embankment surrounding a covered service reservoir, constructed on sloping ground, an analysis was carried out but the incorrect assumption was made that the crack was at the toe of the incipient slip surface instead of at the upper extent of a potential slip. The analysis gave factors of safety of over 1.6, and over 3.0 for different surfaces instead of 1.0. The incorrect analysis led to no remedial action yet the crack in the tarmacadam access track continued to extend.

DRAINAGE

Drains can introduce water into embankment fill, and cases of road embankments are known where instability has arisen from this defect. Such cases sometimes arise from changes in design of drains or changes in use of different fills made during construction when the full consequential effects are not appreciated.

In one road embankment constructed in 1975, chalk material for the 0.9m depth below the road formation was permitted to be used and below this cohesive siltstone was used. The permeability of the top layer was about 8×10^{-4} to 4×10^{-6} cm/sec depending on moisture content and compaction. The permeability of the siltstone was less permeable at 5×10^{-7} cm/sec.

A french drain was provided in the central reserve with another french drain on the north side of the carriageway. Gullies to collect run-off from the road led water into open jointed clay pipes in the french drains. The result was the water passed into the fill and a perched water table was formed above the siltstone leading to slipping of the topsoil layer on the outer face. A change in design had taken place in that the original drain pipe was to be perforated over the top 120°, but open jointed non perforated pipes were used. Even if the perforated pipe was used, run-off water would still have entered the fill. The combined drainage system combining surface and sub-surface water was unsuitable for the embankment fills used, and separate drainage systems should have been designed.

Although this was a road embankment, a similar arrangement could have been used for the roadway and the upper part of a dam embankment.

Good practice relating to drainage could follow from the Code of Practice for Earthworks (BS 6031: 1981) published recently [3]. Typical of its contents is this extract:

> "Open channels should preferably be lined. Unlined channels should not be used where interceptor drains are sited near the top of a cutting in a shrinkable clay, since water seeping down shrinkage cracks may cause instability. Open channel or piped drains should be provided at the toe of the slope on both sides of a cutting, and the formation should be trimmed to fall towards the drains. Care should be taken that the construction of the drains does not weaken the toes of the slopes. All drains sited at the tops of cuttings and designed to carry surface water should be lined. Where both surface and subsoil drainage have to be provided it is desirable to install separate conduits for each purpose. Open channels should not be so deep as to render cleaning difficult, 1.2m being usually a practicable maximum."

The reference to the care needed so that drains do not weaken the toes of slopes, could have been written because of bad practice on one recent highway project where the excavation of a toe drain led to local slipping of the embankment. Open drains at toes need to be constructed before the embankment fill is placed, not afterwards, unless there is no foundation problem and the drain is solely for conveying water from a higher area.

The Code of Practice also refers to the use of perforated drain pipes, but without stating in which direction the perforations should be placed. It would appear obvious to many that the perforations should be upwards, but cases are known of the perforations being downward, or even alternately up and down in adjacent pipe lengths.

In the 1978 'Clay Fills Conference'(4), amongst other examples, Snedker illustrated several cases of water entering embankment fill from drains or cracked roads leading to instability. Such cases could arise where concrete roadways are placed on embankments before an adequate time has been allowed for consolidation and settlement.

Examples of drains not functioning as expected can often be found. Defects can include infiltration of fine materials, too low overall permeability for flow requirements, failure of pipes, penetration by roots and growth, open stone drains (french drains) becoming grassed over, etc. In addition, lack of, or obstruction of, sufficient outlet capacity can prevent drainage occurring effectively. In one case the construction of an outlet drainage chamber led to the temporary plugging of a drain pipe from a piezometer house, which became flooded. An outlet can become flooded by the increased tailwater level at times of overflow thus raising the water levels in the foundation and embankment.

Internal drainage blankets, in clay embankments, which are known to be very effective in dissipation of pore pressure have sometimes been subjected to measures to reduce their cost with adverse effects. In one case, a complete blanket was replaced in the design by 300mm square trench drains at 3m horizontal intervals. Piezometers showed high pore pressures with r_u values of 1 or higher and were not dissipating. Investigation showed that trimming of the outer slope had covered the ends of the trench drain with clay fill thus effectively blocking the drains. With a continuous blanket, local covering is unlikely to restrict the drainage flow. The intermediate stage during construction before the designed outlet drainage work has been completed, can often lead to adverse conditions, which may leave weaker zones in the completed dam.

Relief wells require special consideration, and should be completely separate from base blanket or toe drainage systems. Designs have been seen where water can drain down the well recharging the permeable stratum below the dam rather than solely allowing upward relief. Wells clear of the downstream toe enable separate flow measurement or ground water level observations, when the wells are not overflowing, to be carried out. If the wells are beneath the base drainage system, observations cannot be made of the performance of the wells. Additional wells could easily be drilled outside the downstream toe if required, whereas additional wells on the line covered by the embankment could not readily be undertaken.

A recent article described a road embankment that slipped during construction where the slip plane was in wet fill immediately above the base drainage layer. Insufficient drainage must have the primary cause.

There are many papers and books describing good drainage practice, yet still unsuitable filter gradings may be used. Thirty years ago, column drains of hand placed stones with open joints, were sometimes used in British earth dams. Despite their lack of finer materials they may be more effective in keeping a downstream shoulder drained, than some later designs with thin layers, insufficient carrying capacity and restricted outlets.

SLOPE STABILITY

On most large construction sites, especially for fill dams, local slips occur due to undercutting of slopes for temporary and sometimes for permanent works. Often these could be avoided if cross-sections were taken and analyses and studies made, instead of rushing in with unplanned excavation, and stock-piling of spoil.

At a dam in Cyprus, a borrow pit in the reservoir area was sited at the toe of a slope, and on impounding an area of hillside became unstable. The area extended to a level of at least 80 metres above top water level, and large remedial measures were necessary.

The design of concrete service reservoirs on a steep hillside in an area of possible mining subsidence incorporated several features for stability and involved small tanks on independent foundations with flexible pipework. However, the excavated material was stock-piled on the steep hillside adjacent to the work and the surcharge caused an extensive area of hillside to become unstable, causing damage to houses at the toe of the slope.

INSTRUMENTATION

There are many examples of bad practice in instrumentation where not sufficient consideration is given to the information on performance that will be available. Two points only to establish ground water conditions, or foundation or abutment standpipes installed to a predetermined depth that was not varied when the level of the change in strata was found to be different from the original assumptions,are typical examples. Lack of original datum observations before dewatering during construction, or before impounding, are also known. Settlement gauges not finally installed and read until the embankment is complete have also been reported. Heating a piezometer house for years to avoid freezing but without taking any observations has also happened. Taking observations at a major overseas dam for four years without any plotting, consideration or analysis is also known to the author.

Bad practice in instrumentation can sometimes arise when too many parties are involved and responsibility is divided. The author's preferred method is for the consulting engineer to be fully responsible, and the Resident Engineer to direct the installation, testing and reading of instrumentation, and for equipment to be obtained from nominated suppliers, not necessarily on the basis of least cost. Discussion of the proposed instrumentation layout and details with advisers is recommended, with the engineer preparing the final layout, rather than entrusting the instrumentation programme solely to an outside adviser or specialist contractor. The layout should be considered flexible to allow for changes in progress, materials or even weather during construction.

TESTING

Cole in a discussion in 1979[5] referred to a site in Africa where after each layer within a fill area of a road embankment had been placed and compacted, the fill was checked for acceptable density by at least two sand replacement tests. It is stated that these took two or more days to complete, during which time no further layers were allowed to be placed. The surface of each layer was subject to desiccation under the generally fine weather conditions. Large areas of the downstream face of the road embankment subsequently slipped. Water was found to be seeping from the slipped zone with the level of seepage being the same as the level of the catchment area on the uphill side of the embankment, where ponding occurred after heavy overnight rain. It was concluded that the desiccated top of each layer had become a more permeable zone.

The author stated:

> "the primary cause of the problem could be held to be over rigid adherence to a performance specification and it is likely that had the specification been a method specification, the time interval between placing succeeding layers would have not been sufficient for the degree of desiccation encountered to have occured."

Why testing required "two or more days" is not explained, and would certainly seem to be bad practice. The B.S. drying specification refers to 16 to 24 hours as usually a sufficient time for drying most soils, but micro-wave ovens can be used reducing the time to a few hours. 30 years ago, the author was engaged on control testing on an earth dam, where drying was carried out adequately on a tray over an electric fire over lunch-time - results were all available in 2 hours. Rigid adherence to impractical specifications without on-site checking and consideration have been encountered elsewhere. Placing of clay cores has sometimes been held up because of one moisture content or density test result falling outside the speciciation by 0.1%.

The method of specifying the suitability of compacted clay in clay cores or clay shoulders by means of a shear strength specification has certain advantages over one based on optimum water content and relative density and has been discussed in a recent paper(6).

The method, which would be considered as good practice compared with the method used in the above case history of the road in Africa, has been used in recent years on many British earth dam cores. The principal advantages are that the measured strengths relate directly to the design fill performance and that the results, as well as being fairly quick to obtain, are independant of any variation in optimum water content of the material. The minimum shear strength can be readily related to a stability analysis of the embankment. It is considered that the method is more precise than only a relative density requirement as with modern plant there is usually no difficulty in achieving satisfactory compaction. In specifying the range of strength to be attained, the lower bound is usually governed by the limits of plant trafficability not stability considerations, and the upper bound by the need to ensure a plastic core.

At one site weathered clays, from the previous overburden and now lying in worked out brick pits, were subsequently used in forming embankments up to 20 metres in height. The natural water content of this material was 10-15% in excess of its original water content which was close to Proctor optimum. It is considered a shear strength specification enabled selection of suitable materials whereas a relative density specification would have shown the material to be outside normally acceptable limits.

It is considered that a shear strength specification, combined with a compaction requirement, for controlling and placing clay materials, principally in rolled clay cores of embankment dams, although not perfect, is considered to be an advance on previous methods based exclusively on optimum density and moisture content, and thus in certain situations can be a good practice.

OTHER EXAMPLES

A recent articie described remedial measures to a motorway embankment where a slip had occurred, and which on the face of it, could have been avoided if good practice had been adopted. The article stated that the original slope was 1 in 1.4 (sic) and an analysis carried out after the slip using parameters resulting from samples obtained from boreholes and assuming no pore water pressure gave a factor of safety of 1.08. The top edge of the slip passed through holes which had been dug for tree planting. The holes had been left open over the winter and had allowed surface water to get into the embankment (Did the drawing state "please slip along the dotted line" !). If such conditions exist with recent construction to modern design standards and specifications, there could be worse conditions in old dam embankments.

Local steepening of embankments for topographical reasons would require stronger fill (e.g. rock fill) or improved drainage, having recognised the problem and to maintain at least the same factor of safety as for adjacent flatter slopes. A suspect area should also have suitable displacement observation points, so that early warning of a potential slip could be observed, although it may still be too late to take effective remedial action.

A small irrigation storage reservoir (of less than 5 m.g. capacity) constructed in 1975, after completion leaked in two areas. One leak was found to be due to the inclusion of a layer of hardcore under the spillweir channel concrete slab. This layer was not shown on the drawings. The other leak was found to occur under a pipe that had been laid temporarily for stream diversion during construction, and with the engineer's approval had been left under the embankment provided that a concrete collar was formed around the pipe and that the upstream end pipe length was removed and replaced with clay. After the leak, it was found that the concrete collar had not surrounded the pipe and there was a small void under the pipe. Good practice for concrete collars around pipes for small irrigation dams is well covered in text books and in publications of the Ministry of Agriculture, Fisheries and Food.

Good practice is well documented in books, articles and papers but they are not always referred to at the right time and in some cases dam work is designed by those with insufficient experience of good practice.

CONCLUSION

Although further examples could be given under the given headings or others on the topics of filters, spillways, pipes or concrete, sufficient cases have been included to stimulate discussion and to show that there is scope for better practice in design to improve safety in the construction and performance of dams and embankments.

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