British National Committee on Large Dams (BNCOLD)

BRITISH NATIONAL COMMITTEE ON LARGE DAMS (BNCOLD)

88 RESERVOIR RENOVATION

PROCEEDINGS OF A SYMPOSIUM BY THE BRITISH NATIONAL COMMITTEE ON LARGE DAMS

UNIVERSITY OF MANCHESTER

MANCHESTER, 14–17 SEPTEMBER, 1988

Edited by D. A. K. HUGHES

Proceedings of a Symposium by the British National Committee on Large Dams.

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Dr A K Hughes

THE THEME

Reservoir Renovation is a broad theme, reflecting increased interest being shown in the complex problems associated with the continued safe operation of a stock of dams whose average age is almost 100 years.

The theme of the Symposium is of special interest to engineers in water authorities, inspecting engineers, supervising engineers and engineers working with the enforcement authorities, and all who are employed in or have an interest in the design, construction and operation of reservoirs.

THE EVENT

The Symposium was directed towards all engineers involved with old dams and those actively employed by the universities, the enforcement authorities and government.

The Symposium was intensive, with eight Technical Sessions concentrated into two working days.

The themes selected for the Technical Sessions were

Technical Session 1 :	'Enforcement'
Technical Session 2 :	'The Supervising Engineer'
Technical Session 3 :	'Renewing and Updating Drawoff Works'
Technical Session 4 :	'Overflow Repairs and Extensions'
Technical Session 5 :	'Instrumentation and Drainage of Embankments'
Technical Session 6 :	'Embankment Deterioration'
Technical Session 7 :	'Gravity Dam Deterioration'
Technical Session 8 :	'New Materials for the Renovation of Dams and Reservoirs'

The prestige lecture - the BNCOLD Lecture 1988 - was given by Dr D J Coats CBE BSc FEng FICE FIWEM FGS FASCE, and was entitled 'The Concerns of a Dam Engineer'.

On the third and final day attention was focused on selected local dams, participants visiting Winscar & Longdendale with a supplementary visit being offered to Haweswater.

SYMPOSIUM STEERING GROUP

The Symposium was organised by a Steering Group

R M ARAH	MA DIC FICE MIWEM
A K HUGHES	BSc PhD CEng MICE MIWEM MBIM
A I B MOFFAT	RD BSc CEng MICE FGS

PREFACE AND INTRODUCTORY NOTES

The theme of reservoir renovations was endorsed by the British National Committee on Large Dams late in 1986.

The interest in the theme proved to be much greater than originally anticipated and led to 36 Papers being presented before over 260 participants. It was particularly gratifying to note that some 20 overseas guests attended, representing a span of countries from Europe to the Far East and on to the Americas.

The Symposium proved to be a lively and stimulating affair, not least to the members of the Symposium Steering Group. Credit for this must rest with the Authors and Session Chairmen who contributed so much to the success of the eight Technical Sessions, and also with the participants who so actively involved themselves in discussion, formal and informal.

I would like to record my appreciation to my colleagues on the Steering Group, Messrs Arah and Moffat. I would especially like to thank Mr R M Arah for his assistance in editing the discussion contribution and Mrs E M Upton for her valuable assistance in typing the Discussion Document. Thanks are also due to Mrs G Arah and Mrs S C Hughes for their assistance with the Ladies' Visit and for their patient help with so much of the preparation.

The reports and discussion material presented in these Proceedings are based on tape recordings of the Technical Sessions. Editing of the recordings has been necessary but has been kept to a minimum and every care has been taken to provide an accurate account of what was said.

Many participants illustrated their contributions with the aids of slides, diagrams and other visual aids. Where this was the case the symbol S appears alongside a necessarily edited version of that contribution. In almost all cases Authors made a short presentation (5 mins). Comments which highlighted the main issues contained in their papers have been included in the discussions where they were not included in the Authors' papers.

A detailed Index for each Session immediately precedes the Papers presented in that Session. A list of participants, giving details of their affiliation and, where appropriate, of their contribution to the Symposium, appears at the conclusion of this volume.

The Editor apologizes for any inaccuracies of which he may inadvertently have been guilty.

Dr A K Hughes
November 1988

Note All authors were allowed time for presentation of their paper prior to opening the session to discussion. These presentations have not been reported in the discussion documentation.

Unfortunately some authors were unable to meet the deadline set by the Editor for written contributions and answers.

CONTENTS

BNCOLD LECTURE

INTRODUCTION BY E T HAWS (Chairman of BNCOLD)

David Coats graduated in Civil Engineering from Glasgow University in 1943 and was awarded an Honorary Doctorate by his Alma Mater in 1984. The intervening time started with four years with REME, mostly in India and Major Coats received his C-in-C's commendation. David joined Babtie, Shaw & Morton in 1947 and has been with them ever since. He held the Senior Partner position between 1979 and 1988, only recently relinquishing the Chief Executive's role for that of Senior Consultant.

His early hydro-electric design years were quickly followed by site work at Glenshira and the ER post of Attnalareigh. He soon moved south to the Babtie London Office, where he looked after a shipyard at Lowestoft and report work, before joining a partnership in 1962. Returning to Glasgow in 1963, he looked after numerous projects, including the Llochinvar Dam, Loch Tonn tunnel outlet, the modernisation of lighthouses in Sri Lanka, Spanander Dam and Coursehouse dam until, in 1971, he became Senior Partner in the Water Division. From there, and his Senior Partner position, he had overall responsibility for the Kielder Water Scheme, on which subject he published 7 of his 14 technical papers and won the Telford Medal of the Instutition of Civil Engineers. This project involves Kielder Dam itself, Airy Holm Dam and Bakethin Dam. David is a member of Panel AR under the Resevoirs Act of 1975, following his old Panel 1 membership. He has carried out numerous associated inspections, and is construction engineer under the Act for the reconstruction of Carsington.

Among his Fellowships, David counts those of the most prestigious Fellowship of Engineering, and the Royal Society of Edinburgh, along with 5 major institutions. He has been Chairman of the Association of Consulting Engineers and is a Vice-President of the Institution of Civil Engineers. He has provided services and leadership to ACE and ICE, particularly in Scotland, and has been on steering committees of TRRL, BRE, the Fellowship of Engineering and his old University.

BNCOLD LECTURE - 1988

THE CONCERNS OF A DAM ENGINEER

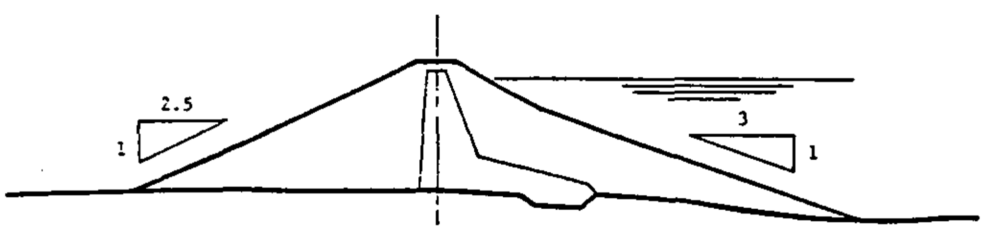
DR D J COATS CBE FEng FRSE

SYNOPSIS

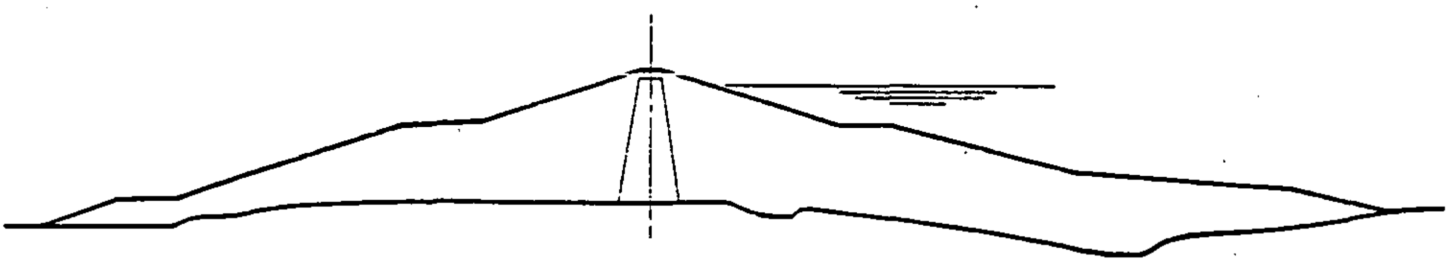
The work 'concern' has at least three meanings (a) anxious or 'of concern' (b) responsibility as in 'that is your concern' and (c) a business organisation. The preparation and practice of a dam engineer is discussed with this as a background, and issues pertinent to the present time of changing attitudes, well-meant controls or restraints and rapidly expanding technical knowledge raised.

INTRODUCTION

1. I was honoured and indeed flattered to be asked to deliver this lecture but, on reflection, foolish to accept. Perhaps it is my connection with Carsington Dam that prompted the invitation since the theme of this Conference is Reservoir Renovation. You are all aware that this dam failed during construction in 1984 and that a contract for its reconstruction is at present out to tender. The original design and new profile are shown in Sketch 1. The reasons for the significant changes might be of interest, but perhaps that should be left for another occasion. However, this set me thinking about other earth dam failures and the modifications adopted.



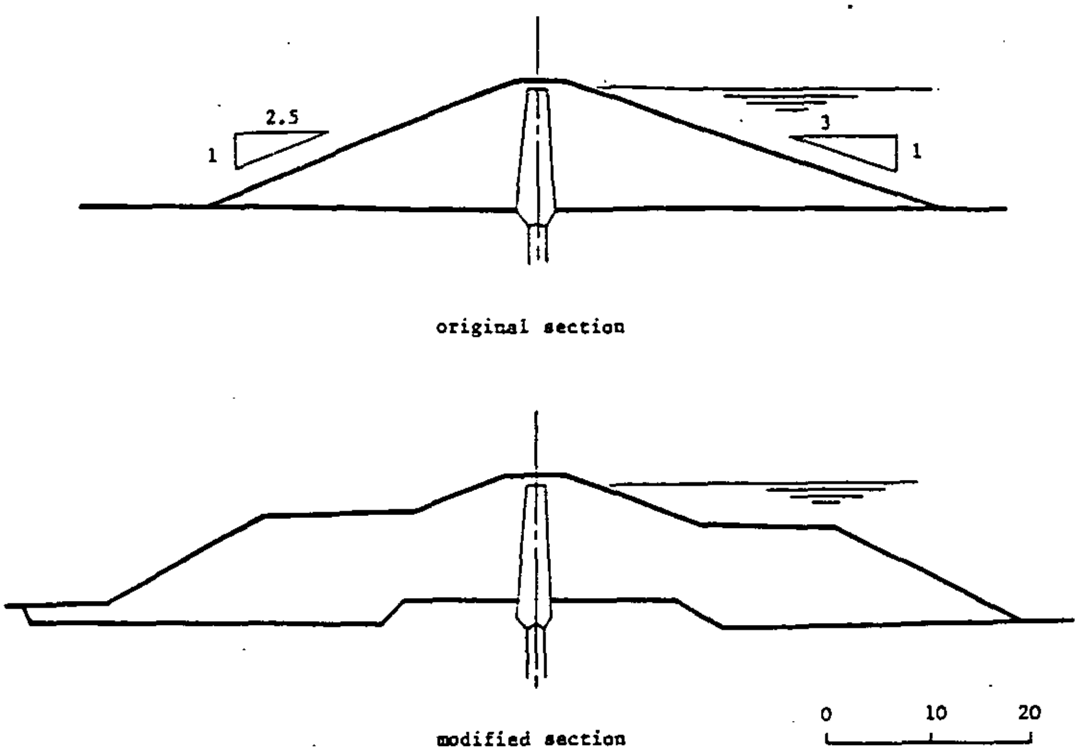
original section



modified section

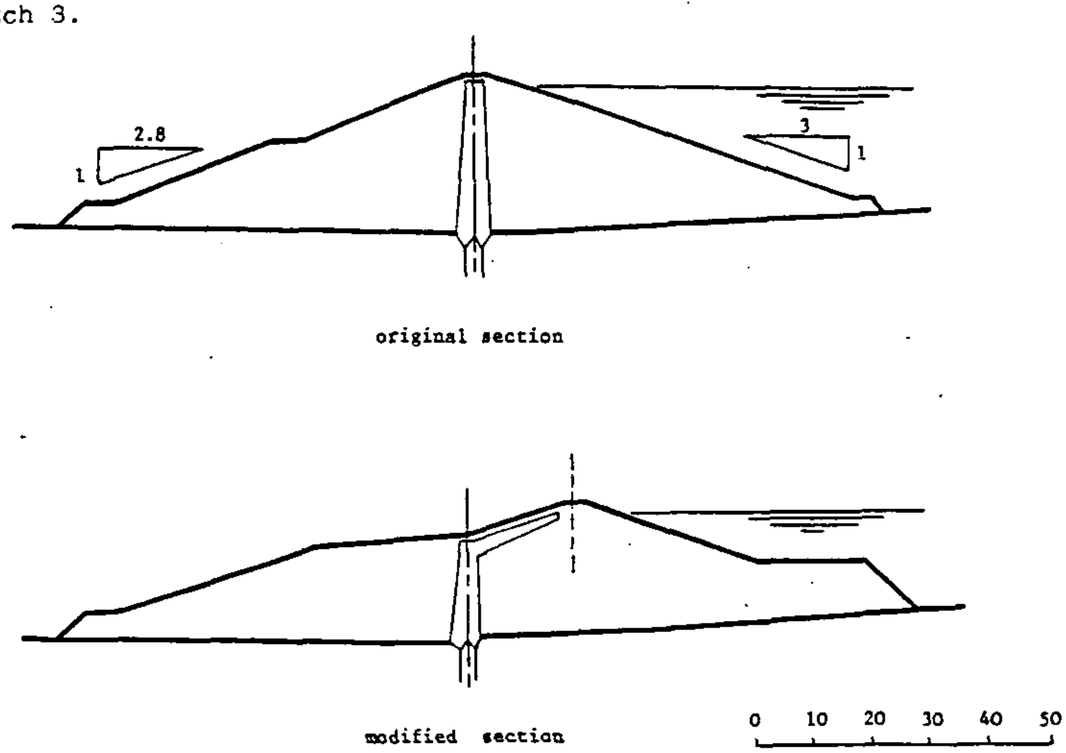
SKETCH 1 - CARSINGTON DAM

2.	Chingford which failed in 1937 (1) sprang to mind. The original section and revised design is shown in Sketch 2.



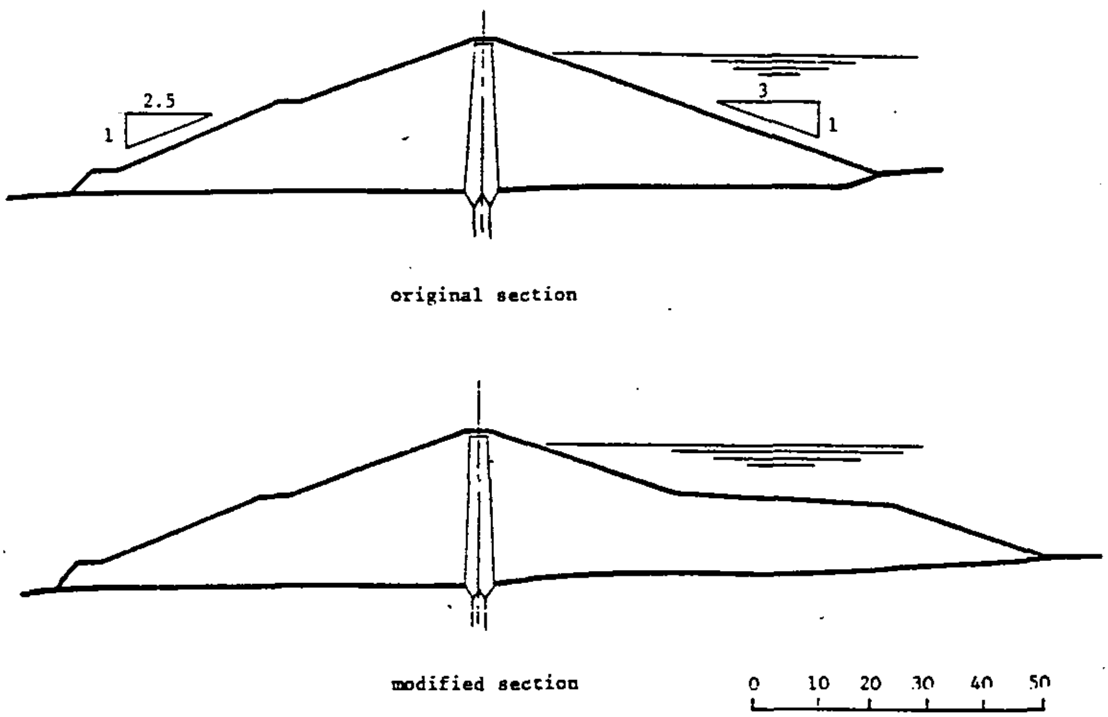
SKETCH 2 - CHINGFORD DAM

3.	Nearer home, as far as I was concerned, was Muirhead Dam which failed in 1941 (2). The original section and modified design are shown on Sketch 3.



SKETCH 3 - MUIRHEAD DAM

4. Now at the time Muirhead failed, another dam of almost identical design was being constructed just 5 kilometers away at Knockendon and it was modified to ensure that it did not fail also. To remind myself of these modifications I had to refer to the paper by Mr Banks in the 1952 Volume of the ICE Proceedings (3). The original design and modified profile are shown on Sketch 4.



SKETCH 4 - KNOCKENDON DAM

5. I introduce these sketches simply to gain your attention! It is not my intention to discuss them no matter how interesting a comparison of the remedial measures adopted might be. I simply use them as a lead-in to the 1952 Volume of the Proceedings of the Institutions of Civil Engineers.

THIRTY-SIX YEARS AGO

6. The 1952 Volume was a revelation. In addition to the Knockendon paper by Banks it also included Fulton on Civil Engineering Aspects of Hydro-electric Development in Scotland; Harding on The Progress of the Science of Soil Mechanics in the Past Decade; Crump on a New Method of Gauging Stream Flow; Rowe on Anchored Sheet-Pile Walls; Scott on a New Method of Tunnelling in London Clay and Rodin on Pressure of Concrete on Formwork - to mention only 7 of the 22 papers. There was also a report from the Glasgow and West of Scotland Association which referred to a student paper on 'Fish Problems on Hydro-electric Schemes' by M F Kennard. It must have been quite a year!

7. These papers seem to encapsulate the atmosphere which I breathed when I became chartered in 1951 and started me wondering about changes since then.

8. Harding's paper was the 58th James Forrest Lecture and he included the following in his conclusions:-

> 'When young engineers first read our Proceedings they may feel that they draw little benefit from them. It is only after amassing some experience that the searcher can see the real wealth of information and value which they contain. The small points in odd Papers add up to a considerable body of fact. They are the work of many minds, and contributions to a discussion, by recording or confirming an experience, can add to the vast accumulation of knowledge which can only be glimpsed by those with the time, interest and experience to delve into the older Papers of predecessors'.

9. I am only just appreciating the wisdom of that remark and I commend to younger engineers the practice of delving into the older papers of our predecessors to a greater degree than I have done.

10. Considering the modified profiles of the unfortunate dams to which I have drawn attention and thinking about other dams it occurred to me that whereas the materials used and the forces and conditions that must be taken into account when designing most works have changed with time, those applying to dams have not, although our understanding of them has improved. Roads and bridges have now to be designed for much greater loadings than previously because of heavier vehicles; a higher quality of supply is required from water treatment works; the standard of effluent from sewage treatment works must be better than in the past; ships, industrial plant and required cranage have changed so that loads on the structures of industrial and marine works have increased; man-made materials used in many structures have changed as new products are introduced; and so on, whereas winds and rainfall are the same as they always have been, the incidence of earthquakes has not materially altered, water has the same density as before and gravity is as Newton found it. These are the forces applying to dams, and, as far as earth dams are concerned, the materials used are natural materials which, by definition, do not change.

11. The changes that have affected the design of dams have been improvements in understanding which not only allow us to use materials previously considered inappropriate and to build dams to heights and at locations not contemplated in the past but also has brought home to us the unexpected behaviour of materials. This understanding combined with a desire for increased safety has made, or should have made, our designs more and more conservative - a feature not always shared by other types of works. It's a thought I leave with you.

12. For the moment I must emphasise that, in my view, dam engineers probably have more need of insights from researchers and the benefits of the experience of others than those in many other branches of civil engineering. It is for this reason that Conferences such as this are so important. To quote further from Harding's paper in the 1952 Volume:-

> 'Let us hope that many more (papers) will be presented, for it is only by the combined efforts of all its members that our great profession can expand its knowledge to the use and convenience of man.'

13. Garth Watson, the former ICE Secretary, tells us (3) that in early 1818, twelve weeks after the founding of the Institution, it was agreed that 'every member shall produce at least one original essay in the course of each session'. There were eight members! On becoming chartered I had signed an important undertaking to the effect that I would ' in the course of my professional career endeavour to present to the Institution an original paper or contribution relating to engineering science or practice'. So the tradition had continued in a modified form and it is interesting to note that in 1952 three future Presidents - Harding (1963), Banks (1965) and Fulton (1969) - fulfilled their obligation. If the ICE List of Members is to be believed, only half of the ten most recent Presidents has had a paper published in the Proceedings before becoming President. Does this suggest that the Institution is paying a reducing regard to its learned society role? If so, others must step into the breach.

14. Certainly opportunities to discuss technical matters under other auspices have increased enormously in recent years with ICOLD and BNCOLD taking a leading role in relation to dams. The Transactions of ICOLD Congress are a mine of information as are ICOLD Bulletins in connection with which BNCOLD members have made valuable contributions.

15. But let me return again to the papers in the 1952 volume and to those of Harding, Banks and Fulton (as it happens, a contractor, a consulting engineer and a client's engineer) in particular.

16. Harding explained that 'the practical application of soil mechanics requires a compromise between the methods of the exact sciences such as The Theory of Structures, and those of the empirical ones, like geology'. This need to marry theoretical considerations with experience is certainly true, in my view, in dam engineering as a whole but how is it to be achieved and to what extent can it be achieved in an individual? Considering the present state of knowledge, the rate at which such knowledge is being expanded, and the restricted opportunities for experience on dams, what should be the preparation of a dam engineer?

17. When considering the failure of Muirhead in 1941 Banks raised the question 'as to whether or not the greatly increased rate of forming the embankment had a bearing on the movement that developed' - an aspect also reflected upon when Chingford failed. He collaborated with Building Research Station and pore pressure observations were made at Knockendon (I believe for the first time at a dam in Britain) using perforated steel tubes driven into the fill. This good example of collaboration between consulting engineer and research organisation has been followed on many occasions and in happier circumstances since, and raises the question for us today of what should be, or can be, the relationship between practicing engineers and researchers?

18. Fulton's paper was presented at a time when virtually all of the conventional hydro-schemes that were ultimately built in Scotland had been identified but, on the basis of anticipated output, only 20% were in operation, 26% were under construction, 10% promoted but not yet started and 44% at the survey or under promotion stages. These schemes represented under half of the potential economic power output in the Scottish Highlands. However, economic concepts seem to change. I like the story of the chap who visited his old university after 30 years and who was shown a recent examination paper in Economics. He saw that the questions were almost exactly the same as those in his final examination paper but he was told that the correct answers were quite different. Short term economic considerations effectively put an end to further hydro development in Scotland although those, such as myself, who greatly benefited from working on schemes in the 1940s and 50s wholeheartedly supported Frank Johnson when he advocated (5):

> 'A long term programme of steady development of our remaining hydro resources since this is a fully proven, indigineous, benign and very attractive source of power, combining it with pumped storage wherever possible'.

19. At one point in his paper Fulton said there were fewer examples of novel civil engineering design in the Scottish schemes than in other countries. This surprised me because a large number of different types of concrete dams were eventually constructed including the Allt-na-Lairige prestressed concrete dam with which I had the privilege of being associated; fly ash and Trieff cement were used; flexible concrete core walls were incorporated in earth embankments and a number of other newish techniques employed. However, it was refreshing to be reminded of a client who obviously wanted to try new things! The present climate would seem to be epitomised by the recent article in an American periodical entitled 'Don't innovate - it is dangerous'.

20. Fulton's paper also demonstrated the engineer's interest in and responsibility for, not only engineering including non-civil engineering, matters but also economic, social and amenity aspects - all of which are being given increasing importance, and rightly so, today. Can or should engineers be credible in these related fields and how can the indisciplinary nature of our industry best be nurtured?

21. With these questions in my mind I decided to consider in this Lecture the preparation and practice of a dam engineer. In my chosen title 'The Concerns of a Dam Engineer' I was conscious that the English word 'concern' has at least three meanings. (a) anxious or 'of concern' (b) responsibility as in 'that is your concern' and (c) a business organisation.

UNIVERSITY EDUCATION

22. As I entered my second year at Glasgow University the nearby Muirhead Dam failed. There was no direct connection and I was not aware of the failure at the time. William John Macquory Rankine had been Professor of Civil Engineering at Glasgow from 1855 to 1872 and it

was he who was instrumental in introducing a degree in science in any department of study. Rankine's own university education at Edinburgh was in Chemistry, Natural History, Botany and Natural Philosphy and, as his one hundred and eleven published papers show, his interests included thermo-dynamics, elasticity, hydrodynamics and shipbuilding as well as the stability of masonry dams and earth pressure theory. He was involved with the promotion of the Loch Katrine Scheme and he also found time to entertain at the piano and to command a volunteer regiment! I am indebted to Hugh Sutherland for all this information (6) and I introduce it to illustrate the broad antecedents of civil engineering at my old University.

23. My own degree course was not quite so wide ranging but it was broadly based and thereafter I spent four years in electrical and mechanical engineering of a military nature. This may have made me a 'jack of all trades and master of none' but, hopefully, it left me with an understanding of the inter-relationship between different facets of engineering and with the tools to develop. This must surely be the purpose of education. I was anything but the complete engineer (and may have deteriorated since!) and I view with suspicion any academic who sets out to impart to students all the knowledge that he will require in his future career.

24. There is much debate at present as to what a budding civil engineer should be taught. Some say that there has been too much teaching of engineering science and not enough teaching of engineering design. Sir Alan Harris seems to take this view as does Sir Donald McCallum of Ferranti who said earlier this year(7):-

> 'The techne of technology comes from the Greek word for act and there can be no art without action. Art is not all intellectual activity on its own. Until there is a product we can see or hear or touch it has no value. Analysis and thought alone do not produce a painting, compose and perform a symphony or produce a statue. Neither do they produce a building, a bridge, an aircraft or a computer. The future for successful education for industry must help to generate in the student the successful combination of thought, decision and action'.

25. Others would add into the university training of engineers related but non-engineering subjects which would no doubt be most useful but which could either extend a course to impossible lengths or dilute the engineering content. One of these is management education. Sir Terence Beckett, lately of CBI, said recently 'the basic requirements of education of literacy and numeracy should be added to them what was called operacy, the art of getting things done'.

26. Our current President of ICE is fond of saying that construction is a management intensive industry but in my experience, civil engineers in particular are disparaging of management theory. On reflection I certainly would have benefited by more and earlier understanding of management, which is the art of making things happen, but I can recall approaching a lecturer when I was a student on this matter to be told that, in his view, students in the engineering faculty were not management material because we always had difficulty in getting someone to act as secretary of the Engineering Society! Certainly, there are some who want teaching in management because they want the status of manager but have no inclination to make things happen.

27. I think that there are similarities between management education and health education. There are some who are naturally healthy; others who keep their health by observing the rules learned at their mother's knee; others who are fortunate enough to have someone keeping an eye on them; and others who have to take great care of their health. But all would benefit (possibly to different degrees) by an understanding of how the body works, what causes disease and how and why certain techniques can be helpful. These are not learned by experience but from those who have studied medicine. I leave you to identify the management counterparts. There are, of course, management hypochondriacs who must try the latest techniques whether or not they are needed, and there are occasions when one must call in the doctor.

28. With so much desirable knowledge, both technical and non-technical becoming available, a re-think of the content and pattern of our formal training became essential. We have heard from Chilver, Finniston and others and now The Engineering Council is proposing a new generalist engineering degree, presumably to provide the foundation for a variety of more specialist second degree courses. This is very reminiscent of the old Scottish pattern for Lawyers and Ministers who graduated in the Arts before studying Law or Theology. This excellent pattern foundered on the rock of availability of student grants. It would be nice to think that this hazard is now removed for I can see advantages in a required two-stage approach to engineering tertiary education where the first stage is broader than the present Bachelor degree and the second stage less narrow than the present Master's degree.

RESEARCH

29. No matter how extensive a professional engineer's formal technical education may have been he is required to keep up to date. Lord Bingham, one of the Appeal Judges on the Abbeystead case, reminded us of this in his (unfortunately minority) judgment when he said:-

> 'A professional man should command the corpus of knowledge which forms part of the professional equipment of the ordinary member of his profession. He should not lag behind other ordinarily assiduous and intelligent members of his profession in knowledge of new advances, discoveries and developments in his field. He should have such awareness as an ordinarily

> competent practitioner would have of the deficiencies in his knowledge and the limitations on his skill. He should be alert to the hazards and risks inherent in any professional task he undertakes to the extent that other ordinarily competent members of his profession would be alert. The standard is that of the reasonable average. The law does not require a professional man that he be a paragon, combining the qualities of polymath and prophet'.

30. In that connection, I was interested to note that, whereas the subject of Question 59 discussed at the Lausanne Congress was 'Rehabilitation of Dams to ensure safety', Raymond Lafitte, the General Reporter for this Question, reported at the closing ceremony that the Question was 'concerned with the rehabilitation of dams in order to enhance their safety'. But the obligation of keeping up to date is an onerous one. In 1939 Terzaghi wrote (8):-

> 'At the beginning of the century the engineer was entitled to consider the misbehaviour of his structure as a deplorable 'act of God' and he was able to justify his claim by producing textbooks containing the essence of the knowledge of contemporary authorities. However, as soon as some members of the profession acquire the capacity for predicting a phenomenon with such a degree of precision as is shown, in many published records, the phenomenon ceases to deserve the title of an 'Act of God'. The same holds true for the effect of pumping on structures adjoining a well, for the failure of underpinning operations to accomplish their purpose, for the failure of earth dams, and for many other phases of practical earthwork engineering. Today there is still some justification for the excuse that the methods for dealing with these problems are new and therefore only familiar to a small group of specialists. One cannot blame a physician in New Orleans for having failed to save the life of a patient by means of a method which had recently been developed in the Rockefeller Institute in New York. This excuse, however, will certainly not hold for ever, and the time is approaching when the Courts will decide against the designer who refuses to take notice of the existence of soil mechanics'.

31. Soil mechanics is now, of course, in the text books but so much more is being revealed almost daily that it is sometimes frightening to consider what other ordinarily competent members of our profession would claim to be common knowledge. A plethora of learned papers and articles by academics, researchers and practitioners is too much of a good thing and presents a problem, which is discussed at some length in a recent ICE Report which suggests that it has the 'effect of recycling undigested material' (10). CIRIA (9) feels that busy professionals need to have trusted but up to date guides to best practice in their intellectual tool kits', BRE are anxious to help with their Technical Consulting Service, DoE are sponsoring the production of Guidance Notes and ICOLD produces excellent state-of-the-art Bulletins. Others go further and seek Codes of Practice, and on this I must comment in relation to dams.

32. Referring to the Code of Practice on Site Investigation, Harding (1952 again) wrote:-

> 'It is as well that such Codes are only permissive, for much damage can be caused to the profession and to industry by ill-informed lesser officials relying blindly upon a Code without the knowledge to apply it with discretion -
>
> 'But man, proud man,
> Drest in a little brief authority,
> Most ignorant of what he's most assur'd
> His glossy essence, like an angry ape,
> Plays such fantastic tricks before high heaven
> As make the angels weep.'

and a typical Alan Harris's explosion reads (11):-

> 'The fences built to contain innovation are what we call Codes. Now Codes as such, the summarising by experienced engineers of sound practice in a mature technique, are wholly praiseworthy; as with so many things, it all depends on how they are used. The prevalence and proliferation of Codes at a time when engineers have never been better educated, their nature, the ease with which engineers can be found to sit unpaid on their editing committees - nay, the pride with which they announce their membership - all point to a deep-seated need on the part of the community of engineers to fence in innovation.'

33. However, there is no doubt that Codes of Practice are essential in certain areas and a distinct comfort in others, but dams, in particular, are so diverse in character that any suggestion of uniformity of design is ridiculous. Even a Code of Practice for the inspection or supervision of dams is hard to imagine and far from protecting the dam engineer, such a Code, if followed, may even put him at risk in certain circumstances which require the exercise of judgment. By the Reservoirs Act, this country, in its wisdom, has placed the responsibility for dams on individuals who need all the help they can get but who in the last analysis must be free to act as they think fit under the prevailing circumstances and not be constrained by other than appropriate knowledge and experience. Any dam engineer will welcome information and consultation which will allow a better understanding of the problems he faces and we must do everything possible to improve communications which will allow this.

34 The American Society of Civil Engineers publishes Manuals on Engineering Practice which they define as:-

> '...an orderly presentation of facts on a particular subject, supplemented by an analysis of limitations and applications of these facts. It contains information useful to the average engineer in his everyday work, rather than findings that may be useful only

> occasionally or rarely. It is not in any sense a 'standard', however; nor is it elementary or so conclusive as to provide a 'rule of thumb' for non-engineers. Furthermore, material in this series, in distinction from a paper (which expresses only one person's observations or opinions), is the work of a committee or group selected to assemble and express information on a specific topic...'

I find this approach interesting although basic.

35. The Reports or Bulletins prepared by ICOLD Technical Committees should be of particular assistance to dam engineers although lack of finance sometimes prevents then being published timeously. They are mainly, although not exclusively, on engineering aspects as the recent Bulletins on environmental aspects demonstrate. If there are areas where BNCOLD members consider a state-of-the-art statement would be helpful, I am sure that ICOLD will be pleased to hear of them.

PRACTICE

36. There are few engineers in Britain who are exclusively employed on the design or inspection of dams and I am not convinced that this would be a good thing anyway. There are many examples of experience in fields other than dams that has benefited dam engineering. My greatly respected former partner, John Paton, was much involved with dams during the hydro-electric era in Scotland (and he still is). When these schemes tailed off he turned his attention to motorway construction and in his Chairman's address to the Glasgow and West of Scotland Association of ICE in 1961 he wondered why dry-lean concrete, which was used extensively and successfully on roads with significant cost advantage, apparently had no application in dams. He raised this matter again at the ICOLD Congress in 1970 and it is now generally recognised that this started engineers thinking about what is now called Roller Compacted Concrete dams.

37. I am also not convinced that Supervising Engineers under the Reservoirs Act, 1975 were ever intended to do that work exclusively - but that is another story.

38. For the design and construction of dams many people with different responsibilities, knowledge, experience and a variety of abilities as well as those who can provide the funds are involved. It goes without saying that this last is the most important. With the present climate nationally and internationally of reducing capital investment, the trend to favour projects with short-term results rather than long-term benefits and the emphasis on 'value for money' or cheapness before excellence, has the time now arrived when we should come out of the wings and take a more central position on the state of community affairs? We are well qualified to make significant contributions towards forecasting and interpreting change and in the selection of better options for the application on our work in the future. Will Howie would ask even more of us and suggests that:-

> 'The training of the engineer in the scientific method is training in a mode of thinking which is ideally

suited to the politics of democracy because it is fundamentally sceptical. The engineer is that supreme pragmatist. He is never likely to become the slave of any ideology nor to believe that only his side can ever be right. If the scientific method lacks the passion of idealism, indeed is incompatible with it, nevertheless the technical daring which is the mark of the good engineer more than replaces it. The engineer contributes the kind of mind which is required for the politics of today.'

39. It certainly is a pity that even in countries where engineers form the most prestigious occupation of all, the majority are generally servants rather than wielders of power. Their work may transform society but only a small minority pull the strings of political power.

40. Someone recently pointed out that 'engineering can be done by slaves, and often is'. We pride ourselves on being a profession, a concept that we must not lose sight of even in the current climate of commercialism, apparently increasing responsibility and liability and lack of appreciation. But we must not be exclusive. The objectives of ICOLD are 'to encourage improvements in the planning, design, construction, operation and maintenance of large dams and associated civil engineering works by bringing together relevant information and studying related questions including technical, economic, financial, environmental and social aspects'. Participation by non-engineers is, therefore, to be welcomed and I hope that more will take part in our activities in the future.

41. Finally, we are expected to be ingenious (i.e. showing cleverness of invention or construction) and we should also be ingenuous (i.e. honourably straight-forward, open, candid or frank) and I, therefore, cannot resist quoting Matthew Arnold's words that:-

'Conduct is three fourths of our life and its largest concern'.

42. As the theme of this Conference is Reservoir Renovation I leave you with this gem (14):-

'Small may not be beautiful.
Conservative may not be safer.
Don't believe the supplier.
Watch the pressures and hydraulic gradients.
Don't throw away your old books.
Read a few good detective stories
and your embankment dam rehabilitation measures
may be successful'.

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A Brief history of 21 years of BNCOLD

1967 to 1988

by M.F. Kennard

General

1. In March 1988, BNCOLD celebrated 21 years of existence in its present form of individual and corporate members electing the British National Committee, of the British Section of the International Commission on Large Dam.

2. Prior to 1967, the British National Committee comprised representatives from the Institution of Civil Engineers, the Institution of Water Engineers, the Asociation of Consulting Engineers, the Federation of Civil Engineering Contractors and others. There were no individual members, only committee members.

Membership

3. The membership of BNCOLD now comprises both individual members and corporate members. For many years the number of individual members remained at about 200 to 250, but there has been a substantial increase in recent years, due to the implementation of the Reservoirs Act 1975 and the encouragement of Supervising Engineers to join.

The number of members in recent years are:

	Individual	Corporate
1979	215	35
1980	222	38
1981	228	38
1982	249	40
1983	276	41
1984	285	40
1985	300	36
1986	345	37

First BNCOLD Committee

4. The Chairman in 1966 was J.A. Banks and in October of that year, he proposed that the existing members serve for different periods of 1, 2 or 3 years so as to provide for elected members to become members of the committee as from March 1967.

5. The original members who continued on the new committee were:

J.A. Banks	R. Le. G. Hetherington
Professor A.W. Bishop	J. Kennard
J. Guthrie Brown	R.H. MacDonald
E.J.K. Chapman	Dr. N. MacGregor
R.H. Cuthbertson	J. Paton
H.H. Dixon	T.A.L. Paton
C.R. Elliott	C.M. Roberts
Dr. A.A. Fulton	C.H. Spens
	Professor S.R. Sparkes

6. The first elected members were:

P.L. Aitken
G.M. Binnie
P.B. Mitchell
Dr. A.D.M. Penman

Chairmen

7. The first Chairman of the newly formed British National Committee was J.A. Banks who continued until his untimely death in 1967.

8. Succeeding Chairmen have been:

H.H. Dixon	(1968-1971)
E.J.K. Chapman	(1971-1974)
R.T. Gerrard	(1974-1977)
M.F. Kennard	(1977-1980)
D.J. Coats	(1980-1983)
R.E. Coxon	(1983-1986)
and E.T. Haws	(1986-)

9. The post of Vice-Chairman was not established until after the death of Mr. Banks. The list of Vice-Chairmen is

Dr. A.A. Fulton	(1968-1970)
E.J.K. Chapman	(1970-1971)
R.Le G. Hetherington	(1971-1973)
R.T. Gerrard	(1973-1974)
C.M. Roberts	(1974-1976)
Dr. D.J. Coats	(1977-1980)
R.E. Coxon	(1980-1983)
E.T. Haws	(1983-1986)
W.J. Carlyle	(1987-)

Committee

10. Since 1967 over 100 members have served on the Committee, and they have involved the majority of the consulting engineers, contractors, public authorities, universities and other organisations active in British dam engineering in these 21 years.

11. The names of these members are:

Committee Members	
P.L. Aitken	1967/70; 1979/82
A.C. Allen	1972/76; 1979/80
L.J.S. Attewill	1987-
Dr. P.A.A.Back	1978-80; 1981-84; 1986-
H.W. Baker	1972-77; 1980-83
J.A. Banks	1967
K.T. Bass	1984-87
R.S. Baxter	1978-80
G.M. Binnie	1967-70
Professor A.W. Bishop	1967-69
J.B. Bowcock	1984-87
R.J. Braybrooks	1980-82
J. Guthrie Brown	1967-76
W.R. Brown	1970-73
R.C. Bridle	1986-
W.J. Carlyle	1975-77; 1980-84; 1986-
E.J.K. Chapman	1967-74
Dr. J.A. Charles	1984-87; 1987-
C.L. Clarke	1973-76
Dr. D.J. Coats	1969-72; 1973-
N.J. Cochrane	1969-72
R.G. Cole	1985-
G.A. Cooper	1986-
R.G. Court	1980-82
J.G. Cowie	1983-86; 1987-
R.E. Coxon	1969-72; 1973-76; 1978-87
J.R. Crichton	1976-79; 1982-85
R.H. Cuthbertson	1967-69; 1971-74; 1978-80
L.H. Dickerson	1967-70
H.H. Dixon	1967-85
D.N.W. Earp	1981-84; 1985-
C.R. Elliot	1967-83
P.A.S. Ferguson	1983-86; 1986
R.L. Fitt	1970-73
J.H. Fleming	1970-73; 1975-78; 1979-82; 1983-86
P.J. Forbes	1985-
D.D. Fraser	1984-87
Dr. A.A. Fulton	1967-70
R.T. Gerrard	1971-77
R. Glossop	1968-71

E.M. Gosschalk	1981-83
F.N. Griffiths	1971-74; 1976-79; 1980-83
P.S. Hallas	1972-75; 1982-85
D.M. Hamilton	1971-74; 1975-78
W.R. Hare	1975-77
E.T. Haws	1969-72; 1974-77; 1978-81; 1982-
R. Le G. Hetherington	1967-69; 1970-73
A.W. Hill	1969-72
J.W. Hodgson	1971-74
Dr.A.K. Hughes	1984-87
Sir Trevor Hughes	1974-75
F.G. Johnson	1973-76; 1986-
H.N. Jones	1981-84; 1987-
M. Kenn	1983-86
J. Kennard	1967-68
M.F. Kennard	1968-75; 1976-82; 1982-86
J.H. Lander	1973-76
J.D. Lawson	1986-
Dr. W. MacGregor	1967-68
P.G. Mackey	1985-
D.P. Maguire	1984-87
R.H. McDonald	1967-68
J.M. McKenna	1985-
W.P. McLeish	1974-77; 1980-83
Dr. A.C. Meigh	1981-84
G.A. Milne	1978-81
P.B. Mitchell	1967-70; 1975-78; 1979-82
A.I.B. Moffat	1974-77; 1978-81; 1987-
Dr. L.J. Murdock	1969-72; 1973-75
D.C. Musgrave	1978-80
Professor J.K.T.L.Nash	1969-71
Dr. J. Newbery	1980-83
D. Ormerod	1982-85; 1986-
D. Palmer	1982-85;
J. Paton	1967-68; 1971-74
Sir Angus Paton	1967-70; 1972-75
B. Pattenden	1975-76
Dr. A.D.M. Penman	1967-70; 1973-77; 1978-83
R.V.C. Phillips	1978-81; 1982-85
F.F. Poskitt	1972-75; 1976-79; 1980-82; 1983-86
W.J.F. Ray	1983-86
E.C. Reed	1971-74; 1978-81
C.M. Roberts	1967-69; 1970-80
R.D. Robinson	1975-78
C.D. Routh	1985-
Sir Norman Rowntree	1968-71
A.E. Seddon	1968-71
R.G. Sharp	1978-81
Dr. G.P. Sims	1985-
J.E. Smith	1987-
Professor S.R. Sparkes	1967-70
C.H. Spens	1967-68; 1969-72
J.R. Stewart	1974-77
T.A. Stoker	1987-
E.H. Taylor	1986-
P.F. Tye	1987-
Dr. P.R. Vaughan	1970-73; 1978-80; 1981-84
T.R.M. Wakeling	1978-81
S.F. White	1975-77
T.E.S. White	1968-71
R.C. Whitehead	1969-72
J.D. Williams	1974-77; 1978-81
A.J.E. Winder	1982-85
Professor O.C. Zienkiewicz	1972-75

12. In 1986 Dr.J.A. Charles was appointed Technical Secretary, and a member of the Committee in this capacity.

ICOLD

13. BNCOLD has provided one President of the International Commission on Large Dams and two Vice Presidents within the last 21 years and these have been:

J. Guthrie Brown	(President 1964-67)
H.H. Dixon	(Vice-President 1971-74)
Dr. D.J. Coats	(Vice-President 1983-86)

14. As officers of ICOLD they may continue as ex-oficio members of the BNCOLD committee.

15. Following the success of the ICOLD Congress in Edinburgh in 1964, BNCOLD issued an invitation in 1980 for an ICOLD Executive Meeting to be held in London. The invitation was accepted and the 51st Executive Meeting was successfully held in London in September 1983.

16. Members of BNCOLD have supported the ICOLD Congresses by writing papers and attending in good numbers.

17. The numbers of BNCOLD papers in the seven Congresses in the 21 years of BNCOLD are shown in the following table.

Year	City	Question	No. of BNCOLD papers
1967	Istanbul	Q 32	2
		33	3
		34	3
		35	4

Year	City	Question	No. of BNCOLD papers
1970	Montreal	36	4
		37	0
		38	5
		39	1
1973	Madrid	40	6
		41	0
		42	3
		43	1
1976	Mexico	44	5
		45	4
		46	2
		47	0
1979	New Delhi	48	5
		49	2
		50	1
		51	2
1982	Rio de Janeiro	52	6
		53	3
		54	2
		55	3
1985	Lausanne	56	4
		57	1
		58	3
		59	2

18. BNCOLD members have played a very active part in the technical committees of ICOLD, including several serving as Chairmen. These have included R.E. Coxon as the Chairman of the Committee on Risks to Third Parties from Large Dams; E.T. Haws as the Chairman of the Committee on the Environment; Professor O. C. Zienkiewicz as the Chairman of the Committee on Analysis and Design; Dr. A.D.M.Penman as Chairman of the Committee on Mine and Industrial Tailings Dams; Mr. R.S. J. Lane as Chairman of the Committee on Earthquakes; Professor R.J. Severn as Chairman of the Committee on Seismic Aspects of Dam Design, and Dr. D.J. Coats as Chairman of the Committee on the Presidency.

BNCOLD "News & Views"

19. The BNCOLD "News & Views" was first published in May 1967 and 31 issues have been produced in 21 years. The Technical Editors have been:

H.H. Dixon & M.F. Kennard	1967-1969	- Nos. 1 to 2
M.F. Kennard	1970-1974	- Nos. 3 to 15
H.W. Baker	1975-1979	- Nos. 16 to 20
J.D. Williams	1980-1984	- Nos. 21 to 28
J.D. Williams & E.A. Jackson	1985	No. 29
E.A. Jackson & J.D. Gosden	1985 -	Nos. 30 to 31

BNCOLD Conferences

20. 1975 the first BNCOLD Conference was held. This was jointly with the University of Newcastle-upon-Tyne.

21. In 1982 a regular series of biennial conferences was started with the second BNCOLD Conference held at the University of Keele; the third at University of Cardiff in 1964; the fourth at Heriot-Watt University, Edinburgh jointly with the Institution of Water Engineers and Scientists. The series continues with this fifth conference in September 1988.

22. From 1984, a speaker has been invited to present the BNCOLD Lecture at each conference. This lecture was given by N.J. Cochrane in 1984 on "Insidious threats to Dams and Reservoirs"; and in 1986 by F.G. Johnson on "Experience with the Concrete Dams of the North of Scotland Hydro-Electric Board".

23. The papers at all these conferences have been published by BNCOLD.

Conclusion

24. The activities and the involvement of a large number of members over the 21 years has contributed to the success of BNCOLD and it is hoped it will continue this successful history in the future.

BNCOLD - 21 YEARS OF SCIENCE-BASED ADVANCES IN DAM BUILDING

A.D.M.Penman, DSc.,C.Eng.,FICE, Geotechnical Engineering Consultant

Introduction

1. Advances in our subject stem from better understanding of dams and their reservoirs. This comes from a combination of detailed observation of dam behaviour, the creation of better models to explain the behaviour, developments in numerical methods which enable more complex models to be used, new approaches to materials testing that give clearer pictures of materials behaviour, together with better hydrological and geological knowledge.

2. The individual can do little by himself without a ready exchange of experience, knowledge and ideas with his peers. ICOLD has given us an international forum since its formation in 1928. The transactions of its congresses have long been recognised as vital milestones in our advancing path and the informal international exchange of ideas which occurs during each congress are of inestimable value.

3. Britain has been a member country of ICOLD since its formation and for many years, official contact was through the British National Committee, a self-perpetuating group made up mostly of senior consulting engineers. British papers to the congresses came mainly from the offices of the leading consultants responsible for the design and supervision of construction of major dams in Britain and overseas. This resulted in numerous excellent accounts of design and construction of dams, providing a historical record through which design improvements could be traced.

4. One of the aims of our Institution of Civil Engineers is the exchange of views on the application of scientific principles to civil engineering practice and many meetings are devoted to discussion of papers on dams. There was, however, no specific opening for exchange of ideas amongst British dam engineers until the 8th International Congress in Edinburgh which gave an unprecedented opportunity for British dam engineers to meet.

4. By contrast, the British soil mechanics fraternity had been meeting to exchange ideas some years before the formation in 1947 of the British National Committee of the International Society for Soil Mechanics and foundation Engineering. It was natural for them to want to continue but in order to meet in the Institution building a recommendation had to come from Council: its agreement on October 1948 led to the British National Section (now BGS) open to anyone with a bona fide interest in soil mechanics.

5. The success of BGS and the enthusiastic attendance at their meetings is legendary within the Institution, and after the success of the Edinburgh Congress, it became clear to BNCOLD that it too should have a British National Section, open to those interested in dams and reservoirs. This Section was established in 1967, 21 years ago, and it is the purpose of

this paper to trace some of the developments in our subject which have taken place during those 21 years.

Grand New Era

6. The formation of the British Section of BNCOLD came at a time of vigorous dam construction in Britain and many other countries. In 1967, there were 15 large dams under construction in Britain. The Water Resources Board published predictions for future water needs in 1970, giving a value of 4.1 million m^3/day as current consumption which would increase to 9.2 million m^3/day by 2001. This increased demand would require construction of 41 inland reservoirs and, because of the difficulty of finding sites, consideration was given to damming estuaries. Detailed proposals were prepared for several schemes in Morecambe Bay: one was for a dam carrying a motorway on its crest as a link between Barrow-in-Furness and M6. Suggestions were also made for Solway and Dee barrages.

7. Numerous schemes were designed for tidal barrages on the Severn estuary with the main aim of electricity production, not storage of fresh water. Two BNCOLD members proposed a link to France with a tidal barrage carrying a very wide motorway and railway on its crest. It could produce electricity but constituted a serious restriction on the movement of Channel shipping. The proposed locking arrangements were not considered satisfactory.

8. During 21 years, the political climate has completely changed. Industry has been run down and the demand for water has fallen far short of the 1970 predictions. The great Kielder scheme which produced what has been claimed to be the largest man-made lake in Europe, with tunnel and aqueduct link to Teesside, was declared by government sources to be unnecessary by the time it was opened in 1982. Had it been possible to transfer water like electricity on a national grid, government calculations indicated that there was no need of further storage facilities until at least the turn of the century. It was only the acute shortage of water in Devon and Cornwall during a dry summer that enabled approval to be granted for Roadford dam: the embankment of this 40.5m high dam reached full height in 1988 and an upstream asphaltic membrane is to be placed during the summer of 1989. Construction of the Queen's Valley dam in Jersey is expected to begin in 1989, but otherwise, no new dams are proposed and activity centres on such works as the reconstruction of Carsington, raising Woodhead and improvements to old dams to ensure their safety and ability to pass maximum floods.

Clay Cores

9. During the 21 years that we are considering, there has come an end to the traditional puddled clay core. It is said that labour costs preclude its use, but standards of living have changed so much that workmen expect to have machines to carry out earthworks and it might prove difficult to recruit puddle gangs such as those who used to follow their work from damsite to damsite. In general, rolled cores have been used since 1960, although Jumbles, the last British dam to use a puddled clay core, was not completed until 1971. Puddling gangs had gone by then and the core was compacted mechanically with a large weight carrying steel cones on its underside to simulate the heels of the puddle gang, dropped repeatedly from a crane. Such an arrangement had been tried during the construction of one of the MWB Lea Valley reservoirs, but the much fatter London Clay had stuck firmly to the spiked weight.

10. Despite their use for well over a hundred years, the behaviour of puddle clay cores was not well understood. There had been little research or detailed observation. Some settlement and pore pressure measurements made in the upper part of Selset core and described by Vaughan (1965) gave sufficient information to quantify arching action and show that the total vertical stress in the core could be calculated from:

$$\sigma_{\prime} = y\left(\gamma - \frac{c_u}{a}\right) + c_u\left(\frac{Z}{a} - \frac{\Pi}{2}\right) + c_u\left(1 + \sqrt{1 - \frac{x^2}{a^2}}\right)$$

where c_u represents the undrained shear strength of the clay
2a " " average width of the core above the point considered
γ " " bulk density of the core.
Z " " depth of possible tension cracks at the upper boundary
x and y " " coordinates of the point, y being measured down from the top of the core

11. A soft puddle clay core is normally capped with other fill forming the dam crest to protect it and prevent drying. This usually removes the risk of tension cracks, so the second term in the above equation can be omitted.

12. If we consider a point on the centre-line of the core, x = a, and since y = h, the height of the core above the point, the vertical total stress can be taken as:

$$\sigma_v = h\left(\gamma - \frac{c_u}{a}\right)$$

13. If it can be assumed that for the soft, wet puddled clay, $\bar{B} = 1$, then an increase of pore pressure, δu, may give an indication of the increase of total stress developing within the core during construction.

14. Beavan et al (1977) have given examples of pore pressures measured in the cores of six dams and have shown that it is not uncommon for $\delta u = \delta\sigma_o$ during the earlier stages of construction when it could be expected that $\delta\sigma_v = \delta\sigma_o$

where δu represents incremental increase of pore pressure
$\delta\sigma_o$ " " " " overburden pressure γh
$\delta\sigma_v$ " " " " vertical stress

15. From this it can be seen that the summation of incremental increases of pore pressure give a good indication of the increment that has occurred in vertical total stress in a core during construction. Care must be taken to begin at the correct zero, which must be a negative pore pressure when the piezometer is installed, and that subsequent falls due to dissipation are subtracted.

16. At one time, low pore pressure in a clay core was thought to be a good thing, indicating rapid dissipation and increase of strength. It is now known to be more likely due to silo action and indicating low total stresses in the core which might lead to risk of leakage and internal erosion.

17. Use of stability analyses, such as Bishop's (1955) more rigorous modification of the Swedish method of slices, showed up the puddled core as a

weakness which, when combined with a soft clay layer in the foundations, as at Chingford, could lead to failure. A rolled core was much stronger, as it had to be to support the placing and compacting machinery of the time. Following the Proctor concept, attempts were made to limit placement water content to values often a few percent below optimum. British weather at many dam sites made winter placing impossible and summer rain could halt work on the core.

18. At Balderhead, there was a limited amount of clay suitable for the core and it was relatively dry. The core was designed to be as narrow as practicable to save volume and specified to be placed slightly above optimum water content, although the borrow pit value was low and some watering would be required. The usual difficulties were encountered over the correct value of optimum for clays from the various parts of the borrow pit: it could be argued that a determination of optimum should be made for every load if an exact specified value of placement water content were to be achieved. A practical solution was found by relating placement water content to the plastic limit. The test for this was somewhat simpler than that for Proctor optimum and through wholesale use in soil mechanics, may be said to be reasonably accurate despite the primitive method of hand-rolling thin threads of clay.

19. Balderhead core was placed during three summer seasons. During the first year, a combination of rain and use of clay from slightly wetter parts of the borrow pit provided a fill that was not too dry. Britain's first installation of a group of five earth pressure cells to measure total pressures in a clay core was made in this fill and readings taken throughout subsequent construction. The second year was much drier. Since rainfall could so easily halt placement, the contractor was naturally reluctant to add too much water and showed willing by provision of small hoses. It was, however, not enough and the middle height of core was placed dry. Unfortunately no more earth pressure cells were placed in the higher parts of the core.

Hydraulic Fracture

20. The leakage and sink holes that developed on first filling (Vaughan et al 1970) were eventually traced to hydraulic fracture through the Balderhead core. This first British example of hydraulic fracture occurred at about the same time as a similar event at Hyttejuvet dam in Norway and was followed, a few years later, by another case at Viddalsvatn, also in Norway. Exchange of ideas and experience with the Norwegian Geotechnical Institute led to research into the phenomenon.

21. An earlier incident with the core of dykes to form evaporation ponds near the Dead Sea had raised the problem of hydraulic fracture in 1966. It had been specified that the core must have a permeability no greater than a given figure and after construction, permeability tests were made from numerous standpipe piezometers. Following best practice, outflow tests were made by lowering the water level in the standpipes and measurements made of rise with time. This was so slow that it was assumed that the intake filters had become blocked and a change was made to inflow tests.

22. Warnings have always been given that great care must be used when de-airing hydraulic piezometers to ensure that water pressure at the intake filter never exceeds the total stress in the soil surrounding the filter, in

case the seal of the filter into the soil should be ruptured. At the dykes, care was taken to observe this warning by not applying heads of water in excess of the overburden pressure at filter level, calculated from vertical height and the known density of the core material.

23. The permeability results were alarming and indicated that remedial work would be required of the contractor over much of the length of core before acceptance. The threatened litigation led to international investigation and experimental work in Britain and Norway, described by Bjerrum et al (1972). It was shown that arching action had reduced the total pressures in the core much below overburden pressure, so that the heads of water used for the inflow permeability tests had in fact broken the seal between soil and intake filter, permitting the flows that had indicated the high values of permeability. Bjerrum and Andersen (1972) suggested that measurement of the excess water pressure required to break the seal, i.e. cause hydraulic fracture in the surrounding soil, could be used as an indication of the minor principal total stress acting on the filter. Penman (1975) carried out numerous hydraulic fracture tests on piezometers which had been placed amongst earth pressure cells in the cores of two dams, to study the relationship between measured total pressures and the critical pressure observed with the.piezometers. His findings indicated that the hydraulic fracture pressure was closer to the average measured total pressure rather than the minor principal stress.

24. It became clear that to avoid hydraulic fracture through a core, the total pressure acting across any potential fracture through the core must exceed the pressure from the reservoir water at that level. Also, of course, the contact pressure between core and abutment, must also equal, or preferably exceed reservoir water pressure.

25. The requirement for a placement water content at about optimum is to ensure that the fill is sufficiently workable yet not too soft to compact. To avoid the problems of measuring Proctor values or plastic limits, it is better to specify required shear strength. This can be measured rapidly during construction so that adjustments can be made before too much more fill has been placed. Experience with this approach has been given by Kennard et al (1979). It enables a core to be placed at a required strength and gives designers the facility to specify a value of c_u which they have calculated will not lead to hydraulic fracture in their particular shape of core.

26. The development of the wedge-foot compactor, particularly the self-driven, four roller type such as the Caterpillar 815, has enabled much softer clays to be used. As at Selset, discussed above, the increase of pore pressure shows the increase that has occurred in the total stress, so that during construction continuous careful measurement of the pore pressure increments can be used to show that sufficient total pressure is developing to avoid hydraulic fracture. Penman (1979) has suggested that if the piezometric level in the core at the end of construction is at or above reservoir level, there will be no danger of hydraulic fracture.

Drainage Layers

27. Where strength is required it can be argued that fill should be placed dry of optimum, although if too dry it may suffer collapse settlement on wetting. A build-up of high pore pressures may endanger stability and if the

fill available for the shoulders is both too fine-grained and wet in the borrow, it can be kept stable by use of drainage layers. The rate of dissipation of pore pressure is proportional to the square of the length of the drainage path, so calculations can be made to determine the spacing required between drainage blankets to ensure that dangerously high pore pressures do not develop during construction. The efficiency of drainage layers has been discussed by Gibson and Shefford (1968) and Sills (1975).

28. Drainage layers were first used in Britain to stabilize the shoulder fill of Usk dam in 1952, and since that time they have become increasingly fashionable. During the period we are considering numerous dams have been built with what at one time would have been considered unsuitable shoulder fill, some on unsuitable foundations. Both have been stabilized by drainage, incorporating near horizontal drainage layers in the fill and vertical sand drains in the foundation, as at Derwent and Selset. At least 20 British dams have drainage layers in their shoulders.

Rockfill

29. Our attitude towards rockfill has changed. At one time fines were regarded as a source of settlement because they prevented good rock to rock contact. When they occurred in quarry run or tunnel spoil, they were often sieved out by passing the rockfill over a screen on its way to the dam. Specifications for dumped rockfill dams in North America often called for the use only of competent, strong rock from the heart of the quarry, free of fines. It was not uncommon for the rockfill to be passed over a 150mm opening size 'grizzly'. Material less than 150mm which passed through and the weathered rock from the quarry was led to waste.

30. It was a mistaken concept. Contact stresses are proportional to the square of the rock size, and angular pieces of hard rock can readily be stressed beyond the strength of the parent material, producing fines by spalling. Experience showed that wetting caused settlement, so it became the practice to sluice dumped rockfill during placing with volumes of water 3 to 5 times fill volume. It was thought that this improved rock to rock contact by washing out the fines.

31. Placement in comparatively thin layers instead of dumping, and compaction, especially with a vibrating roller, improved the properties of the rockfill markedly. The dense, hard, greenish-grey, metamorphic rock used for New Hogan dam was passed over a vibrating screen with a bar spacing of 75 to 150mm, producing a processed rockfill that was completely free of small fines. After placement and compaction with a vibrating roller, however, excavation revealed that voids between the larger pieces of rock were well filled with fragments grading in size from 25mm down to very fine particles, showing that the high contact forces had caused considerable spalling.

32. It is now known that wetting reduces the strength of the parent material, so encouraging spalling, and that the large deformations observed in dumped rockfill dams were largely due to crushing and deformation at points of contact. There is no need for excessive sluicing: the rockfill should be placed at or above an optimum water content to give workability and it should also be well graded so that all voids are filled, producing myriads of contacts, each very lightly stressed. Compaction with a smooth, vibrating roller can develop a high bulk density, producing a rockfill resistant to deformations.

33. A marked advantage of rockfill over earthfill is that no construction pore pressures develop. Penman and Charles (1976) have suggested that fill which is to be classified as rockfill rather than earthfill, should have an in situ permeability greater than 10^{-5}m/sec. This sets a limit to the amount and size of the fines that can be included.

34. There are not very many rockfill dams in Britain. Quoich, 38.4m high, was the highest rockfill dam at the time of the formation of BNCOLD's British Section. It was constructed mainly from a schist rock spoil which came out of the tunnel excavations. The spoil was processed by screening and washing to remove below 10mm sizes, so as to ensure rock to rock contact. It was spread in 0.6m layers, rolled with a 3.5t. vibrating roller, and sluiced. The maximum vertical compression of the rockfill measured during construction was only 1.07% and the dam behaved well. After construction, the need for processing the tunnel spoil to remove fines was questioned with the implication that money could have been saved without diminishing any properties of the rockfill if they had been left in.

35. Quarrying the carboniferous sandstones used for the construction of Scammonden (73m) and Winscar (53m) produced plenty of fines, as well as large pieces of rock, but in line with more modern practice, the fines were left in. Placement trials for Scammonden showed that by tipping the fill well back from the advancing edge of a layer and then bulldozing it over, the larger pieces fell to the bottom and were buried in fines, leaving a relatively smooth surface that was kind to the hauling vehicles and readily accepted a smooth vibrating roller. During very wet weather, the fines could be worked up to a slurry by tyred traffic and a fear was expressed that layers of soft silt might be being built into the dam. To check on this, a coloured sand was spread thinly over the surface as a marker and after the next layer had been placed and compacted, a trial pit was excavated. It was found, not only that all voids between larger sized pieces were completely filled, giving a high density, but the larger pieces from the upper layer had punched down through the lower surface, producing a well integrated mixture of sizes.

36. Layer thickness at Scammonden was specified as 0.9m, but this required larger pieces of sandstone to be broken individually. At Winscar this was avoided by using 1.7m thick layers. The power of vibrating rollers is now such that it is not uncommon to find 2m thick layers specified for well graded rockfill.

37. The traditional approach to the strength of rockfill is to assume $c' = 0$ and to assign a suitable value for an angle of shearing resistance. It has long been known that increased confining pressure reduces the value and emphasis was put on assessing minimum values that would be applicable to rockfill in very high dams. Recent work at the Building Research Station has shown that well graded rockfill develops remarkably high angles of shearing resistance at the low confining pressures near the surface of slopes. Charles and Watts (1980) have shown that it is more realistic to use a curved failure envelope to describe strength and to express this in the form:

$$\tau = A(\sigma')^b$$

where τ represents shear strength
σ' " normal effective stress
A and b " constants for the rockfill

38. This is an approach of major importance to the designers of rockfill dams and analytical methods based on it will be discussed below.

Upstream Membranes

39. It can be argued that rockfill can be used most effectively for a dam by placing the impervious element at the upstream face, so that the whole body of the rockfill is used in resisting the horizontal thrust imposed by the impounded reservoir. Use of an impervious membrane on the upstream slope also attracts some vertical loading from the water which helps to increase resistance against sliding.

40. It was not uncommon in North America (and elsewhere) for the large deformations that occurred in dumped rockfill to damage upstream concrete membranes on first reservoir filling. If the reservoir could be emptied to effect repair, the subsequent performance of the dam could be excellent. This general experience appears to have deterred British designers from the use of upstream membranes, but a change has come about, partly due to the better understanding which has developed of fill properties. Prior to 1967 there were only 5 large dams in Britain with concrete upstream membranes (World Register) and none with asphaltic membranes.

41. First to use an asphaltic upstream membrane was Dungonnel (17m) completed in 1970. This was soon followed at Turlough Hill pumped storage reservoir where the membrane on the slopes of the 24m high dam extended across the floor of the reservoir. These were followed by Winscar (52m), Marchlyn (72m), Sulby (60m), Colliford (28m) and during the summer of 1989, it is expected that an asphaltic membrane will be placed on Roadford dam (40m).

42. During this period, British designers were concerned with large upstream membrane dams overseas, including the 90m high Kotmale rockfill dam in Sri Lanka, and Khao Laem (90m) in Thailand, both with concrete membranes.

Central Asphaltic Membrane

43. An absence of suitable earth fill for a core has caused designers to turn to concrete for a central core, e.g. Daer and the embankment section of the Glen Shira Lower Dam. In a high dam, the concrete can become highly stressed by negative skin friction due to fill settlement. At Daer the core was articulated by numerous bitumen-filled joints. The vertical joints acted as standpipes full of bitumen, which imposed bitumen fluid pressure on the horizontal joints, keeping them full and preventing hydraulic fracture by the reservoir water. Provision was made for topping up the joints at the crest with more bitumen if necessary.

44. The concept of an asphaltic core that could settle with the fill was new to Britain and was used in the first 35m high part of Sulby dam in 1980. The dam was to have been built with provision for raising to 60m with a second stage, by placing rockfill over the downstream slope, made waterproof by an asphaltic upstream membrane connected to the core at the crest of the dam. In the event, the two stages were built together, but the shape of the dam remained as if it had been constructed during two separate periods.

45.Central asphaltic cores had previously been used in Germany and the expertise and machinery of a German contractor was used for constructing the core. A similar arrangement was made for the asphaltic core of the 56m high Megget dam, also begun in 1980.

46. Previously, central asphaltic cores had been used by the British designer of the High Island dams in Hong Kong.

Shears in Foundations

47. It is well known that heavily overconsolidated clays contain fissures and slickensides. Tunnellers in London clay were often made only too well aware of slickensides when they were encountered in the face. The fissures are part of the fabric of the clay and exist in more than overconsolidated clays. Rowe (1972) drew attention to the important effect of the fabric on the properties of many soils from relatively soft, silty clays to sandstones (the fabric largely controls the size and shape of a piece of rockfill won from a quarry), but slickensides are slip sufaces formed at some time during the history of the soil, indicating that there has been relative movement. Slip surfaces can have been formed by the failures of steep slopes or under the stresses imposed by ice loading, long since gone, and may remain under relatively flat sites. In the States, difficulties were experienced with old slip surfaces in the foundations of South Saskatchewan and Waco dams and at Wheeler Lock. The careful vigilance of the British engineers responsible for Mangla dam in Pakistan detected old shear zones in the foundations after construction had begun, causing a change to the designed section to avoid overstressing the soil in those weakened areas. Similar problems were encountered at the site for El Chócon dam in Argentina.

48. In Britain, the heavily overconsolidated Lias clay foundation for Empingham dam contained numerous old slip surfaces, causing the dam to be designed with such flat slopes that they formed ideal pasture for grazing cows. Pre-existing slip surfaces resulting from peri-glacial action led to failures during the construction of dams for Draycote reservoir and they appear to have been a contributory cause of the failure of Carsington dam during construction. A full report on the Carsington failure has been given by Coxon (1986).

49. It is almost impossible to detect old slip surfaces with a borehole site investigation and extremely difficult to detect them in trial pits. A detailed knowledge of the geological history of the site may well provide the best indication of existing slip surfaces. If this suggests that they may be present, then a more detailed investigation, coupled perhaps with loading by trial banks, or oversteepening of slopes in borrow pits, will be needed to isolate them.

Progressive Failure

50. The traditional slip circle analysis is concerned with stress and the aim is to ensure that the imposed stress does not exceed the strength of the soil. The factor of safety is a factor of safety against ultimate failure, but with our modern knowledge, the notion of ultimate failure should be put behind us. It has often been said that we should now be designing for acceptable movement rather than contemplating failure and such a notion brings in the concept of strain in the soil and on a potential slip surface.

51. Many soils when tested in relatively rigid machines at constant rates of strain, exhibit a reduction of strength once a peak has been passed. This

behaviour had been observed since the late 1930's and Skempton had repeatedly drawn attention to the effect this could have on stability analysis. In his Rankine Lecture (1964) he described progressive failure and said that in the limiting condition the strength along the entire length of the slip surface will fall to the residual value. In addition to numerous case histories, he also gave the results of tests on samples containing a slip surface, thereby demonstrating that strength on a pre-formed slip surface corresponds to the residual value.

52. The traditional assumption that the failure envelope for a soil can be represented by a straight line on a plot of shear strength v. normal stress acting on the shear plane, is a convenience which simplifies calculations by making the angle of shearing resistance constant over the range of stresses considered. In practice most failure envelopes are curved. The traditional value of c' is taken from the intercept of the straight line at zero normal load. Use of a curved failure envelope reduces the magnitude of the intercept, in many cases to zero. Whatever the value of "true cohesion" of a clay, in moving from peak to residual, the cohesion intercept c' disappears completely.

53. An allocation of a value to c' can be relatively important in the design of low dams and many designers in the past have chosen to use peak strength values to obtain c' and φ', then relied on use of a large enough factor of safety to avoid any element of soil on a potential slip surface from being strained beyond peak strength.

54. The wisdom of this approach is questionable: its success is dependent on the properties of the various soils through which the surface might pass and the geometry of the embankment section. The Carsington failure has brought a renewed emphasis on the mechanism of progressive failure and new, more complex methods of analysis have been developed.

55. Another type of "progressive failure" was demonstrated at Carsington. The major movements of the slip began at a section of the dam some way along from the major section and the failure than progressed along the dam to beyond the major section, exposing a long cliff, parallel to the dam axis. This behaviour has also been described as "domino action", i.e. as the first section failed, it dragged on the next section, which, it could be argued, might just have been stable at the time and was unbalanced by forces imposed on it by the adjacent sliding mass. Its failure, in turn, triggered the next section and so on until a section of the dam was reached that was so stable it could withstand the drag-down forces.

56. The term "progressive failure" has been used for a great many years to describe the effect of increasing,non-uniform strain along a potential slip surface taking elements of soil beyond their peak strength and so should be reserved to describe this behaviour. The "domino effect" in unbalancing with a relatively small force the next in line, gives a clear indication of the second behaviour and should probably be used - certainly in preference to the expression, "progressive failure".

Electronic Computation

57. Successful design requires vivid imagination - sufficiently vivid to envisage every circumstance that could lead to unsatisfactory behaviour. Supporting calculations at one time limited the scope of imagination that

could be usefully applied. Circular slip surfaces used to be preferred to other shapes because the stability analysis could be readily calculated. Even so, it was common to use a planimeter to measure the area of the slipping section and cardboard cutouts hung from pins to find the position of the centre of gravity.

58. Bishop's (1955) more rigorous approach to the Swedish method of slices was put into a form suitable for analysis by electronic computer by the combined efforts of Little and Price (1958). Since then programmes have been developed that enable analysis of almost any form of slip surface, passing through materials with almost any properties. Programmes that allow of consideration of progressive failure, developed by Potts at Imperial College, are amongst the latest developments.

59. Considerations of the stresses carried by small elements within the body of a soil mass and the way those stresses are transmitted to adjacent elements, making due allowance for the varying, non-linear and non-reversible stress-strain characteristics of the different types of soil encountered can give a picture of stress distribution and strains. Consideration has been given to the effects of the non-linear stress-strain characteristics of soils by Jardine et al (1986). There is a limit to the smallness of an element that can be used to produce a sensible picture but the mass of iterative calculations required made the process virtually impractical before the development of the electronic computer. Nowadays there are large numbers of programmes to enable analyses to be made using the finite element technique.

60. It soon became apparent that the predictions of complex behaviour made by numerical analysis were limited much more by the assumptions that had to be made about boundary conditions and material properties than by the ability to make vast numbers of mathematical calculations. At an international symposium on the criteria and assumptions for the numerical analysis of dams, held at Swansea in 1975, this point was emphasised. It was said that with the impressive array of ever more powerful analytical tools then available to the engineer, a situation had been reached where almost any structural problem could be solved. The very power of the modern methods of analysis had, however, drawn sharp attention to the question of what assumptions should be made in the analysis and by what criteria a design should be judged.

61. The need for more information about the properties of materials required as a basis for numerical analysis led BGS to choose as the subject for the 7th European Conference on Soil Mechanics and Foundation Engineering which they organised at Brighton in 1979, the measurement, selection and use of design parameters in geotechnical engineering. The discussion sessions considered the design parameters for granular soils, fills, weak rocks, stiff clays, soft clays, artificially improved soils and special soil conditions: all of relevance to the design of embankment dams. The value of the information exchanged during this conference can be judged by the interest that has continued to be shown in the proceedings, copies of which have been bought in considerable numbers.

Field Instrumentation

62. During the 21 years of the British Section, instrumentation for dams has moved from the experimental to the established stage. Design predictions require verification by observations of actual behaviour. As the analytical methods available for making predictions have advanced, so there has been a need for more extensive and accurate instrumentation.

63. Traditionally crest settlements were measured as an indication both of dam behaviour and to ensure an adequate freeboard. Measurements of horizontal movements soon followed and today precise surveying from stable reference monuments built into the natural ground at safe distances from the stress field of the dam itself often using both triangulation and trilateration, enable the three dimensional movements of targets on the dam to be measured to an accuracy of ±2mm. Internal gauges such as the horizontal plate gauge enable these movement measurements to be extended to include the movement of marker plates buried in the fill of embankment dams. By placing the plates at positions corresponding to the node points of a finite element grid on a section of the dam, a direct comparison can be made between predicted and actual movements. Such comparisons are of inestimable value for fine tuning the numerical analysis.

64. During construction, the measurement of developing pore pressures in embankment dams and temperature rise due to heat of hydration in concrete dams, were of fundamental importance to the designer. Excessive pressures or temperatures could be reduced by additional provision for drainage or cooling, or by other techniques such as a change in the method of excavation in the borrow pit, or a change to a different cement or use of cement replacement, but a first essential was to know from the field measurements if such measures were effective.

65. Measurements of pore pressures in fill and foundation and temperature measurements in concrete are now regarded as purely routine. In addition, it is common to measure earth pressures within the fill, often in six directions at each measuring position so that the direction and magnitude of the principal stresses can be determined. Internal strains are also usually measured in vertical and also in one or two horizontal directions. Joint movements are measured within concrete dams and in concrete membranes on fill dams, particularly at the plinth where the membrane joins the natural ground.

66. The designer is the most able to decide on the type and positions for the instruments he needs to verify his design assumptions and to check on what he expects to be the most critical parts of his individual dam. Unfortunately, the design of the instrumentation requirements is not given the consideration it deserves. A uniform distribution of instruments on a cross-section does not necessarily provide measurements that will be of maximum help to the designer. So often instrument houses are not built in time, vital zero readings are lost and measurements that could have major significance in revealing unsatisfactory initial behaviour are not taken in time to enable construction techniques to be corrected. There is a danger that instrumentation is provided more because it is expected that a large dam should be fully instrumented rather than for any true desire to observe in detail the behaviour of the structure. Instrument houses, completed, cleaned and lit may look impressive for the opening ceremony, but they do not necessarily indicate that valuable measurements have been or will be obtained from the instrumentation.

67. Field instrumentation in geotechnical engineering was the subject of a Symposium organised by BGS in July 1973. A conference on geotechnical instrumentation in civil engineering projects, organised by the Institution with BNCOLD as a co-sponsor, is to be held in April 1989.

Tailings Dams

68. In Britain, lagoons to store tailings from the chemical industry were built on relatively level ground with surrounding bunds made from a variety of materials including lime-stabilized boiler ash. They were fairly low and appear not to have attracted the attention of the dam designer.

69. The Clean Waters Acts of the 1950's prevented discharge of tailings into rivers by the coal industry and caused the construction of some small lagoons amongst the dumps of colliery waste. One or two of these had failed prior to the disastrous failure of the waste dump at Aberfan in 1966. The structure that failed at Aberfan was a 67m high embankment of colliery spoil and its investigation brought in designers of embankment dams well versed in geotechnical engineering, who later helped to draft the ensuing legislation controlling mines and quarry waste disposal. Although tailings were not involved in the Aberfan failure, the general review of the methods of colliery waste disposal included tailings lagoons, and the manual subsequently prepared by the Coal Board (McKechnie Thomson and Rodin 1973) made recommendations for their safe construction.

70. It could be said that tailings dams were first officially recognised by ICOLD in 1976 when they were included as part of a Question for the Mexico Congress. This Question attracted three British papers on tailings dams, giving details of large dams to store waste from the china clay industry, fly ash from power stations and tailings from fluorspar workings, as well as several examples of failures. It was said at that time that dams for retaining industrial waste had not always, in the past, received the engineering attention they deserved. Because they were constructed piece-meal, the ultimate height they would reach was often not envisaged when construction began and since production could not continue without disposal of tailings, there was a great temptation to keep on adding to the heights of the dams.

71. Today industry is much more aware of the responsibility it has for the safety of its tailings dams and it can be argued that since profitable production would be stopped if tailings disposal were interrupted, then there is a strong incentive to ensure the stability of the tailings dams. There is still a lack of observational evidence of the behaviour of many tailings dams and there is a need for the introduction of instumentation as has become the standard practice with water retaining dams.

Earthquakes

72. The severity of damage that can be caused by earthquake has been demonstrated yet again during the past few months. The earthquake in Armenia on 7 December, 1988 gave a first shock lasting 30 to 40 seconds of Richter magnitude 6.9 at 11.41 local time, followed 4 mins. later by a second shock of M 5.8 that lasted nearly 60 seconds. Severe damage occurred in Spitak and Leninakan, causing an estimated 55,000 deaths. Another event of M 6.0 occurred on 23 January,1989 about 20 miles from Dishanbe, capital of Tadzhikistan, and triggered large mud flows. A smaller event, possibly

one of a self-induced series, affected Nurek, the world's highest dam, but damage has not been reported.

73. Although Britain does not lie in a zone of major seismic activity, it does suffer earthquake shocks. Davison (1924) listed 1,190 events that occurred between the years 974 and 1924. A committee which reported to the Royal Society was set up to study the Chichester earthquakes of 1833-5 and the British Association established a committee to study the earthquakes that were occurring in Scotland after a particularly strong shock near Comrie in 1839.

74. This committee set about the task of designing instruments to measure the intensity, direction and time of shocks. A scientific study of the causes and behaviour of the world's earthquakes sprang from these early beginnings and a centre of expertise has remained in Edinburgh. The Global Seismology Unit of the British Geological Survey is in Edinburgh and during the 21 years we are discussing, the International Seismological Centre has been established at Newbury. It holds a computer store of historical events throughout the world and can supply details of earthquakes that have occurred in any particular area of interest. This can be of help to British engineers, responsible for the design and supervision of construction of dams overseas, particularly in countries where seismic risk is high.

75. The design technique of representing an earthquake by a static horizontal force is being replaced by dynamic analysis. Instead of simply tipping the model up at a small angle to provide the equivalent of a horizontal force of about 0.05 to 0.2g according to the predicted severity of the maximum expected earthquake, consideration can now be given to the expected profile of the shock pattern and allowance made for the cyclic nature of the loading, in vertical as well as horizontal directions. Procedures such as the 'Seed-Lee-Idriss' procedure (Seed 1979) enable the stresses induced in an embankment by the most unfavourable type of base excitation to be computed from a dynamic finite element analysis. The static stresses in the dam can also be computed with the aid of finite element techniques. These stresses, when applied to representative samples of the various fill used in the embankment, enable determination of the pore pressure build-up and deformation characteristics to be expected under worst earthquake conditions.

76. During a London conference on the design of dams to resist earthquakes in 1980, it was said that analytical and computational techniques have far outstripped out understanding of material behaviour. This subject requires much more study before further advance in our overall expertise in earthquake engineering can be made.

77. The behaviour of a dam when shaken can be measured during minor earthquake shocks or by artificial excitation. The response of buildings has been measured while subjecting them to cyclic forces from eccentric-weight exciters and the same technique has been used for dams. Severn et al (1979, 1980) have described tests made with four powerful eccentric-weight exciters on the 50m high concrete Wimbleball dam and the 90m high rockfill Llyn Brianne dam. The results were compared with behaviour predicted by finite element analysis thus enabling improvements to be made to the methods of analysis.

78. The dams most likely to be damaged by earthquake are those constructed from hydraulically placed fill, or those on a foundation of saturated silty-sand that could be liquefied by the earthquake shock. Of the dams constructed today, the most vulnerable are tailings dams. It could be useful to determine their in-situ conditions at various stages during their prolonged construction by subjecting them to controlled vibration with suitable exciters.

Rolled Concrete Dams

79. The advantages of embankment dams over concrete dams stem from the facts that they can be constructed on a wide variety of foundations unsuitable for a concrete dam, using local fill placed with a minimum of labour by machinery which is constantly being developed to have greater power and handle larger volumes of material. To improve their competitiveness, the advocates of concrete dams are now placing their fill continuously with earth-moving machinery.

80. The successful use of dry lean concrete for base courses of roads led Paton (1970) to propose use of this type of material for gravity dams. It is likely to be porous and Wallingford (1970) had suggested use of an upstream wall of normal concrete as an impervious element. Moffat (1973) looked into the properties of dry lean concrete and proposed use of a bituminous impervious membrane to be incorporated in the upstream face.

81. In 1976, Dunstan proposed the use of a concrete, lean in cement but rich in fly-ash, to be placed at a very low water content. Workability would be given more by the fly-ash than by the water and compaction would be by heavy vibrating roller. This would have a low heat of hydration but be so waterproof that no separate element would be required. The outer skins of a dam would be horizontally slip-formed by a kerb-placing machine and the whole structure brought up together in lifts. In preparation for the proposed Milton Brook dam, a trial section, described by Dawson and Dunstan (1979), was built and tested. To check on the bond between successive lifts, various time intervals were used to simulate weekend closedown or a more prolonged plant failure. Cores taken through the joints were tested for permeability by drilling a central hole, and passing water radially. No increase of permeability at the joints was detected.

82. Unfortunately, owing to economic restraints, Milton Brook was not built, but the method of construction was quickly accepted in Japan where numerous rolled concrete dams have been built. CIRIA held a conference on rolled concrete for dams in 1981.

83. Lowe (1988) in a short history of roller compacted concrete dams says that he first used concrete placed by earth moving plant in 1960 for the core of a 65m high cofferdam for the Shihmen embankment dam , Taiwan. In Italy, the 178m Alpe Gira gravity dam was built with a fairly lean concrete using blast furnace cement to minimise heat of hydration, spread in 0.8m layers and compacted by vibrators mounted on tracked vehicles. Open contraction joints were formed by cutting the layers with a vibrating saw blade mounted on a self-propelled chassis. A waterproof element in the form of 3mm steel plate was used on the upstream face. The dam was completed in 1964.

84. Shimajigawa, 89m, the first roller compacted dam to be built in Japan,

was completed in 1980. The first in USA was Willow Creek, 52m, completed in 1982. Since then of those higher than 15m, seven more have been built in USA, three in South Africa, two in Australia and one in China, Brazil, France and Spain, but none in U.K.

The Reservoirs Act

85. The Edinburgh Congress in 1964 took place shortly after the failures of Malpasset and Baldwin Hills dams and the disastrous landslide into the Vajont reservoir. Gruner (1963) had discussed dam disasters and this brought forward a call for review of legislation relating to reservoirs by ICOLD member counries. Although Britain was in the forefront with her Reservoirs (Safety Provisions) Act of 1930, she agreed to look at it again and our Institution set up an ad-hoc committee. This committee agreed with the principle that the responsibility for the safety of a reservoir should continue to be placed on one qualified civil engineer because of the dangers associated with divided responsibility. The 1930 Act had worked well in that no member of the public had been killed as a result of a dam malfunction. It published its recommendations in a Report on Reservoir Safety, 1966. Essentially these were that power to enforce the Act should be given to a Government Department who would keep records of all dams and ensure that they were inspected when required and action taken on the inspector's recommendation. Provision should be made for abandoning reservoirs, consideration given to the storage of fluids other than water, application of the Act to Northern Ireland and the appointment of engineers to Panels to be limited to 5 years, rather than given for life. Some of these recommendations have been incorporated in the 1975 Reservoirs Act which is now being implemented.

Flood Studies

86. Part of the success of the 1930 Act can be attributed to the "Interim Report of the Committee on Floods in Relation to Reservoir Practice", published in 1933. This provided a more rational approach to the design of spillways, and largely overcame the dangers of failure by overtopping.

87. During consideration of the 1930 Act, it was logical also to see if the Interim Report should not now be put in a more final form. In 1967, the committee on floods recommended that the whole subject should be re-examined. Subsequently, the Institute of Hydrology of the National Environmental Research Council, supported by the then Water Resources Board, undertook this study. It published the result of its work in 1975; a work which it is claimed enables all the characteristics of a flood on any river in the United Kingdom and the Republic of Ireland, to be calculated i.e. mean annual flood, flood with 150 year return period and the Probable Maximum Flood.

88. This was discussed during the BNCOLD Symposium on Inspection, Operation and Improvement of Existing Dams, in 1975 and the Institution published "Floods and Reservoir Safety: an Engineering Guide" in 1978.

89. The Institution organised a conference in 1980 to discuss the Flood Studies Report, when it was said that it was of immense importance to engineers responsible for the design, inspection and operation of dams in the United Kingdom: it was probably the largest single hydrological study ever undertaken in the British Isles.

90. It was suggested that some 170 spillways would require enlargement, at a cost in the order of £50 million. It was also suggested that analysis to show the extent of floods resulting from dam failures should be made to assist in identifying dams that present most risk and should receive priority in carrying out remedial works.

Reinforced Grass Spillways

91. Many dams over a hundred years old had experienced no trouble with their spillways, yet they have been shown to be completely inadequate to take the PMF. Not only is the cost very high to increase spillway capacity, but in many cases it is almost impossible to increase width because of existing restraints in the valley. As it is, there have been cases of very large slips started by excavations into a hillside as part of spillway widening works.

92. In order to get over some of these difficulties, it has been suggested that embankment dams could be reinforced so as to permit exceptional floods to overtop the crest. A research project by CIRIA included field trials of several types of open grid type concrete block work placed on the downstream slope of a dam, through which the grass could grow. The appearance of the grassy slope was not affected by the blocks but they prevented erosion. Mackey (1985) has described the use of flexible independent concrete slabs placed on the crest and downstream slope of the 60 year old Stanford dam: one of the first U.K. applications of this approach to cater for the possibility of large floods.

Aeration

93. Perhaps because flows through the spillways of British dams are not particularly high and no doubt as a result of ingenious design, the water always appears to contain plenty of air and severe erosion damage is not usual. Many dams overseas, however, have to deal with large annual floods, and the damage that occurred to the Tarbela spillway focused attention on cavitation problems. It does not appear to be very practical to demand surface finishes on large concrete shutes that will enable solid water to pass over them at velocities in excess of 40m/sec.

94. In general, cavitation erosion can be virtually eliminated if the air concentration at the solid surface can be kept above about 8%. For existing and proposed spillways, modern methodology can forecast potential cavitation damage, the extent of natural aeration and where artificial aeration by slots or otherwise, might be necessary.

Old Dams

95. A recent survey of U.K. embankment dams (Technical Note 1 in these Proceedings) has indicated that 70% were built prior to 1900, a large majority having puddled clay cores and below ground cut-offs. An earlier review by Charles and Boden (1985) relating to nearly 100 cases of unsatisfactory performance had shown that of dams in service, retaining a reservoir, 24% of the cases were associated with overtopping, 14% due to slips and slides and 55% resulted from internal erosion.

96. Flood studies have enabled overtopping problems to be controlled and developments in the art and science of geotechnical engineering have removed

uncertainties about slips and slides. The problems of internal erosion, however, are more difficult.

97. The general principles discussed above relating to clay cores show that a sufficient total pressure must be developed from the weight of the core on all planes through the core and on all boundary surfaces that pass from upstream to downstream, to resist the pressure of the reservoir water.

98. In order to assess the condition of old puddled clay cores, a research programme is being carried out by the Building Research Station in conjunction with owners and consultants. Total pressures acting horizontally in axial and upstream-downstream directions are being measured together in some cases, with vertical pressures. Some of this work has been described by Charles and Watts (1987). An ingenious device has recently been developed which enables total stresses in the vertical direction to be measured. This has been described by Watts and Charles (1988).

Acknowledgement

99. BNCOLD News and Views has formed a valuable record of the activity of the British Section since its inception. Mr Hugh Dixon, then Chairman of the General Purposes Committee, in a short note in the first issue (May 1967) said: "On the crest of the wave of enthusiasm engendered by the undoubted success of the Eighth Congress, the proposal was made that a British Section, whose membership would be open to all engineers interested in dams in Great Britain, should be formed. A draft Constitution was drawn up, the task having been given to the Writer on the grounds that it was he who had raised the matter in the first place!" We are glad to report that Mr Dixon, past BNCOLD chairman and now 86, is enjoying retirement on Mallorca.

100. In preparing this Paper, reference has been made to all the issues of News and Views, and the loan of a complete set from Dr D J Coats is gratefully acknowledged.

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Pre-visit presentation on Winscar Dam

Presenter: J D Humphreys (MRM Partnership)

DISCUSSION:

R Melbinger

J D Humphreys

W J Carlyle

PRE-VISIT TALK ON WINSCAR DAM

J D HUMPHREYS (MRM Partnership)

Just a few facts and figures first of all. Winscar dam is a vibro compacted rock-filled dam, about 54 metres in height, situated on and built of the millstone grits sandstone which contains a little shale. The downstream slope is grassed, the upstream slope is covered with an asphalted concrete deck constructed basically in two layers of dense asphaltic concrete.

I am proud of saying, that it was the first in Great Britain. We did make a few mistakes, and I thought it would be most useful if I tried to accent the things that I think we did wrong.

The upstream slope is 1 on 1.7 and the downstream slope 1 on 1.4 in general terms, but with additional landscaping fill near the toe which comprised the spoil from the necessary excavation.

The slope of the landscape fill by the way was consciously made roughly the same as the slope of the valley sides, so that it looked as natural as possible. One of the reasons for stacking the spoil against the downstream slope was to try to diminish the apparent size of the dam, as seen by the villagers, immediately downstream.

Another feature is that there is no valve tower. There is instead a sloping access gallery. There didn't seem to me to be any virtue in having a valve tower if you didn't have to and it worked out to be just as cheap to build one instead on the rather steep left bank side of the valley, immediately upstream of the toe. You'll see that the dam is slightly S-shaped in plan.

The honest reason for the shape was, quite simply, that I was tired of seeing British design, the dams straight in plan. I thought they looked nicer if they were a bit curved, and that is the honest reason.

The official reason was that it so happened that the shape of the dam was such that, owing to the topography of the valley, the upstream toe reduced to almost two straight lines, so that one could at least tell one's conscience that the upstream toe works were roughly as short as possible, and therefore most economical. It doesn't happen to be true. I am sure that if that hadn't worked out, we'd have found some other reason!

The grout curtain; I once heard it said that half the money spent on advertising is wasted, but you never know which half, and I have always felt that perhaps the same applies to grouting. We felt that particularly at the wings of the dam, where you'd have to go on for a long way before the bedrock actually rose to reservoir level. We had to curtail the grout curtain, but we erred, if you like on the un-conservative side, on the grounds that it was fairly easy afterwards to come back and do more grouting if necessary. More was needed in the left bank.

The design flood, at the time of the design when we were still working on the old interim report of the committee on floods of 1933, was 95 cubic metres per second. This was based on twice the so-called maximum flood, as then defined. It was, in fact, checked later against the FSR and it was found to be adequate.

The design was assisted by hydraulic model testing, carried out at Wimpey Laboratories Ltd. The spillway crest length is 40 metres and the acceptable performance of the weir in channel, according to the model, went up to 135 cumecs. The flow, before actual over-topping of the embankment occurred was found to be 190 cumecs. Damage to the spillway and downstream works, which were reckoned to be acceptable in extreme conditions appeared to begin to occur at about 75 to 95 cumecs.

Probably the most interesting defect was a crack in the asphaltic concrete which was discovered a few years ago. This is well described in a paper in the proceedings of the San Francisco ICOLD. It was not a massive crack: the actual leakage area, the area through which water was passing through the asphaltic concrete was probably equivalent in size, say, to a matchbox, but under something like 50 metres head of water, that does represent quite a nasty flow, and I can assure anyone who has not yet had this experience, that when you receive reports of leakage increasing as the reservoir level is slowly dropping, you don't get an awful lot of sleep.

The causes, again, are described in the paper. I would sum them up, very briefly, by saying that firstly some of the design details were not properly implemented. The double layer of asphaltic concrete was sitting upon a binder and levelling course of asphaltic concrete which, according to the drawings, should have thickened as it approached the toe. Again, this was a fairly standard detail. This, when we finally investigated the crack and dug it all, was found to be not the case.

The upstream slope I've already told you: 1 on 1.7, downstream slope 1 on 1.4. I felt at the time that a lot of British engineers really weren't quite up to date on rock-filled dam properties and were still thinking in terms of slopes on 1 on 3. I remembered, before we embarked on that design of Winscar, a paper in the ASCE proceedings about Lewis Smith dam, a rock-fill dam about 300 feet high, which is a good deal higher than Winscar, with downstream slopes of 1 on 1.3, even though it was sitting on coal measures with a very simple fill material and I didn't really see any virtue in going for the old-fashioned flatter slopes.

The draw-off gallery has draw-off valves at three different levels and one bottom one, which you could call a scour valve. In fact, there are two, twin scour valves and scour pipes, running down the culvert, from one of which is drawn the compensation flow, with a concrete plug at the upstream end.

I would draw your attention to the fact that the toe wall, with this type of design, is of course an integral part of the cut-off. I regarded it as a kind of grout cap, which forms a joint between the grout and the asphaltic concrete. The point is that where you have a culvert or any concrete structure that's got to stick out of the face of the dam, then the necessary profile, forming the joint between the asphaltic concrete and the concrete has got to be carried over the culvert.

This detail, therefore, takes the form of a kind of a collar around the culvert and cast integral with it, but whose upstream profile matches the detail of the toe wall. The crack when it was discovered in fact was vertically over that collar having to stand proud.

Going back to the causes of the crack, if you imagine yourself looking immediately downstream of this collar, then in the plane with the collar you've got a bit of concrete; immediately behind it of course you've got more fill, so you have an abrupt change from concrete to fill material. Now, we were conscious of this at the time of the design, and that differential settlement could take place. This is one of the reasons why the binder and levelling course of asphaltic concrete thickened as it approached the toe wall and, of course, the collar.

The next most important detail was the downstream face of this collar, designed with a batter downstream, for the fairly obvious reason that this would encourage any fill material on settling to become compressed; in other words there was a wedging action resisting differential settlement.

When the crack was opened up and the damage asphaltic concrete was removed and the collar was exposed, it was discovered that it had been in fact cast with a vertical face. I have never had the slightest idea why that was done, but it was obviously a mistake on somebody's part. I can't believe it was something of which we would ever approve and I can never find any reference to either the contractor requesting any change or us issuing one.

There were other points of detail, which had perhaps not been properly carried out, to do with the nature of the fill material in that region, but I think if we were designing the dam again, I would want to pay very much more attention to the detailed treatment of the fill material, wherever a concrete structure sticks out at the face. If anybody is considering a similar design of dam I would advise them very strongly to give that a lot of thought.

Another point is that discovery of the leak was very very difficult indeed. It is very easy, as most people say from time to time, to draw straight lines on drawings and imagine the most clean situations. If you have a dam or a reservoir that has been filled with water for 2 or 3 years, inevitably there is going to be a fair amount of sediment lying around. If you imagine the condition at the toe of a dam like this, with a fairly steep slope, and perhaps 2 or 3 inches of sediment against the bottom, and you have just emptied it because you are looking for a leak, you really are in an awfully messy situation and the discovery of such a small leak is extremely difficult.

I said that the cross-sectional area of the leak was roughly equivalent to that of a matchbox. This is because, although the crack had developed across the whole width of the culvert, or the asphaltic concrete above it, most of the crack, in fact, was very, very tightly caulked with mainly organic material, decayed grass or what have you, bits of mud and there was a fairly short length of the crack that was exposed and it was simply by chance really that the thing was discovered. I was plodding around in the mud, almost despairing of ever finding any leak and looking at this caked sediment when I saw this little round hole, about the size of a mouse-hole. I stuck my finger into this hole and realised that my finger had gone in further than the actual thickness of sediment, so then I plucked off the sediment and found this crack underneath. It really was by luck.

In the discussion at the San Francisco ICOLD about this type of dam, three main points were being made about asphaltic concrete.

One was that there was a lot of discussion of the material to be placed on the face of the dam immediately underneath the asphaltic concrete, from the point of view of a possible crack and the damage that it might then do to the fill material. It seems to me that this is most relevant when one is contemplating putting this sort of facing on to a sand-fill or other kind of earth-fill dam, rather than block-fill. It is all to do with the problem of prevention of erosion, but people contemplating this sort of dam might do well to study the discussion on that aspect in the ICOLD proceedings.

Secondly there was much discussion about the pros and cons of stacking over the toe, as it were, a reservoir of material - of filter material or silt - in order to bung up any defects that might occur at the interface between the asphaltic concrete and the concrete toe wall. I came away with the impression that most speakers were in favour of this and there was a lot of varied discussion about the grading of such a material. With the experience of Winscar behind me I am not sure that I would go along with that. It seems to me its very much better to try to create a situation in which any defect would most easily be discovered, rather than to bury it with something that you hope will be self-sealing.

Thirdly, there was some discussion at San Francisco about safe slopes for asphaltic concrete, and what was meant here, it slowly dawned on me, was safety from the point of view of the actual operators working on the face. Now it seemed to me that a slope such as 1 on 1.7 is so steep that you've got to be daring to try and work on it at all without ropes. Then I remembered, what I should have remembered all my life, that there was, in fact, a fatal accident at Winscar during construction. Sadly, the man who was in charge of the team constructing asphaltic concrete stepped on to the top of the slope one day, after rain when it was wet, lost his footing, fell down the slope and was killed instantly on arrival at the toe. It had never occurred to me before that a safe slope in asphaltic concrete from this point of view was, in fact, a design criterion.

Another problem was some degree of blistering where the upper layer of asphaltic concrete, given access to water from above, did tend to blister here and there and that was due to imperfect connection between the layers.

Of the other things that I would do differently - the upstream toe wall was specified as being 2 metres thick and to penetrate sound rock by 2 metres. I think we really cut the dimensions down to a bare minimum here. I think I would be rather more conservative next time.

There is also one badly leaking joint in the culvert immediately upstream of the toe collar, just downstream of its junction with the gallery. The movement of the culvert sections have been continuously monitored and it is probable that a water bar has been stretched to something approaching its limit, and there seems to be some leakage around one of the pairs of waterbars that were installed around the joints.

R MELBINGER (Federal Ministry of Agriculture, Forestry and Water Management, Austria)

Do you measure the seepage flows at Winscar and if so how great are they?

J D HUMPHREYS

Yes, it is being measured all the time and it was this measurement of seepage flows that alerted us, first of all, to the fact that the grouting needed supplementing and, later, that there was a problem with the asphaltic concrete. I think that, as the inspecting engineer, Bill Carlyle might like to comment on the quantities. My own view is that they are nothing to worry about.

W J CARLYLE (Binnie & Partners)

I would say that they were at one time high when John Humphreys was doing the secondary grouting and they progressively reduced, and I am only going to guess now an integration of a number of different drainages, some from the grouting in the rock abutment, some from the seepage collection points beneath the dam, and some from the drainage pipes laid within the fill; I think somewhere between 15 and 20 litres per second, something relatively quite small.

The gallery is built well into the rock; the rock foundation level is up near the shoulder of the gallery, so one would have expected it to be solidly integrated. In fact, it does appear that on filling the reservoir, the whole dam and its rock foundations moved downstream, opened that joint by 20 mm or thereabouts and, at the moment, it opens and closes roughly a millimetre for every metre of change in the water level of the reservoir, so obviously a crack has formed in the bedrock coincident with the intersection of the upstream toe. The joint was perfectly well able to take this, but the leakage is thought to be due to an imperfection in one of the welded joints in the water bar and it has stretched a bit; maybe some little work needs to be done to make sure that the water bar does not give way any further. It's not a major problem, it's just rather interesting that it's not something you would have expected to happen. If anything, you would have expected the upstream toe to move to the left and the downstream toe to move to the right, as the embankment spread. It appears not to have worked that way.

OPENING SESSION AND INTRODUCTORY ADDRESS

PROFESSOR MONTAGUE (Head of Dept of Civil Engineering, University of Manchester)

Good morning, Gentlemen. I have been looking out for the ladies and I can't see a single one. You know about 20% of our civil engineering undergraduates these days are ladies. What do you do to them?

I imagine that the majority of the 260 or so delegates attending this conference are practising engineers. You are most welcome into the heart of this great university. As an academic, teaching and conducting research in civil engineering, I say that word of welcome with particular sincerity because one of the most important developments during the last decade for the academic engineering community has been the rapid growth and the now close association with the civil engineering industry. The old 'ivory tower' image has been completely demolished and we now get more than half of our research income by means of research contracts with the civil engineering industry. The industrial contributions to our undergraduate courses and the most welcome and very generous contributions from people coming in to help us, particularly with design, all this is particularly well-established and greatly appreciated by us. There is presently something approaching a national crisis in the provision of civil engineering graduates. I have never known a year when the number of requests I have received from industry for graduates has been as great as it has been during 1988. Of course, when people call and ask me if I know of a graduate who is looking for a job in this or that kind of civil engineering, they implicitly mean a good graduate and that leads to a point.

Whilst the demand for our product is very pressing, the number of young people choosing civil engineering as an undergraduate study continues to fall. At the present time, the number of applications for civil engineering is just about equal to the number of places available in the universities, but because the universities demand a certain entry standard and not all those applicants achieve that standard and therefore do not come, we have a surplus of places. However, because universities themselves are under considerable pressure to attract students because our resources depend on numbers, there is a temptation to lower the standard of entry and that, of course, has the inevitable effect of diluting the level of talent in the profession.

So we have a national problem which is only part, although a particularly difficult part, I think, of a bigger one, namely, the movement of young people away from science and technology - a sort of rejection of an economically-approved culture. We can't find the reason in demographic trends because, although our 18-year old population is going to reduce by about 35% between now and 1995, the vast majority of students coming to the university come from socio-economic groups 1 and 2 and actually there is a 9% increase in those groups between now and 1995.

Now, I feel the solution to this problem must be provided largely by the profession. Two things, I think:

- it must pay better salaries and, somehow,

- it must convey to young people the excitement and the satisfaction and the enjoyment of engineering.

Now, being concerned with dams, I think you have a particularly good opportunity to cast an image of the profession which is attractive to youngsters. And I don't suppose for a moment that you would disagree that you should be paid more. Well, that's my message. I hope the conference is a great success. Let me repeat my very warm welcome to the university and, right across the road opposite this building, stands the engineering department. The engineering department here consists of, in one department, aeronautical, civil, mechanical and nuclear. If you should feel like popping in any time during your stay here, you are most welcome to do so and you can have a look round and see what is going on in research, and so on.

Thank you for inviting me to come along and say these few words and, again, welcome to Manchester.

MR E T HAWS (Chairman, British National Committee on Large Dams)

Professor, thank you very much indeed for your welcome and thank you for letting BNCOLD have the use of your university facilities for this conference. We do value it very much indeed. As you know, BNCOLD dams are all about water - according to folklore so is Manchester, but we seem to have done a great deal better than that today. I hope you all enjoyed your 'constitutional' coming along from the Hulme Building and I hope the ladies enjoy this weather and we get it for our site visit on Saturday, too.

The student shortage - yes, indeed it is quite a problem for the future. We have it now in the industry - we find great difficulty in recruiting even now and, with the shortage of students being produced from the universities in future, that is obviously going to become increasingly difficult and increasingly vital for this country to overcome. We shall certainly do everything we can in the profession to show how attractive it is and to show that there are other things than large sums of money in service industries; creation of resource is an activity to be proud of and we, as BNCOLD members, will be selling that message to young people. It is particularly sad to see students who have taken an engineering course tempted away immediately they graduate to higher earnings in the service industries and that's something we certainly as a profession, have to face up to: to make it attractive; to sell ourselves to these young people.

It's very impressive to see the numbers here present this morning and we are absolutely delighted at the numbers who find it worthwhile to be with us.

In particular, I must welcome our foreign visitors. We have representatives from Austria, Belgium, Hong Kong, Italy, Spain, Switzerland, USA and West Germany and that's a very impressive list of which BNCOLD is extremely proud. Welcome most sincerely to you all. We know you are going to contribute greatly to our discussions. You have already contributed in terms of papers presented. We have papers from six countries and three continents and that's not including the UK - and I daresay some of my colleagues think that should be at least three, there are papers from all these countries. We do look forward to contributions of the visitor. I would please ask them and also the Chairmen of the sessions - your English is excellent, our foreign languages are rather pathetic in this country - if, however, there is anything in the discussions of a technical nature that you would like additionally clarified, please stop the Chairman and we will do our utmost to help you if there is any help needed. That's not done in any patronising way, I can assure you. We do wish for your full participation.

The other very exciting thing for BNCOLD is to see the full representation from everyone with interest in dams: the owners, the consultants, the contractors, the panel engineers, the universities, the enforcement authorities and government. This is the first time, to my knowledge, that the enforcement authorities have participated in such a function. We do welcome you. We hope your participation will grow over the years. Perhaps we shall hear from you how the Act is working. We look forward to that soon. This is an auspicious year for BNCOLD. This is our 21st milestone - our 21st anniversary. The presence here shows the strength and growth of the organisation and how it is thriving.

Michael Kennard has taken on board the task of writing a history of BNCOLD and that is going to appear as the preamble of the proceedings of this conference. So we look forward to seeing that from Michael in the very near future. Another auspicious circumstance for this particular conference, in my view, is the fact that, as I count them, there are at least four new dams under construction in the UK. That hasn't been happening for rather a long time and perhaps there are more. But I know of Roadford, Queen's Valley about to start, Carsington and Gale Common Stage 2: all major construction efforts and new works. I don't think we will be scaring the Japs with a list of four under construction, but it's a promising sign of growth, and also that British engineers continue to show imagination in these projects. There are features in them that would have been considered uncommon a little while ago. Roadford with the upstream bituminous membrane, Queen's Valley with the central bituminous core and Gale Common with an extensive plastic membrane protecting the aquifers against leaching.

So there are new and exciting things going on. If people involved with those new projects feel that anything on those is relevant, please do have your say, in spite of this being a conference on renovation of old reservoirs. We do like to see where new ideas are relevant, particularly in the context of the old dams we have to deal with.

Then, another exciting development, I think, for our group of dam engineers is the great interest in the country in estuarial structures. Again, to my knowledge, we have studies and designs going on for the Severn, the Mersey. the Humber, the Towey, the Taff, Usk and Deptford Creek. Surely, here, there must be a major subject for BNCOLD in the years to come and another, as I say, important development for their engineers and hydraulic engineers.

Another thing that we have to face up to in the fairly near future in the industry is privatisation and clearly this will be in mind during our discussions during the next couple of days. What will be the new pressures on maintenance? Will there be pressure from shareholders for profit, so that maintenance items are pushed a bit further in the background and works other than in the interests of safety reported by an inspection engineer - are they going to be delays and deferments? Will there be a premium on the services of less demanding inspection engineers? There's simple food for thought, and will there be more dams per supervising engineer with the big organisations? Maybe you would care to put some thought and make some remarks on some of these things during the next couple of days.

Another circumstance which we meet is the increasingly vociferous outbursts against dams. I was moved to write to "The Times" this week. I haven't managed to get a copy today - I understand it is to be in, but there have been articles, letters blaming dams for some of the flooding that has been occurring around the world. Quite inaccurate and misleading reports and associations, but they are being made repeatedly. Those that were in San Francisco will have seen the great activity by the anti-dam lobby there to the extent of making a proclamation for a moratorium on all dam construction. That's something we as a profession must face up to. It's the policy of BNCOLD as declared at San Francisco and likewise of BNCOLD to be "pro-active" not "reactive". Take the initiative: point out to the society and the community the value of the work we are doing in providing good water supply and clean energy.

I haven't talked at all about the papers yet. We have a great variety of subjects: discontinuance, spillway and draw-off augmentation, monitoring and supervision, drainage instrumentation, sealing, slug protection, even RCC. Papers by Hughes and Milmore and Charles quote some facinating statistics. Here we are faced with 2,450, no less, large dams of a great variety of age and state which we have to confirm are safe and deal with for the future. In many cases, the ownership is in question; loving care is no longer available and it's a great problem with some of these old dams which are the subject of the papers we are to hear about. I find it infinitely sad that there should be a potential loss of an amenity like an old millpond, an old mill pond in my old home county of Sussex - been there for a couple of hundred years, used for angling, watersports, other amenities and, because of the Act and requirements related to spillway capacity in particular, the things are, on occasion, threatened with discontinuance. I believe that's something we, as a profession must deal with, we must try and preserve those amenities. People want them, they have been there, they are enjoying them. So let's do something about it that's cheap and reliable and yet conforms with the Act.

I am delighted to see grass spillways and collapsible gates suggested as possible solutions in some of these circumstances. I, myself, have suggested old-fashioned flashboards in the recent past. There are answers that are cheap that will preserve these old dams and ponds and we should be looking at all of those.

If we go too far with safety, the next step obviously relates to qualification for earthquake and I won't dwell on that subject very much, but it is a thing that certainly hasn't been taken to the lengths of flooding. We know dams are generally rather good at earthquake resistance but we haven't looked at return-period earthquakes anything like return-period floods. What about the Lake District earthquake last week?

Thank you very much. Thank you, Professor, for having us at your university.

PROCEEDINGS: TECHNICAL SESSION 1

ENFORCEMENT

PAPERS:

DISCUSSION:

THE ENFORCEMENT AUTHORITY AND THE RESERVOIRS ACT 1975

W S Maconochie BSc CEng MICE MIWEM

Scottish Development Department

SYNOPSIS

The Enforcement Authority and the Reservoirs Act 1975

The paper outlines the initial effect in Scotland of the Reservoirs Act 1975 as indicated by the first reports to the Secretary of State by the enforcement authorities. Matters of interest raised by the enforcement authorities and others are discussed in the context of the Reservoirs Act 1975 and Scottish legislation.

INTRODUCTION

1. The events leading up to the earlier legislation of the Reservoirs (Safety Provisions) Act 1930 are well known. Its implementation brought about an improvement in reservoir safety, and while it was in force there were no major dam disasters in this country involving loss of life. It introduced important concepts such as the panel system of engineers qualified under the Act, safeguards for the design and inspection of large raised reservoirs, and statutory procedures for record keeping. Nevertheless the lack of qualified supervision between inspections, and the absence of any authority with a duty to enforce the Act were among factors that weakened its effectiveness.

2. The Reservoirs Act 1975 replaced and improved on the provisions of the 1930 Act. It applies throughout Great Britain, and provides that in Scotland, regional and islands councils are enforcement authorities with a duty to enforce the 1975 Act and to compile a register of large raised reservoirs in their area. For the purposes of the 1975 Act a large raised reservoir is one designed to hold or capable of holding more than 25,000 cubic metres (5.5 million gallons) above the natural level of any part of the adjoining land instead of the 5.0 million gallons under the 1930 Act. The main changes are:

i. the introduction of enforcement authorities with stronger and more explicit powers than those previously available to local authorities, including powers to satisfy themselves that the Act is being complied with, to have measures in the interests of safety carried out (section 15) and to take emergency action (section 16) and to recoup their reasonable expenses from the undertakers.

ii. the enforcement authority in whose area a reservoir is situated, if they are not themselves the undertakers, secure that the undertakers observe and comply with the requirements of the Act; (section 2(3))

iii. the enforcement authorities have a duty to establish and maintain a register showing the large raised reservoirs situated wholly or partly in their area; (section 2(2))

iv. appointments of panel engineers to design, inspect and supervise reservoirs are limited to terms of 5 years, subject to reappointment; (section 4(3))

v. a panel engineer must be appointed when a large raised reservoir is to be discontinued or abandoned; (sections 13 and 14)

vi. without reasonable excuse, an undertaker's wilful non compliance with the safety provisions of the Act is a criminal offence; (section 22). Undertakers are defined in section 1(4). If the reservoir is not used or intended to be used for the purposes of any undertaking, the undertakers are then taken to be the owners or lessees of the reservoir.

REPORTS TO SECRETARY OF STATE

3. In Scotland the Act required regional and islands councils to report to the Secretary of State on 1 April 1987 on their actions as enforcement authorities and as undertakers for reservoirs wholly within their areas. Further reports have to be made at two yearly intervals. (SI 1985 No 177 Reg 4).

4. By public advertisement, map searches, use of local knowledge and correspondence with undertakers, enforcement authorities have endeavoured to identify the large raised reservoirs in their area, and to inform undertakers of their duties under the Act. In spite of the earlier reservoir legislation it is apparent that many private undertakers were unaware of their duties under that legislation. Where the enforcement authorities have found undertakers not complying with the Act, the authorities have therefore generally sought to persuade them to comply. Nevertheless in some cases formal action has been, or is likely to be taken by authorities to:

a. serve notice on undertakers to appoint a qualified engineer or to carry out safety works recommended by a qualified engineer;

b. appoint a qualified engineer on behalf of an undertaker;

c. if necessary to consider the possibility of criminal proceedings against undertakers wilfully not complying with the Act

5. Compared with the 590 or so large raised reservoirs known before implementation, the reports indicate a total of about 730 large raised reservoirs in Scotland, and confirm that a number of mainly private reservoirs that should have been inspected under the 1930 Act are now being inspected for the first time. (For Great Britain the total number of large raised reservoirs is now estimated at about 2450 compared with the estimate of about 2000 before the 1975 Act came into force). As expected, the regional and islands councils, and major undertakers such as the North of Scotland Hydro Electric Board are complying with the Act and there is no doubt that they will make every effort to continue to do so. The table at Annex 1 gives numerical information gathered from the reports. Because numerical information was prescribed by statutory instrument for items (i) and (ii) only of the table, the remaining information is based on subsequent assessment in some cases, and is therefore imprecise.

Nevertheless, by September 1987, a good indication of the degree of compliance had been obtained. Although about 100 reservoirs were without supervising engineers, and 60 had overdue appointments of inspecting engineers, the continuing efforts of the enforcement authorities should see a reduction in the number of instances where undertakers are failing to meet their obligations.

6. In my view the reports indicate that in Scotland enforcement authorities are discharging their duties responsibly and that the Act is being enforced in a realistic way. The enforcement authorities in Scotland will next report to the Secretary of State on 1 April 1989.

7. I must take this opportunity to thank publicly the Scottish enforcement authorities for their patience and cooperation in providing me with information for this paper beyond that prescribed by regulation.

8. The information called for by statutory instrument does not provide categories of ownership other than those necessary to prevent double counting of reservoirs. An approximate allocation of the ownership of Scottish large raised reservoirs in 1987 is however as follows:-

Regional and islands councils	356
Other public bodies	148
Others	226
Total	730

MATTERS OF INTEREST THAT HAVE ARISEN

9. Interpretation of legislation is ultimately a matter for the courts. The opinions expressed in this paper therefore do not purport to be an interpretation of the Act, nor do they necessarily represent the view of the Scottish Development Department.

Section 1. Definition of large raised reservoir

10. The question most frequently asked is what water level should be used in calculating the capacity of a raised reservoir for the purposes of the Act. Reference is sometimes made to the definition of "top water level" given in SIs 1985 No 177 and 1986 No 468, and arguments have been put forward that that definition should be used in calculating the above capacity.

11. SI 1985 No 177, as its title indicates, is concerned with registers, reports and records, and the object of item 4 in particular of Part 7 of the Form of Record is to establish 3 capacities of the reservoir. For the purpose of prescribed records there must be no doubt about the points from which measurements are taken, and that is why definitions such as that of "lowest natural ground level" and "top water level" were given in the statutory instrument. The definitions are however prefixed by words such as "In this order" and "In this Form". In my view the instrument cannot give a definition for the purpose of assessing whether or not a reservoir is subject to the Act, and I suggest that the wording of section 1 of the Act only, defines the capacity of a large raised reservoir for that purpose.

12. Similarly, in SI 1986 No 468, which deals with certificates, reports and

prescribed information, the definitions of "lowest natural ground level" and "top water level" in SCHEDULE 3 allow information about the prescribed distances, capacity and area in item 8 to be established without doubt. In this case the prescribed information is that to be given in a section 21(1) notice served by undertakers on the authority or authorities in whose area the reservoir lies or will be situated, ie notice of intention to construct or increase the capacity, of a large raised reservoir, or bring back into use a large raised reservoir.

13. The water level to be taken when considering the stability or safety of a dam would, of course, take into account the amount of water that could reasonably be anticipated above "top water level" as defined by either statutory instrument.

14. Notwithstanding my views on the legislation, I have no quarrel, with those who (as long as they do not found on the statutory instruments) adopt the spillway level in straightforward cases to assess a reservoir's capacity under the Act.

15. It has been suggested to me that by introducing a pipe below the existing spillway level, then, almost regardless of the pipe size, the reservoir capacity for the purposes of the Act could be measured up to the pipe invert only. Although to add to existing spillway or overflow capacity can be, a proper exercise of engineering judgement, the object in this instance apparently was to remove the reservoir from the Act, without greatly effecting its actual capacity. It is unlikely that the proposal was implemented, but in any case such an approach would not have been compatible with the interests of safety, nor with the spirit of the Act.

16. Before leaving the subject of capacity it is worth considering those reservoirs with facilities for stop logs, particularly as it is important in "borderline" capacity cases to decide whether the capacity retained by stop logs should be included. At least one enforcement authority has taken the view, with which I agree, that if there are facilities for fitting stop logs then it should be assumed that the logs could be fitted and therefore contribute to the reservoir capacity. It should make no difference whether or not the stop logs are available on site.

17. There is, of course, an understandable emphasis, on establishing if a reservoir is subject to the provisions of the Act, nevertheless it is important that undertakers whose reservoirs fall outwith the Act should be aware of their duty of care to the public, and obligations under the Health and Safety at Work etc Act 1974.

Section 4. Qualifications of Engineers

18. It is becoming increasingly difficult for engineers to gain the necessary experience of large reservoir construction or major alteration to prepare them for membership of an inspecting engineer panel. More than half of the All Reservoirs panel members are aged over 60. Although applications continue to be made for appointment to the inspecting engineer panels, it is hoped that senior engineers will be aware of the need to give their staff the opportunity to gain experience suitable for membership of those panels.

Sections 6 and 10 etc. Relationship between construction and inspecting engineers.

19. The question of the "independence" of the inspecting engineer is sometimes raised. An inspecting engineer must be an "independent" engineer (section 10(1)). Briefly "independent" in the Act means not employed (except as a consultant) by the undertakers and not connected in any way with the construction engineer in a civil engineering business (section 10(9)).

20. For the purpose of the Act a qualified engineer is a reference to a member of a panel constituted under section 4. Qualified engineers under sections 8, 9, 25 (via section 8) and 27 (via section 8) act in similar ways to construction engineers and can provide certificates that have effect for the purposes of the Act as though they were certificates of a construction engineer. Nevertheless, it is only in sections 6 (construction, or enlargement of reservoirs) and 7 (certificates of construction engineers) that an engineer is called a construction engineer. These sections are also referred to in sections 25 and 27.

21. Because section 6 deals only with new, enlarged or restored reservoirs, then an inspecting engineer would only be barred from inspecting a particular reservoir if his connection with the reservoir related to those categories. If an engineer has doubts about whether or not he is independent, it would be a matter of common prudence to decline to undertake the inspection of the reservoir. There should be no obstacle, however to an inspecting engineer who is not independent of the construction engineer subsequently designing and supervising work which does not come into these categories.

Section 10 etc. Inspections and subsequent safety recommendations

22. The main concern of the Act is safety and formal recommendations as to measures to be taken by the undertaker in the interests of safety have the force of law. As stated in the Department of the Environment's letter to panel engineers, of 26 February 1986, it is important therefore that such recommendations should be confined to those which the inspecting engineer considers to be essential in the interests of safety. Other measures in the interests of good management should be conveyed to the undertaker separately.

23. Some enforcement authorities have expressed concern about the lengthy time that sometimes occurs between an inspection and the subsequent report. Section 10(3) states that the report should be made as soon as practicable after the inspection. There is therefore no quantified time before submission of the report.

24. Such delays may well be a recognition of the early effectiveness of the Act, and the subsequent increased work load being placed on inspecting engineers. If a report is unduely delayed a concerned enforcement authority or undertaker might wish to seek confirmation prior to the submission of the report that safety measures are not urgently needed, and (although not set down in the Act) to ask the engineers to submit the report within a set time.

25. Where an inspecting engineer includes in his report any recommendations as to measures to be taken in the interests of safety then, subject to the referee provisions, these measures are to be carried out under the supervision of a qualified engineer (Section 10(6)). <u>That</u>

engineer subsequently gives the certificate that the recommendations have been carried into effect.

26. It has been suggested to me that a supervising engineer could carry out these functions of supervision and certification. Section 12 however, which relates to supervising engineers, contains no such powers and section 10 is referred to, only to allow the supervising engineer to recommend an inspection. I believe therefore that a section 10(6) qualified engineer cannot be a member of the supervising panel only. To allow good housekeeping measures to be carried out outwith section 10(6) is therefore another reason to ensure that recommendations in the interests of safety are separately conveyed to the undertaker.

Sections 10 and 12. Relationship between inspecting and supervising engineers.

27. An engineer who is a member of the supervising panel only, cannot properly carry out a section 10 inspection. A supervising engineer's duties however do not lapse during a section 10 inspection and it would obviously be helpful to the supervising engineer and the inspecting engineer if the former were present during the course of the inspecting engineer's site visit.

Section 11. Directions given by a construction or inspecting engineer.

28. There can be difficulty when the last involvement of the appropriate panel engineer preceded commencement of the Act and therefore the directions referred to in section 11(2) have not been given. Nevertheless, undertakers must now keep records as prescribed in SI 1985 No 177 and I suggest in such cases employ either the previous frequency of record keeping, or seek directions from an appropriate panel engineer who has been connected with the reservoir. Although it is common to refer to the inspecting engineer of a reservoir as though his duty were continuous, it is of an episodic nature and it may be that a panel engineer would take the view that his directions would have no standing unless he were appointed as inspecting engineer for the reservoir. I hope however that a panel engineer with a good knowledge of the reservoir would be prepared to make recommendations about record keeping, without seeking appointment as the inspecting engineer for that purpose only.

29. There is a view that the inspecting engineer has discretion to waive the undertaker's duty to keep section 11 records. The engineer's discretion however relates only to the "intervals" and to the "manner" of record keeping (section 11(2)), and in my view a variation in the "interval" or "manner" is unlikely to extend to a complete absence of records. I accept however that although the undertaker's record keeping duty is clear section 11 does not say the engineer has a duty to supply directions relating to record keeping. If the engineer is satisfied that for a particular remote reservoir it is safe to do so, he may direct that records be taken at fairly infrequent intervals. This would comply with the Act and at the same time place little burden on the undertaker.

30. Form F must no longer be used. Unlike uncompleted reports based on a 1930 Act inspection, record keeping is a continuing situation and there is a legal requirement for section 11 records under the 1975 Act. This applies regardless of the legislation under which the last inspection was made.

Sections 13 and 14. Discontinuance and abandonment

31. Before a large raised reservoir can be discontinued and removed from the register (section 13(3), a certificate, or copy certificate under section 13(2) must be submitted to the enforcement authority.

32. Abandonment is not defined in the Act, and any meaning can only be obtained from the wording of section 14. Because the measures (if any) that ought to be taken in the interests of safety, require that the reservoir is incapable of filling accidentally or naturally etc an abandoned reservoir probably means that there has been a permanent reduction in water level. Whatever the definition, because an abandoned reservoir may in some circumstances fill to a level and capacity not quantified in section 14 then the reservoir still remains within the Act, requiring supervision and statutory inspection, unless and until it is discontinued under section 13.

33. Although strictly not related to the Act, undertakers should consider other factors before discontinuing or abandoning a reservoir. Planning and environmental considerations may be significant, and there may be an existing requirement to provide compensation water which could not always be satisfied after the reservoir had been discontinued or abandoned. In the case of a reservoir for which the water authority is undertaker, there may be a local enactment relating to the supply of water, with provision for compensation requirements. Under section 107 of the Water (Scotland) Act 1980 the Secretary of State may, by order, repeal or amend any local enactment relating to the supply of water. Such an order may include such transitional, incidental, supplementary and consequential provisions as the Secretary of State may consider necessary or expedient. The provisions of Part 1 of Schedule 1 to the 1980 Act would apply, ensuring that the necessary advertising and notification procedures would be carried out. In addition to the requirements of the 1975 Act the discontinuance or abandonment of a reservoir not relating to the supply of water would have to be considered in the light of the powers under which it was constructed.

34. In cases where there is agreement only, or indeed where there is no record of powers or agreement, the undertaker would no doubt wish to reach agreement with known interested bodies. It may also be prudent to consider whether or not some form of advertisement should be carried out before discontinuance or abandonment.

35. Concern has been expressed by some private reservoir owners, about the effect of the 1975 Act on their reservoirs and on the environment if their reservoirs had to be discontinued on financial grounds. Undertakers had of course, obligations under the 1930 Act but, nevertheless, any concern for the environment must be recognised.

36. Though the main purpose of the Act is to ensure reservoir safety, it is important that environmental considerations should be taken into account in keeping with the spirit of the National Planning Guidelines. Proposals to drain reservoirs or lower water levels can have implications for nature conservation, landscape and recreation. Scottish local authorities should therefore consult the Nature Conservancy Council (NCC) and/or the Countryside Commission for Scotland for appropriate advice wherever these interests are likely to be affected by such proposals irrespective of the

location of the reservoir. Breaching of a dam would normally require planning permission which would therefore require consultation with the NCC under Article 13(1)(g) of the Town and Country Planning (General Development) (Scotland) Order 1981 in respect of dams within sites of special scientific interest. In addition section 28(5) of the Wildlife and Countryside Act 1981 places a requirement on owners or occupiers of land within sites of special scientific interest to notify the NCC of any operations in areas of special scientific interest.

Section 17. Powers of Entry

37. A person duly authorised in writing by the enforcement authority may at any time enter the land on which a reservoir is situated, for the purposes listed in sections 17(1)(a) to (e).

38. An interesting case was recently drawn to my attention in which the undertaker was the enforcement authority for the area in which the reservoir was wholly situated, and therefore the provisions of the Act relating to enforcement by the authority did not apply to that reservoir (section 2(6)). Under section 10(3) the inspecting engineer had recommended safety work to the earth dam, the downstream toe of which was in private ownership. The undertaker was willing to carry out the work at his own expense, but was not allowed access to the embankment toe. This may seem surprising particularly as the safety operations would have benefitted the land downstream from the dam.

39. Normally it may have been possible via section 15(2) (Powers of the enforcement authority, to cause safety recommendations to be carried into effect) to use section 17(1)(d) to gain power of entry to the toe. In this instance because of section 2(6) the undertaker could not seek authorisation from an enforcement authority to obtain access. It was suggested to the undertaker that he should either look for the necessary powers in the legislation under which the undertaking was being carried out, or under which the local authority operated.

40. It would be prudent therefore for enforcement authorities who are undertakers, or propose to become undertakers for such reservoirs, to ensure as far as possible that they have power of access to carry out safety work on their reservoirs, or indeed work on adjacent land if it is needed to protect the safety interests of the reservoir.

41. Section 25(1)(b). Large raised reservoirs without a final certificate under the 1930 Act

A large raised reservoir constructed or altered to increase its capacity between the commencement of the 1930 and 1975 Acts could lack a final certificate. If so, section 25(1) of the 1975 Act states that sections 6 to 8 shall apply as if the construction or alteration were carried out wholly after the commencement of the 1975 Act. These sections refer to the need for a construction engineer, to certificates of construction engineers, and to the powers of an enforcement authority in the event of associated non compliance.

42. If the reservoir has been constructed or altered in the last few years then there is unlikely to be difficulty in obtaining in due course a section 7(3) final certificate, and the section 7(6) supporting information and drawings.

43. In the case however of a reservoir constructed or altered in say the 1930's and for which there is no final certificate the original construction engineer will probably be unavailable. This has been perceived by some as a problem, because a panel engineer, who has little knowledge of the reservoir, could not, if he were asked to do so, give the unqualified assurances and detailed information called for in section 7. Section 25(4) of the 1975 Act anticipated the problem by stating that if at the commencement of the 1975 Act there were no construction engineer for such a reservoir, then the undertakers had to appoint a qualified engineer for the purposes of section 8, without being required to by a notice from the enforcement authority. If the undertakers fail to do so within 6 months of the commencement of the Act, then sections 15 (reserve powers) and 22(1) (criminal liability) apply as if the undertakers had been served with a section 8(6) notice to make the appointment by the end of that 6 months. With the exception of section 15, this also applies to enforcement authorities as undertakers for reservoirs wholly within their area.

44. The main practical advantage of section 8 is that there is provision in section 8(6) for a final certificate in which the engineer need not state that he is satisfied that the reservoir or its addition is sound. In addition the information to be provided about the execution of the works, and site conditions, is qualified by such phrases as "so far as has been able to ascertain" and "giving such information as he can". (section 8(7)).

45. In this paper I have tried to explore some of the areas of the Act which have caused concern, or which have given rise to differing opinions. I am sure that opinions will still differ but perhaps subsequent debate may go some way to establishing a common view, where none is yet available from the Courts.

ANNEX 1

RESERVOIRS ACT 1975
LARGE RAISED RESERVOIRS IN SCOTLAND

i.	reservoirs for which the enforcement authority is undertaker, and which are situated wholly in the authority's area	314
ii.	reservoirs for which the authority is the enforcement authority	416
iii.	large raised reservoirs in Scotland, total of (i) and (ii)	730
iv.	pre-implementation estimate of large raised reservoirs in Scotland	590
v.	reservoirs without supervising engineers	101
vi.	reservoirs where appointment of inspecting engineers is overdue	60
vii.	reservoirs where formal enforcement action has already been taken	6

Note: Information obtained from local authority reports of April 1987 to the Secretary of State, updated and augmented to September 1987.

Items i and ii are prescribed requirements; items iv - vii are estimates only.

GWYNEDD COUNTY COUNCIL AND THE RESERVOIRS ACT 1975

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Gwynedd County Council

SYNOPSIS

Gwynedd County Council's involvement with the Reservoirs Act started from a base of no previous direct experience. The establishment of a register and procedures were undertaken in a manner probably mirrored in many other enforcement authorities. However Gwynedd had the additional experience of exercise of emergency powers and has applied that knowledge in formulation of proposed emergency procedures.

BACKGROUND

1. The introduction of the 1975 Reservoirs Act involved the newly designated enforcement authorities intimately with large raised reservoirs where in many cases there had been no previous involvement. This paper outlines some of the experiences and lessons learnt within one such enforcement authority, Gwynedd County Council, since the inception of the Act.

2. The lack of specific funding necessitated the establishment of the required administrative and organisational system within the existing authority structure and without prior experience of the problems. Thus the situation has arisen where the enforcement duties are performed jointly by the legal section of the County Secretary's staff and designated members of the County Surveyor's staff whilst the role of undertaker devolves upon the Snowdonia National Park Committee. An unexpected bonus of this diverse structure has been the different outlook of staff from various disciplines when formulating our new procedures and appropriate responses during the formative period of establishing and maintaining a register. In reality the County Secretary's staff keep the official register and deal with all normal correspondence, whilst the County Surveyor's staff maintain the register and ensure compliance both in administrative and technical terms. This system provides a dual check on compliance and has served us well in general but could prove cumbersome where a great deal of purely technical correspondence is required and in such cases the County Surveyor would write directly (as in the discontinuance of Llyn Cwm-y-Foel discussed later).

ESTABLISHMENT AND MAINTENANCE OF THE REGISTER

3. Creation of a register was the first problem and as an enforcement authority we had to take "steps they think are reasonably required to inform undertakers of the requirement". We decided on a 'broad brush' approach and accordingly took a reservoir list provided by the Welsh Office, added all bodies of water held to be of sufficient size which were

visible on the 1:50000 ordnance survey sheets and used this as a starting point. This provided a list of some 162 bodies of water and, in the absence of a Land Register, local knowledge of the area staff was used to ascertain as many of the landowners/undertakers as possible. All parties identified at this stage were notified with an appropriate letter considering the fact that the information may have been inaccurate or the body of water only a natural lake. In the meantime consultation with the Welsh Water Authority and the Central Electricity Generating Board clarified which of their reservoirs were affected and also provided the first series of required information to test the adequacy of our proposed register format. In addition it provided the first real idea of the magnitude of the task since although their reservoirs had invariably been inspected, notices issued and works undertaken even here the odd report or certificate had gone astray. The format chosen for the register was a loose leaf, A3 sized, sheet which has not required any revision since its inception. The sheet or sheets for each reservoir are held in a plastic wallet and the wallets are gathered together in a binder. The flexibility this gives has been of advantage in coping with varying amounts of data ranging between a simple reservoir with just ten yearly inspections and a situation as at Llyn Trawsfynydd where reports were being made every six months together with recommendations for measures in the interests of safety.

4. The next major operation was to visit all the outstanding sites to formulate our interpretation of their standing with regard to the Act. If the body of water was not wholly natural then possible maximum water area and retained depth were used together with a reduction to allow for the cross section in order to classify all bodies of water as a) definitely within b) uncertain c) definitely outside the Act. This operation led to the removal of a large number of possible sites which were too small, natural lakes or, in one case, discontinued and built upon. At this stage the full 9 month period for establishment had elapsed and so the initial register was compiled incorporating all sites in classes a) and b) comprising a total of 70 reservoirs, many of which had solely a grid reference and a name. The reservoirs and other bodies of water outside the Act have been maintained on a supplementary list to enable a check against any future query as to their status.

5. Work since the establishment of the register has been threefold in maintaining complete records, work to obtain records from undertakers in default of their provision and attempts to clarify the position of reservoirs where we were uncertain of their true status. Uncertainty has been due to i) uncertainty as to exact volumes ii) doubt on interpretation of capacity to hold and iii) the possibility of 2 reservoirs being lagoons under the Mines and Quarries Act. Our efforts to resolve the uncertain reservoirs have been through in-house surveys of volume and where required the appointment of an All Reservoirs Panel Engineer to advise on their status. The work has not always been straightforward as in the case of one semi-drained reservoir where unexploded mortar shells hampered safe surveying or the interpretation of fully silted and habitually empty reservoirs. However these exercises have removed a number of reservoirs from consideration some 7 being removed from the register as a result.

6. The other cause of a decline in numbers on the register is the tendency to carry out works sufficient to reduce the retained volume below 25000 cubic metres and obtain a certificate of discontinuance. This trend has

been quite strong in the county but perhaps because of the influence of the National Park Authority there has been little tendency to completely drain reservoirs and often the reduction has had little effect on surface area. However it may be an expression of the perceived effectiveness of the new enforcement authority structure that undertakers are taking measures to remove themselves from the ambit of the legislation where under the previous Act no action was taken. At present 6 reservoirs have been discontinued since January 1986 whilst several others are being considered.

7. In terms of maintenance of the completed register we have decided to allow a two month period after the due date of inspection before, initially politely, reminding an undertaker that it is overdue. Much discussion as to what was a reasonable time for an Inspecting Engineer to complete and submit a report preceded the decision, the longest period we have currently recorded is 15 months!. Obviously less time will be allowed before enquiring about important works required in the interests of safety and it is of considerable assistance where the Engineer stipulates a time period for their completion. Public safety is always a prime concern for a local authority and as such brought Gwynedd direct involvement with a large raised reservoir at an early stage in the implementation of the Act and before full inception of all of its sections.

LLYN CWM-Y-FOEL DISCONTINUANCE

8. Llyn Cwm-y-Foel was a large raised reservoir of some 80000 cubic metres capacity situated at SH 655 466 in the Snowdonia National Park above the valley and village of Croesor. It was constructed at the turn of the century as a masonry dam at to provide electricity for Croesor slate quarry but had been disused for many years. On the 7th of November 1985 a walker reported to the North Wales Police that the dam was leaking, it is probably no coincidence that a programme 60 years after the 1925 Coedty failure outlining its effects on Dolgarrog had just been televised. The police contacted the Welsh Water Authority and National Park Wardens who visited the site in the dark. The police were unaware of the County Council's new role and in any event no special emergency procedures for reservoirs yet existed (the emergency powers had not yet been enacted). The following day, in the afternoon, the Council was informed; the reservoir was already listed as being within the Act and our previous visit to assess capacity gave a rough volume estimate however the question of ownership had not been resolved. Our initial reaction and response was hampered by a lack of previous knowledge of the state of the structure and the possible effects that failure might have on the valley and village, this point will be further considered in the description of our present envisaged emergency procedures. As the Authority was unable to trace the persons responsible we arranged for an engineer on Panel 1 of the 1930 Act to inspect the dam as a matter of urgency. Mr R. M. Arah, Binnie and Partners was appointed and inspected the dam the following day.

9. In terms of an immediate emergency Mr Arah concluded that there was no visible indication of any fundamental structural distress but local deterioration of the upper masonry was allowing leakage. In his report he included the possible effects of failure and further made recommendations in the interests of safety that either

a) The dam should be repaired to prevent further deterioration and the draw off pipes and control valves put into working order, or

b) the reservoir should be safely discontinued by progressive breaching until the dam was no longer capable of holding 25000 cubic metres above the natural ground level.

Mr Arah further recommended that until one or other of the above measures had been carried out that the dam should be inspected and reported upon at monthly intervals and particularly after floods or periods of severe cold. The Council opted to retain Binnie and Partners to carry out the monthly inspections rather than involve the County Surveyor's staff. The initial report and subsequent monthly inspections were forwarded to the two parties in possible ownership as they were received.

10. The monthly inspections continued until the summer of 1988 by when it was apparent that the matter of ownership would not be easily resolved and that the inspections would represent an ongoing cost to the Authority. Additionally and more importantly deterioration of the dam had been observed during the monthly inspections carried out over the winter. Bearing in mind the isolated location of the dam the necessary works were required to be undertaken in the summer if further deterioration was to be avoided. The Panel Engineer recommended discontinuance and the Authority decided to execute its powers under section 16 and proceed to discontinuance in accordance with section 13, therefore Mr Arah was further appointed in July to supervise the discontinuance. It was first necessary for the Authority to consult with all bodies and persons who could be affected by the discontinuance of the reservoir and consequent release of water into Afon Croesor. Thus, for instance, at a meeting with the Welsh Water Authority it was agreed that the water level and water quantity would be monitored during water release to safeguard against possible flooding, erosion and pollution due to suspended solids or lowering PH values. All houses in the vicinity were visited to inform their occupants of the work and the probable owners were also kept informed.

11. The dam is in an extremely isolated location some 300 metres above and 1.2 kilometres away from the nearest point accessible to vehicles (on the valley floor) from where the ascent is across open hillside without even a footpath to the site. We were extremely fortunate to obtain the assistance of the Army on exercise and National Park Wardens to carry the required equipment up to the site, even so there were only light hand tools and a small portable hydraulic generator (which was to prove invaluable). The dam is a gravity structure comprised of masonry bedded in mortar and has three distinct sections, the highest section extends about 30 metres from the left abutment and spans the deepest part of the valley some 6 metres below crest level and also has the drawoff pipes. The middle section 20 metres long, at an angle to the first contained a lowered overflow section some 3 metres above the bedrock whilst the final section, roughly parallel to the first is 20 metres long and generally only 2 metres high. The intended method of breaching was to open a small slot in the third section and progressively deepen it to 1.5 metres a similar slot would then be taken in the centre section down to 3 metres with a final opening in the highest section if required to achieve the reduced volume.

12. The work was begun in August and undertaken by a gang of four men from the Council's Direct Labour Organisation, in the event it called for much more strenuous efforts than perhaps was anticipated. On the reservoir face a concrete skin with minor mesh reinforcement presented no particular problems but the stones in the front metre were a resistant igneous rock

which was particularly difficult to remove. Behind this first metre the rest of the structure comprised a local shale which could be cut through with the help of hydraulic breakers. Forming a slot by removing the stones involved all four men and the compressor working all day to produce a slot 0.5 by 0.5 metres. It became progressively harder as the working space reduced in the slot with depth and a constant flow of water through the opening after the first day made work difficult. Additionally weather conditions were unpredictable and strong winds meant the men had to work with safety lines and lifejackets in case they were blown off the structure. A control centre manned by Council staff was maintained in the village during this period with radio contact with an engineer overseeing work on the dam and monitoring as agreed with the Water Authority was undertaken by their staff. This phase progressed close to schedule despite the difficulties the men encountered and at the completion of two weeks work a satisfactory level for the reservoir had been achieved. The volume was checked by a land survey using electronic survey equipment whilst the depth of water was checked by soundings. A finished level with the centre section of dam removed to bedrock was found to reduce the volume below 25000 cubic metres, a situation that made producing a satisfactory structure and spillway an easier task.

13. The work received a lot of publicity locally and as a result further information as to the history of the structure was drawn to our attention by members of the public. The dam had been renovated in 1949 when the landowner was Clough-Ellis of Portmeirion. Several old press cuttings had been kept because the works had been billed as the first commercial demonstration of the use of a helicopter to lift materials (during which one of them had crashed at the site). They also revealed that the structure had been gunited/grouted in some form accounting for its resilience during discontinuance. This type of information is likely to be lacking for many of the old reservoirs around the country and although it is outwith the strict requirements of the Act a desk study of local records, archives etc such as would be undertaken prior to a site investigation may well prove fruitful in bringing to light significant information relevant to some of the more obscure reservoirs.

14. Further work was required to widen the narrow slot in the centre to form a gap sufficient to satisfy the Engineer it was no longer capable of impounding more 25000 cubic metres. Additionally pointing of loose stones on the remaining structure was required. We were again fortunate to receive assistance from the Army with transport of materials to the site, this time utilising helicopters, as a training exercise. We also employed a team of divers to investigate the now defunct drawoff pipes. Large stones had to be removed from the vicinity and the three pipes examined, two had gate valves which were badly corroded whilst the third pipe was blanked off in the reservoir. All the pipes were blanked off in the reservoir by the divers and subsequently plugged with cement grout.

15. On completion of all these measures Mr Arah issued a discontinuance certificate (22nd October 1986) and the reservoir was removed from the register.The costs incurred by the Council amounted to some £3600 for the initial and monthly inspections and £15000 in connection with the discontinuance, and we are currently seeking to recover some of these costs from the owners. Subsequent to these events we assessed what was required to meet our obligations under section 16 of the Act and decided that

specific procedures should be laid down and I would like to deal with these next.

ESTABLISHMENT OF EMERGENCY PROCEDURES

16. One of the features that became apparent at an early stage of the incident at Cwm-y-Foel was that contact with someone with previous knowledge of the reservoir condition together with an idea of the likely effects of failure would have been invaluable in gauging the required response. Therefore all Supervising Engineers of reservoirs for which the county is enforcement authority have been approached to provide an emergency 24 hour contact to be incorporated into our emergency procedure. Additionally the risk category of each reservoir as considered by the Inspecting Engineer has been noted. This information is provided to all persons on the County Surveyor's staff involved as possible contacts for reservoir emergencies. We have also ensured that the North Wales Police Operations Room is aware of our role and will initially refer all possible reservoir emergencies to the Authority and have accordingly mapped out the following strategy to dovetail into the pre-existing County emergency plan.

1) N. W. Police contact the County Council emergency contact using the normal emergency procedures with as much detail as possible. The contact will then contact nominated staff on the County Surveyor's staff who will be supplied with a list of registered reservoirs and a supplementary list of Welsh Water Authority reservoirs. The officer contacted will consult the lists and act in accordance with 2) 3)or 4)

2) a) For reservoirs not included on either list the police and Emergency Planning Officer will be informed of the fact and that the reservoir is not therefore the Council's responsibility.

b) For a non-registered W.W.A. reservoir the police and E.P.O. will be informed the reservoir is not registered but believed to be owned or operated by W.W.A and the police may contact them.

In either case the officer may offer the services of the County Surveyor's staff for further assistance (road closures etc) through the normal 24 hour area system.

3) For reservoirs of category D on the register no further action is required until the following day. The police and E.P.O. will be informed of its status. The officer will have responsibility for ensuring the matter is fully followed up the next day. If the police believe an additional hazard exists they may recontact the officer who will then treat the reservoir as a higher category.

4) For all other reservoirs the officer will contact one other of the nominated officers and they will proceed to the Council's headquarters where the relevant documents for reservoirs are held. In the case of category A reservoirs the E.P.O. will also attend to assist. They will attempt to contact the Supervising Engineer and Undertaker. One officer will proceed to site and arrange to meet the police, National Park Wardens and establish mobile communication as necessary. In the case of W.W.A. or C.E.G.B. reservoirs it may not be necessary to attend site if early contact can be made with a responsible officer.

5) If a threat to persons or property appears to exist the Supervising Engineer will be asked for his proposals to deal with the situation. If the Supervising Engineer or Undertaker cannot be contacted or appear not to be acting sufficiently quickly or competently the Council will contact an independent member of the All Reservoirs Panel and if appropriate appoint him as an Inspecting Engineer under

section 16.

6)The E.P.O. will remain responsible throughout any emergency arising out of the above for liason with press and public. The County Surveyor's staff will remain responsible for conduct of operations at the site until either the Supervising Engineer confirms his intentions or an Inspecting Engineer is appointed. Any request for assistance from the Supervising Engineer/Undertaker will be given on the grounds that costs will be recorded and may be recovered later. Following appointment of an Inspecting Engineer by the Council the County Surveyor's staff will be responsible for ensuring his requirements are carried out as promptly and efficiently as possible. On completion of the alert a full report will be made of the nature of the incident and the action taken.

17. We obviously hope that we will not have occassion to test the effectiveness of these arrangements which have taken full advantage of the lessons that were learnt during our connection with Llyn Cwm-y-Foel. The clearest impression we received during that incident was the need for early information, a good organisation with knowledge of where assistance may be available and ,in a mountainous region with poor access, excellent communications. This paper is presented by the kind permission of Mr H. E. Davies, County Surveyor and Mr H. Ellis Hughes, County Secretary.

ENFORCEMENT OF THE RESERVOIRS ACT 1975 - EXPERIENCE IN LANCASHIRE

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SYNOPSIS

In common with the other non-metropolitan counties in England and Wales, Lancashire County Council acquired new duties and powers as a local authority and an enforcement authority under the Reservoirs Act 1975 which became operative in stages between 1983 and 1986. The paper describes the investigations carried out to establish the likely scale of the County Council's involvement, the organisation set up to deal with obligations under the Act and some of the problems, technical, legal and financial, which have become apparent during the first two years since the Act came into full operation in the County area.

INTRODUCTION

1. The Reservoirs Act was placed on the Statute Book in 1975, but then remained inoperative for eight years awaiting the making of secondary legislation. The first step towards implementation of the Act came in November 1983, with the making of a Statutory Order enabling the Secretary of State for the Environment to make further Orders and Regulations and authorising the setting up of panels of qualified civil engineers for the purposes of the Act.

2. Subsequently, the remaining sections of the Act were brought into operation within the shire counties in two phases. Registration of all existing large raised reservoirs was required during a nine month period commencing on 1 April 1985. On that date local authorities as defined by the Act, were given power of entry onto land and power to take emergency action under Section 16.

3. All the remaining powers and duties, including enforcement powers, came into operation in the shire counties in April 1986. Simultaneously, registration commenced in the areas of the former metropolitan counties, where the Borough Councils now became local authorities under the Act. Full implementation in the metropolitan areas followed in April 1987.

Powers and Duties of Local Authorities

4. Prior to the implementation of the 1985 Act, County and District Councils had permissive power, under the Reservoirs (Safety Provisions) Act 1930, to call for information from undertakers. In cases of default they could seek enforcement of the requirements of that Act through the Crown Court (originally the Quarter Sessions). They had no obligation to take such action and their powers were, in fact, no greater than those of any resident or property owner whose interests were likely to be at risk from an escape of water. Very few enforcement actions appear to have been taken under the 1930 Act and these can hardly have been encouraged by the scale of sanctions available to the Courts.

5. For those bodies defined as local authorities under the 1975 Act, the major change following its implementation lay in the specific duty which they now had to secure that all undertakers (other than themselves) should observe and comply with all the Act's requirements.

6. It is not proposed in this paper to rehearse in detail all the various powers and duties arising under the Act and its dependent Orders and Regulations. Suffice it to say that, in any particular set of circumstances, careful study is often needed to establish the procedures required of the undertaker, the inspecting engineer, the construction or supervising engineer, the enforcement authority and any other local authority involved.

County Council involvement

7. It is not readily apparent why the County Councils should have been chosen to act as local and enforcement authorities under the Act. Historically they have had no water supply functions and they are unlikely to have any staff with directly relevant engineering training or experience. However, it appears to have been conceived that the local authority's task would be purely administrative. This view was stated in a consultation paper issued by the Department of the Environment in 1982 (1) and it appears to be implicit in the final section of the DoE Circular (2) explaining the workings of the Act. The latter, not unusually for government documents placing additional duties on local authorities, states that no additional staff or resources will be necessary and that the cost will be insignificant. No supporting data is given for this convenient conclusion.

8. In fact, experience in Lancashire has shown that a certain degree of civil engineering input is essential, and other professional services are also involved. The enforcement duties under the Act do not fit conveniently within the traditional departmental organisation of a County Council and yet, even in those counties with the largest number of reservoirs, these duties are not of sufficient extent to justify the establishment of a special unit. In consequence, in individual authorities, the reservoirs function has been undertaken by a variety of committees and chief officers. This appears to have led to some difficulty for the local authority and chief officer organisations in responding to government consultations and initiatives with respect to the Act. It has largely been left to BNCOLD and the Institution of Civil Engineers to provide a forum in which local authority representatives may discuss the working of the Act with undertakers and panel engineers.

Preliminary Studies in Lancashire

9. County Council involvement with reservoirs began in Lancashire as far back as 1970. In that year the County Council (for 'old' Lancashire) agreed to undertake, on behalf of the District Councils, a survey of information available on reservoirs coming under the provisions of the 1930 Act. This was done solely by correspondence with the District Councils and known undertakers. A schedule was prepared giving the names of reservoirs, the names and addresses of undertakers and any available information on inspections and certificates. Unfortunately, in the light of later developments, map references were not included, and the schedule was not subsequently kept up to date.

10. In March 1986, following the passing of the 1975 Act, Lancashire County Council delegated its new responsibilities to the Land and Buildings Sub-Committee. No further action was necessary at that time as the Act remained dormant. In February 1982 the Department of Environment wrote to District and Borough Councils reminding them of the provisions of the 1930 Act and stating that the Secretary of State was satisfied that most problems of reservoir safety could be dealt with by its proper application. These Councils were asked to check on reservoirs in their areas to ensure that the 1930 Act was being complied with and that they were adequately using their powers. They were also asked to complete a questionnaire giving details of all relevant reservoirs. The letter was copied to County Councils, but no action was required of them. According to Goode (3), response to the questionnaire was poor. It was intended to issue a follow-up questionnaire in 1983, but this was not done, presumably because it had been overtaken by preparation of the Consultation Paper (1) on the 1975 Act which was issued in October 1983.

11. Following receipt of this Consultation Paper, the County Council initiated a study to establish the likely number of reservoirs to be registered and the organisation necessary to carry out registration and enforcement duties. A desk study of the County area from the 1/50,000 scale O.S. maps revealed 600 'areas of blue' which might be reservoirs. This was followed by a more detailed study on the 1/250,000 O.S. which, combined with local knowledge, reduced the number of possible large raised reservoirs to just under 200.

12. An analysis of the tasks involved in ensuring that all large raised reservoirs were identified and registered suggested strongly that the whole operation should be in the charge of an experienced civil engineer who should, in default of experience of earth dams, have a soil mechanics background. This was because, in cases where an undertaker did not or could not supply the basic information for registration, an early inspection would be necessary. The object was threefold; firstly to establish whether there was a raised reservoir, secondly to make a first estimate of its capacity and thirdly to check for any obvious signs of danger, so that specialist advice could be sought immediately. In connection with the last point, it was known that almost all the large raised reservoirs in the area were retained by earth dams or embankments, hence the requirement for soil mechanics experience.

13. It was therefore decided that the reservoirs function should be centred in the County Surveyor's Department, which would supply the engineering and administrative staff. Associated legal, planning, land survey and referencing work would require input from the Chief Executive/Clerk, Director of Property Services and County Planning Officer, whilst the County Treasurer would have to deal with any question of finance. Initially the work was to be carried out wihtin existing resources but, from the outset, it was considered unlikely that this state of affairs could continue indefinitely and that eventually some formal allocation of staff resources would be necessary.

14. In March 1984 action was taken to draw up a Provisional List of known or suspected large raised reservoirs. The District Councils were asked to make available any information they held, the North West Water Authority were asked to identify their reservoirs and all previously listed undertakers were asked to confirm their interest or give information on any change of ownership. A press release was also issued asking private reservoir owners to contact the County Council, but with no response.

The 1970 schedule was found to be of little use because the administrative area of the County had been changed so considerably at local government re-organisation in 1974. Many of the reservoirs listed were found to be now outside Lancashire, whereas the areas of the former County Borough were not covered. However, information on those reservoirs formerly in the West Riding of Yorkshire had been handed over. The amount of information held by the fourteen District Councils varied widely. One or two had already carried out a full survey of their area and were able to give information, not only on reservoirs affected by the 1930 Act, but also others which were outside the Act. Other Districts were able to supply incomplete lists and some had no information. A review of the County Council's own land holdings showed that these did not include any registrable reservoir.

15. In setting up a referencing system for all reservoirs on the Provisional List it was found that the whole area of the County lay conveniently within a single 100 km square (SD) of the National Grid. It was therefore possible to allocate each reservoir a four figure reference number representing the lowest numbered 1 km grid square within which the reservoir lay. (This is normally the square containing the SW corner of the reservoir). In a few cases, where two smaller reservoirs would otherwise have the same number, a further two figures were added to give the 100m reference within the 1 km square and thus distinguish the two. This reference system is now used for the Statutory Register and all files, reports and records.

Registration

16. The value of the preliminary work became apparent when the Reservoirs Act 1975 (Commencement No 2) Order was made in February 1985, only six weeks before it came into force. Local authorities affected were then required to take steps, within three months of the commencement date, to inform undertakers of their duty to register. In Lancashire, this was done by press advertisement, by the issue of a further press release and by letter to all the undertakers previously identified.

17. Since the North West Water Authority had by far the greatest number of registrable reservoirs in the County, consultations were held on the form in which their registration information was to be submitted. The objective was to facilitate the transfer of information from the NWWA computer printout to the computer input for the Statutory Register to be set up on one of the DRS terminals in the County Surveyor's Department. In the event, due to delays in the setting up of the County Council database, for an interim period the Register consisted of the NWWA printouts with the addition of manual entries for other undetakers.

18. By the end of the initial registration period 99 reservoirs had been registered, of which 91 were owned by the Water Authorities and the British Waterways Board, whereas only eight were in local authority or private hands. A further seven reservoirs had been inspected and on survey evidence were found not to need registration. Thus there were a further 90 to 100 entries on the Provisional List which had not been registered, although it was recognised that a substantial proportion of these eventually be found not to come under the Act.

19. With the limited resources available it was not found possible to deal with all the outstanding cases in the three month period before the Act came fully into operation. However the first report to the Secretary of State, made in April 1987, showed that 118 reservoirs had then been registered and emergency action had been taken in one case.

20. New registrations can be expected to occur very seldom, and the total number of registrations is likely to decrease since discontinuance works are planned or in progress in a number of cases.

COMPLICATIONS ARISING DURING REGISTRATION

Cross-Boundary Reservoirs

21. In spite of the efforts of successive Boundary Commissions, most of our present local government boundaries are several hundred years old, although the administrative units they separate may have changed several times. These old boundaries frequently followed the course of a stream. When the valley of that stream has been subsequently dammed, the boundary will be found to bisect the resulting reservoir which, if of sufficient capacity, must be registered with both local authorities. They must then agree which of them will assume the enforcement powers. Nine such cases occur in Lancashire. Eight of these lie on the boundary with a Metropolitan Borough and were affected by the interim arrangements whereby the County Council was required to act as enforcement authority until the Act was fully implemented in the metropolitan areas. In one case this ruling applied even though only a very small area of the reservoir, remote from the dam, lay within the County. Agreements have now been reached in all nine cases, mainly by applying the principle that enforcement should lie with the authority whose area would suffer most in the event of a dam failure.

Definition of a Large Raised Reservoir

22. Section 1 of the 1975 Act defines a "large raised reservoir" as one designed to hold, or capable of holding, more than 25000 cubic metres of water above the natural level of any part of the land adjoining the reservoir. Thus a distinction is drawn between the natural ground surrounding the reservoir and the artificial dam or structure forming it. In most cases this definition works very well, although there could be argument over the meaning of "adjoining". However, much of the surface of these islands has been disturbed or altered by man and cannot be called natural. In consequence, any water filled depression in an area of made or disturbed ground is, if of sufficient size, subject to the Act. This will include large lakes or lagoons in, for instance, a restored open-cast mining site or an area of land reclaimed by tipping. In such cases there is often no discernable retaining structure and negligible risk of an escape of water.

23. There must also be some doubt about the applicability of the Act to a marine lake, separated from the sea or estuary by an embankment, where the water level is maintained at a level within the tidal range and the adjoining natural ground is below high water mark.

24. It appears to be firmly established that an embanked flood control reservoir, even though only occasionally used, is subject to the Act.

Marginal Cases

25. It is difficult to judge the capacity of a reservoir by visual inspection or even by measurement of the plan area and the height of dam. In cases where the undertaker claims that the reservoir does not require registration, the DoE Circular (2) suggests that a joint survey, or a survey by the local authority alone should settle the matter. This problem has not yet occurred in the author's experience, but it must be pointed out that the undertaker may not agree to a joint survey and the Courts may not accept the authority's survey. It would have been better if the legislation had required that, in such cases, a survey by an independent and properly qualified third party should be commissioned.

Divided ownership of a reservoir

27. In the field of public or industrial water supplies, the operator or user of any reservoir will normally control the whole of the reservoir and its associated works. This is not invariably the case with ornamental waters or other reservoirs not currently used as a source of supply. It is possible for ownership boundaries to have been drawn up without consideration of the need for maintenance works and possibly even without knowing that the water was an artificial reservoir. In one such case in Lancashire the ownership of the water area is divided between two parties, whereas the earth dam, except for the submerged part of the upstream face, is in a third ownership. In this case all three parties have been required to register as undertakers and, upon legal advice, are held to be jointly and severally responsible under the Act.

Content of the Statutory Register

28. Under the Registers, Reports and Records Regulations 1985 the Register maintained by each local authority is required to contain a summary of all certificates or reports received under the 1930 and 1975 Acts, including four specified items. It is unclear what, if any, additional information is intended to be included. To precis every engineer's report would be a considerable task and would add greatly to the size of the Register. On the other hand it is doubtful whether any part of an engineer's report other than the Register entry should be open to public inspection. At present, the Register for Lancashire contains only the details specifically listed in the Regulations. So far, there have been no requests for further information.

29. The section of the Register entry relating to the physical details of each reservoir is still incomplete in many cases, since this information was not required to be notified at registration. Complete records are not likely to be available until after 1996, when all reservoirs have been inspected under the 1975 Act and reports received.

ENFORCEMENT

30. In the case of the great majority of large raised reservoirs the undertaker is either a public body or a large industrial concern. There should be little doubt of the intention and ability of these undertakers to comply with the Act although, due to Government restraint on capital expenditure, the public bodies may have difficulty in programming major works. It is of considerable assistance to the enforcing authority if undertakers will keep them informed of progress towards the completion of works recommended in the interests of safety.

Otherwise, to take for example the normal inspection procedure, the authority has only to be formally notified of three events: appointment of an inspecting engineer, issue of his report and/or certificate and, subsequently, the issue of a certificate of satisfactory completion. The intervals between these events may be considerable. The authority, if not kept informed, will have to enquire whether an inspection has occurred and that measures are being taken to carry out any recommended works.

31. Where an enforcement authority considers that an undertaker is not carrying out works recommended in the interest of safety, they are empowered to serve notice requiring the undertaker to do so without delay. Before doing so, however, they must consult an appropriate panel engineer as to the time to be allowed for completion of the works. This presents a certain difficulty, since such works are frequently recommended to be carried out "as soon as possible." Few panel engineers will then be willing to go on record as stating that the works need not be completed for, say, twelve months. In such cases the authority may have to fall back upon the judgement of their own officers.

32. The experience in Lancashire has been that by far the greatest part of the enforcement workload arises from a comparatively small number of reservoirs in the hands of private individuals, members clubs, and property companies. A few of these reservoirs were created as ornamental lakes, but the remainder are relics of the industrial revolution. Typically, they at one time served a textile mill, now defunct, whose premises have been re-developed or split up into small units. The ownership of the reservoir may remain with a developer or have been sold off to an angling club.

33. Most of these reservoirs have had no inspection or maintenance since their original use ceased. When registration and inspection have been effected it is likely that the necessary work will be completely beyond the resources of the undertaker. These cases present the authority with a considerable problem, since the pursuit of formal enforcement procedures is unlikely to produce any effect apart from the winding-up of the club or company involved. No specific government grant is available in order to make a reservoir safe although, means may be found to include the reservoir works in some wider programme of environmental improvement or industrial redevelopment which can attract grant aid. If the authority commissions the necessary work, the cost will rank as prescribed expenditure under the Local Government, Planning and Land Act 1980 and will be chargeable against the authority's capital allocation. Under such circumstances the authority could only consider the minimum cost scheme which would probably lead to the discontinuance of the reservoir and the loss of any environmental value it may have possessed.

34. At the time of writing, the Lancashire County Council has so far not had to commission any works directly but, in several cases, negotiations with undertakers and possible funding agencies are still in progress. Should the authority find it necessary to fund any substantial part of the cost, it could only be at the expense of some other element of their capital programme.

CONCLUSION

35. The initial tasks of locating large raised reservoirs, identifying undertakers and securing registration and inspection is now virtually completed. Registrations of new reservoirs are likely to be few but discontinuance of registered reservoirs will continue as undertakers take action to minimise future maintenance costs.

There will remain a continuing workload in monitoring the cycle of inspections and remedial works. The effect of privatisation of the water industry upon the local authorities has yet to emerge.

36. The outstanding major problem resides in ensuring the safety of those reservoirs of no commercial value and now in private hands. This problem seems unlikely to be solved until some adequate form of central funding is made available either to undertakers or to the enforcement authorities.

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DISCONTINUANCE: THE ALTERNATIVE TO RENOVATION

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SYNOPSIS

Faced with unpalatable costs of repairs to old dams to comply with the recommendations of inspecting engineers, reservoir owners have increasingly turned to discontinuance.

Discontinuance must render the dam incapable of impounding more than 25 000 cubic metres of water above the lowest level of the surrounding land. It is often cheaper to dispense with the impoundment completely, and the methods are discussed.

The paper describes the discontinuance of five reservoirs in the Pennines, based on substantial breaching or total removal of the dams. There have been numerous problems: planning approvals, silt control, conservation interests, landscaping, affected landowners, downstream flood aggravation and the stability of the re-established water course through the reservoir basin and dam site. In each of the schemes different problems have predominated.

INTRODUCTION

1. There are over 2000 reservoirs in Great Britain. Most were built in the last century and the vast majority are formed by earth embankments. It is not surprising that some of these reservoirs have become unsafe and that, where they are also redundant, the owners should seek relief from future risks and maintenance costs.

2. It took several major dam failures before reservoir safety became the subject of legislation in the form of the Reservoirs (Safety Provisions) Act of 1930, which laid down requirements for the design and construction of reservoirs under the supervision of authorised civil engineers and for periodic inspections at intervals not exceeding 10 years.

3. Although it was not brought into effect until recently, the Reservoirs Act 1975 was passed to overcome the shortcomings of the 1930 Act in three main respects: in placing reservoirs under continuous supervision, giving local authorities the duty of enforcement, and in the keeping of registers of reservoirs. The 1975 Act applies to those reservoirs capable of holding at least 25 000 m^3 of water above the level of the adjoining land. The process by which a reservoir can be taken out of the ambit of the Act, by rendering it incapable of holding that volume, is called "discontinuance". The Act lays down procedures to be followed; and the works required to achieve discontinuance have to be carried out in a manner approved and supervised by an engineer qualified under the Act.

THE CASE FOR DISCONTINUANCE

4. The decision to discontinue a reservoir is commonly influenced by the costs of the repairs needed to restore the structure to an acceptable standard of safety. If the present value of repairs and future operating costs exceeds that of discontinuance by more than the present value of future benefits from water abstraction, angling and so on, then the economics favour discontinuance.

5. The principal defects requiring repairs and improvements can be summarised as follows:
- distress in the dam embankment, including excessive seepage or settlement, or instability of the dam shoulders;
- collapsing culverts or corroding pipes buried in the embankment;
- insufficient means of emptying the reservoir or of lowering the water level in an emergency; and
- inadequate facilities for the safe discharge of floodwater from the reservoir.

6. Excessive seepage through a dam embankment or high porewater pressures could result in a loss of material from the dam, leading to a progressive enlargement of the seepage passage, instability of the downstream shoulder, and eventually a rapid washout of the embankment. Instability of the dam shoulders carries with it the risk of a loss of support of the central impermeable core (if the dam has one), leading to fracture and seepage, or overall instability of the embankment as a water-retaining structure.

7. In many old dams, the drawoff pipework is buried directly in the dam fill, or at best in a shallow trench below the dam. Such pipes are vulnerable to tension if the base of the dam spreads as the embankment consolidates, or the valves may have seized up through lack of use, or the inlet blocked with silt. Although it may be possible to perform a limited internal examination by CCTV, the external condition of the buried pipework cannot be certain. Moreover, it might take weeks to significantly lower the water level in the reservoir, should it become necessary in an emergency.

8. Inadequate overflow weirs and spillways are frequent problems with old dams and are often aggravated by settlement of the embankment, reducing the freeboard between the overflow weir level and the dam crest. The science of hydrology has advanced considerably as additional data on rainfall and floods have been collected, and the consensus regarding the acceptable risk of a dam being overwhelmed by an incoming flood appears to have become more conservative. A several-fold increase in discharge capacity of the existing overflow weir and spillway channel may be required.

METHODS OF DISCONTINUANCE

9. There are two principal options for rendering the reservoir incapable of holding more than 25 000 m^3 of water:
- retaining a lake small enough to be outside the ambit of the Act; and
- dispensing with the reservoir completely.

10. Retaining a small lake has the advantage that it can act as a silt trap, so that less of the sediment which has accumulated since the reservoir was formed will be liable to be washed downstream, or require removal or treatment. It will also probably involve less of the dam having to be removed and may be preferred on amenity grounds. On the other hand, the remaining lake is likely to require a new overflow and spillway, as the original ones will have been left high-and-dry by the lowering of the water level. Although liability under the Act will have been removed, the owner will still be faced with a degree of maintenance and will remain liable in tort for injury caused by any escape of water.

11. Dispensing with the reservoir will often restore the watercourse through the reservoir basin to its original bed, so that major channel works should not be required. There will also be no risk of an escape of impounded water. On the other hand there can be problems with reservoir sediments.

12. If the decision is made to dispense with the impoundment completely, there are still choices to be made regarding how much of the dam to remove. If the dam is situated in a steep-sided valley, it will have to be removed in its entirety, or nearly so. With a longer dam on gentle valley slopes, it would be possible to breach the dam effectively by cutting a vee at or near its highest section, probably on the line of the original watercourse, leaving the flanking embankments, perhaps containing the majority of the dam volume, in place. Clearly aesthetics and landscape considerations can have a major influence in this decision.

13. The feasibility of discontinuing a reservoir by forming a culvert or tunnel through the dam may also be considered. However, this method is generally more costly for the smaller dams likely to be discontinued, and has several significant practical and safety drawbacks:
- it is necessary to achieve an effective seal around the culvert where it crosses the core of the embankment, to prevent erosion of the material surrounding the culvert if the inlet becomes surcharged during a flood;
- there will be a risk of the inlet becoming blocked by debris or by landslips on the face of the dam or the sides of the valley just upstream;
- maintenance of the embankment and culvert will be required in perpetuity; and
- there may be difficulties in providing for the safe passage of floodwater during the relatively lengthy period required to carry out the works.

PRINCIPLES OF DAM BREACHING OR REMOVAL

14. Clearly the process of dam breaching or removal has to be accomplished in a safe manner, without creating risks greater than those already caused by the existence of the deficient reservoir. The principal problem is that runoff from the catchment will continue throughout the operation and that the risk of floods will continue too. The situation may be relieved if the reservoir has a bywash channel, but most candidates for discontinuance have not. The reservoir may have pipework capable of emptying it, but certainly not of preventing it from impounding in the event of a flood.

15. As breaching is undertaken the effective height of the dam will progressively reduce. At a very early stage, the existing overflow will become unusable, leaving the low level pipework as the only means of discharging any inflow. This is potentially a very vulnerable condition, as floodwater overflowing the partially removed earth embankment would lead to rapid erosion and the risk of a washout.

16. In designing the breaching operation, it is vital to carry out a detailed hydrological appraisal of the reservoir and catchment, to evaluate the probability of the remaining capacity of the reservoir basin being refilled and the embankment being overwhelmed during each stage of the operation. This will take account of the discharge which can be passed through the outlet pipework, any additional discharge facilities proposed, the capacity and behaviour of the bywash (if any), and the anticipated duration of each stage of the operation. In the light of these studies it may be necessary to alter the design of the operation, make provision for pumps and siphons, or impose severe progress requirements, in order to reduce to an acceptable level the risk of the basin refilling with water.

17. Prior to commencing the breaching or removal of the dam, it is clearly desirable that the reservoir be emptied. As the operation proceeds, the diminution of the reservoir capacity means that the risks of the working area being overwhelmed will increase, but that the volume of water which might escape will reduce. As the height of the dam is reduced, the potential velocity and erosive power of any overflowing water will also be reduced. Ultimately the decision as to the acceptable level of risk and the assessment of the likely consequences of overtopping during each stage of the operation is a matter of engineering judgement, which has to be shouldered by the civil engineer responsible under the Act for the design and supervision of the operation.

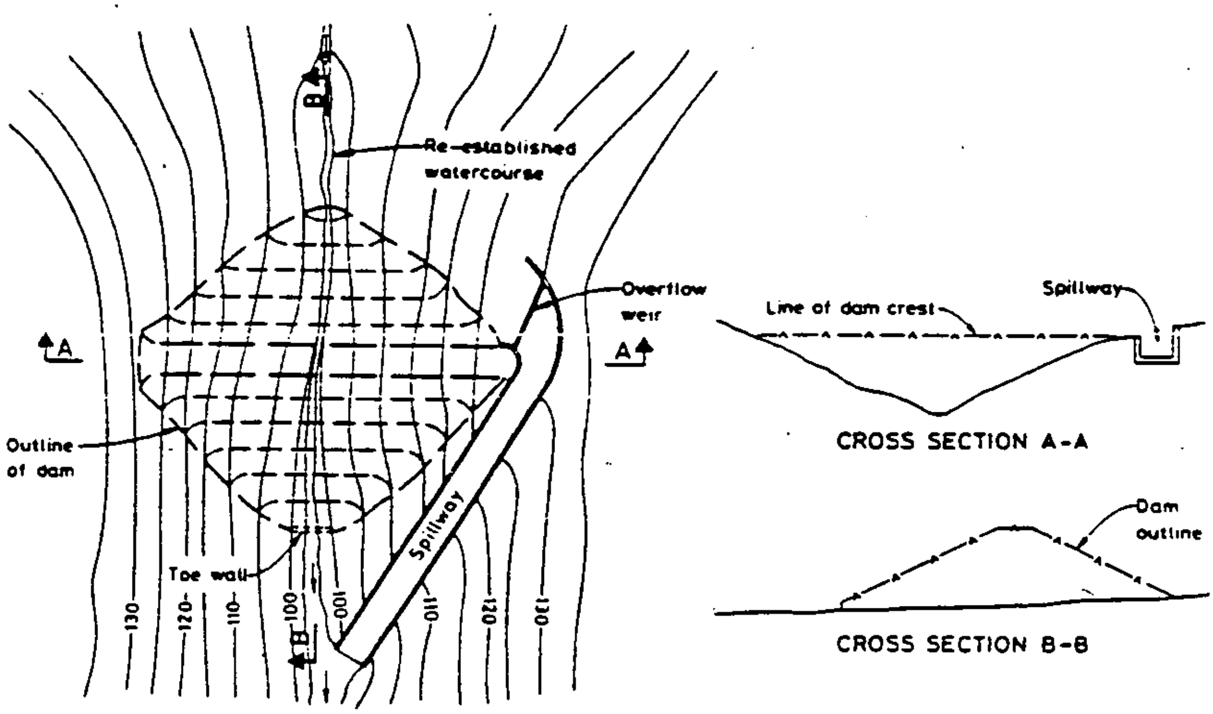


Figure 1: Removal of imaginary dam

18. Figure 1 shows an imaginary plan and sections for the removal of a dam in a steep-sided valley. The volume of earthmoving is a major factor in determining what plant is practicable, the mechanics of the operation and how long must be allowed for it to be undertaken. In the case of a breach only 4m high, the volume of earthmoving involved will probably be no more than $1000m^3$, based on a typical dam geometry, breach slopes of 1 in 4 and a base width of about 5m. This volume can be excavated in one or two days, so the recommended method is to empty the reservoir and start excavation early in the morning when there is a good weather forecast, placing minimal or no restrictions on the sequence of excavation.

19. For higher dams, the operation will last longer, and a more rigorous approach to the scheduling of the excavation is required. The procedure which we have adopted is to excavate the breach in a series of essentially horizontal layers, generally with a slight fall back towards the reservoir basin. By maintaining a horizontal crest, the area over which floods would flow is maximised so that, in the event of the basin refilling, the depth and velocity of the flow are minimised.

20. In the early stages of excavation, the working area will be long and narrow along the centre line of the dam crest, therefore awkward to work in but with easy access from the abutments. As the excavation proceeds the width quickly increases until the working area is roughly square at mid-height. Eventually, at the later stages, the working area is again long and narrow but with its axis running with the stream bed.

21. Although unnecessary restrictions on the contractor in the choice of plant and method of carrying out the excavation should be avoided, the designer needs to be well aware of the practicability of the breaching sequence and what rate of progress can reasonably be achieved. He can then use this information in estimating the risks of refilling, as described above, and determining what additional flood discharge provisions should be made.

ENVIRONMENTAL AND OTHER CONSIDERATIONS

22. A reservoir which has been in existence for more than a lifetime may form a familiar and much-loved feature of the landscape, or a forgotten, inaccessible and overgrown relic. In any case, opposition to discontinuance can come from a variety of sources, many of whom would be unsympathetic towards the owner's economic reasons for discontinuance, or unaware of the dangers inherent in prolonging the reservoir's existence.

23. The main forum for formal public representations over the discontinuance plans is when the owner applies for planning permission under the Town and Country Planning Act 1971. Although demolition of a building does not generally require planning consent, the removal or breaching of a dam usually does. The planning committee and their officers are principally concerned over the physical appearance of the finished scheme, including the area affected by the disposal of the spoil from the breach, but may also require reassurance regarding the impact of the scheme on the downstream watercourse and that the interests of the general public and any affected landowners have been properly safeguarded. The unusual nature of the application can cause considerable delays in the planning process.

24. In some instances a public right-of-way exists along the crest of a dam. The formalities of temporary and permanent footpath diversions, extinguishment and creation orders can be frustrating and complex and appear to vary according to locality. There are usually footpath user groups to consult, as well as landowners and tenants affected by the diversions.

25. The most difficult problems to solve are often associated with the watercourse downstream of the dam. Prior to the formation of the reservoir this would have carried all the runoff from the catchment, but since then it will have benefited from a degree of flood control. This benefit could be quite substantial if the surface area of the reservoir is more than a few percent of the total catchment or if the reservoir has often been depleted at times of flood inflow. Significant streamside developments may have occurred since the construction of the dam, taking advantage of its flood control benefits. The removal of the dam introduces new threats to those properties, unless compensatory flood protection measures are taken.

26. The volume of sediment accumulated in an old reservoir can sometimes be in excess of 50% of its original water capacity, although this will depend greatly on the catchment characteristics and the operational practices of the reservoir's owner. Upon removal of the dam the watercourse across the reservoir basin will erode through the sediment down towards its original bed, greatly increasing the load of suspended solids in the stream below. In many cases it will therefore be desirable to take preventive measures in advance of breaching or removing the dam. These measures can include:

- emptying the reservoir well in advance of the breaching operation to encourage drying, hardening and natural seeding of the sediments; and
- where access is practicable, removing sediments from the bottom of the basin, particularly from the vicinity of the watercourses which will be re-established across the basin.

27. Further sediment control measures can be incorporated in the final design of the scheme, including:

- keeping any suitable toe wall which the dam may have, to form a small settling pond and limit the stream gradient and hence velocity of flow;
- opting for a scheme which retains a reservoir whose capacity is small enough to keep it out of the ambit of the Act;
- constructing shallow arresting weirs across the watercourses through the basin; and
- artificial seeding or afforestation of the reservoir basin.

28. Even with the benefit of several of these measures, some problems with sediment should be anticipated.

SOME RECENT SCHEMES

29. Particular features of some recent schemes are described below and summaries of the costs and other information are presented in Table 1, at the end of the paper.

The Horsforth reservoirs

30. The three Horsforth reservoirs - Upper, Middle and Lower - were on the outskirts of Leeds. They were constructed by the Horsforth Waterworks Company between 1866 and 1886, and the main defects identified in the 1980 statutory inspection report were the inadequate hydraulic capacities of their overflow weirs and spillway pipes. Feasibility studies indicated that the cost of new spillways, other repairs and future running costs, exceeded the likely cost of discontinuance, including establishing alternative supplies to their service area. The reservoirs were taken out of service in late 1984 and work on discontinuance plans started in early 1985.

31. Each of the dams had a relatively long crest in relation to its height, so to limit earthworks volumes the preferred scheme was to breach the highest section of each dam, rather than removing the dams in their entirety. Minimising the excavation volumes meant that the spoil could be disposed of on site within the reservoir basins, avoiding road haulage, but this proposal was met with some opposition from the planners, who felt that the remaining flanks of the embankments might form discordant features on the landscape. This opposition was overcome for the Upper reservoir as its site was likely to be acquired by Leeds City Council, incorporated into the surrounding golf course, and then remodelled during redevelopment of the course.

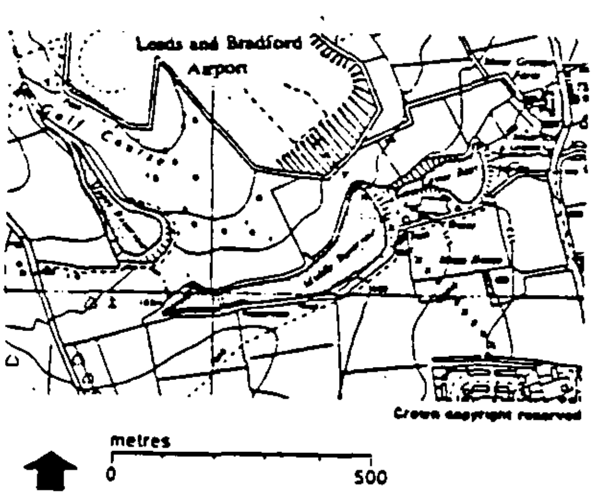


Figure 2: Location of Horsforth reservoirs

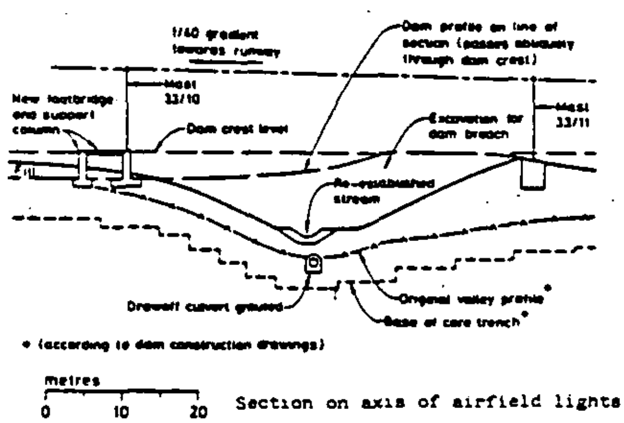


Figure 3: Section of Middle dam breach

32. There were stronger reasons for accepting a narrow breach and retaining the flanking embankments at the Middle reservoir, because a few years earlier the authorities for Leeds & Bradford Airport had positioned several substantial landing light masts on the dam (Figure 2). Consultations quickly revealed that the lights could not be interrupted for long enough to allow them to be dismantled, the dam to be removed and the lights re-erected on taller masts. Figure 3 shows the location of the breach in relation to the lighting masts. Interruptions to the lighting to allow cable diversions and minor modifications to the masts were confined to a few hours in daylight during the summer.

33. The Lower dam was shorter than the other two, with a steep abutment on the north side of the valley, so only the southern flanking embankment was to be left. However, it was later decided to remove most of this also, to improve the final appearance of the site and aid access into the basin.

34. A major concern in the planning of the scheme was that downstream flood risks should not be unduly aggravated and, to that end, substantial improvements were proposed to several sections of the downstream watercourse. In spite of these measures, there have been significant flood problems at a farmyard and stables built across the valley bottom about 500m downstream of the reservoirs, mainly resulting from blockage of culvert screens. Further improvements have been proposed, but are in abeyance pending agreement with the affected landowners.

35. The bulk of the discontinuance works were carried out between July 1986 and February 1987. The critical dam breaching operations were carried out by a variety of methods, using bulldozers (sometimes towing scraper boxes), backhoes and dump trucks, and the contractor achieved earthmoving rates of about 500 m^3/day at the Upper dam and 1000 m^3/day at the Middle and Lower dams. Each dam took about two weeks to breach effectively.

Greenfold reservoir

36. Greenfold reservoir was situated about 4km north of Rawtenstall, in Rossendale, Lancashire, and is thought to have been formed in about 1860. The dam was found to be unsafe when inspected in 1980, so the reservoir was taken out of service and kept empty pending repairs. The principal defects were the inadequate flood discharge performance and the risks associated with the single outlet pipe, valved only at the downstream end and buried directly within the embankment fill.

37. The decision to discontinue the reservoir was made in 1984 and the scheme chosen was removal of the entire dam, because of the relatively steep valley sides. The spoil disposal areas were within the reservoir basin just upstream of the dam and on fields up to about 300m distant.

38. Work on site commenced in August 1986 and was substantially completed in January 1987. The dam removal was carried out by tracked dozers and scraper boxes and it took about 5 weeks to remove 32 000 m^3, at a peak rate of around 2000 m^3/day.

39. A major problem at Greenfold has been the steepness and geology of the watercourse through the site of the dam. Rock exposures immediately downstream of the dam had led to the expectation that a durable original stream bed would be revealed by removal of the dam, but it turned out to be boulder clay. Provision had been made for placing stones recovered from the upstream face of the dam on the bed and banks of the watercourse, but the gradient proved to be too great over part of its length, and the stones were too small to withstand the resulting flow velocities. It is now planned to monitor the rate of erosion and review the progress towards a steady regime.

Ilton reservoir

40. Ilton dam was built in about 1890 to augment the water supply to a series of ornamental lakes in Swinton Park, near Ripon, North Yorkshire. An inspection report in 1980 pointed to inadequate freeboard between the overflow level and the dam crest, deficient hydraulic capacity of the spillway channel and to unsatisfactory arrangements for drawing water from the reservoir. The improvements recommended were found to be too costly in relation to the value of the supply, particularly at a time of depressed demand. The decision to discontinue the reservoir was made at the beginning of 1987 after consultations with Swinton Estate regarding their remaining interests in the reservoir.

41. The planning application was submitted at the end of April 1987 and approved by the county planning committee within five weeks, subject to conditions regarding a scheme of landscaping. The reservoir site is surrounded by a young conifer plantation and the county's proposal to introduce some permanent deciduous woodland into the reservoir basin was agreed to by the Water Authority and is to be implemented during 1988.

42. Because of the steep valley sides at the dam, the discontinuance scheme for Ilton reservoir was based on the removal of the entire dam, except for its toe wall. This involved about 10 000 m^3 of excavation, which was disposed of on one side of the valley within about 100m of the dam, much of it forming an extension to a natural ridge. Some felling of the conifers was needed to provide a sufficient area for spoil disposal.

43. The bulk of the discontinuance works were carried out between July and September 1987. The excavation of the dam was carried out by an 18-tonne backhoe with a bucket capacity of a little under a cubic metre, working with a team of two 15-tonne capacity dump trucks, and a small bulldozer on the spoil disposal area. The maximum rate of excavation was about 1000 m^3/day and the dam was breached in four weeks.

44. A significant problem at Ilton reservoir arose after removal of the dam. At the instigation of the YWA, an attempt was made to limit the amount of sediment carried downstream by installing revetment boarding to protect the unstable banks where the stream passed through the deposited silt in the reservoir basin. Unfortunately a particularly severe rainstorm occurred soon afterwards and the streamflow washed out large quantities of bank material from behind the boards. As a result of this experience, no further attempts are proposed at stabilising the deposits. These are already supporting substantial natural growths of vegetation which, if not further disturbed, should soon provide protection to the sediments.

ACKNOWLEDGEMENTS

45. The authors thank the Engineering Manager (East) of the North West Water Authority, the Area Manager (Harrogate & Dales) and the Development Manager, Central Division, Yorkshire Water Authority, for permission to include information on their recent discontinuance projects, and for the cooperation and advice of their staff during the formulation of the schemes. They also thank Binnie & Partners for permission to prepare this paper.

Table 1 Principal details of recent schemes

	Horsforth			Greenfold	Ilton
	Upper	Middle	Lower		
Catchment area (ha)	26	79	87	250	540
Reservoir area (ha)	1.1	2.4	0.9	5.7	2.5
Reservoir capacity (Ml)	39	127	36	292	90
Dam height (m)	11	15	12	20	13
Breach volume (m^3)	4800	8400	9000	32 000	10 000
Contract cost (£ x 1000)		318		127	62
Apportionment of works costs (%)					
Dam breach and spoil disposal		31		72	50
Access improvements		3		6	4
Minor structures		8		3	8
Other works at dam site		7		19	
Downstream watercourse improvements		37			
Airfield lighting modifications		4			
Treatment works demolition		10			
Sediment stabilisation					38
Unit cost of dam breach and spoil disposal (£/m^3)		4.44		2.86	3.10
Tender date		May 1986		June 1986	May 1987

DISCUSSION: TECHNICAL SESSION 1

ENFORCEMENT

Session Chairman: W J Carlyle
Partner, Binnie & Partners

Well, Gentlemen, Professor Montague and Mr Hawes. This is Session 1 on Enforcement. We have four papers from very authoritative authors. Unfortunately, Mr Maconochie is ill and I am grateful to Mr John Phillips who is going to stand in for him. I believe Mr Wight of Gwynedd County Council is unable to be here. I have a volunteer in Mr Arah who is well-known in Gwynedd and knows Mr Wight very well. Mr Morris is to present a paper on Enforcement in Lancashire and Mr Dunn and Mr Ackers on the De-Commissioning of Reservoirs.

J PHILLIPS (Department of Environment)

My introduction will be pretty brief.

As you know, Bill Maconchie is not well and having prepared the paper some little while ago, he fell ill between now and then and was not able to attend.

Perhaps I could just say briefly that, as you know, the Act was passed in 1975, but for various reasons it was not implemented until 1983 and then, because of the intricacies of the Act itself and another Act which was passed at the same time, it wasn't fully implemented until 1987. At the present moment, we have only had one report from the enforcement authorities which should give us a much clearer picture of how the Act is working. So far, as far as we know, it is proceeding well; we have heard of one or two proposed court cases to enforce the Act itself, but none that we know has actually come to court as yet.

To put it is perspective, I could only find two enforcement cases in the whole 45 years of the 1930 Act, although there were other cases, principally related to the ownership of a reservoir which, as you know, is quite a contentious issue in many places.

So, as I say, we look forward to the next series of reports. We don't anticipate any major problems in that respect.

W J CARLYLE (Session Chairman)

Thank you Mr Phillips. It was unfortunate that Mr Maconachie wasn't able to present his paper because there are significant differences in the style of the enforcement authorities in Scotland and his experience there would have been very valuable.

R M ARAH (Binnie & Partners)

Gentlemen. This is an unexpected pleasure, but as I was involved with the case of discontinuance in an emergency which is described in Mr Wight's paper, and as I pressed Gwynedd to write a paper and make a contribution to this conference, I feel I have no option but to attempt to field the questions and introduce the paper. I think I can say little on the matters of registration and the procedural matters. We have with us Mr Greatrex of Welsh Water Authority - he was the engineer from the Welsh Water Authority who was called out in the emergency and had a very exciting time in the darkness. I am sure that Mr Morris, who deals with a similar topic in his paper, will also help to cope with those matters.

The details of the discontinuance of Cwm-y-Foel make this a very helpful paper. All panel engineers have a duty to try to explain to their undertakers that these extraordinary emergencies, which never seem to happen to them, do in fact happen sooner or later. I hope that this paper will be followed by others in later BNCOLD conferences because I feel that, for instance, the Lluest-Wen papers did a great deal to help us to explain to undertakers that the time to plan for emergencies is long before they happen, when you really have to sit and quietly imagine what might happen.

When I last saw it some persons unknown had begun to close the Cwm-y-Foel notch which had been cut through the dam for discontinuance. It does bring up the question of whether, once a dam has been discontinued, the enforcement authority should go around looking to see that it remains discontinued. I suppose that there is a continuing duty on the enforcement authorities to go around and see that there are no new dams coming up, whether they are official or unofficial.

I think another point of interest is that the enforcement authority looked in the first instance to the water authority for advice. It is now their intention (in the procedures described in the paper) to go in the first place to the supervising engineers. Now I suppose that is extra-statutory, but I hope that no undertaker would ever refuse the services of his supervising engineer to an enforcement authority for comment in any circumstances. It would be interesting to know if this is the practice generally or whether people are sensitive to the crossed responsibilities of a supervising engineer talking direct to an enforcement authority.

W J CARLYLE (Session Chairman)

For those who may wish to make comments on Mr Wight's paper, there are a couple of quite interesting statements in it. One relates to the length of time for inspecting engineers to make inspections and reports, which is certainly an interesting topic, and whether they should in all circumstances stipulate a time during which matters recommended in the interests of safety should be carried out. Secondly he almost suggests that inspecting engineers have a duty to quote the risk category for each reservoir and that's a matter, I think, which we should consider. There is no particular statutory requirement to do so and, as you know, the risk category at the moment is only enshrined in the 'Engineering Guide.'

A A WOODHEAD (Sir Alexander Gibb & Partners)

I would like to pass some comments on Mr Wight's paper. I am one of the supervising engineers for the dams in his area and, in particular, for Eigiau dam which was the source of the Dolgarrog dam disaster and, to some extent, the reason for us all being there.

Most of you will not be aware, that despite having failed in 1925, it is still a reservoir. The dam remains and when it came up for its inspection under the 1975 Act recently, the inspecting engineer declined to give it a certificate and requested its final abandonment. Plans were duly drawn up and they were refused planning permission by one of Mr Wight's colleagues in a separate section of his authority. This has now been resolved and tenders have been invited. In the meantime, I felt it was my duty to advise the enforcement authority that the reservoir was being operated without a certificate. The enforcement authority's response to that was to require me to inspect it monthly. I think probably that's extra-statutory, that they cannot require that, but nevertheless it has been done.

I would commend to all budding engineers planning to build a concrete dam to make a pilgrimage to the site of it and see just how badly some ancestors actually built a dam.

T A JOHNSTON (Babtie Shaw & Morton)

Referring to Mr Wight's paper and the procedures that have been set up to deal with the situation when there may be 'a threat' he says: 'if a threat to persons or property appears to exist, the supervising engineer will be asked for his proposals', and it seems to me that this is perhaps an overlarge burden to place upon a supervising engineer. He has been appointed to a panel to watch over reservoirs and if he has considered that there is a problem, he should then advise the undertaker. It seems to me as though somehow the cart has come before the horse. If the police or the local authority has already decided that there is a threat to persons or property then the introduction of the supervising engineer here could be an unfortunate link in the chain which might delay emergency action.

A point which is perhaps more pertinent to Mr Arah and the advice which he had to give very quickly regarding the safe discontinuance of the reservoir, I wondered, why he tied it into 25,000 cubic metres, because it could well be that the danger was related not so much to the volume of water but to the height of the dam. I know that there are a number of people, owners, who quite wrongly think that provided they get down below 25,000 cubic metres, they have made a reservoir safe. That is quite often not the case at all, as we all know. So I think there is a danger of those who are not engineers being misled by the significance of the figure of 25,000 cubic metres.

Mr Morris in his paper refers to the Act being for the storage of water. In Lancashire there's a couple of reservoirs which don't store water, but they store dilute hydrochloric acid. Now, is hydrocloric acid water? I do know that the owner had the reservoirs inspected and they are on Mr Morris's register. I wonder whether this is a common feature in other authorities? Certainly I am not clear in my mind as to whether they should properly be covered by the Act.

I was very interested in the slides which Mr Ackers showed of demolition of dams. It confirmed my experience that the easy bit is getting rid of the dam, the difficult bit is what to do with the reservoir basin because once you get rid of the dam you are left with an unattractive silt-filled basin that can cost many times more expenditure to convert to an attractive amenity.

R M ARAH (Binnie & Partners)

On the question of the magic value of 25,000, I agree entirely. It so happened at Cwm-y-Foel that if we cut it down to bedrock near the abutment, the remaining structure was about 2 metres high and about 4 or 5 metres wide at the base, and it really appeared to present no risk at all. It was really coincidence but I do agree that, to discontinue a dam, you may very well have to take it down to where it started from.

On the question of whether the supervising engineer can be required to advise the enforcing authority, I do believe that any panel engineer, being given the slightest opportunity to give the situation a twist in the direction of safety, really ought to do it. Of course there may be questions of liability and treading on other people's toes but I think that would be well understood, particularly in emergency situations. I believe, in this particular case, there wasn't a supervising engineer and the reference to the water authority caused no delay at all: it probably even increased the sense of urgency.

On the question of acid in reservoirs, I remember Withens Clough where the reservoir was purporting to contain water for water supply and the pH was 3.2: it was virtually dilute sulphuric acid. There is a very fine line to draw between impure water and extremely weak chemicals, but again I would have thought the real question is - is it going to act as a reservoir? If so, I believe it should be dealt with under the Act.

DR D J COATS (Babtie Shaw & Morton)

May I just make general comments. The first is that there seems to be in a number of papers the inference that the enforcement authority has an engineering input. There was reference to wondering what category was set by the inspecting engineer and querying it. Also there's questions of the supervising engineer having an engineering input, being asked what should be done in the case of an emergency, or whatever. As I understand it, and please correct me if I am wrong, the enforcement authority has no responsibility for engineering input and the supervising engineer's responsibility for engineering input is for supervising; observing and advising and getting someone to do something who has got the authority to do it later on, by way of inspection.

In that connection, I am not all that clear why some people are inferring that you can do work without an inspection engineer's report. This may be something that we will discuss later on in the second session, because Mr Earp discusses this as well, where rather than have an inspecting engineer's report, you should just get on and do it, and the supervising engineer is asked to do something and maybe he goes to a panel man and they do it between them. There's no backup as I understand it for this in the Act. You can't touch a dam in a way that might affect safety unless you have, first of all, an inspecting engineer's report.

The second point I want to make is in connection with discontinuance and abandonment. I think it is agreed that discontinuance, it is out of the Act; abandonment, it is not out of the Act, even though in abandonment you have the reservoir level brought down below 25,000 cubic metres. I was wondering when Mr Arah was talking about his slot, whether this small slot ensures that the water level will never rise such that there is an impounded volume and that there may be a dangerous situation. As was pointed out in Mr Maconochie's paper, to have just a slot or a pipe at a level low enough to keep it below the Act is to outwit the intention of the Act although, maybe legally and pedantically, it may be so. Also I think that there was one case discussed by Mr Ackers about a culvert when he was saying one way of doing a discontinuance would be to put in a culvert. Now my understanding of that again is that the culvert first of all must be sufficient to take the full flood flows without any problem at all; but also it must be capable of not ever becoming blocked because, if it does become blocked then you have a situation when it is really abandonment, and if a reservoir is abandoned it still remains under the Act. This overcomes the problem which Mr Arah was mentioning where reservoirs were discontinued and therefore not then looked at all. If they are certified as abandoned, then they are looked after from there on because they continue to remain in the Act.

On Mr Maconchie's paper, we do have a problem in the Town and Country Planning Act. Dams are almost being treated as listed buildings. A building is listed, they cannot take it down, they cannot improve it, they cannot do anything with it. They are stuck with it and no one will pay for doing anything with it, other than themselves. That is the situation very often, except that in a dam it could be a danger to the public and I am quite sure that we must not say if the thing is dangerous that we have got to get Town and Country Planning permission for this and wait six months, or whatever, to get it. We have the authority, as I understand it, to go ahead and get emergency measures taken immediately, and that may mean breaching the dam.

Mr Maconochie's paper in paragraph 38 I couldn't understand. He seems to think that the dam and the reservoir are two different things, and that the reference to the reservoir does not include the dam. Maybe I am misreading him and I am sorry he is not here to correct me.

A G MORRIS (Lancashire County Council)

With regard to Dr Coats' comment on the need for technical knowledge within the enforcement authority; as the act lays down, an enforcement authority can take no action with respect to a reservoir without consulting an inspecting engineer. In every case, they have to ask an inspecting engineer for advice before taking any enforcement action, but it is necessary for someone in the enforcement authority to understand what the inspecting engineer is saying. I would not expect an enforcement authority at any time to take action without having the approval of the inspecting engineer and I am myself rather surprised to hear it suggested that an undertaker might put works which affect safety into operation without a further inspection, to their own design or on the advice of the supervising engineer.

R M ARAH (Binnie & Partners)

At Cwm-y-Foel there was a reference to the water authority before an inspecting engineer was called in, really to establish whether there was a need for an inspecting engineer: was it a false alarm or was it not? I think that was quite sensible and the right person to ask is very probably the supervising engineer, if he is prepared to answer and if the undertaker is prepared to let him. I hope he would be because it seems to me it is helpful. On Dr Coats' point about the size of works to ensure abandonment or discontinuation, I think it is horses for courses. In the case of Cwm-y-Foel for instance, the upper part of the dam was the trouble and we cut it down to a quite small, quite wide structure, which presents no threat. As it happens, it drops out of the Act - which was what the county was trying to do. On the question of how large a pipe or culvert through a dam needs to be for discontinuance, I believe it is very much a function of the type of dam and the problems involved, and I think it is eminently a matter for an inspecting engineer's opinion.

Cwm-y-Foel could indeed have been certified to have been abandoned if it was thought that there was an ongoing risk. I think the remaining risk is that somebody closes the notch and restores the previous situation. It is a large notch, some 4 m wide, but people do strange things.

D J GREATREX (Welsh Water)

I am the person involved in this little incident at Cwm-y-Foel.

As supervising engineer for the Welsh Water Authority, I was contacted by the police in the first instance. This is before people really realised what the Act was about. In fact, I was asked to inspect this reservoir, which I declined first of all, because it was not a Welsh Water reservoir. However, the particular policeman then had to report to his superiors, who told him to make sure that the dam was inspected in case it did fail. That policeman then came back on the phone to me to ask me to inspect it. Of course, as in all incidents, they always happen late in the evening, or on the weekend, when people are not available, and so it was agreed by my superiors that I should go and inspect this reservoir. I was told that a helicopter was not available. I arrived eventually at the local police station some miles away, and there was no landrover available. Off we set on shanks's pony on what was considered to be the best route by the National Park Warden.

It was pouring with rain and dark. We had torches which eventually began to run out, and eventually came to a track, which just abruptly ended. We could not go on, and we could not go up because it was too steep. We then had to go back down the valley and climb up on the other side which is very steep. On the way up the torches were already running out, and I decided to switch mine off. The next moment I slipped and I nearly went headlong down this very steep valley. Fortunately, there was a pipe, which I managed to grab hold of. We arrived eventually at this particular dam at midnight. It was pitch black, there was cloud cover, and there I was as the supervising engineer, inspecting somebody else's dam to give what was, obviously, an engineering opinion. I looked at it and it did not look too bad to me. So I just turned to the policeman and said "Well, I do not think it is a particular emergency", and then we just clambered back down the mountain. Mr Arah was contacted the next day, via the County Council, who did not find out until the following day; Mr Arah looked at it on the day following this inspection and happily, he agreed with myself. It does point out the difficulties that supervising engineers can get themselves into, but I would draw attention to the fact that the supervising engineer must be contacted first, because he has to give some kind of engineering opinion as to whether he thinks that this is an emergency or not or as to whether the inspecting engineer should be called in.

W J CARLYLE (Session Chairman)

I am disappointed that nobody is discussing the length of time to make inspections and reports. Mr Wight quoted that the record was 15 months, and I think I could certainly beat that, any day! It may be quite easy to make an inspection and write the report on the train going home, and get it in to he owner the next day, but it is equally the case that when there are difficulties, you develop the whole inspection procedure, and it can take quite a long time. Has anybody got any views on that, or on whether you should stipulate time for completion of the necessary works in the interests of safety? 'As soon as possible' means nothing at all and it would in my view be difficult to put a specific time in your report because money has to be raised and powers have to be obtained in certain circumstances. Finally, also in Mr Wight's paper, the question of whether written categories should be notified. Has anybody any views on that?

J D HUMPHREYS (MRM Partnership)

With regard to the time within which remedial action should be taken I always take the view that if the inspecting engineer really feels that something is vital, and he wants to apply a bit of pressure, he says that the time of his next inspection should be, let us say, two years or after the completion of the remedial works. In other words, he has the ability to use his recommended date of the next inspection as a kind of pressure to apply to the undertaker, as we all know.

F G JOHNSON (North of Scotland Hydroelectric Board)

When we inspect our reservoirs we categorise our defects into two categories; I which should be done in a season and D which should be done as early as convenient.

Since the new Act was introduced we have asked our inspecting engineers to state a date by which essential safety works should be carried out - only the safety works - because we find this is helpful in ensuring that safety measures are carried out properly and to the right programme. We find that if this isn't done, when the next inspection comes along the work hasn't been done, and it's very embarrassing and I think against the spirit of the Act. So we are asking our inspecting engineers to specify a time by which just safety work has got to be carried out and we ask them also to be reasonable in specifying that time.

D GALLACHER (R H Cuthbertson & Partners)

On this question of setting a time for safety recommendations, I must say that I do normally set a time, obviously taking account of how serious one considers the recommendation is. It is worded in such a way that when in fact the recommendation is carried out, it can then be struck off by a certificate and the next inspection would then take place in say, 5 or 10 years, so that it doesn't then require a further inspection.

WRITTEN CONTRIBUTIONS

D A THOMAS (Welsh Water)

Many and varied are the suggested draw down rates to be found in text books, published papers and Inspecting Engineers' reports. One source recommends 1m in 6 hours whilst another suggests 1m in 6 days would be acceptable. It is not unreasonable to expect some rational thought to be behind the derivation of these rates and that acceptable draw down rates should be published as a guide incorporating the consensus of opinion.

Any criteria for draw down rates should endeavour to take into account the following influences:-

i) The category of the reservoir - Obviously it is of greater concern to draw a reservoir down more quickly if loss of life is at stake.

ii) The height of the dam - Leakage rates or discharge rates on failure are related to the height.

iii) The volumetric shape of the reservoir - For the same volume and depth an inverted pyramid will draw down quicker than a cylinder.

iv) The inflow into the reservoir - In some cases reservoirs could not be drawn down during a severe storm.

v) The feasibility of draw down arrangements - Any suggested rates must be achievable without prohibitive cost implications and yet must alleviate a disaster with all reasonable haste.

Based on the criteria the following table of draw down rates is suggested. The table provides the number of days to draw the reservoir down to half its height during an average Winter 6 months inflow.

HEIGHT / CATEGORY	A	B	C	D
0 M - 10 M	10	12	14	17
10 M - 20 M	9	11	13	16
20 M - 30 M	8	10	13	16
30 M - 50 M	7	9	12	15

It should be noted that rapid draw rates may be restricted by the upstream slope stability, by the flow that the pipework/conduits can structurally sustain or by maximum discharges that will not cause severe flooding downstream.

Space does not permit the full justification of the table in meeting the criteria but out of 32 reservoirs considered 6 lie outside the suggested rate. At two of these reservoirs neither pipe modification nor imported pumps and syphons would provide sufficient draw down capacity and another large conduit through the dam would appear to be the only answer.

So here is a table of suggested draw down rates, can anyone improve on this?

J L HINKS (Sir William Halcrow & Partners)

It may be of interest to record some of the considerations influencing the choice of capacity for the drawdown outlet for the Mrica Hydro-Electric Power Project in Central Java. The dam is a rockfill dam about 100 metres high with an inclined clay core. Key levels are:

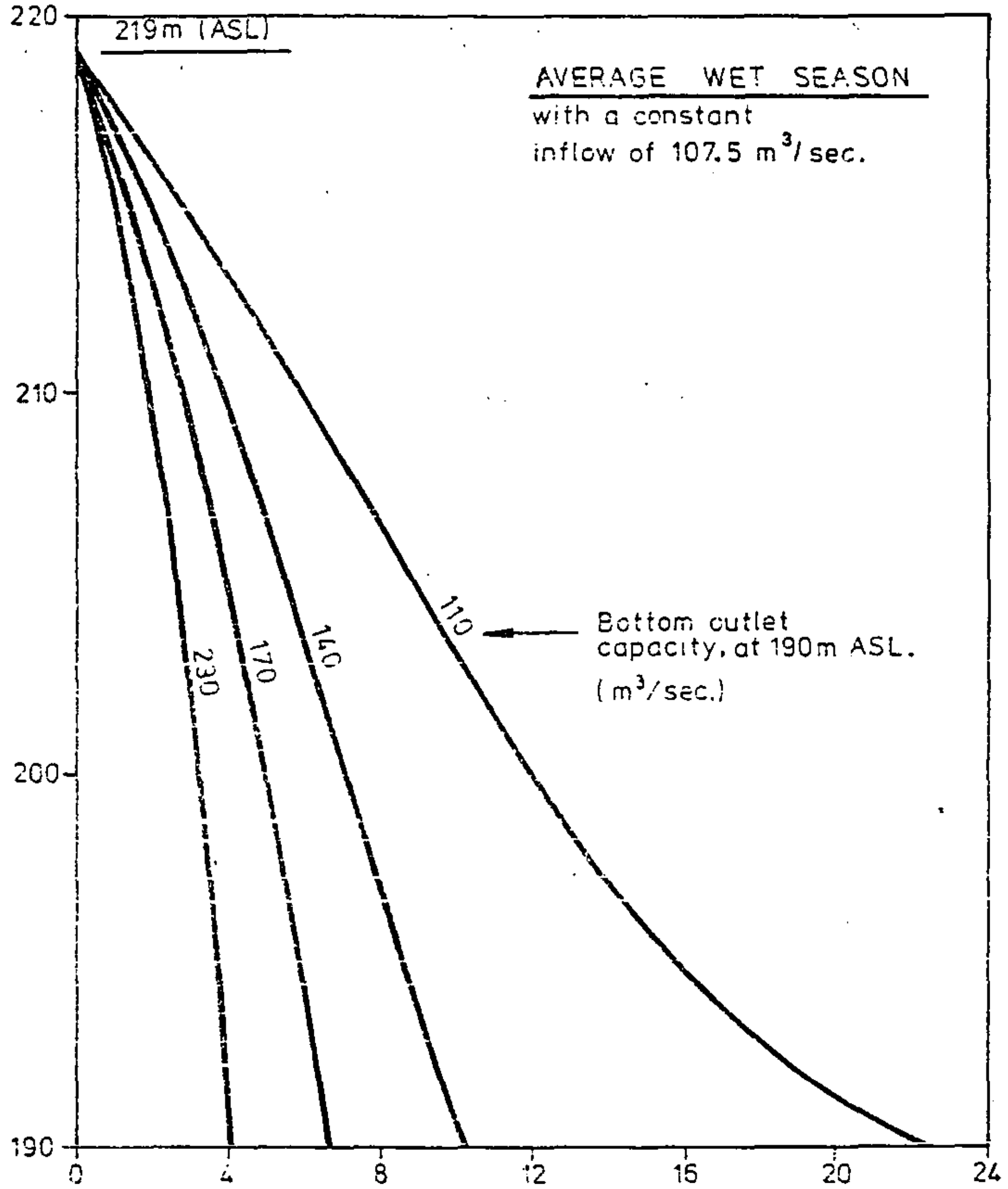
Dam Crest	235m ASL
Full Supply Level	231m ASL
Sill Level of Gated Spillway	220m ASL

The purpose of the drawdown outlet is to control first filling of the reservoir, to prevent the build up of sediment around the power intake and to permit drawdown for repair to the dam or spillway should this ever be necessary following an earthquake or a flood in excess of the design capacity of the spillway energy dissipator.

In order to flush sediment from around the power intake, without wastage of water, it is envisaged that the drawdown outlet will be opened during the passage of each flood. The spillway gates will be opened only if the drawdown outlet is unable to contain the flood rise. Clearly the control of first filling and the potential to discharge sediment are both improved by increasing the capacity of the outlet. On the other hand an excessive rate of drawdown could lead to instability of the reservoir banks.

Bearing these factors in mind it was felt that the outlet should have a sufficient capacity to permit the reservoir to be drawn down from the level of the spillway sill (originally planned to be at 219m ASL) to a level of 190m ASL (10 metres above the intake sill of the drawdown culvert) within a reasonable period of time during any season of the year. It should also be possible to prevent excessive flood rises with the reservoir drawn down during the progress of any repair works. The table shows drawdown times for an average dry season, and average wet season and a 1 in 5 year wet season.

	Time for Drawdown from 219m ASL to 190m ASL (Days)		
Capacity of Drawdown Outlet (Reservoir at 190m ASL) M^3/sec	(a) Average Dry Season	(b) Average Wet Season	(c) 1 in 5 year Wet Season
110	10.0	22.8	-
140	6.6	10.1	87.0
170	5.1	6.7	59.4
230	3.4	4.1	56.2



The figure shows graphically the above drawdown times for an average wet season and it is clear that a worthwhile improvement is achieved as the outlet capacity (at a water level of 190m ASL) is increased from 110m^3/sec to 140m^3/sec. After this improvements are more modest.

The finally selected capacity of the outlet was 280m^3/sec at Full Supply Level (231m ASL) or 160m^3/sec at a water level of 190m ASL. The latter figure is about 150% of the average wet season reservoir inflow and is also expected to give an acceptable degree of control over initial impounding as well as facilitating sediment flushing and the ability to hold the reservoir at a low level for repairs.

J HAWKES (Hampshire County Council)

Referring to para 28 of Mr Morris's paper I believe that a precis of each panel engineer's report should be included in the Register with particular attention paid to matters in the interests of safety, since the purpose of the Act is to ensure public safety.

In this respect, why should not an engineer's report be made available for the public to see if they wish? If I was negotiating the purchase of a property downstream of a large raised reservoir I would want to read every word of all the reports available - wouldn't you?

To deny access to any report could be construed as an attempt to keep from public knowledge some unpalatable fact, whether this was true or not. Just suppose for example, that a dam failed and it was subsequently revealed that a panel engineer had made recommendations for remedial works that had been suppressed. The damage to the standing of the civil engineering profession could be incalculable. Would it not be better to make reports and certificates available?

A ROBERTSHAW (Yorkshire Water)

I note what appear to be apparent anomalies between the papers of Mr Maconachie and Mr Earp regarding the certification under Section 10(6) of the Reservoirs Act 1975 of recommendations made by an inspecting engineer in the interests of safety. Mr Maconachie states that, in his opinion, no such works can be certified by a supervising engineer (paragraph 26), whereas Mr Earp states, again in his opinion, that this is satisfactory where the inspecting engineer has specifically delegated this work to a member of the supervising engineer panel (paragraph 4).

According to my interpretation of the act, supported by others within Yorkshire Water, the Act allows the supervision and certification of such works to be carried out by a 'qualified civil engineer' (within the meaning of the act as defined in Section 4(1)) and, unless the inspecting engineer has specifically made reference to the panel of the qualified civil engineer, it is up to the undertaker and, ultimately, the relevant enforcement authority to decide who is acceptable to be appointed. Obviously, a responsible undertaker will ensure that the person appointed will be appropriate for the scale of the works that are involved and I would also think that a supervising engineer would bear this in mind when deciding whether or not to accept such an appointment.

J W PHILLIPS (Department of the Environment)

The following points are made in response to various contributions on Mr Maconachie's Paper.

1 Supervising Engineer

A Supervising Engineer's powers under the Act are limited by Statutory Instrument to those given in Section 12. It is for engineers to decide what matters should be referred to the undertaker as affecting reservoir safety, with particular attention to the observance of the provisions of Sections 6(2) to (4), 9(2) and 11. They may give useful advice on other matters but it is not statutorily enforceable.

2 Any works under Section 6(1) should be designed and their construction supervised by a construction engineer. No powers have been given to members of the supervising panel either in the Act or by Statutory Instrument to act as construction engineers or to issue certificates (any works not covered by Section 6(1) but which affect reservoir safety should be inspected under Section 10(2)(b)).

3 The time interval between visits of a supervising engineer is a matter to be agreed between him and the undertaker. The only guidance given in the Act is that where the inspecting engineer has noted in the annex to his report matters to be watched by the supervising engineer, the latter shall make a written statement of the action he has taken to do so not less often than once a year (Section 12(2)). It is of assistance if the inspecting engineer is able to give some guidance on timing, but this has no statutory force.

4 Dams

The word 'dam' is not mentioned in the Act. It may reasonably be assumed that it is included in the works comprising a reservoir.

5 Other Relevant Acts.

Two other Acts which may be consulted if a reservoir does not appear to be covered by the Reservoirs Act are the Health and Safety at Work Act 1974 and the Mines and Quarries (Tips) Act 1969. Both of these Acts are operated by Inspectorates of the Health and Safety Commission.

6 Reserve and Emergency Powers

It is for enforcement authorities to determine whether works are required under Sections 15(2) and 16. The use of either Section requires the services of an appropriate panel engineer.

A MORRIS (Lancashire County Council)

The following points are made in response to the written contribution by Mr J Hawkes.

Schedule 1 to the Registers, Reports and Records Regulations 1975 requires that the information recorded in a local authority's register shall include a summary of all certificates and reports received under the 1975 Act. It specifies four items which must be included, but none of these provides any technical information. No guidance has been given as to what, if any, further information is intended to be registered and open to inspection and the Schedule contains no reference to works recommended to be carried out in the interests of safety.

There are obvious dangers inherent in the preparation by non-specialist staff of a precis of the technical content of an inspecting engineer's report for public consumption. It is the author's view that this should not be attempted, since such a summary or precis would be open to further interpretation not only by members of the public but also by the media.

With regard to the status of the full text of such reports, DoE Circular 5/85 requires local authorities to provide secure storage for all reports and certificates which, it is specifically stated, will not form part of the register and will not be available to the public. It must be remembered that final certificates and first inspection reports on pre-1930 reservoirs under the 1975 Act will be accompanied by detailed drawings and descriptions of the works, including dimensions, levels and geological information. The Circular explains that unauthorised access to plans and operational details of reservoirs could constitute a security risk.

Clearly, therefore, it is not intended that such information should be available to the owner or a prospective purchaser of downstream property, however relevant it might appear. As the matter stands, the Secretary of State has the power to institute an inquiry in circumstances where it appears that a local authority has failed in its duty under the Act. In the event of a reservoir failure, an inquiry would seem inevitable, and the actions of all the parties involved would be subject to examination.

DR A K HUGHES (North West Water)

For the difficulties experienced in obtaining information on our 'stock' of dams I would again suggest that a central registry of dam information equipt with all necessary databases may be a much more efficient method of policing the Reservoirs Act 1975 and giving professional engineers information on which to base many of their decisions.

PROCEEDINGS: TECHNICAL SESSION 2

THE SUPERVISING ENGINEER

PAPERS:

DISCUSSION:

RESERVOIR SUPERVISION 1986-88

D N W Earp MA FICE FIWEM

Consultant, Binnie & Partners

SYNOPSIS

The paper draws on the author's experience of reservoir supervision during the first two years after the implementation of the 1975 Act. It highlights differences between the attitude of various categories of reservoir undertakers and also the differences of approach between several enforcement authorities. It also describes some of the practical aspects of a supervising engineer's work.

INTRODUCTION

1. The author has inspected 40 reservoirs under the 1975 Act. All but five of these reservoirs are in Wales, where there is only a handful of privately-employed members of the Panel of Supervising Engineers. The author has been appointed to act as supervising engineer for 22 of the above reservoirs, and for a further 13 reservoirs which had been inspected by other panel engineers. With one exception, the reservoirs concerned are all impounding reservoirs with capacities between 25,000 and 16,000,000 cubic metres. His clients have included four public, two large industrial and 17 small private undertakers. The author has been involved with seven Welsh and four English enforcement authorities. The comments and conclusions in this paper are based on his own experience and do not necessarily represent the views of his colleagues in Binnie and Partners.

THE WORK OF THE SUPERVISING ENGINEER

Matters to be watched by the supervising engineer

2. Some inspecting engineers' reports issued under the 1930 Act during the period 1975 to 1986 included a list of matters to be watched by the supervising engineer, but more of these reports did not include such a list. Most publicly owned reservoirs were subject to regular supervision on a non-statutory basis before April 1986, and in these cases the absence of an inspecting engineer's list presented no problem to the formally appointed supervising engineer when the Act was implemented. The position was different for a supervising engineer newly appointed to supervise an old, privately owned reservoir which had never been inspected under the 1930 Act, when he was expected to commence his duties in advance of the first statutory inspection under the 1986 Act. This situation arose because many enforcement authorities were more concerned to see supervising engineers appointed than to ensure that statutory inspections were carried out. In these circumstances the supervising engineer was faced with the necessity to carry out what almost amounted to a statutory inspection on his first visit to site and to draw up his own list of matters to watch on future visits.

Frequency of visits

3. A recommended minimum frequency of visits is rarely stated in inspecting engineers' reports under the 1930 Act, although this is normally given under the 1975 Act. In the absence of formal guidance, a reasonable working basis appears to be quarterly visits for Category A and B reservoirs, six-monthly for Category C and annual for Category D. When two visits are made annually, it is much more useful for these to take place in winter and summer than in spring and autumn. The supervising engineer for a Category D reservoir who cannot justify the cost of more than one regular visit is nevertheless well advised to pay both a winter and a summer visit during the first year of his appointment in order to see the dam under widely differing conditions.

Supervision of works under Section 10(6) of the Act

4. Inspecting engineers sometimes state that the execution of certain of their recommendations on "measures to be taken in the interests of safety" may be supervised and approved by the supervising engineer, to save the costs of employing a Panel AR engineer for what are often quite straightforward matters. Some enforcement authorities have objected to the delegation of this responsibility to a supervising engineer, but others have accepted it and in these circumstances it is in the interest of all parties for the responsibility to be entrusted to the supervising engineer.

Adjudication on capacity of border-line reservoirs

5. Some enforcement authorities have given positive encouragement to the owners of reservoirs whose capacity in relation to the qualifying volume of 25,000 cubic metres is in doubt, to employ a member of the supervising panel to carry out a survey and state his opinion on the capacity. In such cases the enforcement authority has usually accepted the supervising engineer's opinion as to whether or not the reservoir should be added to or taken off the register.

Time spent on supervision

6. The supervising engineer is likely to spend several hours on site on the occasion of his first visit. At large, well-documented reservoirs there is much to familiarise himself with, whereas at small privately owned reservoirs there are usually no drawings and no documentary evidence of any kind. Future visits may take anything between 30 minutes and 3 hours on site. It is rare to carry out any routine survey or instrument measurements on small privately owned reservoirs and on the larger publicly owned reservoirs such work is normally undertaken by technicians. In most cases the major element is the time spent on travelling to and from site, the cost of which must be covered by the fee charged for the supervising engineer's services.. Some private owners, who see their supervising engineer for only an hour or two each year, still appear to resent having to pay a fair fee for the service they receive. Others full appreciate the situation and are very co-operative.

Paper-work

7. The appointment of a supervising engineer who is not a member of the undertaker's own staff is normally on the basis of an exchange of correspondence, but occasionally an undertaker requires the provision of a formal

agreement. There is no experience to date of such an appointment having been terminated by decision of an undertaker because he was dis-satisfied with the service provided. On the other hand there has been some reluctance by firms providing supervising engineer services to introduce an annual cost-of-living rise in fees which are charged on a lump-sum basis.

8. The completion of the initial entries in the new prescribed form of record is normally carried out by either the supervising engineer or the inspecting engineer, depending who is first on the scene. It is often necessary to make assumptions or approximations in order to complete these entries within a reasonable period of time.

9. Although a formal annual report by the supervising engineer is only required where "matters to be watched" have been listed by an inspecting engineer, the undertaker is clearly entitled to expect to receive such a report in all cases. A single page report, reassuring the undertakers that all is well, is often sufficient.

Action when trouble occurs

10. The only formal remedy provided in the Act for dealing with trouble at a reservoir is for the supervising engineer to call for a full statutory inspection. Indiscriminate use of this remedy would put undertakers to much unnecessary expense. In practice, supervising engineers usually seek the informal advice of a Panel AR (or Panel I) engineer, often but not necessarily the last inspecting engineer, on appropriate measures to overcome the trouble in question. If the supervising engineer happens also to be a member of Panel AR he can of course provide the informal advice himself. Only if the undertaker refuses to take action on the informal advice provided in these circumstances, does it become necessary for the supervising engineer to recommend the undertaker to appoint an inspecting engineer, sending a copy of his recommendation to the enforcement authority.

11. On a number of occasions, the owners of small private reservoirs have, when faced with the need to carry out expensive measures in order to restore their reservoirs to a safe condition, chosen to incur the cost of "discontinuing" their reservoirs in order to avoid probable future maintenance costs as well as supervision and inspection fees. The most usual cause of such a decision is the discovery of leakage through the dam which appears to be increasing with time.

RELATIONS WITH UNDERTAKERS

Publicly owned undertakers

12. It is well known that Water Authorities, Water Companies and the other public bodies owning significant numbers of reservoirs had taken their responsibilities for reservoir safety seriously for many years before the implementation of the 1975 Act. It has been encouraging to find that the public owners of small reservoirs are taking their responsibilities just as seriously. For instance, one community council, which has taken over a former water authority reservoir for amenity purposes, regularly sends three council members to meet the supervising engineer during his site visits. There is now a limited use of supervising engineers from consulting engineers by Water Authorities as well as other public owners. The supervising engineer can normally rely upon publicly owned undertakers to maintain

accurate and up-to-date records of water level, instrumentation etc, and also to carry out inspecting engineers' recommendations within a reasonable timescale.

Industrial undertakers

13. Industrial undertakers in Wales also appear to have taken their responsibilites under the 1930 Act seriously. One company owning three old dams with a chequered history located above a centre of population has for many years encouraged its inspecting engineer to carry out annual informal inspections and has acted on the advice which they received after these visits as though this advice had the force of statutory recommendations. The same company has willingly accepted the need for their supervising engineer to visit their reservoirs quarterly. This is a case where the appointment of a Panel AR engineer able to give advice on problems as they arise on a group of ageing reservoirs has proved to be of advantage to the undertaker.

Small private owners

14. Many of the small reservoirs in Wales which escaped the 1930 Act by default, are owned by private householders, farmers and angling associations. Without exception, these owners resent the fact that the 1975 Act has caught up with them after 50 years of ignoring the 1930 Act. A few owners have chosen to lower the top water level to reduce the capacity below 25,000 cubic metres in order to escape the provisions of the Act, even in cases where the reservoir in its existing form has been found satisfactory when inspected. Others have chosen to remain within the provisions of the Act unless and until trouble occurs, as mentioned in para. 11 above.

15. Whilst a reservoir which has once been registered must be left in a "safe" condition after discontinuance, a number of cases have come to light of "borderline" reservoirs which are found after survey to contain less than 25,000 cubic metres but nevertheless pose a safety threat. In these circumstances all that the panel engineer responsible for the survey can do is to draw the owner's attention to his potential liabilities under "Rylands v Fletcher".

16. Many private owners are showing great reluctance to record water levels, even when required to do so as infrequently as once a month by the inspecting engineer. In the case of reservoirs which overflow continuously, it is easy to understand why these owners consider such a requirement to be an unnecessary waste of time and effort.

RELATIONS WITH ENFORCEMENT AUTHORITIES

17. Each enforcement authority appears to have used a different system for establishing its register, with a wide spread of manual and computer-based systems in use. Some authorities include in their registers only those reservoirs the capacity of which is acknowledged by the undertakers to be greater than 25,000 cubic metres, whilst maintaining separate lists of "borderline" cases and of reservoirs which have been removed from the borderline list as a result of their own investigations or of submissions by the respective owners. Other authorities initially included all borderline cases on their registers and placed the onus on the undertaker to provide a survey report showing the capacity to be less than 25,000 cubic metres before agreeing to remove the reservoir from the register.

18. The County Surveyor's department has in most cases been given the responsibility of deciding which reservoirs are large enough to be included on the County register. However, in one County with a large number of borderline reservoirs, all correspondence with undertakers has been conducted by the County Secretary and Solicitor's department and in some other counties the County Clerk's department appears to have inhibited the activites of the County Surveyor in compiling the register. At least one County employed a firm of Consulting Engineers to handle the initial compilation of its register.

19. Most County Surveyor's departments have sent engineers to carry out visual surveys of all small blue patches representing water areas larger than about 2 hectares, in order to ascertain whether the water is held back by a dam and if so to estimate the height of the dam and hence the potential volume of water stored above original ground level. At least one County has called on the services of a panel engineer to help resolve disagreements between the County and a number of owners on the status of borderline reservoirs.

20. One County sends an engineer to look at every reservoir for which an inspecting engineer's report recommends that measures should be taken in the interests of safety.

21. Some Counties always acknowledge receipt of panel engineers' reports and certificates, some do so from time to time and some never do so. One County has queried the flood category chosen by an inspecting engineer and the storage capacity determined by another. A different County declined to accept the appointment of the same individual as both supervising and inspecting engineer for a reservoir, on the grounds that the former MUST be employed by the undertaker and latter CANNOT be so employed.

22. One County has asked supervising engineers to supply their home telephone numbers to the County for emergency use. This is not "prescribed information", although both the home and business numbers are included in the statutory form of record, this being a document to which only the undertaker, supervising engineer and inspecting engineer have right of access.

23. At least one County has required undertakers to provide retrospective Final Certificates for reservoirs completed between 1933 and 1986 in all cases where no such certificate was issued under the 1930 Act, even in those cases where a Certificate as to Execution of Works had been issued. Compliance with this requirement has necessitated the appointment of vicarious Construction Engineers to carry out inspections under Section 8, following which they could in most cases immediately issue Final Certificates.

24. In another County there is a situation which has been unresolved for two years. An angling association asserts that because it owns the water in five reservoirs but not the dams which hold back the water, it is not the undertaker and cannot therefore be required to appoint an inspecting engineer or a supervising engineer.

RELATIONS WITH INSPECTING ENGINEERS

25. Whilst the 1930 Act was in operation, it was common for both the undertaker and the inspecting engineer to consider that the latter had an ongoing "duty of care" in respect of any reservoir inspected, until the time of the next inspection. On occasion, the inspecting engineer has been known to

act as a self-appointed unofficial enforcement authority when he considered that the undertaker was perpetuating an identified public safety hazard by failing to carry out one or more of the "measures recommended in the interests of safety" in his report. The 1975 Act has formally removed this ongoing relationship between undertaker and inspecting engineer by placing the day-to-day responsibility for the well-being of the reservoir on the shoulders of the supervising engineer.

26. In practice, in many cases the old relationship has been perpetuated, as one leg of an informal new tripartite relationship between undertaker, supervising engineer and inspecting engineer. As indicated in para. 10 above, the first person to whom an anxious supervising engineer turns for advice may be the previous inspecting engineer. On the other hand, the employment by some of the major reservoir-owning authorities of ten or more full or part-time supervising engineers has created sources of in-house expertise which have reduced the number of occasions on which inspecting engineers' advice is sought. Nevertheless, there is no delay in calling in an inspecting engineer when a supervising engineer considers that a real safety hazard exists on a publicly owned reservoir.

27. The owners of many private reservoirs continue to be perplexed by the need to employ both an inspecting engineer and a supervising engineer. This confusion is compounded when they discover that the supervising engineer is in most cases not deemed to be competent to supervise the implementation of inspecting engineer's safety recommendations.

CONCLUSIONS

28. This paper is to a large extent a recital of facts which contain few surprises and little cause for concern. They do however indicate that the implementation of the concept of regular supervision of British reservoirs has proceeded in a relatively straightforward manner.

29. As expected, the function of supervision is being carried out by empanelled employees of the major undertakers, for whom the duty represents a significant part of their workload, and by some consulting engineers' staff for whom this work is generally an insignificant part of their total activity. Generally the provision of supervising engineers for clients' reservoirs is a relatively unprofitable part of the overall service provided by consulting engineers to the community of reservoir owners.

30. Two years after the full implementation of the 1975 Act throughout most of Britain, the process of registration, the appointment of supervising engineers and the first round of inspections of reservoirs which escaped the 1930 Act is still incomplete in some areas. Perhaps not surprisingly, there are wide variations in the practices of different County Councils in the exercise of their role as enforcement authorities.

ROUTINE MONITORING WORK ON RESERVOIR SAFETY IN HONG KONG

KU Chi-chung Damien BSc(Eng) MICE MIWEM MHKIE

SYNOPSIS

1. In maintaining dams and reservoirs, the Water Supplies Department of Hong Kong engages qualified panel engineers to carry out inspections at regular intervals, and maintains a team of staff to undertake continuous monitoring and surveillance. Attempts have been made to establish effective systems of data collection, recording and analysis to facilitate reservoir safety examination. This paper gives a brief account of these activities in Hong Kong, addressing particularly the problems encountered and how they are being tackled.

INTRODUCTION

2. In Hong Kong maintenance of reservoir safety follows generally the spirit of the U.K. legislation, although this practice is not a statutory requirement in the territory. Since the mid-1970's, the Water Supplies Department has engaged panel engineers from three different consulting firms, at approximately 5 year intervals, as "inspecting engineers" in line with the U.K. Reservoirs (Safety Provisions) Act, 1930 and later the Reservoirs Act of 1975.

3. In the Department there is a Reservoir Safety Section comprising 2 chartered civil engineers and 13 inspectorate and technical staff to provide the necessary support for the inspecting engineer during his visit to Hong Kong, and to undertake follow-up actions on his recommendations including the monitoring requirements and matters to be watched subsequent to his inspection. Routine monitoring is therefore a major task of the Reservoir Safety Section and is carried out as a continuous exercise at all times.

4. The current monitoring programme covers 28 impounding reservoirs which have a total of 47 dams including 21 embankment ones, 24 gravity ones and 2 composite ones having features of both types. Some of the dams have been in existence for more than 100 years while others have been constructed progressively since the turn of the century, the most recent being only about 10 years old. Also included in the programme are 30 service reservoirs all having a storage capacity larger than 25000 cubic metres. (The smaller service reservoirs not covered by this programme are also regularly inspected and maintained under a separate exercise of the Department). The work involves direct inspection and instrumentation, and covers the dam structures as well as ancillary works associated with the safe operation of the reservoirs, such as the inlet controls, drawoff facilities, overflow spillways, etc. The task of the monitoring work of the Reservoir Safety Section is generally described below.

DIRECT INSPECTION

5. Direct inspection on reservoir structures is the basic method of observing external signs of deterioration. Each reservoir is inspected at least once a year by one of the engineers in the Reservoir Safety Section. Besides going through those areas of concern highlighted by the last inspecting engineer the Engineer/Reservoir Safety also takes note of any anomalies that he considers to be related to reservoir safety, and refers the necessary remedial measures to the works sections for implementation. Subsequently he will prepare specific guidelines for his supporting staff to carry out further surveillance about once a month for updating the information.

6. In order to ensure that nothing significant is overlooked, a check-list is drawn up and continuously updated for each reservoir. Major external defects, such as cracks, seepage points, vertical or horizontal movements, etc., are given reference codes showing the location, type and serial number. The observed conditions are recorded in prescribed formats supplemented by sketches and photographs. This system has proved to be very convenient for updating the information, which is largely descriptive, and for tracing the history of development at a later date.

7. Inspections by the Engineer/Reservoir Safety are often arranged in co-ordination with operational or maintenance activities. As a general rule, the internal inspection of a service reservoir is scheduled to tie in with the cleansing programme. For dams with significant leakage problems opportunities are taken to make special visits when operational plans can allow the reservoir storage to be drawn down progressively from a high water level to an exceptionally low level. More frequent readings of the monitoring instruments are taken during the drawn down period. Major leaking points at some old dams have been located successfully in this way.

8. Under circumstances of severe events of nature, such as typhoons and rainstorms, the Reservoir Safety staff would make special site visits to observe the characteristics and performance of certain components of reservoirs that bear significant safety implications, e.g. syphon spillways, discharge channels, embankments subject to severe wave action, etc. A portable video camera is sometimes used to record details of interest.

9. Visual inspections will be difficult and information unreliable if the area to be inspected is covered up, inaccessible or in a dark environment where details cannot be clearly seen. Prior arrangement is therefore always made to cut short the grass on earth embankment dams before inspection, and the surveillance staff are equipped with binoculars, powerful torches and portable flood lights to enable them to see fine details even under very unfavourable conditions. To facilitate close inspection on the upstream side of a dam the staff are also provided with an inflatable dinghy for use on the reservoir surface.

FIELD MEASUREMENT

10. While some evidence of deterioration can be detected by direct inspection, the underlying causes and the trend of behavioural changes cannot be fully understood and quantified without field measurements. This aspect of monitoring includes engineering surveys and instrumentation.

11. Routine land surveys are undertaken by the Surveying Section at regular intervals on 9 impounding reservoirs and 3 service reservoirs where settlements are expected or signs of movements have once been observed. Survey points are established on the dam bodies, reservoir walls and surrounding areas where movements would be of significance in the interest of safety. On embankments which have shown evidence of being disturbed by wave actions surveys are requested after severe rainstorms and typhoons to check the consequent effects.

12. Aerial survey is adopted in one particular case to check the disturbance on a wave protection structure at High Island East Sea Coffer Dam which is to protect the main dam against wave attacks from the Pacific Ocean. The protection work is made up of more than a thousand pieces of interlocking concrete anchors called "Dolosse", each of which is cast in the shape of a twisted letter "H". It is not practical to use ordinary land surveying techniques in this case because of the large number of points to be checked and the difficulty in walking up and down the slope formed by these anchors.

13. With regard to instrumentation, the older dams have a general lack of original monitoring equipment. To facilitate observation of their performance some piezometers and observation wells have been installed later, but the number is not great. In the newer dams, on the other hand, many more monitoring instruments are generally provided and most of them were installed during the construction stage. Table 1 shows a summary of typical monitoring measurements adopted in the existing dams.

TABLE 1

MEASUREMENT	CONCRETE DAMS		EMBANKMENT DAMS	
	No. of Instruments	No. of Dams	No. of Instruments	No. of Dams
Rainfall	8	8	10	10
Seepage	39	6	49	15
Joint & crack movement	19	4	120	3
Horiz. or vert. movement	24	1	23	5
Phreatic level & pore pressure	48	6	752	17
Stress	Nil	Nil	146	2
Strain	Nil	Nil	20	2

14. There are 9 concrete dams and 1 composite dam without any monitoring instruments. Most of the service reservoirs are also not equipped with instruments. Scanning of defects for these reservoirs relies mainly on visual inspection on site.

15. The adequacy of monitoring instruments and frequency of taking readings are topics to be reviewed by the inspecting engineer and his recommendations in this respect are closely followed. When significant changes of the potentially hazardous conditions are noticed, extra readings are normally taken to provide further information until a predictable or stabilised situation is observed.

16. For economic use of manpower resources, the assistance of operational staff outside the Reservoir Safety Section is called on to take simple instrument readings at remote reservoirs. All staff members taking instrument readings are reminded to report immediately to the Engineer/Reservoir Safety on any abnormal reading noted, and be watchful of any factors that may affect the accuracy or reliability of the readings. Some of these factors are easily noticeable, e.g. rainfall or draining through cracks in the collection channel, which will obviously result in false seepage readings. Some others, however, are not so obvious, e.g. a broken piezometer tube will give misleading readings that may not be easily verified or even noticed. The method of measurement could be another factor, e.g. different V-notch flow readings may be obtained if the measurement is taken by different methods and at different positions.

DATA PROCESSING & ANALYSIS

17. Collected data and information would have very little value if they are not systematically recorded and analysed. In this respect, computer aids are almost indispensable. Computerisation in the Reservoir Safety Section is still at a developing stage, and a micro-computer system is being used.

18. Monitoring data are analysed to serve one or more of the following purposes :-

(a) to make anomalies more easily noticeable among the norm;
(b) to show development trends of potentially hazardous conditions;
(c) to verify the parameters assumed in theoretical analyses; and
(d) to check the effectiveness of remedial measures.

19. Almost all observed data are displayed in graphs. To show correlations among the data, various combinations of these graphic plots are produced. In most cases the measured readings are plotted together with reservoir water levels, and in some cases together with rainfall readings, on a specified time scale up to 20 years.

20. For piezometers of embankment dams the following sets of graphs are usually plotted :-

(a) readings at upstream and downstream embankments, core and foundation across each monitored section of the dam are plotted in selected groups against time;

(b) readings at corresponding cross sectional positions along the length of the dam are plotted in groups against time; and

(c) readings of (a) plotted against the chainage of the dam at a specific point of time.

21. For seepage flows the following typical graphs are plotted :-

(a) flow readings are plotted together reservoir water levels and rainfall readings against time; and

(b) flow readings with no rainfall are plotted against reservoir water levels over a specified period of time.

22. In examining a combination of graphs, the greatest difficulty is to identify the tangling lines. In this regard it is highly desirable to include in the hardware system a colour monitor and a colour plotter, and to incorporate in the software library a 3-dimensional graphic package.

23. Analysed results and observations are incorporated in the annual report of the Engineer/Reservoir Safety on the performance of each reservoir, which also includes his other findings based on inspection records and information on system operation and maintenance. Progress of recommended remedial measures is also reviewed. This document will form a very important reference for the next inspecting engineer.

STAFF TRAINING

24. Training of staff on reservoir safety monitoring is basically provided on the job. In the course of working together with the inspecting engineers and through discussions with them, the staff members are exposed to the latest concepts and thinking in this field. Basic understanding of the design and functions of various elements of the reservoirs and the instruments installed can be acquired from the inspecting engineers' reports which set out their observations, deliberations, findings and recommendations. Arrangements have also been made in the past for the inspecting engineers to share their experience with the Department staff, not just confined to the Reservoir Safety Section, in the form of a series of talks.

CONCLUSION

25. The Water Supplies Department of Hong Kong has established a practice of engaging a qualified panel engineer about every 5 years to carry out detailed examinations of the reservoirs in the territory. The task of routine monitoring and surveillance undertaken by the Department is a continuation of the inspecting engineer's work.

26. While a lot of guidance is provided by the inspecting engineer, successful monitoring can only be achieved with the persistent vigilance and adequate experience of the staff doing the job. Some of the reservoirs may remain almost in the same condition for many years before any significant changes can be observed. It is natural that attention to these reservoirs will tend to become slack after some time. Systematic surveillance, data collection, recording, processing and analysis are therefore essential to ensure that the safety conditions are always sufficiently understood and nothing significant are overlooked.

ACKNOWLEDGEMENT

27. The author wishes to thank the Director of Water Supplies of Hong Kong for permission to present this paper and to the staff of the Reservoir Safety Section for their assistance and contributions to the preparation of this paper.

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TECHNICAL OVERSIGHT OF THE OPERATION AND MAINTENANCE OF RECLAMATION WATER PROJECTS FOR THE 21st CENTURY

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SYNOPSIS

In the last 85 years, the United States Bureau of Reclamation has constructed 715 storage and diversion dams including over 70,000 miles of canals, pipelines, drains, tunnels, and laterals. Many of these projects are nearing 100 years of age and sound, farsighted O&M (Operation and Maintenance) standards must be applied to ensure satisfactory operation as these facilities enter their second hundred years of existence. To ensure that the management, operation, and maintenance of these facilities is adequate, Reclamation has established a number of technical oversight programs including the RO&M (Review of Operation and Maintenance), Rehabilitation and Betterment, Water Systems Automation, Pesticide, and others that are to be discussed in this paper. In addition to these programs, Reclamation conducts workshops and publishes operating standards.

TECHNICAL OVERSIGHT OF THE OPERATION AND MAINTENANCE OF RECLAMATION WATER PROJECTS FOR THE 21ST CENTURY

by

Joseph L. Miller 1/ and Fred J. Gientke 2/

1. The Bureau of Reclamation has instituted a number of operation and maintenance oversight activities to ensure that Reclamation and water-user operated projects are operated and maintained in a manner that protects the Federal investment. In addition to these oversight activities, Reclamation conducts training on the operation of dams and a workshop on Water Systems Operation and Maintenance. Technical manuals are also being prepared or are published which discuss field examination guidelines, herbicides, pesticide safety, guidelines for preparation of standing operating procedures, and others. The major programs are discussed below.

RO&M (REVIEW OF OPERATION AND MAINTENANCE) PROGRAM

2. Under the RO&M program, technical specialists examine about 100 major structures and about 100 carriage and distribution systems annually. These examinations are performed to determine and document the general condition and level of maintenance being performed on storage dams, diversion dams, tunnels, canals, pumping plants, powerplants, pipelines, and other structures on Reclamation projects. Most of these facilities are operated and maintained by water-user organizations under contract with the Bureau. Since title to these

facilities remains with the Federal Government, scheduled examinations enable the Bureau to fulfill its obligation to protect the Federal investment. Deficiencies in operation and/or maintenance are documented and pursued until corrective measures are implemented. In this program, scheduled preventative maintenance and full cycle testing of mechanical equipment is stressed and encouraged.

R&B (REHABILITATION AND BETTERMENT) PROGRAM

3. The R&B program, as authorized by the Rehabilitation and Betterment Act of 1949 and by special legislation, is administered by the Secretary of the Interior through the Bureau of Reclamation to facilitate the R&B of irrigation systems on projects governed by Federal Reclamation law and projects constructed under the Small Reclamation Projects Act. Federal funds are loaned to water-user organizations for the repair, replacement, or improvement of irrigation structures and systems which have become obsolete or deteriorated to the extent that improvement costs are more than can be funded by the water-user organizations.

4. Benefits that can be derived from an R&B program include better use of the project water supply, improved water distribution through modern control structures and measuring devices, reduction in frequent and expensive maintenance, improved safety for operating personnel and the public, and environmental enhancement. Since the program began, 108 R&B programs have been approved.

5. Although front-end capital costs for an R&B program may be high, the long term benefits and savings in water and related O&M costs can be substantial and depend on the existing irrigation system efficiency and other factors.

WMC (WATER MANAGEMENT AND CONSERVATION)

6. The WMC program was developed as a nonstructural means to promote improvements in project and onfarm water systems and management practices. Authority to conduct the WMC program is contained in the general provisions of the Reclamation law which authorized the Secretary of the Interior to plan, construct, and operate works for the storage, diversion, and development of water in the 17 Western states and Hawaii. The WMC program is concerned with the privately owned lands served by Reclamation projects.

7. In addition to reducing water use and losses, program objectives include energy conservation and the protection and enhancement of water quality. Considerable emphasis is placed on water measurement and accounting, evaluation of irrigation efficiencies, and evaluation of the program itself. State-of-the-art minicomputers and associated software are used in implementing these objectives.

8. The program includes three general activities as follows:

1. Developing or adapting new technologies for efficient use of water and energy and protection of water quality in irrigation operations.

2. Promoting the use of improved technology and management practices by water users.

3. Providing technical services and assistance to water-user organizations and to allied Reclamation programs.

9. The Bureau has trained technicians and management support personnel in all regions and in a number of project offices to assist districts in developing water management programs. During the most recent irrigation season, about 40 districts were developing or operating irrigation scheduling and WMC programs with assistance from the Bureau.

10. A prime objective of these cooperative programs is to establish "system scheduling" techniques which incorporate onfarm irrigation scheduling with the operation of the distribution system. The Bureau has increased system scheduling activities within a number of these districts and plans to demonstrate to others that scheduling onfarm demands with distribution system operation is an integral part of the WMC program. More than 100,000 acres are involved in system scheduling programs. An irrigation guide service that provides the daily evapotranspiration rate for the major crops grown in an area is used for more than 2 million acres, with over 90,000 acres receiving a detailed field-by-field irrigation schedule.

RO&M Guidelines

11. The amount of maintenance required on irrigation facilities generally increases with age. However, if proper preventive maintenance measures are taken, costly breakdown maintenance can usually be avoided.

12. It has been shown that visual monitoring and formal examinations (such as those conducted under the RO&M program) performed on a regular basis are valuable and economical aids in ensuring the safety and long life of structures and in avoiding costly breakdown maintenance. Such periodic monitoring and examinations can allow the examiners or operating personnel to evaluate the need for preventive or corrective O&M measures and to make a generally accurate assessment of the overall condition of the structures.

13. The Bureau of Reclamation has published field examination guidelines to provide professional personnel with a technical overview and general instructions to follow during field examinations conducted under the RO&M program for such structural and equipment components and features as dams, spillways, gates, valves, canals, tunnels, etc. Applicable portions of these guidelines can also be utilized by operating personnel and other individuals responsible for the O&M of an irrigation project or facility. These guidelines provide the minimum observations (important and critical) which are to be made during onsite RO&M examinations. Other observations may be necessary at specific structures or facilities to properly assess their condition.

14. Incorporation of equipment service life data or other similar information into the guidelines would enhance the document and provide irrigation operators with criteria to plan replacements.

Training for Dam Operators

15. The Bureau requires that all dam operators receive formal training on the proper operation and maintenance of a dam. In addition to this, refresher training is required every 3 years.

16. Several objectives are sought in training dam operators. They need to be acquainted with the full range of operation that is required for dams of all sizes. This general overview should encourage an appreciation of the interrelated character of projects and provide a broader perspective on operation and maintenance of dams. More specifically, onsite training is a necessary adjunct to ensure that adequate and safe O&M procedures are followed at the facilities for which each operator has responsibility and that the chance of operator error is minimized. Thus, the training is two-part: general classroom exposure, and onsite at the dam operator's work area.

ERF (Emergency Reserve Fund)

17. The purpose of the ERF is to ensure that operating entities maintain adequate financial capability to meet unforeseen extraordinary O&M costs; extraordinary repair or replacement costs; and betterment costs (in situations where a recurrence of severe problems can be eliminated) during periods of special stress caused by damaging droughts, storms, earthquakes, floods, or other emergencies causing interruption of, or threatening to interrupt, water or power service.

18. The amount of the ERF required will, in part, depend on the size and complexity of the project and the type of facilities and conditions involved. Projects with long supply canals, or which are dependent upon pumping plants, will require a larger ERF than ones with relatively short canals, pipelines, and/or simple gravity works. In general, the ERF requirement will be related to the annual O&M expenses. In line with that objective, the initial amount of the ERF for amendatory or supplemental contracts being sought by existing or potential contracting entities could be based on the 5-year average annual O&M costs (actual or representative project O&M costs), or adjusted to the current cost level excluding pumping plant power costs (see table below). Care must be taken to assure that 1 year of unusually high or low costs, for whatever reason, do not drastically impact the final determination. For new projects, the operation, maintenance, replacement, and energy cost estimate prepared for the planning report will be the basis for the amount of the initial ERF.

19. The magnitude of the fund should reflect consideration of the financial, manpower, and equipment resources of an operating entity to cope with emergency situations. Important considerations, besides the complexity and costs, relating to the size of the ERF are topographic, weather, watershed, seismic factors, and the condition and vulnerability of the project works. An analysis of the above may indicate that the suggested maximum reserve shown in the table is not sufficient and exceeding that amount is permissible. By the same token, the minimum reserve suggested could be excessive and a lesser fund applicable. If available, actual repair costs of recorded emergency situations could dictate the magnitude of the fund. Judgment should be based on case-by-case conditions.

20. Contracts should provide for such funds to be accumulated as rapidly as possible considering estimated payment capacity, O&M costs, repayment obligations, miscellaneous sources of income, and up-front financing plans. Accumulation should begin with contract execution, or as soon thereafter as practical, and be accomplished within the first 10 years of the repayment period. Contracts should also require that funds be maintained in interest-bearing accounts and provide for accumulation of interest. Further, contracts should contain provisions for periodic review and adjustment and require an annual status report on the fund from the contractor. The current standard reserve fund article, which contains such provisions, should continue to be used unless there are unique conditions that warrant revision of the article.

Emergency Reserve Fund Determination Guideline

5-year Average Annual O&M Costs*	Minimum Reserve	Maximum Reserve
20,000	20,000	30,000
30,000	25,000	40,000
50,000	35,000	50,000
75,000	45,000	70,000
100,000	55,000	85,000
150,000	70,000	110,000
200,000	75,000	120,000
300,000	80,000	155,000
400,000	100,000	200,000
500,000	112,500	225,000
1,000,000	200,000	400,000
2,000,000	300,000	700,000
3,000,000	400,000	900,000
4,000,000	500,000	1,000,000

* Exclusive of water and pumping power costs.

21. The introduction of probabilistic methods in this program could reduce the amount of the ERF that needs to be maintained by the water user.

Water O&M Bulletin

22. The Water Operation and Maintenance Bulletin is a technical publication prepared by the Bureau of Reclamation for use by its personnel and water-user groups for maintaining project facilities. The principal purpose of the bulletin is to disseminate Reclamation standards, criteria, and directives on acceptable operation and maintenance practices for storage and diversion dams or structures, conveyance facilities, and other appurtenant works. The bulletin is available for overseas irrigation system managers and operators.

SOP'S (Standing Operating Procedures)

23. Bureau of Reclamation SOP's for dams and reservoirs are prepared to establish in one primary controlled document--with associated supporting documents--the complete, accurate, current, and structure-oriented operating instructions for each storage reservoir and its related structures. The

purpose is to ensure adherence to approved operating procedures over long periods of time and during changes in operating personnel. The instructions also will permit responsible persons knowledgeable in reservoir operation, but unfamiliar with the conditions at a particular dam, to operate the dam and reservoir during an emergency situation and at times when regular operators cannot perform their normal duties.

24. To ensure compliance with operating procedures and regulations, the Bureau has issued a guide for the preparation of SOP's which gives the essential requirements. Some of which are:

- An Emergency Preparedness Plan.
- A Communications Directory.
- General information and instructions concerning administration of the dam.
- Electrical, mechanical, and structural information as to detailed descriptions and instructions for operation and maintenance of the dam and its appurtenance structures and equipment.
- Special instrumentation at the facility; and, if instrumentation installations are significant, the extent of installed instrumentation, monitoring, and maintenance requirements.
- Detailed instructions and information on all aspects of reservoir operation.
- Drawings, maps, photographs, charts, copies of selected supporting documents, and related reference material.

25. The SOP is prepared primarily for operating personnel located at or nearest to the dam and their immediate supervisors who are assigned the responsibility for the physical operation and maintenance of the dam. As a minimum, the SOP should contain all information and instructions necessary for operators to perform their duties.

<u>Case Studies of O&M Incidents at Reclamation Facilities</u>

26. This manual consists of a set of case studies of exceptional and extraordinary O&M incidents which have occurred at Reclamation's major facilities. These case studies do not represent a full range nor all of the incidents experienced at Reclamation projects. Rather, they represent a selection of the exceptional problems documented in the E&R Center's Water O&M Branch files. These files consist of RO&M Examination Reports, Travel Reports, SEED Reports, Project Data Books, and other sources of information. The review team provided additional references which are noted.

27. The purposes of this O&M manual are as follows:

1. To provide an easy-to-access information data base for designers and O&M personnel on significant operational problems and incidents.

2. To describe symptoms of the problems to enable engineers and O&M personnel to recognize similar problems that may repeat at other projects.

3. To be aware of design, construction, operation, or maintenance practices which should be avoided.

28. The case studies should serve as a quick reference for future problems of parallel situations as those documented which may be readily recognized, diagnosed, and solved using this manual. The manual should prove useful to O&M personnel, designers, planners, and reviewers. The information in each case study has been summarized and readers who desire additional information can contact the Bureau of Reclamation's Water O&M Branch in Denver, Colorado.

Pesticide Program

29. Extensive infestations of obnoxious terrestrial plants and certain aquatic animals cause problems on irrigation systems such as reduction in carrying capacity of the system, increased evaporation and seepage, clogging of structures, adverse environmental impacts, and water loss through transpiration. Because of the many problems created by these growths, steps are usually necessary to control them or prevent their occurrence.

30. The methods employed to control these plants and animals differ widely due to differences in the organisms such as growth habitat, organism metabolism, size, growth stage, and genetic characteristics. Correct identification of the organism is therefore important to the proper control recommendation.

31. In support of the above, three publications have been published to provide answers to the growing concerns of pesticide management:

1. Aquatic Pests on Irrigation Systems
2. Herbicide Manual
3. Pesticide Safety Manual

Water Systems Operation and Maintenance Workshop

32. Each year, the Water O&M Branch conducts this workshop for staff from Reclamation offices, operating projects, and private water districts. The subjects include pump maintenance, water systems automation, water measurement, and a variety of other irrigation system subjects. Experienced speakers from the west are invited to conduct and lead the sessions. Also included is a tour of Reclamation's laboratory facilities in Denver.

O&M Research

33. About $1,500,000 per year is being budgeted for about 36 research programs related to O&M activities. Examples of these research projects include investigations of open and closed conduits, ground water well design, foundation drain rehabilitation, mechanical equipment systems canal lining, cathodic protection, herbivorous fish, and other areas.

Artificial Intelligence

34. About $1,000,000 per year is budgeted for the development and implementation of a center for advanced decision support systems within the

Bureau of Reclamation. At the present time, the Bureau has developed several prototype applications such as well design and seepage investigations for embankment dams.

Mechanical Systems Investigation and Replacement

35. Over the last several years, Reclamation has been investigating and temporarily modifying its large diameter needle valves. These valves were designed and installed many years ago and have exceeded their operational life. They are also difficult and expensive to maintain. As a result, about 48 needle valves are being replaced over a 5-year period with jet flow gates. This program is considered the first of a number of related investigations of large mechanical systems to determine if they should be modified and/or replaced. Similar investigations are anticipated or are occurring in drum gates, sleeve valves, and other areas.

Landslide Surveillance

36. Each year Reclamation's water projects throughout the west report on the status of known landslides in their project areas. Information collected includes the size and rate of movement of the mass, potential impact on project facilities, and other data. This information is intended to maintain an awareness among operating personnel of the dangers associated with landslides and minimize damages to project facilities and/or loss of life.

Vegetation Clearance Criteria

37. Reclamation recently developed comprehensive vegetation clearance standards which require that a clearance zone of from 25 to 50 feet be maintained around dams, structures, canals, and other facilities. The purpose of these criteria was to ensure that vegetation does not damage the facility, conceal seepage, or in any way contribute to other types of operational problems.

O&M Organization

38. Development and implementation of the above programs is the responsibility of Reclamation's O&M organization. Generally, Bureauwide technical policy is finalized at the E&R Center with the assistance of the regional and project offices. The direct and daily operation and maintenance of Reclamation irrigation projects is the responsibility of either Reclamation or a private irrigation district. Typically, multipurpose projects are operated by Reclamation and single-purpose facilities are transferred to the water users to operate. Whether or not the facility is operated by a Reclamation project office or a water user, the facilities are reviewed frequently to ensure that the operation and maintenance is satisfactory and in accordance with Reclamation standards. These reviews are conducted by the project offices (annually), the regional office (every three years), and by the E&R Center (every 6 years). Responsibility for the satisfactory operation of all facilities remains within the regional offices whereas the E&R Center staff has technical oversight responsibility.

CONCLUSIONS

39. In addition to the above programs, Reclamation also manages other O&M programs such as early warning systems, hydromet, mechanical equipment testing, and other programs. These programs as well as the 16 discussed above are designed to minimize operational interruptions at Reclamation and water user-operated facilities. Although the dollar benefits of these programs are difficult to quantify, these programs are known to positively contribute to well being of irrigation facilities in the west.

THE SUPERVISION OF NEW RESERVOIRS
A COMPARISON BETWEEN FLOOD ALLEVIATION AND WATER SUPPLY

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SYNOPSIS

Southern Water has four reservoirs in Kent covered by the Act, a major source for water supply and three for flood alleviation. All have been completed within the last 14 years. The wide variety in method of operation and importance of the different types of reservoir is described, together with differences in the flood standards adopted for the flood alleviation reservoirs, and their method of operation, these aspects influence the timing, content and frequency of the Supervising Engineer's inspections.

THE EMBANKMENTS

1. The size, whether length or height, of the Authority's embankments in Kent that are covered by the Act vary greatly and are shown for comparison in Table 1.

Table 1 - Reservoir Basic Data

Location	Purpose	Construction	Crest Length m	Maximum Height m	Capacity Ml	Flood Control Device
Bewl Water, Lamberhurst	Water Supply	Clay Core Sandstone Shoulders	928	32	31,400	Circular Drop Spillway
R. Medway Scheme, Tonbridge	Flood Allev-iation	Clay Core Granular Shoulders	1300	4.7	5,580	Automatic Radial Gates
Hall Place, Bexley	Flood Allev-iation	Clay	366	3.2	132	Siphon (Inlet) Penstock (Outlet)
Lamorbey Park, Bexley	Flood Allev-iation	Clay	175	2.4	56	Fixed orifice culvert/notch

2. The method of dealing with flood control is different in each case, three relying on fixed devices and one on automatic control by electrically-operated gates. All the flood alleviation embankments are designed to allow overtopping when the design flood is exceeded, Bewl Water is not.

3. The flood alleviation reservoirs are all designed to relieve urban and industrial areas from flooding but to different return frequencies and have catchment areas significantly different in both size and character (Table 2).

Table 2 - Flood Alleviation Reservoirs

Location	Design Return Period years	Catchment		Peak Discharge
		Area hectare	Type	m^3/s
R. Medway Scheme	100	54200	rural	181
Hall Place	30	river flow above $12.2m^3/s$		4.5
Lamorbey Park	30	1104	urban	8.5

4. Hall Place and Lamorbey Park receive water from urban catchments with short times of concentration and the rapid response to summer storms, in particular at Lamorbey Park, has resulted in the maximum stored levels of water being only rarely observed at that site. In contrast, the River Medway Scheme has a large rural catchment and upstream rain gauges and river level monitors give twelve hours or more notice of flood impounding; while at Bewl Water, a pumped storage reservoir with a small catchment area of 1900 hectares, the impounding from the natural catchment has not yet caused the spillway to be used in anger.

CONTROL DEVICES

5. The smallest flood storage reservoir, at Lamorbey Park, has the simplest control, an armco culvert discharging through a rectangular notch into a small stilling basin. At Hall Place, flow into the closed storage area is via a siphon - the peak of the flood is siphoned from the river and discharged after the peak of the flood has abated by manually opening an outlet penstock. Both these reservoirs are small and have limited capacity to control the floods.

6. A different approach was necessary at Tonbridge, (1) where the objective is to actively intervene and control flood events up to the 100 year return period without overtopping the embankment. Here three electrically-driven radial gates, with a capacity to discharge a controlled flow of up to 150 m^3/s each, operate automatically via water level sensor controls to keep the discharge to a preset value. The preset discharge depends upon the predicted value of the peak flood derived from a mini computer.

7. The drop shaft spillway at Bewl Water has been artificially tested to obtain the Construction Engineer's final certificate and has been designed to pass the maximum inflow to the reservoir with a nominal rise in water level above the cill.

8. The design emphasis has been to limit operational problems and maintenance liabilities wherever possible by the use of a 'fixed orifice' device for passing flood flows. Trash screens which guard the entrances to two of the control devices are regularly cleared and no other routine work

is necessary, at Bewl Water the entrance to the spillway crest is protected by a ring beam whose soffit becomes submerged befor T.W.L. is reached.

9. Table 2 indicates the significant difference in the peak discharge for the design return period for the flood alleviation reservoirs, and these must be considered in relation to the downstream channel capacity. The urban schemes allow for channel flows to be kept within banks at the design flood whereas at Tonbridge this is only the case up to about the five year return period; hence the need to be able to vary the controlled discharge to maximise the use of the storage area and minimise flooding downstream.

INSPECTION

10. Southern Water uses a standard format for the Supervising Engineer's Report, which, no doubt like all other undertakers, has been designed for impounding reservoirs that continually retain water.

Flood Alleviation Reservoirs

11. The report format has to be adapted for the flood storage reservoirs as they depend upon an entirely different philosophy. They are usually empty, and retain some water for perhaps as little as part of a day only per annum, or at most fifteen/twenty days per annum.

12. It is this basic point which requires a fundamental difference in the Supervising Engineer's approach to his inspections. Quite apart from routine visits, the most important inspections for flood storage reservoirs take place during, and immediately after a flood event on the comparatively rare occasions when the embankments are in use. It is this aspect of supervision which is crucial to the proper operation of a flood reservoir and differs from the way in which impounding reservoirs are dealt with. In this respect, the Supervising Engineer knows that there will be a demand in the flood season for inspections in addition to the regular routine visits, which are not emergencies, but cover normal operation, and that he will be involved in the operation of the reservoir during an event alongside staff who have the responsibility for its day-to-day care.

13. Opportunity to assess the performance of this type of reservoir is very limited and every opportunity must be taken to monitor the embankment and the control devices under a head of water during this critical time. Generally, the embankment lies dormant across the valley with the control devices passing the base river flow and it is therefore impossible to consider the integrity of the embankment, except for surface condition, or the performance of the control devices, stilling basins or the associated gates/penstocks until operating under design conditions.

14. It is therefore necessary to regularly check the performance of all electrical and mechanical plant throughout their full range of operation, together with their back-up systems, both during and immediately before the anticipated flood season, which is broadly defined as 1st September to 31st April, to ensure that the reservoir functions without a hitch when required. This operation is onerous at Tonbridge where it takes a full day and several members of the operational staff to check standby generation start up, penstock and radial gate operation throughout their full travel (including hand operation) and their limit switches, accuracy of remote control probes and water level indicators and the functions of the control

panels and computer support. However, at Hall Place there are bar screens and a penstock only to check and operate and at Lamorbey Park a bar screen only.

15. The embankments and the structures are easily routinely inspected when the reservoirs are not in use. Conventional visual methods are employed, supported by annual settlement readings of the embankment crest and photographs of any defects noted, followed by comparison with earlier records. It could be argued that these inspections are of little value to the Supervising Engineer or the Undertaker since the interest of both parties is in design performance.

16. On the one hand, when the reservoirs are in use as a result of heavy rain, embankment inspection is complicated by surface saturation, a toe drain network that has submerged outlets and control devices sometimes partly blocked with debris; on the other hand, they are made simpler by the fact that the control devices are in use and their performance can be monitored and recorded.

17. I consider that the most important of the inspections follows at the end of a flood event. It provides an opportunity to inspect for damage - to check on the condition of spillways, stilling basins and channel armouring, to note whether the toe drain system is flowing, and also to follow-up malfunctions of automatic equipment if any have occurred during the event.

Impounding Reservoirs

18. This type of reservoir is under constant use and electrical and mechanical plant is monitored during its regular operation on a daily/weekly basis, and by the nature of its use, is less liable to cause problems provided there is a proper programme of maintenance.

19. The inspections are nonetheless time consuming, but while checking plant operation, the inspection concentrates on entirely different issues, which, as these do not involve operational staff, can be more easily carried out. The inspection, monitoring and record photography of the access and spillway tunnels and shaft, and the underdrain system and relief wells is more comprehensive at Bewl Water than at the other Kent sites as a result of the scale, size and purpose of the embankment and the maximum volume of water that can be stored. Hence the continuing need for monitoring of piezometric pressures using a system established during construction to check for embankment stability and to provide an early warning system. This work is carried out monthly by contract and monitored in-house.

New Reservoirs

20. All the reservoirs in Kent can be described as new in terms of an anticipated life of 100 years or more, with the oldest being completed in 1974, see Table 3.

21. The Supervising Engineer needs to be on his guard and to make a conscious effort to observe. It is all too easy to assume that because a scheme is new there is nothing that will need to be done to maintain the security of the embankments.

Table 3 - Reservoir Construction

Location	Construction	Engineer's Final Certificate
Bewl Water	1972-75	October 1982
R. Medway Scheme	1978-80	November 1980
Hall Place	1976	January 1977
Lamorbey Park	1973	February 1974

Reports

22. If the philosophy of paying attention to detail goes hand in hand with the well documented procedures for reservoir inspection, then the Supervising Engineer's reports will be of significant advantage to the Undertaker and his operational staff. The 1975 Act formalised the arrangements for monitoring dams, and in many instances set up for the first time a system of regular visits. This often revealed a legacy of neglect even to the point where there were no records of any kind and reports should be prepared such that this cannot happen in the future.

23. The reports must pay careful regard to everything, from the condition of concrete structures to the presence of all the coping bricks at a drain outfall, and from the condition of a draw-off tower, to the maintenance of a settlement point, and the amount of diesel in the fuel tank.

24. This attention to detail is not as time consuming on site as it might appear to be, although it can make annual reports rather lengthy and there is a need to concisely summarise items for action regularly throughout the reports. These summaries enable operators to plan the remedial work rapidly and to have the advantage to follow through a series of reports to project a trend that can result in future expenditure being planned and the appropriate allowance made in the budget.

25. I do not hesitate to report one copying brick missing from a brick headwall, or to describe and photograph a crack in a concrete wall, or to comment on even a short length of french drain that is becoming grassed over with a recommendation that either remedial work or further investigations should be undertaken. If items of this kind are left out of reports their condition can only deteriorate and ultimately the safety of the reservoir can be put at risk.

Floods

26. Floods often occur at the least convenient times and have a habit of being at their worst at night or over weekends/public holidays. The timing of the arrival of the design flood cannot be predicted. Unlike a conventional impounding reservoir the flood alleviation reservoirs are not in continuous use and have to reach their full design performance very

rapidly - in Kent, this time scale can vary from a few hours to about 36 hours depending upon the storm and the catchment and reservoir characteristics.

27. In order to deal with the controlled outflow from the storage area, the control devices, stilling basins and protective works all have to be in a fit condition or the integrity of the embankment may be affected and it is for this reason that any structure or items of plant required to deal with flood flows have to be monitored in detail, particularly when the flow could reach 300 m^3/s.

28. In principle, there is no difference between the type of reservoir in need of comprehensive supervision and inspection, although the impounding reservoir enables this work to be carried out while continuously performing to its design requirements. Perhaps this is an advantage in that defects of a serious nature may develop and be monitored over a long period enabling careful consideration of remedial measures. A serious defect occurring in a large flood alleviation scheme however could be catastrophic, and this is a major influence on the Supervising Engineer's involvement on site during a flood event.

FLOOD EVENTS

29. The Kent Division responds to a warning of a flood event (or emergency event at Bewl Water) from a central control room monitoring rainfall via telemetry and weather radar. Operational staff man local control rooms and notify the Supervising Engineer who is on permanent call.

30. The response by the Engineer is to inspect at the commencement of impounding, and thereafter at least once every 24 hours, with a final inspection once base river flows are returned. The Supervising Engineer's duties take precedence over normal duties.

THE UNDERTAKER

31. Southern Water has taken a positive view of the 1975 Act and indeed implemented it for some reservoirs before required to do so by statute. Appropriately qualified staff were actively encouraged to submit themselves for appointment to the Reservoirs Act Panels and the decision taken that the Authority would carry out supervision "in-house". Difficulties could have arisen from this decision in implementation where the Supervising Engineer would be cutting across his normal day to day channels of communication, making comment on operational matters outside his normal work and going above people's heads.

32. I have always approached reservoir supervision with three basic intentions, firstly, to keep the operations staff aware of what I am doing and involving them during inspections/visits as often as possible, secondly, to discuss the final draft report with the managers responsible for the reservoirs so that they are aware of its contents, and thirdly, to submit the final draft report unamended to the Divisional Manager only, as the Undertaker, for his action on the recommendations in the report. The independence of the Supervising Engineer is acknowledged and actively endorsed by the Divisional Manager.

33. This approach has been successful since a working relationship has now been built up with operational staff, who, after being initially somewhat apprehensive of the Supervising Engineer and his powers, now see the advantage of regular independent inspection. The Authority's approach to safety in its widest sense has helped by giving an awareness to the operations staff but they now realise that the Supervising Engineer can be of great use. He is available to provide technical input into operational problems, as and when they occur, which was not previously available, he provides technical support on site during a flood event and also provides an additional member of staff but with special training and skills.

34. The reports can also make funds available for maintenance work to be carried out earlier than would perhaps have been the case, to require the use of outside specialists for investigation purposes and to ensure special arrangements are made on a regular basis (albeit perhaps at 3 or 5 yearly intervals) to carry out checks which would otherwise not be possible to undertake using the Undertaker's own staff.

35. The relationship between the operational staff and the Supervising Engineer is a delicate one and has to be constructed carefully to produce a team effort. It would be easy for the Engineer to give the impression of taking over everything and for operators to become reluctant to involve the Engineer. The team approach takes time but once established has the great advantage of openness with a free exchange of views and mutual support for a common objective.

Future Works

36. Southern Water is currently seeking Parliamentary Powers by way of a Private Bill for two further category 'A' flood storage reservoirs near Ashford to alleviate flooding in the town centre. Each reservoir will control floods up to the 100 year event by means of a fixed orifice discharge controlled by a Hydrobrake. More severe events are catered for by a grass reinforced spillway which will allow overtopping to take the excess inflow up to the P.M.F.

Reference

37. The Control Structure of the River Medway Flood Relief Scheme K.J. Shave and M.F. Kennard International Conf. 1984 Univ. of Southampton.

Acknowledgement

38. The Author acknowledges the support of B.A.O. Hewett, Divisional Manager, Kent Division, and his permission to present this paper and reproduce details of the Authority's reservoirs in Kent indicating the approach of this Undertaker to reservoir supervision.

BNCOLD CONFERENCE 1988

THE SUPERVISING ENGINEER AND THE RESERVOIRS ACT 1975

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THE SUPERVISING ENGINEER AND THE RESERVOIRS ACT 1975

A K HUGHES BSc PhD DMS CEng MICE MIWEM MBIM
GROUP MANAGER, NORTH WEST WATER

SYNOPSIS

The paper presents information obtained from a survey of Enforcement Authorities and outlines some of the problems experienced by Enforcement Authorities and Supervising Engineers since the implementation of the Reservoirs Act, 1975.

INTRODUCTION

1. Dale Dyke (Bradfield) dam failed in 1864, with heavy loss of life and damage to property. The Coroner's jury, at the conclusion of the inquest on the 245 victims suggested that all dams and reservoirs should, by an Act of Parliament, be subject to "frequent, sufficient and regular" inspection. In the following year a Parliamentary Select Committee was set up, but unfortunately no Parliamentary action was taken for over 60 years.

2. Just prior to the failure at Dale Dyke, a dam at Holmfirth (Bilberry) had failed and resulted in 18 fatalities, heightening public awareness to the dangers posed by dams and reservoirs.

3. Apart from Common Law it appears that the only legislation which was specifically written for reservoirs, was the Waterworks Clauses Act, of 1863, sections of which gave a concerned party the power to complain to two justices of the peace.

4. The events at Dalgarrog and Skelmorlie in 1925 are now well known, but it was these events which prompted Mr. Edward Sandeman to express his concern over reservoir safety in a letter to The Times in December 1925. Under the pressure of public concern following the St. Francis disaster in 1928 and no doubt following Mr. Sandeman's letter, the Reservoirs (Safety Provisions) Act, 1930 came into being. Public and professional awareness was again raised in the 1960's when a number of international failures occurred. (Malpasset, Vaiont, etc.) and following the ICOLD Congress in Edinburgh (1964) an ad hoc committee of the Institution of Civil Engineers was formed. This reported in 1966 and sowed the seeds of the Reservoirs Act, 1975, which as we know was not implemented until 1986/87.

5. A "potted history" of reservoir legislation is given below

1845	Bilberry dam fail	-	18 killed
1863	Waterworks Clauses Act		
1864	Dale Dyke dam failure	-	245 killed
1865	Select Committee		
1925	(Apr) Skelmolie failure	-	5 killed
1925	(Nov) Dalgarrog failures	-	16 killed
1925	(Dec) Mr. Sandeman wrote to 'The Times'		
1930	Reservoirs (Safety Provisions) Act, 1930		
1960	'International Concern' over reservoir safety - following Malpasset, Vaiont etc.		
1966	Report of the Ad Hoc Committee on Reservoir Safety		
1969	Lluest Wen Incident		
1970	Warmwithens failure		
1975	Reservoirs Act, 1975		
1986	Implementation of Reservoirs Act, 1975/87		

GENERAL CONSIDERATIONS

6. As one might expect, following drafting of the Act, and prior to implementation many criticisms were made but in general the Reservoirs Act, 1975 was seen to be an improvement on the Reservoir (Safety Provisions) Act, 1930. Some of these criticisms of the 1930 Act included;

(1) The exclusion of canals

(2) The number of dams to which its provisions applied was not known.

(3) The Inspecting Engineers' recommendations could not be enforced.

(4) Dams may not be examined by anyone with any knowledge of dams for up to ten years.

(5) Details of dams, inspections etc. should be kept by a Government Department in a central registry.

7. The Reservoirs Act, 1975 addressed most of these problems but some issues have yet to be resolved and many feel that improvements can still be made. Obviously still excluded are canals. A further criticism perhaps can be levelled at the Act in respect of having 66 different enforcement authorities which have and do apply very different standards of enforcement.

8. Still we apply reservoir legislation arbitrarily to reservoirs of a particular capacity with very little regard to the risk posed by the structure and its stored contents.

9. The Central Registry was not adopted although the Department of the Environment are developing a database of information on incidents/accidents and this study, described later, has established a database of information on the 'stock' of reservoirs.

10. We still do not know, and will not know until all registers are complete, the number of dams to which legislation applies although we now have a much better idea.

SURVEY OF ENFORCEMENT AUTHORITIES

11. A survey of all enforcement authorities was carried out in late 1987 and early 1988. Although not quite complete at the time of writing this paper, responses were obtained from the majority of Enforcement Authorities. They were asked to outline any problems experienced in implementing or enforcing the Reservoirs Act, 1975 as well as provide information from their Register of Large Raised Reservoirs. Appendix 1 gives details on the information registered.

12. The majority of Enforcement Authorities gave the information willingly, some after greater explanation of the need for the information and the use to which it was to be put, most supplied the information without charge - only two charged for photocopying.

13. Telephone enquiries to Enforcement Authorities often proved to be difficult. Many main switchboards or enquiry offices had very little knowledge of the requirement of their Authority to hold a register and certainly little idea of which section held and maintained the register.

14. In practice the majority of Authorities gave the task to their engineering department, (transport/sewage) while only a small number used the secretary and solicitor's department. Most felt it necessary to use sections which had engineering resources despite the DOE stating their role should be administrative.

PROBLEMS ENCOUNTERED

15. In general, it appears that few Enforcement Authorities have experienced problems in implementing the Reservoirs Act, 1975.

16. However, more than one Enforcement Authority experienced problems in ascertaining ownership. As one Enforcement Authority Engineer said - "Some private owners were sufficiently well informed about the Act to either deny all knowledge of ownership or disappear over the nearest hill!"

17. For those reservoirs where ownership could not be traced and the Enforcement Authority has 'adopted' the reservoir the Enforcement Authority worry is whether it has the ability to recover the costs incurred.

18. One Shire County observed a reluctance by private owners to employ Supervising Engineers or Inspecting Engineers on large ponds and lakes in rural areas where there is perceived to be no danger to life or property. In these instances initial enforcement has had to be taken.

19. Concern over silt lagoons and dredging grounds not being included in any legislation other than Health and Safety legislation and Common Law was expressed by one Enforcement Authority who had a large number of these structures within their area where the retained substance was described as 80% water and 20% silt.

20. Arguments over capacity appear to have been common with surveys by Supervising Engineers, by Consultants' staff, and jointly between Enforcement Authority and Undertaker all being cited.

21. Charges made by Supervising Engineers have been made on the basis of lump sums, time charges and a combination of both and it is true to say that charges have become very competitive.

22. The larger public undertakers (Water Authorities, BWB, CEGB, NSHEB etc.,) have usually selected supervising engineers from within their own staffs, whereas the private owner has more often than not used Consultants' staff. Most owners have recruited Supervising Engineers who are based 'locally' (i.e., within say 1 hour travelling time) where these structures pose a significant risk to life, but some structures are 'supervised' by engineers who are based or live as much as 250 miles from the site in England and in the North of Scotland the problem is more extreme.

23. Most Supervising Engineers have been recruited from the Supervising Engineers' Panel; only a limited number recruiting Supervising Engineers from other panels.

24. One Enforcement Authority is known to have retained a Panel AR engineer to interpret the recommendations of other Panel AR engineers.

25. Supervising Engineers appear to have experienced some difficulties carrying out their duties when a report under the Reservoirs Act, 1975 has not been written or when they have particular concerns. For those Supervising Engineers who frequently meet other Panel Engineers then informal contact is the obvious and common method of resolving problems without the Supervising Engineer having to call for another statutory inspection. For those not having this contact this facility will have to be developed with time.

Analysis of Results

26. Information on 2378 reservoirs/dams is currently held on the database. This is not the total number of large raised reservoirs in England, Scotland and Wales. It is estimated that there are approximately 2450 reservoirs/dams subject to the Reservoirs Act, 1975 at the moment. The information on the other 70 or so dams together with those for Northern Ireland as presented by Cooper (1) is being assembled to include in the analysis. It is true however, that the number of dams subject to legislation will be for ever changing as a result of new construction, discontinuance, and when other existing structures are discovered!

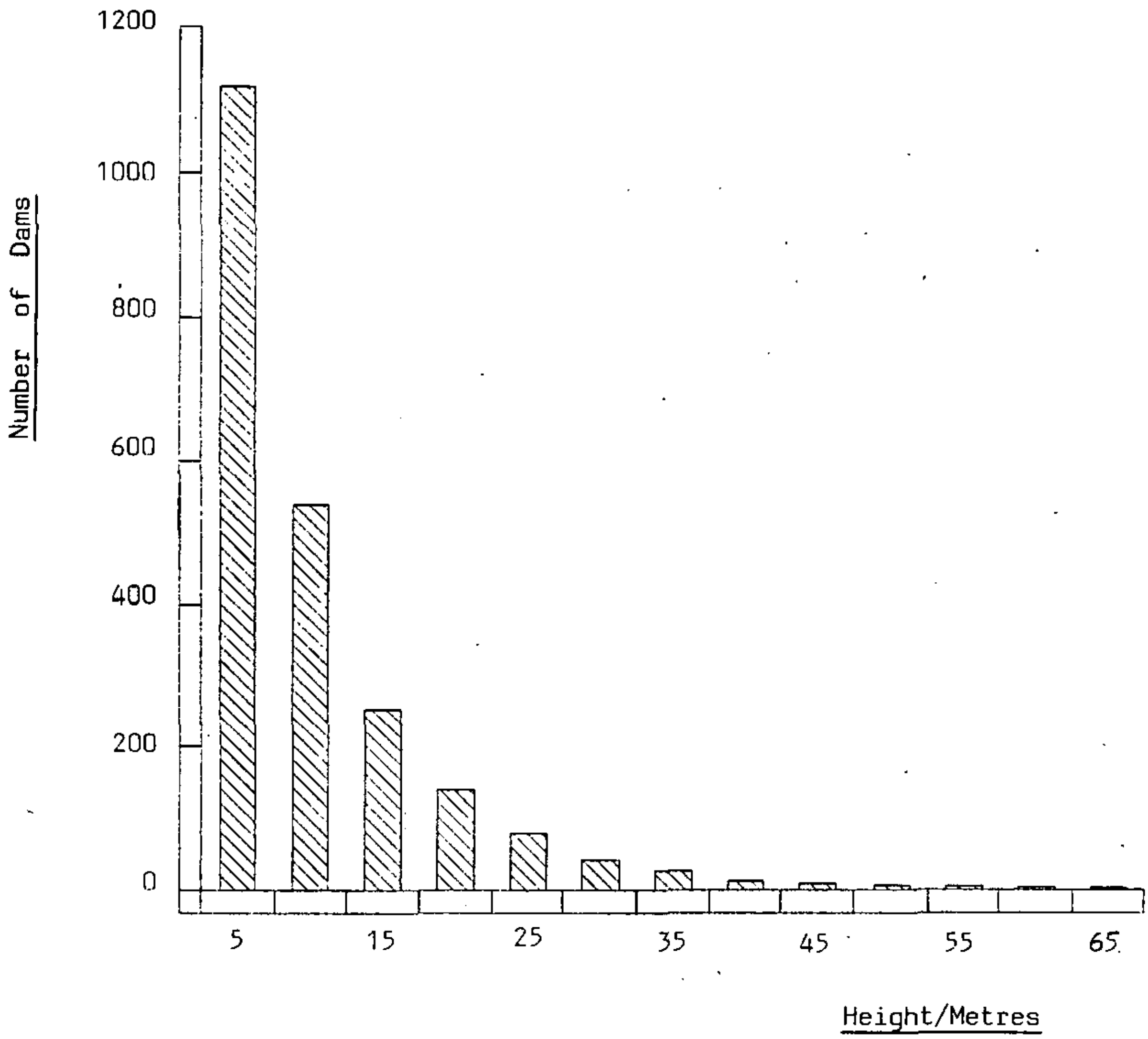

Figure 1 : Results of Analysis of the Height of Dams subject to Legislation

27. The majority of dams, some 83%, are classified as impounding and most, some 78% are of earthfill construction.

28. Figure 2 illustrates the steady growth of dams subject to legislation for the period from 1750 to the present day. Of the stock of dams 42% date back to the nineteenth century or even earlier with a steady growth over the period 1800 to 1930. The oldest dam on the register is stated to have been built in 1150 but many structures do not have a date of construction stated in the return of information.

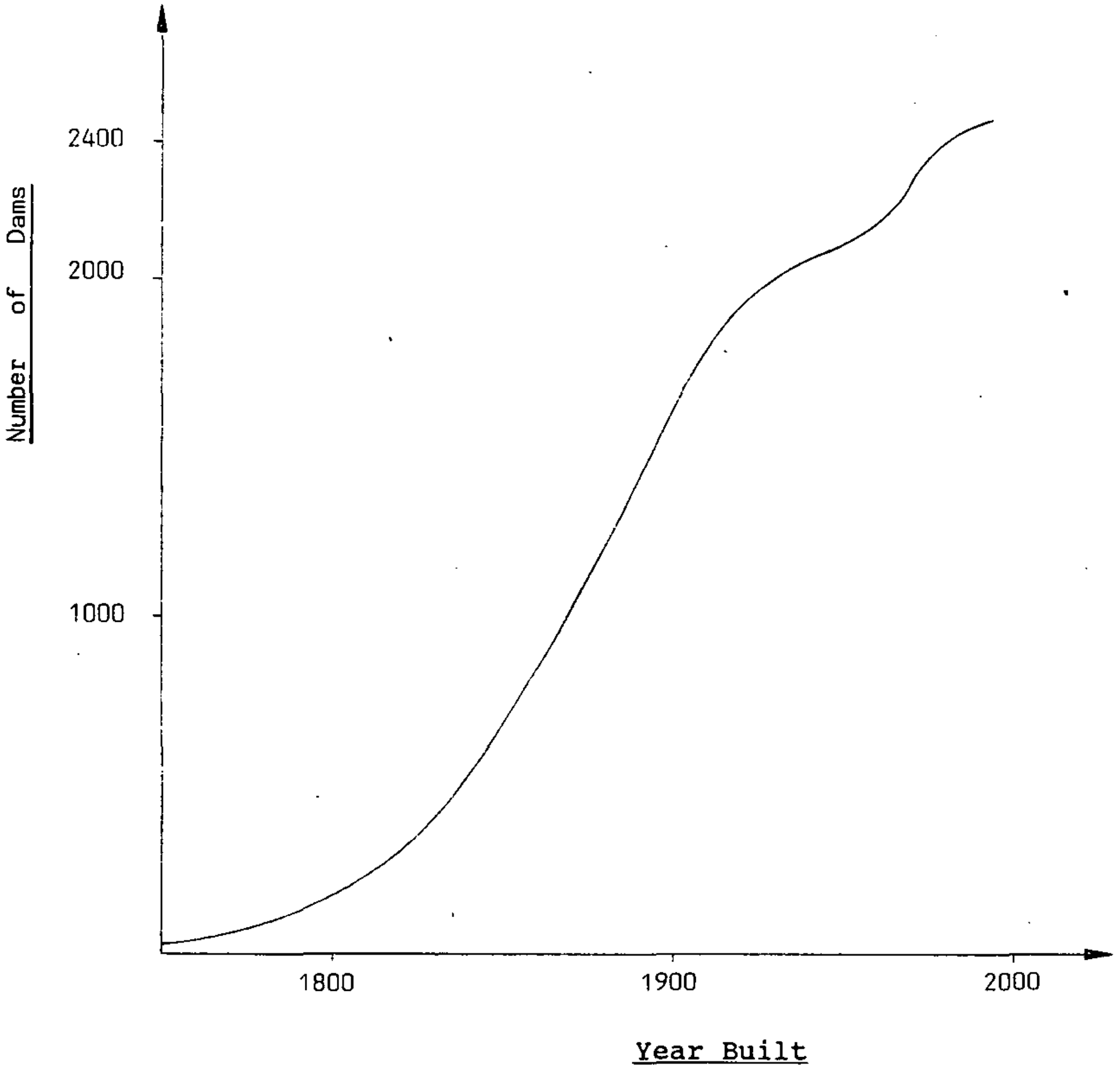

Figure 2: Cumulative 'stock' of dams

29. The average age of dams subject to legislation is now about 96 years.

30. About 70% of dams subject to the Reservoirs Act, 1975 are in public ownership - the remainder being owned by a variety of organisations in the private sector, ranging from large organisations to an individual householder.

31. A very large proportion (71%) of the stock of dams need not be subject to a statutory inspection for a period of 10 years following the last inspection, while 11% were given 5 years, and the remainder spread fairly evenly over the other years.

32. As one might expect, with the creation of the Supervising Engineer and improvements to structures/recommendations being carried out, this 10 year period between statutory inspections is becoming more common, perhaps because of the confidence that examinations by the Supervising Engineer will take place at least once a year.

OTHER MATTERS

33. The Health and Safety Executive have declared an interest in dams not subject to legislation. They are known to have asked owners of dams in the private sector, which are not subject to the Reservoirs Act, 1975, to prove that their dams are able to pass the design floods as defined - Table 1 of 'Floods and Reservoir Safety: an Engineering Guide'.

CONCLUSION

34. The compilation of registers is now largely complete, with very different levels of involvement by the Enforcement Authorities. Most dams and reservoirs are owned by the large public undertakers with most supervising engineers drawn from the staff of those organisations.

35. There remain a number of dams, either in private ownership or where ownership is unknown, which will continue to pose a threat to safety, due to lack of funds on the part of the owner or which will require the Enforcement Authority to carry out works with little or no chance of recovering the costs.

ACKNOWLEDGEMENTS

36. The Author wishes to thank Mr. P. L. Horton for his assistance in assembling the database of information from the registers and the subsequent analysis.

REFERENCES

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2. ICE Floods and Reservoir Safety; an engineering guide 1978

DISCUSSION: TECHNICAL SESSION 2

THE SUPERVISING ENGINEER

Session Chairman: W J Carlyle
Partner, Binnie & Partners

Session 2 is on supervision of reservoirs. We have Mr Earp from Binnie & Partners talking about reservoir supervision in the last couple of years. Mr Ku is responsible for monitoring the safety of the reservoirs in Hong Kong, and we are grateful to him for coming over and presenting such an interesting paper.

We have a paper by Mr Miller and Mr Gientke of the USSR. I don't believe anyone is here to present that paper, so it will just have to be taken as read and discussed. They present some useful and interesting ideas and bring in the possibility that they are going to take probabilistic risk assessment seriously as a means of reducing their emergency reserve funding. We have a paper on the flood reservoirs in Kent by Mr Shave, and Dr Hughes, on the enforcement survey details.

D J COATS (Babtie Shaw & Morton)

So that there are no lingering doubts that there is any disagreement between Dennis Earp and I: what I was saying was that by all means the supervising engineer should be in touch with the inspecting engineer in an informal way if that is necessary, but that no work carried out in the interests of safety should be carried out, or can be carried out as I understand the Act, without an inspection report to justify it, and I think we are stuck with that. When we require something done in the interests of safety then, as I understand it, you need an inspection report to actually ensure that it is done and to allow it to be done.

Secondly, this business of category. People now say: 'we've a Category A dam, a category B dam', whether it is anything to do with the spillway capacity or not; it seems to be some categorising which has been used and extended in a way that was never intended, and I think this is dangerous. In fact I think all categorisation of anything is dangerous, and I think codes of practice are dangerous in many senses.

The business of disagreements as to capacity. It's astonishing to me that we cannot agree whether it is 25,000 cubic metres and if in doubt it is out of the Act, in my view. For a particular reservoir I did an inspection survey with the reservoir empty and I came to 24,500m^3 at 3 inches below the spillway and 25,000m^3 at 3 inches above the spillway. I didn't check levels on the spillway, but I wrote saying that this was clearly not under the Act. The enforcing authority said: "Oh yes it is. We have done a different calculation using your survey and we come to 25.3 and therefore it is under the Act." This whole niggling, legalistic kind of approach which is creeping into engineering is something to be thoroughly deplored.

Could I ask Dr Hughes why he thinks there are works which should be carried out by the enforcment authority with little chance of getting any costs recovered under paragraph 35?

DR A K HUGHES (North West Water)

It's a case of the enforcement authority not being able to find an owner, having to carry out work under emergency powers, and not being able to recoup money from anybody.

P JOHNSON (Thames Water)

I'd like to express my thanks to Mr Shave for a very interesting paper and presentation. Thames also have quite a number of these flood storage reservoirs which as Mr Shave quite ably demonstrated are quite difficult to monitor because of the limitations to frequency of use and operation. I would ask how does he ensure that the supervising engineers actually get to the sites of the smaller flood storage ponds that clearly respond very quickly to urban storm rainfall conditions? Clearly, the Medway site cited by Mr Shave is monitored and controlled fairly well, but the smaller sites must pose major problems and it is causing difficulties for us at present to get the supervising engineer to the site at the right time. Can he comment accordingly?

K SHAVE (Southern Water)

It is a distinct problem, in that there are no men on site and there are no men based close to some sites, and it would be pure chance that the supervising engineer happened to be in the area when it was pouring with rain.

At Hall Place filling of the reservoir takes place when the River Cray reaches a certain flow. The storage area is filled by means of a syphon and the water is released after the flood has passed, so at that point you have an opportunity to go to the reservoir and inspect it and then say to your operations staff: 'Thank you very much, wind the penstock up and let it go.' At Lye with the large scheme there and the investment of something over £3½M in the late seventies, the case is different again. The supervising engineer's notification is triggered by a rainfall gauge or series of rainfall gauges: as soon as they indicate 18 mm over any 24-hour period that is the sign that the reservoir is likely to be used. We then have a snowball system from our emergency control room that notifies the operations staff who man the local control room there and myself that the reservoir is likely to fill.

A BIRTLES (Thames Water)

Referring to Mr Earp's paper and the frequency of visits by the supervising engineer. I feel he has another question to be taken into account: the arrangements which the undertaker has for operating and for his own surveillance of the reservoirs. I would not be satisfied myself that a supervising engineer's visit itself was adequate surveillance for any of the reservoirs which which I am a supervising engineer, and I need to be satisfied that arrangements are in force to ensure that surveillanace takes places between my visits.

Secondly, the question of the relationship with the inspecting engineer. My authority does retain inspecting engineers between statutory inspections so that they are available for ad hoc advice for the supervising engineers and this is a very profitable arrangement for both parties in a technical sense. The inspecting engineers also receive copies of the supervising engineer's reports to the undertaker and are invited to comment upon them.

Picking up Dr Hughes' challenge on the availablity of the supervising engineers, I believe that this is an important point and I think that the operators of reservoirs need to be able to contact and have on site quickly the supervising engineer upon whom they are relying.

D J KNIGHT (Sir Alexander Gibb & Partners)

I have the following questions for Mr Shave, relating to Paper 2.4, page 4, paragraph 19:

1 Do the designs have any particular reference to the repetitive undrained loading conditions involved, including rapid drawdown?

2 Is any instrumentation read during the presumably infrequent impounding periods?

3 How many times has impoundment taken place, and over how many years?

4 What conclusions have you drawn in respect of the attainment of an equilibrium piezometric state for the embankments?

K SHAVE (Southern Water)

Water is against the embankment for such a short time that changes would not be recorded. The reservoir might fill over 40 hours, remain full for 10 hours, and drain down over another 36 hours. We optimise the storage area and minimise inundation downstream.

M F KENNARD (Rofe Kennard and Lapworth)

Im my opinion these reservoirs come within the Act but even if not they should be properly designed. Hall Place is an off river storage pond and extreme events would be rare but the other two will I think be overtopped on occasions and this has been taken into account in design - even though they are category A dams the embankments are designed for rapid drawdown with gravel shoulders and a wide clay core. Hall Place has never been filled in the 11 years since construction. It is much easier to inspect a reservoir if it has been used and tested to full capacity. With regard to supervision I say in inspection reports usually at six month intervals and immediately following a flood event.

With regard to the paper on dams in Hong Kong I am pleased to see that computer plotting has been used to record and analyse instrumentation results.

C C D KU (Water Supplies Dept, Hong Kong)

The computer plotting was introduced two years ago and is still being improved. Training is provided by lectures given by visiting inspecting engineers and by contact with the inspecting engineer.

C J A BINNIE (W S Atkins)

Sometimes there is a problem of communication. I have had a situation where work was going on when a flood caused overtopping which caused a slip. Before I could get to the dam after being called by the Supervising Engineer the farmer had used a machine to reform the bank and made the situation worse before returning to market.

With regard to Mr Shave's dams you appear to have dams which have clay cores which stand empty for considerable periods. Have you found any evidence of cracking of these cores?

With regard to Mr Ku's computer programme I assume your Inspecting Engineers do stability analyses based on assumed hydrostatic and piezometric criteria. Do you feed these assumptions into your model so that you can get a direct comparison of assumed conditions with actual conditions?

C C D KU (Water Supplies Dept., Hong Kong)

We record all readings and feed them into the computer and check these against theoretical data.

K SHAVE (Southern Water)

On the Lye scheme we have gravel shoulders with a clay core and on the others they are all clay embankments. There is no evidence of cracking in the core or surface instability on any of the dams.

T A JOHNSTON (Babtie Shaw & Morton)

As an answer to Dr Hughes I believe supervising engineers should be as close as practicable. In one or two cases I know the distance is over 200 miles which is obviously not ideal but this is as close as practicable.

Mr Earp's paper highlights an unfortunate inconsistency regarding the attitude of enforcement authorities and inspecting engineers as to who can give approval to measures taken in the interests of safety. I wonder if he knows whether any enforcement authority has sought advice of the Department of the Environment giving some definitive advice on this. The reservoirs do not have any instrumentation at all. All are comparatively low: no more than about 4.5 metres and full for a very short period. They may be full at their base over perhaps half a metre or a metre for four days; at the most about five days unless you have events following one behind the other, which we have actually experienced.

D N W EARP (Binnie & Partners)

In law there is no doubt that if an inspecting engineer has made recommendations in the interests of safety then the necessary certificate under 10(6) can only be given by a member of the same panel as the inspecting engineer. I am sure that's what the DoE would say. It's just that in the case of relatively insignificant recommendations, where the inspecting engineer has been happy for the supervising engineer to certify the work, some enforcement authorities have accepted it; but this is a case where one does not want to press for a definitive answer because, if the definitive answer were given, it could completely close the door on supervising engineers ever performing that function.

R M ARAH (Binnie & Partners)

Mr Earp makes the point of getting level records kept by some undertakers. In many cases, I think they are of very little value unless the reservoir is spilling and I believe we are rather too demanding in having a record kept at weekly intervals when sometimes six-monthly or yearly might do.

Mr Ku mentioned the policy in Hong Kong of inspecting during severe natural events. I believe that is very helpful indeed and I do think supervising engineers should try to get to sites during floods, high winds, droughts and extreme frost.

On Dr Hughes's paper I think that with well over a thousand dams less than 3 metres high on the register in this country, I believe there really should be a process for excluding dams from the Act, with the blessing of perhaps 2 or even 3 inspecting engineers. I think it's a defect in the present legislation.

W J CARLYLE (Session Chairman)

With that last remark, I thoroughly agree. Indeed, in the discussions leading up to the 1975 Act, it was a bit of a toss-up whether the limit would be raised to 50,000 cubic metres. If it had, my goodness what a difference in the numbers there would have been and really it is a question of hazard.

Dr J A CHARLES (Building Research Station)

I would like to question Dr Hughes on that figure.

DR A K HUGHES (North West Water)

I will review the statistics and make any corrections if necessary.

M F KENNARD (Rofe Kennard & Lapworth)

M F Kennard has provided the following information in reply to Mr D J Knight's questions on Mr Shave's page.

As rapid drawdown of a flood alleviation storage reservoir will follow a rapid filling of the reservoir, there is unlikely to be high pore pressures in the upstream shoulder fill prior to the rapid drawdown. At Leigh Reservoir, the upstream shoulder was of gravel with relatively high permeability so that rapid drawdown was not a critical loading case.

The frequency of filling and drawing was anticipated to be less frequent than annual events.

At the flood alleviation reservoirs which are empty for very substantial periods, an aquilibrium piezometric stage is not due to reservoir level and seepage and so conclusions have not needed to be drawn.

WRITTEN CONTRIBUTIONS

K SHAVE (Southern Water Authority)

In response to Mr Knight I would comment as follows:-

The Leigh embankment does not contain any instrumentation apart from crest settlement points which are surveyed annually, and which showed minimal movement after the first year following completion of the earthworks.

There are records existing for each significant event at Tonbridge as the site is manned and flood impounding warnings issued to landowners to enable them to move stock from the land. Eleven significant events have occurred since 1960 where a depth of 1.5 m or more of water has been impounded and on two occasions the reservoir has been filled to its design level. The two smaller reservoirs in Bexley are not manned and although the one at Lamorbey Park has impounded, this is to an unknown degree since it has not been witnessed - the rise and fall in the small storage area taking place relatively quickly at a site at the extremity of the Division's boundary.

PROCEEDINGS: TECHNICAL SESSION 3

RENEWING AND UPDATING DRAWOFF WORKS

PAPERS:

DISCUSSION:

Index 3 (cont)

RENEWING AND UPDATING DRAWOFF WORKS

C G Gregory BSc FICE FIWEM (Associate)
J Hay BSc MICE FIWEM (Associate)

Rofe, Kennard & Lapworth

SYNOPSIS

The refurbishment and conversion to electrical operation of the existing drawoff and scour valves and their operating mechanisms at two reservoirs in Yorkshire, each about 100 years old, are described. Replacement of spear rods, inspection of pipework below water by CCTV and conversion of the headstocks to electrical operation are included.

MIDHOPE RESERVOIR

Introduction

1. Midhope Reservoir is situated about five kilometres to the west of Stocksbridge in the County of South Yorkshire and was constructed in the period 1897 to 1904.

2. It is of conventional construction comprising an earthen embankment with central clay core above a concrete cut-off carried down into the underlying shales.

3. The embankment is 254 metres long with a maximum height of 31 metres and its storage capacity is 1,850,000 cubic metres.

4. The valve shaft consists of two rectangular section dry wells, one upstream and one downstream, separated by a diaphragm wall.

5. The wells are directly above a brickwork culvert from the forebay and each well houses an 800mm (2'-8") diameter vertical cast iron standpipe which connects into the culvert.

6. The reservoir was inspected under Section 2 of the Reservoirs (Safety Provisions) Act 1930 in January 1975 and the Inspecting Engineer included the recommendation in his report that in the interests of safety of the reservoir, "all valves be serviced as soon as possible."

7. The valves had not been operated for some years and the Inspecting Engineer was of the opinion that the valves could stick either in an open or closed position.

8. The Authority commenced the servicing of the valves in October 1981 by inviting Rofe, Kennard & Lapworth to comment upon the most economic method to achieve this by one of the following alternatives:

(a) Refurbishing the existing valves.

(b) Modifying the existing valves, shaft and pipework.

(c) Constructing a new valve shaft with pipework and valves.

9. After initial investigations it became apparent that the most economic solution would be to refurbish the valves, including headstocks, if practicable.

Description of Valves

10. The valves are two in number, one to control the discharge of water to the nearby treatment works and a scour valve for discharging water into the river downstream of the dam via the tailbay.

11. The valves were manufactured by Guest & Chrimes Ltd and were identical, being controlled by massive headstocks.

12. Each valve gate is suspended from its headstock inside an 800mm diameter cast iron standpipe by 225mm x 225mm teak spear rods joined together by scarfed joints, wrought iron plates, nuts and bolts. The rods operated partly in wet and partly in dry conditions depending upon the level of water in the reservoir.

13. Spider guides are provided every 2.4m to locate the rods centrally in the standpipe.

14. The valves are of the single faced type and rely upon the head of water upstream to hold the gate onto a matching machined seat in the valve body. See Figure No. 1.

Renovation Works

15. Various firms were approached to enquire if they would be able to undertake the removal of the spear rods, valve gates and headstocks followed by overhaul of the gates and headstocks and replacement of the rods and spider guides in stainless steel.

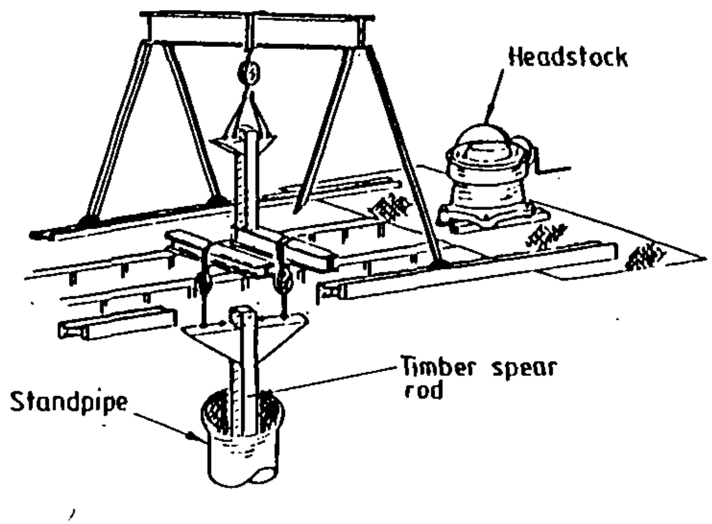
16. Messrs. Guest and Chrimes, the original manufacturers of the equipment, replied saying that they could not undertake the work.

17. J. Blakeborough & Sons Ltd, however, who had carried out similar works at Damflask Reservoir in 1954, were the firm who gave the most satisfactory response to the enquiry and their quotation was accepted as the basis for carrying out the refurbishment.

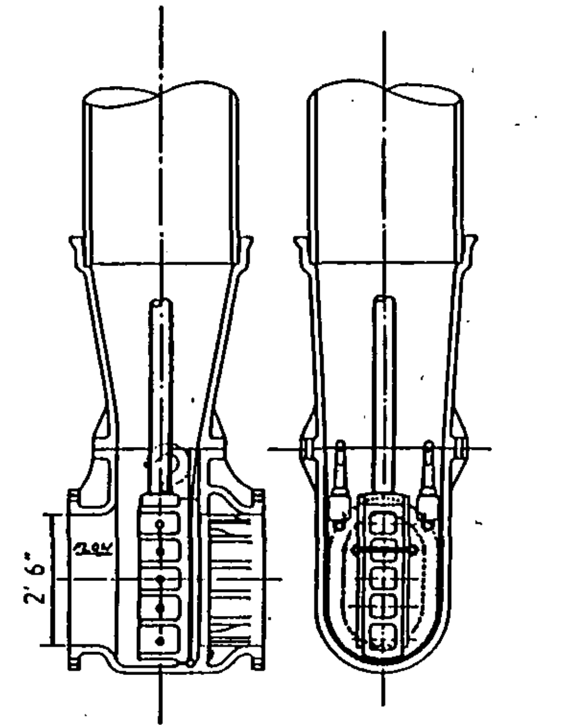
18. The cost of the remedial works was £55,000 including CCTV inspection and conversion of the headstocks to electrical operation by fitting actuators with gear boxes to the lay shafts.

Description of Site Operations

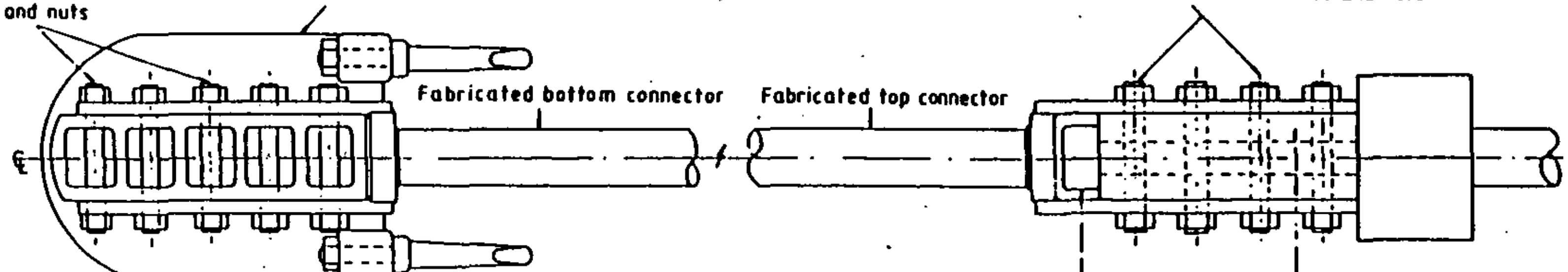
19. The sequence of operations for each valve was as follows.



LIFTING FRAME



GENERAL DETAILS OF VALVES



GATE CONNECTOR

HEADSTOCK CONNECTOR

DETAILS OF CONNECTORS

19.1 Secure lifting brackets to the sides of the rod below the headstock, raise the rods via the headstock, transfer the weight of the rods and gate to the existing beams below the headstock by chocking between beams and brackets.

19.2 Erect purpose made mobile lifting frame (SWL 10 tonnes) over the headstock and remove the headstock for overhaul followed by removal of the spear rods and gate. See Figure No. 1.

19.3 Make CCTV inspection of the standpipe and valve seating.

19.4 Reassemble with new spear rods 2.4m (8'0") long 125mm (5") OD/100mm (4") ID with spider guides, all in stainless steel. See Figure No. 2.

19.5 Reinstate headstock and fit electrically operated actuator and gear box to lay shaft. See Figure No. 2.

20. No undue problems were encountered during removal of the valve gates. Headroom in the valve house was limited and the gantry frame had to be sized accordingly.

21. The lift available determined the lengths into which the teak spear rods were cut for removal and the length of the new rods.

22. The CCTV inspection showed the standpipe to have approximately 25mm of incrustation and that the bronze valve seating for the drawoff valve was in good condition.

23. The seating for the scour valve, which could be examined physically from the tunnel via the pipe through the brickwork plug, was found to be satisfactory.

24. The valve gates were refurbished on site, including painting, and the inside of the standpipes were also cleaned and painted down to water level.

25. The headstocks were overhauled at Blakeborough's works, the main thrust bearings being skimmed and O ring seals and greasing points being fitted to the lay and main shafts. Guards were also fitted to the exposed spur gear trains as subsequent operation would be under electrical power and not by hand.

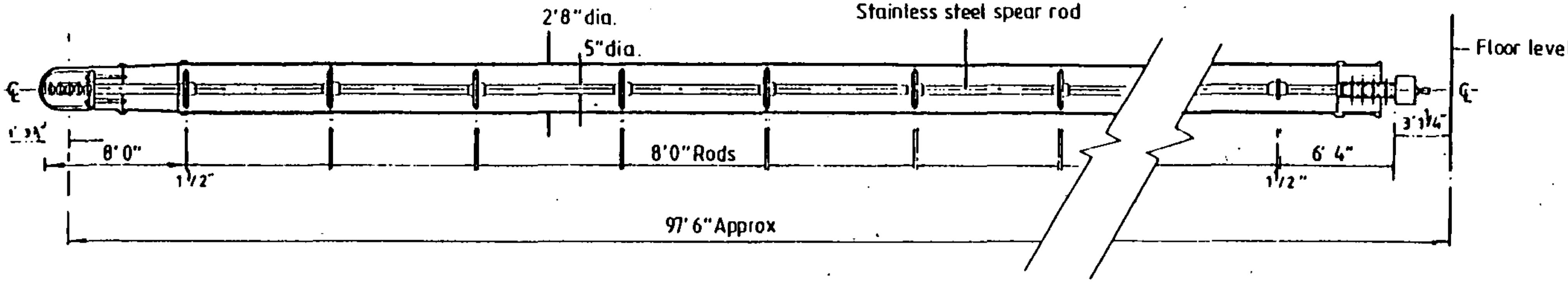
Commissioning

26. The scour valve was the first valve to be commissioned.

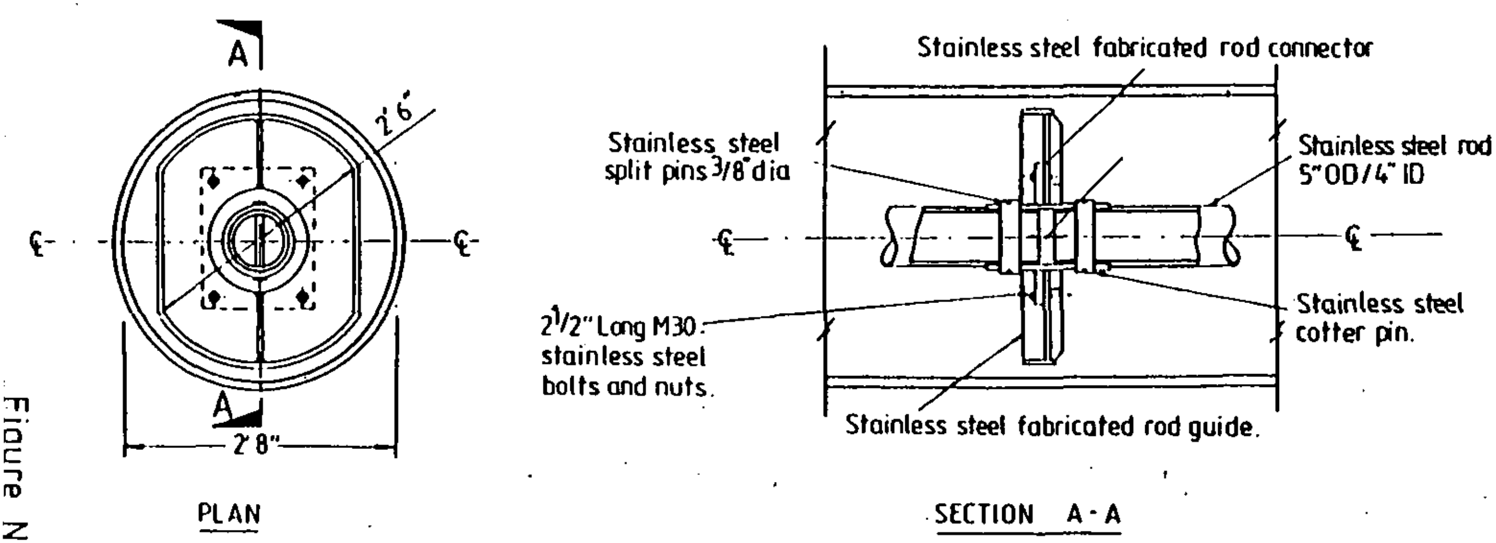
27. The gate was checked from the upstream end of the tunnel and was found to hang freely just clear of its seating.

28. The gate was swung in an upstream and downstream direction and it was confirmed that the spider guides did not touch the standpipe before the machined face of the gate touched its mating face.

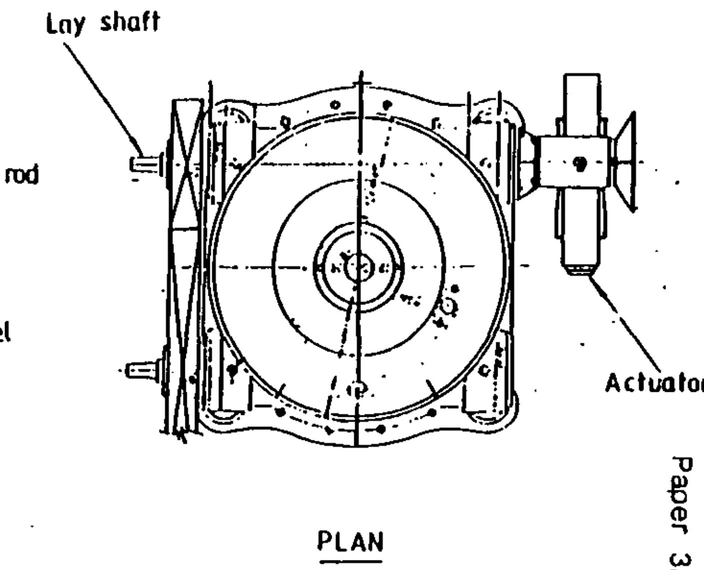
29. The scour valve was checked for travel under a non-differential head condition across the gate with the draw-off valve closed and was found to move freely without any resistance.



GENERAL DETAILS OF SPEAR RODS



DETAILS OF SPIDER GUIDES



HEADSTOCK WITH ACTUATOR

Figure N°: 2

30. The scour valve was then closed and the standpipe was filled with water by cracking the draw-off valve. This valve opened in a series of jerks as it travelled across its seating.

31. The scour valve gate was then examined physically at the end of the tunnel under the head of the reservoir and was found to be leaking very slightly. It was then opened a little and reclosed and the leak was eliminated.

32. Subsequent operation of the scour valve to a 75mm/100mm open position was accompanied by jerking as the gate travelled across its mating face with the jerking ceasing after about 75mm of travel.

33. The scour valve was closed against the full head of the reservoir and as it travelled through the "lap" position, i.e. when the 75mm wide machined face of the valve gate travelled across the 75mm wide mating face downstream, it was noticed that excessive play in the travel motion in the headstock was present. This was apparent because the main spur gear around the threaded upper section of spear rod travelled up the rod for a distance of approximately 7mm before a positive downwards thrust could be imparted into the rod. This play was taken up by inserting a suitably sized distance piece in the motion.

34. The draw-off valve was then opened to its full extent of 750mm (2'6") and closed, all without any differential head across the valve gate. The valve operation was smooth.

35. The draw-off valve was then opened approximately 100mm in a full differential head condition and it was noticed that the operation was in a series of jerks until the valve seating at its upper and lower extremities was cleared, the operation thereafter being much smoother.

36. In the light of experience gained on the scour valve it was concluded that the jerking motion of the draw-off valve in the differential head condition was due to overcoming the frictional resistance to movement rather than the scraping of spider guides in the wet well. Under a full reservoir head of approximately 28.5m it is estimated that a force of about 13 tonnes is required to start the valve gate moving from a static position.

37. During commissioning various modifications to the actuators, which were fitted to the lay shafts on the headstocks, were found to be necessary.

38. These included increasing the setting capability from 550 turns to 1220 turns and the provision of limit switch stopping in lieu of stopping on torque which could damage the valve if the full torque of the actuator were to be transmitted to the valve through the reduction gearing in the headstock.

DALE DIKE RESERVOIR

Introduction

39. The second Dale Dike Reservoir, situated in the Parish of Bradfield in the District of Sheffield in the County of South Yorkshire, about 13 kilometres west northwest of Sheffield Town Hall, was completed in 1875 to the design of Thomas Hawksley.

40. The reservoir is formed by an earth embankment with a puddle clay core carried down in trench to a shale stratum in the underlying gritstone series. The embankment is 275m long with a maximum height of 22m and impounds a total storage of 2,209,000 cubic metres.

41. Discharge of water from the reservoir is controlled by two valves in series housed in adjacent brick lined shafts positioned upstream of the core of the dam. To the body of each valve there is connected a cast iron vertical standpipe which accommodates a line of wooden spear rods for raising and lowering the valve gates. The valves, together with their associated pipework, are bricked in for a height of some 9m above the base of the shaft. Water is led to the valves from the reservoir, and from the valves to the downstream side of the dam, in a 2.13m internal diameter brick lined tunnel 375m long driven in solid ground around the south east end of the dam.

42. The upstream (guard) valve is normally kept fully open and had not been operated for many years. The downstream valve is used to regulate discharges and is normally only cracked open. At the downstream end of the tunnel, discharges are either conveyed by pipe to the local treatment works or released to the river.

43. The reservoir was last inspected under Section 2 of the Reservoirs (Safety Provisions) Act 1930 in December 1985. Regarding the existing draw-off arrangements, the Inspecting Engineer recommended that "both valves controlling the discharge of water from the reservoir be made fully serviceable". Two earlier inspection reports had advised maintenance or replacement of the draw-off valves in the foreseeable future because of leaks past the valve seatings and the Inspecting Engineer was concerned that corrosion might cause resistance to movement and inhibit the operation of the valves.

44. In August 1986, Rofe, Kennard & Lapworth were commissioned by the Yorkshire Water Authority to prepare a scheme for overhauling and motorising the valves along similar lines to the works which had recently been carried out at Midhope Reservoir.

Description of Valves

45. The manufacturer of the valves could not be confirmed although the design was very similar to those at Midhope Reservoir, which were manufactured by Guest and Chrimes (See Figure No. 1).

46. The valves are cast iron of the single face type, the waterway being 1.37m high x 0.46m wide with an invert and crown radius of 0.23m. The valve gates are suspended from headstocks located in the valve house at ground level by rising spindles linked to timber spear rods, 200mm x 200m

in cross-section and reinforced by 30mm thick wrought iron straps. The spear rods are kept central within 860mm dia standpipes by spider guides and the gates hang freely within the valve body, relying on upstream water pressure to seal them against the valve seat.

47. Each valve can be manually operated from the valve house through the lay shaft of the headstock which gives a 2:1 gearing reduction to the main shaft. The latter drives a worm wheel and thrust operating nut threaded to receive the rising spindle of the valve. The flat bearing surfaces of the rotating nut are fed by oil reservoirs.

Refurbishing of Valves

48. Following preliminary consultations with the Authority and J. Blakeborough & Sons Ltd, it was concluded that the remedial measures could be less extensive than had been undertaken at Midhope. Taking into account the satisfactory results of tests on the timber recovered from the spear rods at Midhope, and following a visual inspection of the rods above water level at Dale Dike, it was decided that the latter rods could be left intact and refurbishing could be restricted to servicing of the headstocks.

49. In October 1986, the downstream valve was inspected by the Authority from the tunnel with the upstream valve closed. The phosphor bronze seating face was reported to be in good condition and the gate itself, although heavily encrusted with iron deposits, also appeared sound. The upstream valve was tightly sealed with no water passing.

Motorising of Valves

50. It was decided to fit the valves with electric motors and gear boxes to enable them to be operated with fewer personnel in an emergency or during routine maintenance. Lights would be provided in the valve house and at intermediate platform levels in the shafts to assist in routine weekly inspections.

51. A test was carried out by the Authority to measure the torques required to operate the valves under maximum head conditions so that the appropriate size of electric motor could be determined. A torque wrench was applied to the lay and main shafts, and the torques necessary to move the valves from a stationary position at various stages of opening were recorded. Maximum torques (approximately 230 kNm on the lay shaft) were recorded when starting to open the valves from the fully closed position.

52. During the test both valves were operated by hand through their full range of travel and were found to run smoothly. The upstream valve was closed in 22 minutes and the downstream opened in 32 minutes. Four men operated the valves during the test.

53. In the light of the test, which showed that the valves could be manually operated via both the lay and main shafts, consideration was given to fitting the motors directly on to the main shaft at a reduced output speed. However, it was decided that use of the lay shaft would give an added factor of safety if increased effort was found to be needed to operate the valves at a later date.

54. The principal features of the scheme for motorising the valves are as follows:-

54.1 Each valve is driven by a 3-phase electric motor, manufactured by Brook Crompton Parkinson, and rated at 2.2 kW with a nominal operating speed of 1250 r.p.m. The motor is linked to the lay shaft of each headstock by a reduction gearbox, manufactured by David Brown Gear Industries Ltd, reducing speed type CAM 337 Adaptable, giving an output speed of 100 r.p.m. The motors, which are fitted with embedded thermistors to guard against overheating, are also protected against overload by an electronic shear pin, manufactured by Fenner Electronic Controls, and by a mechanical torque limiting device. The motors and gear boxes are mounted on prefabricated steel pedestals.

54.2 The motors are powered by a mobile (wheeled) diesel electric generator, manufactured by the Forest City Electric Company Ltd, fitted with a "Lister" single cylinder diesel engine. The generator output is to two 3-phase and two single-phase weatherproof sockets. The engine exhaust is vented through the wall of the valve house.

54.3 Starters, pushbutton controls and electronic shear pins are contained within a wall mounted control panel with protection to IP 55 standard. When the generator is running, lights on the panel indicate whether valves are in the open or closed position or in the process of opening or closing.

54.4 Limit switches are weatherproof, industrial type, "Snaplock", manufactured by Sigma. They are fixed to a new steel guard fitted over the rising stem of each valve.

54.5 All moving parts are fully enclosed within protective guards, and removable covers are fitted with automatic isolating switches.

54.6 The facility of manual operation of the valves has been retained. Slackening an expanding cone on the motor drive shaft and the removal of a protective cover to the lay shaft spindle enables the original turning handle to be used.

55. A quotation submitted by J. Blakeborough & Sons Ltd, inclusive of refurbishing, motors and ancillary equipment, and inspection lighting was accepted in the total sum of £17,300, of which approximately half was for new equipment and the remainder was for works on site, provision of temporary lifting gear and clamps, production of working drawings and operation and maintenance manuals.

Description of Contract Works

56. Following an initial survey to measure up for motor and gearbox pedestals and protective guards, Blakeborough produced manufacturing drawings.

57. Before dismantling the existing gearboxes it had been intended to clamp the rising spindles and suspend the valve gates and spear rods from the main steel beams under the valve house floor. The transfer of load on to the top of the cast iron standpipe, which had run lead joints, had been prohibited.

58. In the event it was possible to dismantle and inspect the gearboxes with the valves closed. Although this was preferable from a safety point of view in that the risk of impact damage to the valve gate or body if the temporary support were to fail, was reduced, the Authority were involved in extra costs in meeting the water supply demand from more expensive sources.

59. After the lay shafts had been removed for machining, dismantling of the headstocks proved to be less straightforward than had been hoped. On the upstream valve, a cotter pin on the stop nut at the top of the rising spindle had seized and attempts to dismantle the bearings in the downstream headstock had to be curtailed.

60. The bearings of the upstream headstock were found to be in excellent condition with no evidence of wear other than some slight pitting which may have been present in the original casting. No remedial measures were considered necessary either to the bearings or the moving gears. In the light of this, and to avoid any further disruption to supply, it was decided that there was no need to fully dismantle the bearings in the downstream headstock for inspection. There was concern that any damage to the bearings incurred during dismantling could have led to lengthy delays whilst replacement parts were being manufactured.

61. The control panel, guards, pedestals, mountings and motor drives were manufactured at Blakeborough's works before being assembled on site. Pedestals were bolted to the top flange of the main beams under the valve house floor.

62. The top and bottom limit switches on the valves were set to stop the gates approximately 20-25mm before the end of their travel, final opening or closing being achieved manually. This was felt to be a prudent measure in the closing mode to guard against possible impact damage to the valves if they were driven fully into the seat.

Commissioning

63. For final commissioning, the supply to the treatment works was closed, thus enabling both valves to be operated through their full range of travel. The valve under test was initially closed with the full reservoir head acting on the gate. The other valve was set approximately 300mm open.

64. The rates of opening and closing and the relative resistance to movement were recorded. The latter was measured on the electronic shear pin device where the actual motor load was indicated as a percentage of the rated load.

65. It was decided that the occasional trip of the motor would be a useful indicator that the shear pin device was functioning satisfactorily. It was therefore adjusted to trip the motor when an obstruction or tight spot in the valve travel was encountered at a load just in excess of the rated load.

66. As they were raised, both valves were found to be relatively tight and the motor tripped out several times over the first 100-125mm above the bottom as the gates cleared the seats. Thereafter they moved smoothly. The downstream headstock (which was operated more frequently) was slightly noisier than the upstream one.

67. The upstream valve was raised a second time over the first 150mm from the closed position and the motor tripped only once. Regular routine operation of the valves will probably lead to smoother running at the bottom end of the travel.

68. Rates of opening and closing were 73 mm/min and 75 mm/min respectively, equivalent to a time of approximately 20 minutes to fully open the valve from the closed position and vice versa. Measured motor loads were 100% of rated load in the initial 100-125mm of raising, settling to 60 to 70% thereafter. To lower the gates, motor loads were 30 to 40% of rated load increasing to 50 to 55% as the gate neared the fully closed position. Closure of the downstream valve was easier, probably because it had been regularly operated in the past, and the manual operation over the final 25mm was very smooth.

69. During commissioning, the valves were inspected from the tunnel with the upstream valve closed. Some erosion was noted at the bottom of the machined face of the downstream valve gate and the top of the seat on the valve body was slightly rough. There was a slight leak from the top of the upstream valve. However, it had been reported following other recent inspections that a tight seal could be achieved on both valves, and no further repairs were considered necessary at this stage.

70. During final commissioning the equipment met the Authority's operational requirements both for maintaining supply and for routine maintenance. In achieving this valuable information was obtained from the earlier Midhope contract, particularly in assessing the extent of refurbishing works that would be necessary and in choosing the most appropriate method for motorising the valves.

Acknowledgements

The authors wish to thank Yorkshire Water Authority, Southern Division, and Messrs. J. Blakeborough & Sons Ltd for permission to publish details of the manner in which the refurbishment of the valves and their operating mechanisms was carried out.

REMEDIAL AND IMPROVEMENT WORKS TO RESERVOIR DRAWOFF WORKS

D Gallacher MSc DIC CEng FICE FIWEM (Partner)

Robert H Cuthbertson & Partners

SYNOPSIS

Draw-off arrangements at many reservoirs require refurbishment and improvement. Access facilities within them are often difficult if not dangerous, unless safety measures are taken, which in turn inhibit inspection and maintenance work. Problems and remedial measures at draw-off towers for two reservoirs, and construction of major new draw-off arrangements at a third are described. The reservoirs are Loch Cote and Drumbowie (both Central Region, Scotland) and Gladhouse (Lothian Region, Scotland).

INTRODUCTION

1. This Paper describes remedial works carried out to two reservoir valve towers in Central Region, Scotland and installation of new draw-off arrangements to a reservoir in Lothian Region, Scotland. In the first two cases the same basic fault was found - viz. substantial leakage into the towers through structural cracks and/or open joints. The circumstances giving rise to the cracks differed in the two cases although certain common features were evident. Different solutions to the problem were adopted since there were fundamental differences in the nature and construction of the two towers. In the third case additional draw-off capacity was required as part of the development of the Megget Scheme which provided for substantial additional inflow to this reservoir.

LOCH COTE RESERVOIR

Original Works

2. Loch Cote reservoir is near the village of Torphichen in West Lothian and was the principal water supply source for Bo'ness in Central Region. The reservoir was constructed about 1899.

3. The valve tower is of the dry type originally 12 metres high from base to top and 1.83 m internal diameter (Fig.1). The lower 5 m is buried within the upstream face of the embankment and comprises engineering brickwork varying in thickness from 685 mm at the base to 457 mm at the embankment interface. The exposed height of tower above the embankment face was smooth dressed sandstone 457 mm with 3 mm joints. A single 380 mm cast iron stand-pipe runs the full height of the tower with 2 No. 280 mm draw-off pipes connecting to the reservoir at approximately 3 m intervals below overflow level. The stand-pipe connects to a 380 mm bottom draw-off which continues through a D-shaped brick culvert under the reservoir embankment. A separate reservoir scour pipe discharges into the base of the tower. The top of the tower was 0.92m above overflow level and was connected to the reservoir embankment by a 16.90 m span steel lattice girder bridge.

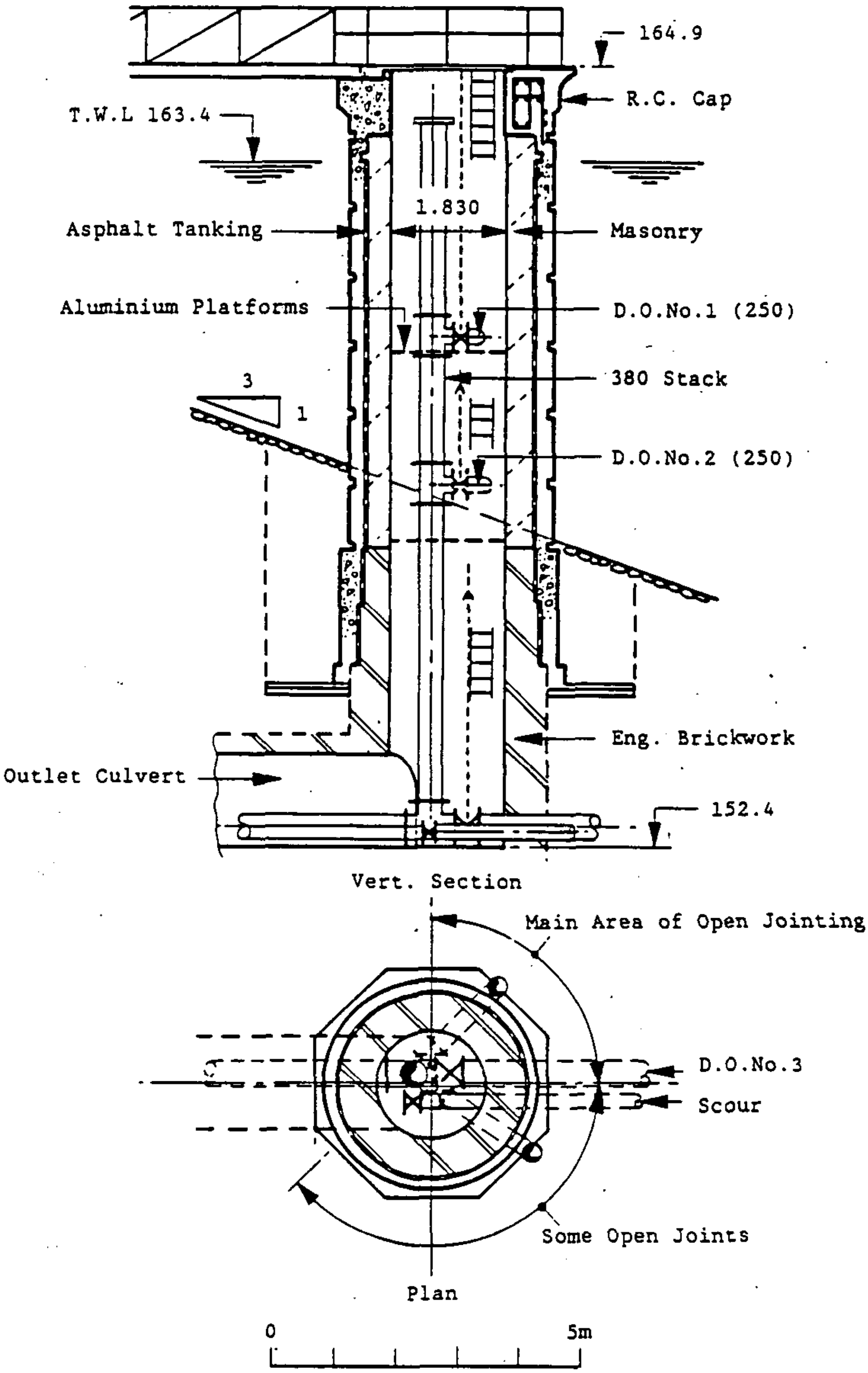


Figure 1: Valve Tower

LOCH COTE

4. Inspection of the tower internal surfaces revealed substantial areas of efflorescence indicative of long term leakage or seepage, and at several positions water was jetting into the tower. Leakage appeared to be more prevalent on the south-east quadrent of the tower but there were significant leaks on the diametrically opposite quadrent. There were extensive signs of previous remedial pointing of joints. The reservoir was drawn-down to permit inspection of the outside of the tower which revealed a number of joints caulked with lead-wool and concrete bagwork at the interface between the tower and embankment.

5. A number of possible mechanisms which might have been responsible for the opening of joints were postulated. Among these were:-

(i) differential thermal expansion of tower.
(ii) ice thrust.
(iii) thermal expansion of bridge.
(iv) lateral displacement of bridge.
(v) eccentric loading.
(vi) hydrostatic uplift.

The pattern of open jointing was such as to be explicable only by a combination of some of the factors noted above.

6. An analysis of the static loading on the tower including the bridge showed that, even on the worst assumption of 100% uplift, stress on the joints should have been compressive throughout the height of the tower and should have increased with depth: factors (v) and (vi) above did not appear to be the primary cause of the problem.

7. It was observed that the tower appeared to be out of plumb with the top deflected 16 mm towards the reservoir. There were signs of shear in the upper courses of masonry and it was concluded that the apparent inclination of the tower was due to displacement of these courses. The pattern of open jointing occurred mainly on the reservoir side of the tower which was inconsistent with the apparent observed deflction.

8. Distortion of hand-rails at the embankment end of the bridge showed that the bridge abutment had settled. Levelling indicated that the bridge had also twisted. The bridge was effectively fixed at both ends being firmly attached by rust to the sliding bearings. It seemed possible, therefore, that slight rotation of the bridge and settlement of the embankment end had been sufficient to pull the tower top towards the embankment causing opening of joints on the opposite side of the tower. Displacement of the masonry at the top of the tower was probably due to thermal movement of the bridge and not directly connected with the mechanism which caused the lower joints to open.

9. The possibility of acid water attack on the mortar jointing was considered but was ruled out as the reservoir water was slightly alkaline. No definite single cause of the open jointing could be identified although displacement of the bridge and eccentric loading were thought likely to be the primary causes.

Remedial Works

10. The original steel access bridge weighed 5 tonnes and was in poor condition with serious corrosion in vital areas and was replaced with a lighter structure. The replacement is a 19.8 m span standard aluminium triodetic structure weighing 1.5 tonnes. The new bridge is located with its bearings slightly closer to the tower axis than the original bridge and this reduction in eccentricity combined with the substantially reduced weight results in the eccentric tensile stress being reduced to about a quarter of its previous value.

11. The outside of the tower over a height of 8 m below overflow level was provided with a 20 mm thick coat of mastic asphalt trowelled on in 3 coats. The mastic asphalt was painted with a lime wash and encased in a 230 mm thick reinforced concrete "jacket". The concrete, Grade 25 air entrained with a minimum cement content of 350 kg/m^3, was provided primarily as a means of preventing mechanical damage to the tanking and securing it against long-term slumping. It was, however, designed to hang from a ring-beam constructed on top of the original tower and thus effectively provide an element of pre-stress to the masonry (Fig.1). Allowing for the reduction in concrete weight due to submergence the nett additional load on the tower is some 47 tonnes giving an effective pre-stress of about 0.14 N/mm^2 on the masonry. The pre-stress is equivalent to the pressure resulting from a head of water of about 14 metres which is about 33% greater than the highest possible reservoir head. The concrete jacket could have been made thinner but the actual thickness was selected to ease construction.

12. The remedial works described above were carried out in 1981/82 and shortly after the access arrangements within the tower were improved with the installation of new aluminium ladders and platforms. The remedial and improvement works were carried out at a total cost of £70,000. The tower was inspected in October 1985 when it was found to be dry apart from some sweating of the brickwork at the base.

DRUMBOWIE RESERVOIR

Original Works

13. Drumbowie reservoir is near Denny, Stirlingshire. It is primarily an off-stream storage reservoir formed by an earthen embankment with puddle clay core constructed c1901 (Fig.2).

14. The valve-tower was of the wet-type constructed in mass concrete 2.80 m internal diameter and about 18 m total height. The walls were 610 mm thick over the bottom 11 m and 457 mm thick above. The lower 11 m was buried within the upstream face of the embankment and was surrounded by puddle-clay.

15. Two 305 mm draw-off pipes at 4 and 7.6 m below overflow level connected the tower to the reservoir, the lower being housed in a recess in the embankment face. A 610 mm reservoir scour pipe discharged into the base of the tower where it was controlled by a geared sluice valve. Draw-off from the tower was by means of a 610 mm cast-iron pipe encased in concrete below the embankment and controlled by a valve at its downstream end: this pipe was thus under full reservoir head.

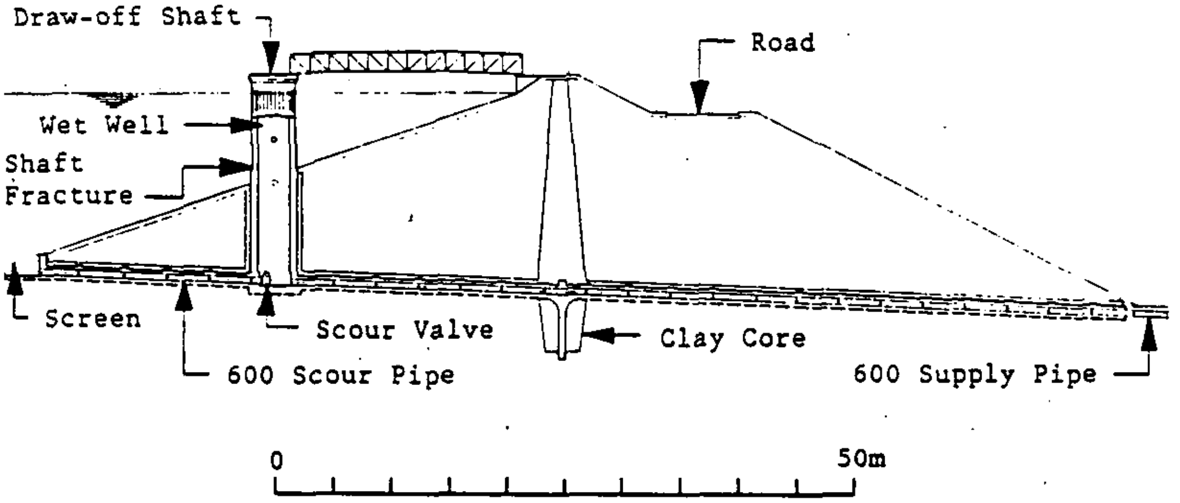


Figure 2: Embankment Section

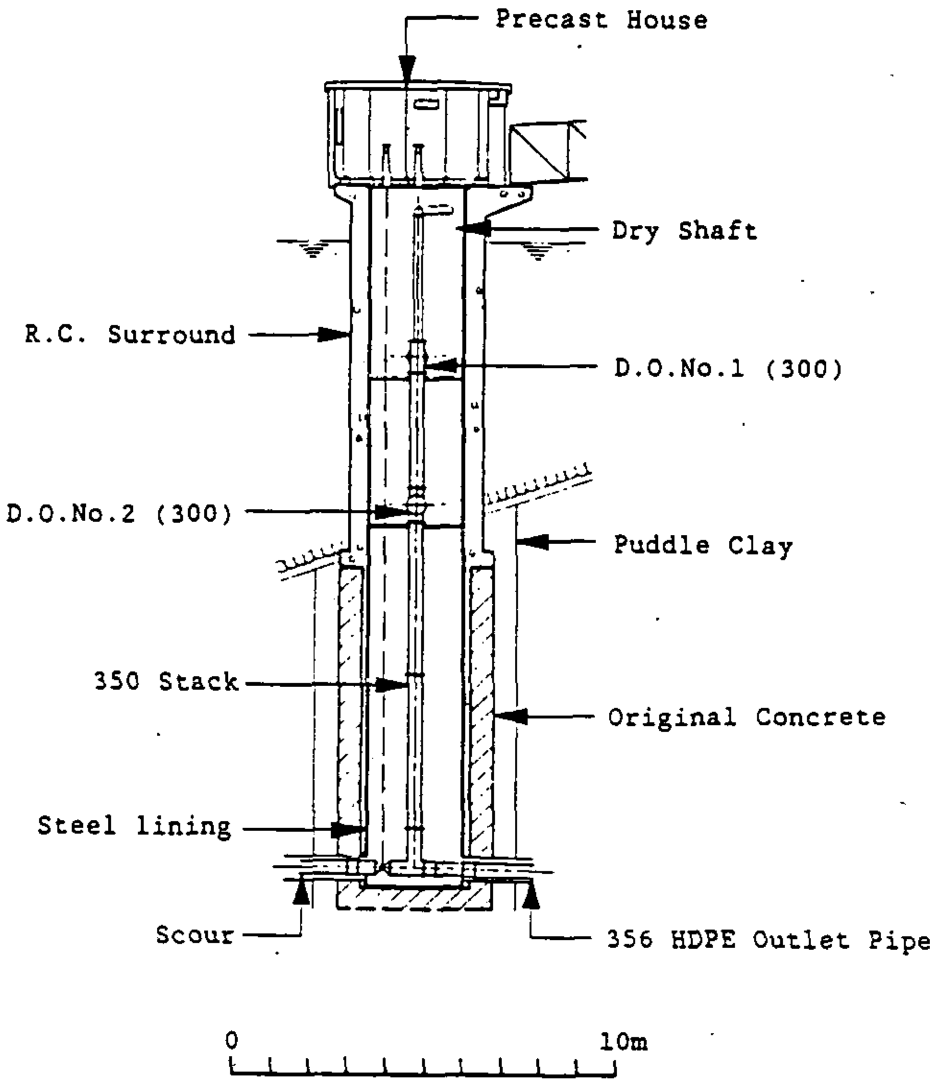


Figure 3: Draw-off Shaft

DRUMBOWIE RESERVOIR

16. The scour valve and the draw-off valves were operated by headstocks mounted on top of the tower. A ladder ran the full height in a single flight with one intermediate platform at the level of the lower draw-off. The tower was connected to the embankment crest by a 19 m span steel N-girder bridge constructed in 1974: the bridge had fixed bearings on the tower top with sliding bearings on the landward abutment.

17. Leakage into the tower had been a problem of long-standing and had reached a stage where it was leading to operational difficulties. Inspection revealed various faults such as cracking and spalling of concrete which could be attributed directly to poor construction. The concrete was poorly graded with an absence of small aggregate. In the upper part of the tower mortar patching was evident but behind the mortar skin the concrete had little matrix as most of the cement had been leached out.

18. The principal point of leakage was found at the level of the lower draw-off, i.e. at 7.6 m below overflow level where a horizontal crack extending right round the tower was found. Water was jetting into the tower over a 180° arc on the reservoir side: the other half of the crack was below the embankment face and within the puddle clay surround. At the crack location the draw-off pipe and 2 No. RSJ platform supports were built into the walls in the same horizontal plane providing a ready-made crack-inducer.

19. The lower part of the tower, i.e. that part buried within the embankment, was generally leak free although there were some signs of seepage at lift joints. Pipework, ladder, platform and supports within the tower were all badly corroded. There were deposits of sand and gravel in the base of the tower similar to those which were found in the embankment recess at the lower draw-off.

20. Apart from the poorish nature of the construction materials it was concluded that the primary cause of the crack giving rise to the principal leakage was deflection of the tower by thrust imposed by thermal expansion of the bridge.

Remedial Works

21. The nature of the tower foundations was unknown and since the outlet pipe under the embankment was rigidly connected to the tower it was considered undesirable to effect repair by any means which would radically increase the weight of the tower. A solution similar to the one adopted at Loch Cote was thus ruled out. The tower was converted to the dry well type and all pipework and internal steelwork was replaced (Fig.3).

22. The upper part of the tower to a level just below the embankment interface was demolished. A steel tube 2.45 m internal diameter and 18.20 m long was lowered into the remaining part of the tower after cleaning and scabbling of the concrete surface. The steel tube was located centrally in the tower and the 160 mm annular gap between the tube and the tower wall filled solid with Grade 25 concrete containing 10 mm maximum size aggregate. The upper part of the tube was surrounded in concrete 488 mm thick to produce a finished outside diameter of 3.45 m.

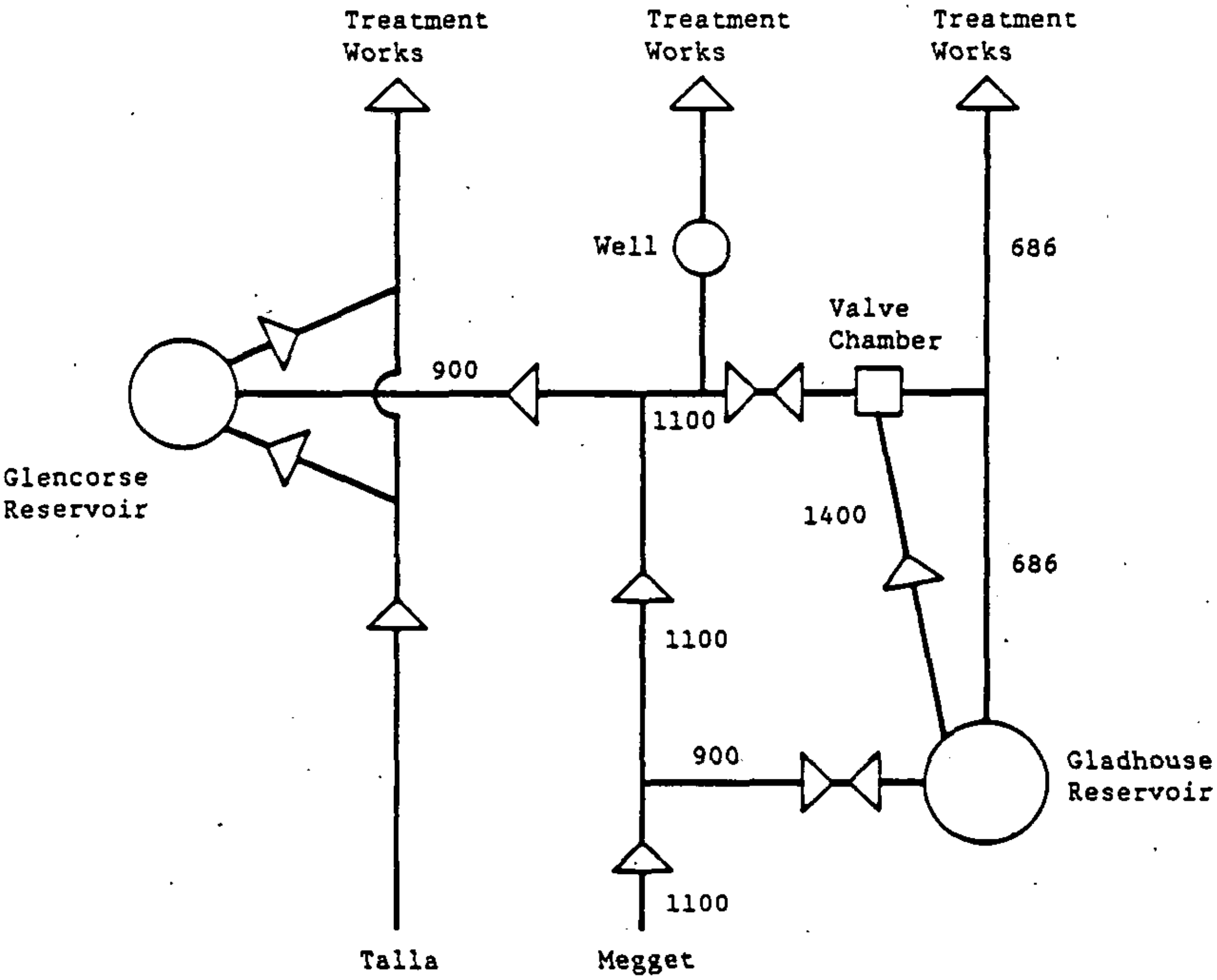


Figure 4: Supply Distribution

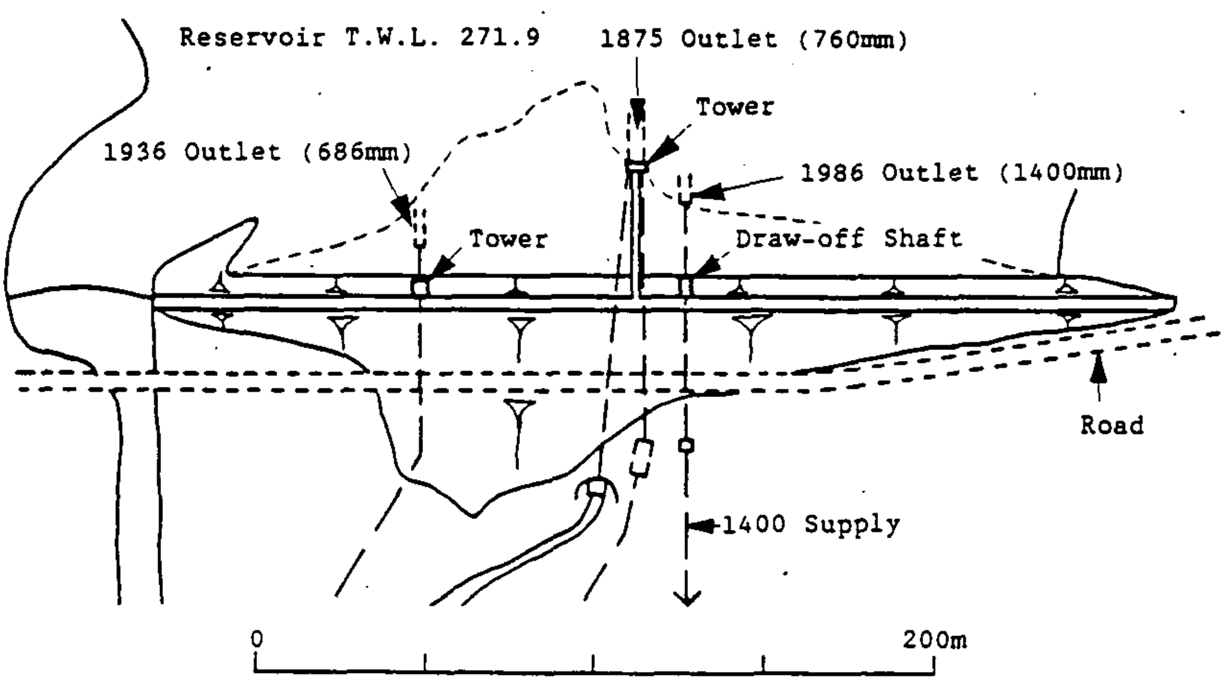


Figure 5: Plan of Gladhouse Dam

23. The steel liner tube was fabricated in one piece generally in accordance with BS 534 using 12 mm thick plate of Grade 43A steel. It was protected internally with Metalife System 508 and externally with 2 coats of Metalife 44 epoxy coating. The base of the tube is of 20 mm thick plate stiffened internally with steel channels: the barrel was stiffened over the lower 600 mm by a 20 mm thick external wrapper. The tube was provided with cut-outs for the scour, supply and draw-off pipes and with internal support angles for new platforms and ladders.

24. Pipework within the tower was renewed in ductile-iron with a 350 mm vertical stand-pipe and 300 mm draw-offs controlled by butterfly valves. The lower draw-off was raised slightly and relocated at 90° to its previous position to keep it clear of the embankment face and remove any possibility of gravel entering the pipe.

25. It was proposed to slip-line the original 610 mm base inlet and outlet pipes with 556 mm steel pipe 6 mm thick connected to the upstand pipe within the tower (Fig.2). Descaling of the supply pipe through the embankment, however, revealed that it had been relined over a length of about 5 m immediately upstream of the embankment core and the effective internal diameter had been reduced to 465 mm: this repair was not noted on any drawing or the reservoir record book. It was also discovered that there were significant vertical deflections in the supply pipe under the embankment and it was apparent that it would be impossible to install the steel pipe as originally proposed.

26. The supply pipe and scour pipe were slip-lined using 355 mm HDPE pipe. The annular space between the HDPE pipe and original cast-iron pipe was filled with sand/cement grout. A 63 mm diameter HDPE drain pipe from the base of the tower was incorporated alongside the 355 mm HDPE pipe under the embankment.

27. The reconstruction works to the tower were completed by installing new ladders fitted with safety cages, new platforms at about 5 m intervals and lighting facilities. A pre-cast concrete house was erected on top of the tower to deter vandals and the access footbridge was cleaned, repainted, and re-erected in its former location. The remedial works were constructed in 1984 at a cost of £175,000.

GLADHOUSE RESERVOIR

History

28. Gladhouse Reservoir is in Lothian Region, Scotland about 15 km from Edinburgh. The dam is an earthen embankment with a central puddle clay core.

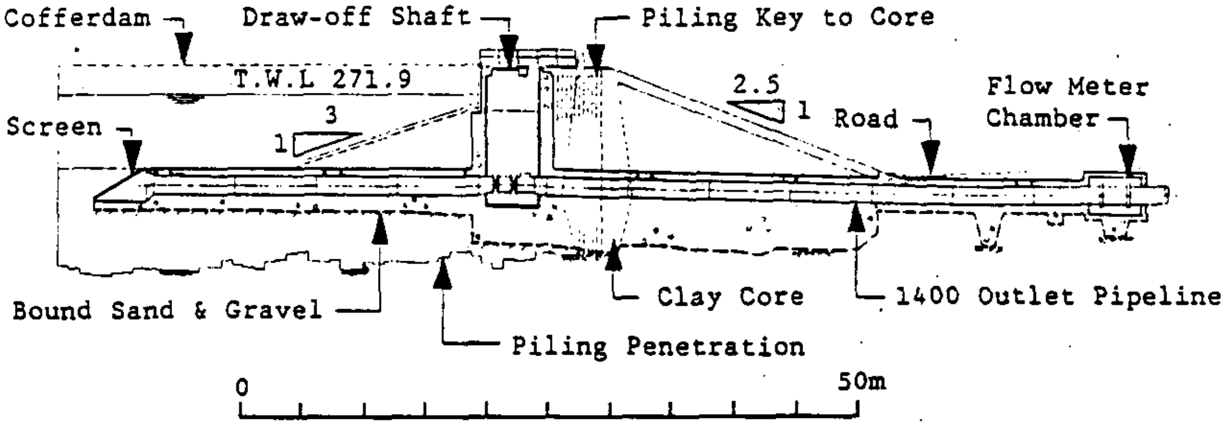


Figure 6: Embankment Section

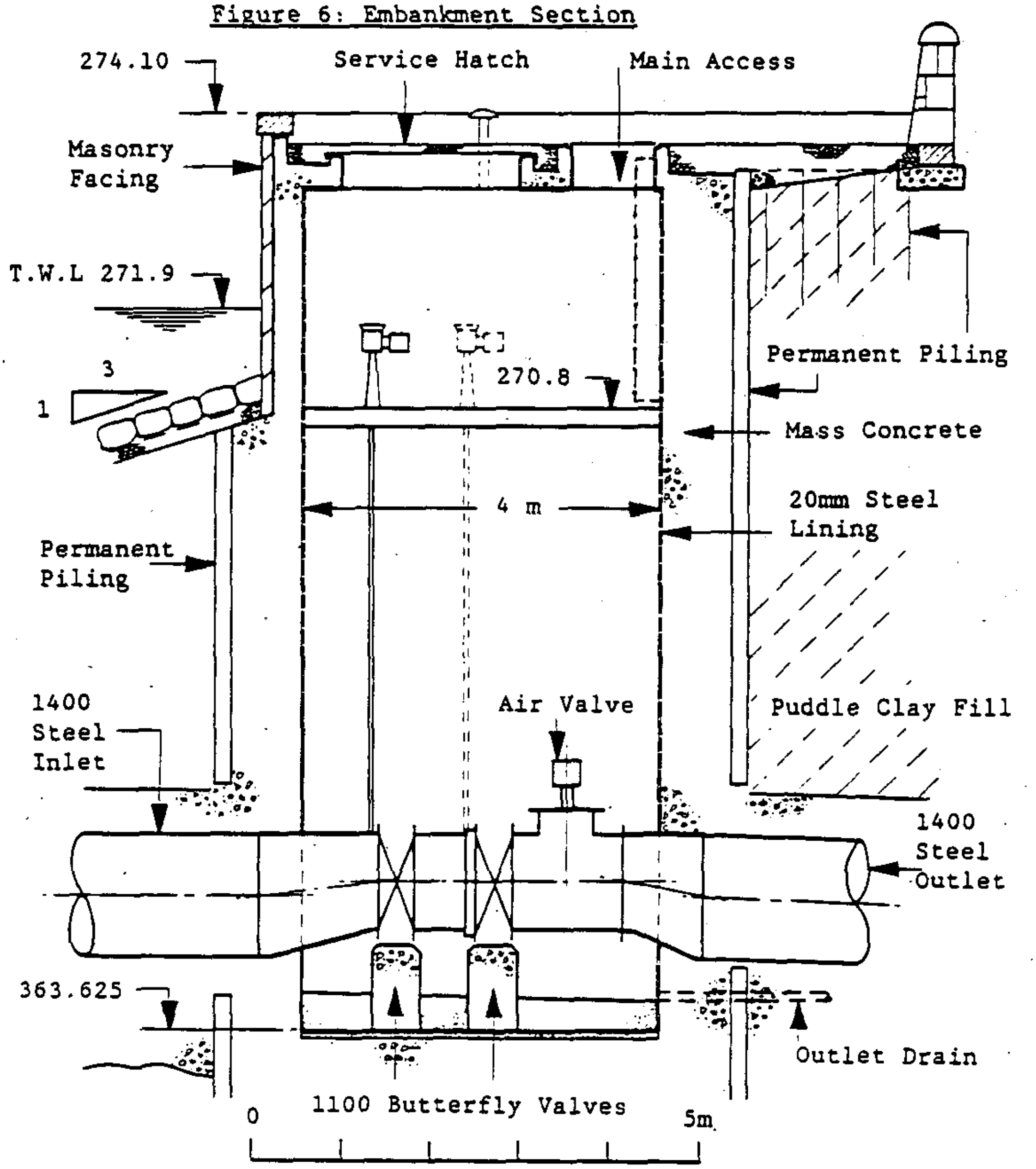


Figure 7: Draw-off Shaft

GLADHOUSE RESERVOIR

29. The draw-off works at Gladhouse were augmented as part of the Megget Scheme. Megget Reservoir completed in 1983 is Lothian Region's largest reservoir with a capacity of 61,400 Ml. The supply from Megget is conveyed by gravity to existing reservoirs at Gladhouse and Glencorse, and to other supply points local to these reservoirs feeding direct to treatment works (Fig.4). The capacities of Gladhouse and Glencorse Reservoirs are 8,240 and 1,671 Ml respectively and the reservoirs provide reserve and balance storage near to the main supply areas. Glencorse Reservoir also acts as a balancing reservoir for the supply from the Talla Scheme which is Lothian Region's other major supply source. As there was a substantial imbalance in storage between Gladhouse and Glencorse in terms of days supply available it was decided that transfer facilities from Gladhouse to Glencorse be provided. This was effected by laying a connecting aqueduct about 2.8 km long from Gladhouse to the existing Megget aqueduct which feeds Glencorse and providing new draw-off works at Gladhouse where the existing arrangements were inadequate. The draw-off works and the connecting aqueduct were completed in 1986.

Original Works

30. The embankment dam at Gladhouse is 25 m high with upstream and downstream slopes of 3:1 and 2.5:1 respectively. A public road crosses the downstream embankment slope on a berm. The safe yield from the Gladhouse reservoir catchment is 33.8 Ml/day and there were two draw-off towers from the reservoir with outlet pipes through the embankment (Fig.5).

31. The original outlet constructed in 1875 comprises a wet well draw-off shaft with three draw-off points each controlled by penstocks with a single 760 mm outlet pipeline feeding a measuring house and masonry conduit. Apart from the scour from the base of the wet tower these draw-off works are largely disused. An additional outlet constructed in 1936 comprises a 915 mm inlet pipeline connecting to a 686 mm supply pipeline feeding two treatment works. There is a single 686 mm sluice valve located in a dry shaft within the upstream embankment. The existing draw-off arrangements were inadequate to meet the predicted demand from Gladhouse in the event of failure of the Megget aqueduct and therefore there was no reserve outlet capacity available for transfer of water from Gladhouse to Glencorse.

Improvement Works

32. The first stage yield from the Megget Scheme is 102.5 Ml/day, one half of the total estimated yield on completion of stage 2. Schemes for providing an additional outlet capacity from Gladhouse in the range 140 to 280 Ml/day were considered. The transfer capacity of the preferred option was in the range of about 200 to 300 Ml/day depending on reservoir water level and on completion of duplication of a section of aqueduct feeding Glencorse Reservoir. There was a requirement for operational reasons to maintain the security of supply from Gladhouse to the local treatment works and therefore it was necessary to keep the reservoir in service during construction of the new draw-off works.

33. The new draw-off arrangements are constructed within the reservoir embankment about 30 m to the north of the original outlet and located at a level to utilise 87% of the reservoir capacity (Fig.6). A single 1,400 mm inlet pipeline reduces to 1,100 mm within the draw-off shaft where two in-line butterfly valves are housed; the supply pipeline from the shaft is 1,400 mm and passes through the embankment core and downstream shoulder. The draw-off shaft is within the upstream shoulder clear of the puddle clay core, and both inlet and outlet pipelines are surrounded with concrete. The specification required that all works within the reservoir embankment be constructed in cofferdams.

34. The draw-off excavation was divided into three sections for cofferdam purposes with bulkheads between the inlet pipeline, draw-off shaft and outlet pipeline. It was a requirement that the former two cofferdam sections be constructed with steel sheet piling; the latter could be either steel sheet piling or timber sheeting. The cofferdams were to be driven to rock to form a base seal. The bedrock level was estimated from record drawings supplemented by borehole information through upper part of the upstream shoulder, core, and the downstream shoulder. The excavation method within cofferdams was not specified and therefore the excavation to the cofferdam within the reservoir could either be carried out in the dry or underwater. To ensure the security of the embankment during the cofferdam works there was a requirement that only one internal bulkhead be cut at any one time to permit pipework including concrete surround to be completed.

35. The inlet and outlet pipelines through the embankment comprise steel pipes with short sleeve welded joints, mortar lined internally and with an external concrete surround. The butterfly valves are electrically operated from headstocks located on an upper platform within the shaft. An air valve cluster is positioned on an access hatch downstream of the valves.

36. The draw-off shaft is 4 m internal diameter and 10.5 m high with a steel lining to the floor, walls, and roof (Fig.7). The void between the permanent steel sheet piling and the steel internal lining, and the upfill below the lining from rock level is mass concrete. The exposed section of the shaft above the upstream embankment slope is faced in masonry with access through a roof manhole at crest level. The roof incorporates a large access hatch for removal of valves or pipework. The steel lining was fabricated and protected to a similar specification to the steel lining described for Drumbowie Reservoir. The lining is 20 mm thick and short steel angles are provided externally to connect to the surround concrete. The floor is reinforced with steel beams and inlet and outlet pipes are welded to the walls to form waterproof seals. The prefabricated steel lining to the shaft was provided to give a leak free environment and for speed of construction.

Construction

37. Construction commenced with the draw-off shaft where the permanent sheet piling was driven to rock about 1 m below the estimated level. Shaft excavation was carried out in the dry by hydraulic grab with piling bracing added as the excavation proceeded. A good seal was obtained with the bedrock sandstone and the shaft was infilled with mass concrete to the steel lining formation level. The steel lining was supplied in three sections with connections made by butt welding insitu. The concrete filling around the lining was carried out in lifts to avoid lining deformation.

38. Sheet piling to the upstream and downstream cofferdams was advanced from the draw-off shaft (Fig.6). Downstream of the shaft the permanent piling was returned into the puddle clay core to form a water barrier. Excavation to these cofferdams was not commenced until shaft construction was substantially complete to minimise risk to dam security. Some ground movement occurred during excavation in the core area requiring an additional temporary sheet piling buttress across the downstream cofferdam at the clay core limit.

39. Piling penetrated to a greater depth than anticipated particularly in the upstream cofferdam requiring the freeboard above overflow level to be made up with temporary boarding for wave protection. Excavation in the upstream cofferdam commenced using a hydraulic grab operating below water level and with divers to fix walings and supports to piling. The excavation method was changed to carry out the work in the dry to improve progress; this required the piling support system to be redesigned to incorporate additional supports. The reservoir water level was also lowered about 2 m, pressure relief drains installed within the upstream cofferdam, and piezometers in the foundation to check uplift pressures. Investigation holes showed that the embankment was underlain by layers of bound sand and gravel, and dense clay/mudstone overlying the sandstone bedrock. The hard bound sand and gravel layer provided a suitable foundation for the inlet pipeline and its concrete surround.

40. In the downstream cofferdam excavation was continued to the sandstone bedrock and mass concrete make up was provided to the outlet pipeline which was also surrounded with mass concrete all contained within permanent piling. Above the inlet and outlet pipeline surrounds embankment materials including upstream protection pitching were reinstated and the temporary piling removed. A satisfactory final test of the outlet works was carried out with the reservoir at overflow level.

41. The draw-off works were completed by installing ladders and platforms within the tower and lighting facilities. Drainage from the tower was also provided. The draw-off works within the reservoir embankment were completed in 1986 at a cost of £390,000.

CONCLUSIONS

42. Draw-off arrangements at many old reservoirs are often the major elements requiring refurbishment and/or improvement. The access facilities in many cases are poor which inhibits inspection and maintenance work. Internal conditions of draw-off towers and access tunnels are also often damp or wet and with poor ventilation causing extensive corrosion of pipework fittings, ladders and platforms. Lighting is only provided in a minority of cases.

43. Structural integrity of the draw-off towers at Loch Cote and Drumbowie has been re-established and the internal conditions including access improved to such extent no special safety measures are necessary for entry. Space within the towers is still limited by the original tower sections retained within the embankment but not to such extent as to impair their function.

44. Use of all welded and corrosion protected steel cylinders to the Drumbowie and Gladhouse towers has provided an economic structural lining which is also completely watertight. The lining costs were about £20,000 (Drumbowie) and £30,000 (Gladhouse) which represents about 11 and 8% of the total cost of the remedial and improvement works.

45. Conversion of the Drumbowie tower from a "wet" to a "dry" shaft permits access for inspection and maintenance of pipework and valves, and the grouting of the new outlet pipeline within the old cast iron pipeline through the embankment has improved its security. The supply pipeline can also be isolated at the draw-off shaft if required in an emergency.

46. The Loch Cote and Drumbowie cases demonstrate different problems which can occur with draw-off towers with access bridges particularly where provision for bridge movement is not adequate or has not been maintained. The solutions in these cases are different but both require the reservoir to be emptied. The draw-off attention to the value of regular inspection if remedial measures are to be avoided or undertaken at an appropriate time.

47. The Gladhouse case illustrates additional difficulties caused by installation of major draw-off works in an existing reservoir embankment particularly where it is necessary to keep the reservoir in service. The importance of maintaining good construction records of all works is emphasised.

ACKNOWLEDGEMENTS

The cases described in this paper are published with the permission of Central Regional Council and Lothian Regional Council and their co-operation is gratefully acknowledged. The assistance provided by the Author's colleague, E McKenna, is also gratefully acknowledged.

REFURBISHMENT OF THE FOEL TOWER INTAKE TO THE ELAN AQUEDUCT

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SYNOPSIS

The 85 year old Foel Tower, located in Caban Coch Reservoir, Powys, is the intake to the Elan Aqueduct. Through this aqueduct, Birmingham receives over 95% of its water supply.

The paper describes an appraisal of the tower initiated in 1985 to assess the condition of the structure and its hydraulic equipment. Following the appraisal a programme of works was started and contracts let for replacement of the main control valves, replacement of the wet well draw-off and structural refurbishment. All the works were planned to be carried out without disrupting supply.

INTRODUCTION

1. The Foel Tower was constructed as part of the Elan Valley scheme authorised under the Birmingham Water Act 1892. The scheme embraced the construction of several dams in the upper catchment of the River Wye, together with a 118km long gravity aqueduct and storage and filtration works at the Birmingham end.

2. Work on the Elan dams started in 1893, but the Foel Tower was one of the last works to be constructed, being completed by 1902. The dams, Foel Tower and the upper reaches of the aqueduct were all constructed by Birmingham Corporation's Direct Labour.

3. Since its commissioning in 1904, the Elan system has been the main source of supply to Birmingham, and its capacity has been upgraded progressively to meet the increase in demand.

4. In the Water Act 1973 ownership of the reservoirs, including the Foel Tower intake and the aqueduct within Wales passed to Welsh Water while the remainder of the aqueduct was transferred to Severn Trent. A bulk supply agreement between the two Authorities ensured the future supply to Birmingham. In 1983 ownership of the aqueduct which lies in the Welsh Water area also passed to Severn Trent. A plan of the Elan catchment above the lowest dam, Caban Coch, and the five reservoirs, is shown on Figure 1. Thus the present situation is that the Foel Tower is owned by Welsh Water whilst the tunnel which leads from it is the property of Severn Trent.

THE FOEL TOWER

5. The Foel Tower is located in Caban Reservoir just upstream of the normally submerged Garreg Ddu dam. At times of drought, the water level at the tower can be maintained close to the crest of Garreg Ddu, either by discharging water from the upper reservoirs, Pen-y-garreg and Craig Goch or by transferring water from the Claerwen Reservoir via the Dol-y-mynach Tunnel. Figure 2 shows the relative levels and locations of the reservoirs and the Foel Tower.

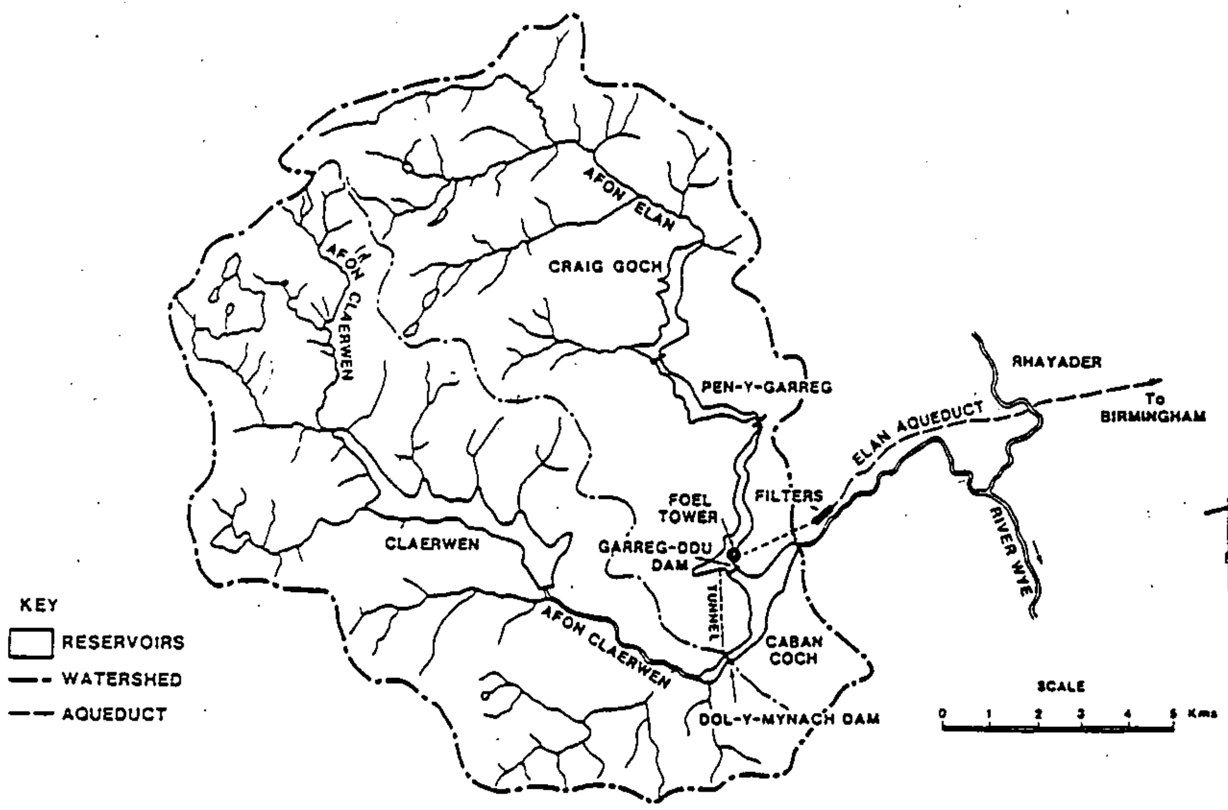

Figure 1: Elan Valley catchment and location of the Foel Tower

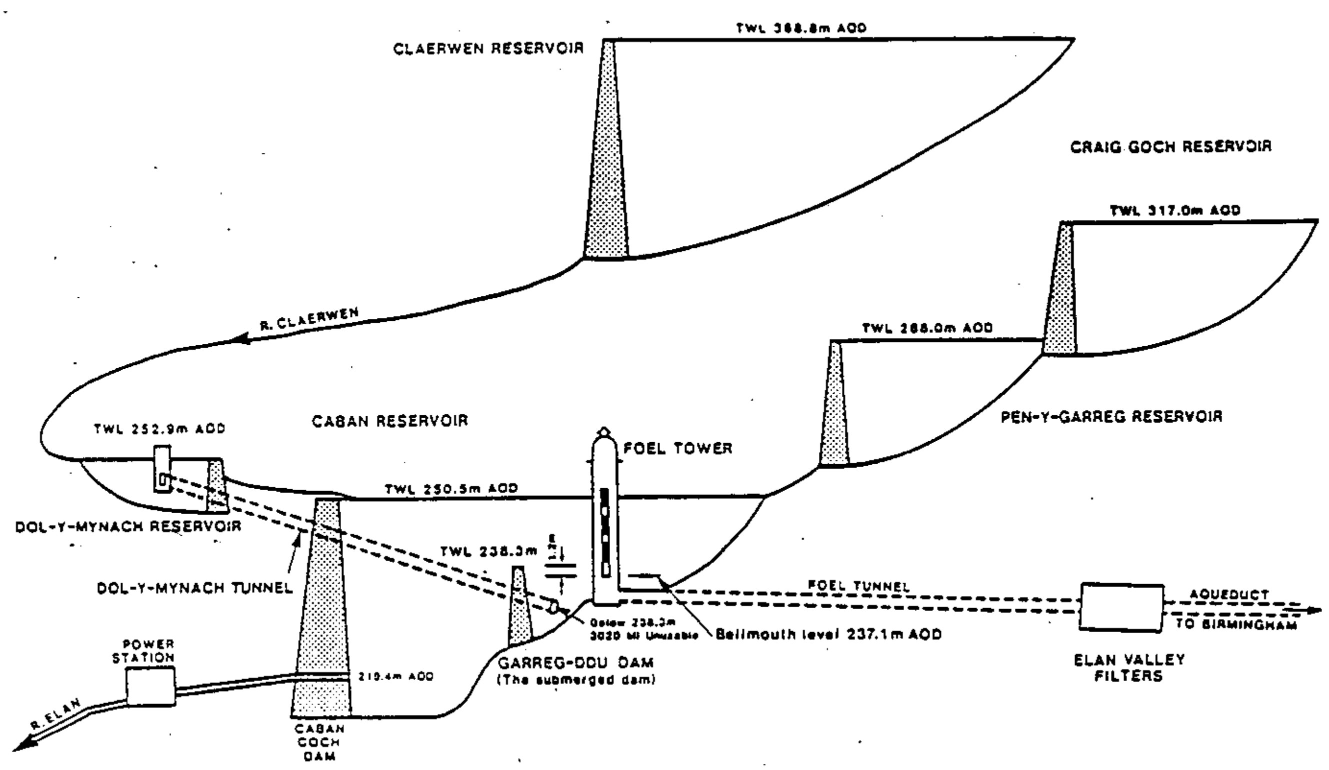

Figure 2: Levels of the Foel Tower and the Elan Valley reservoirs

6. In view of the strategic importance of the Elan supply, Severn Trent commissioned a thorough engineering appraisal of the Elan Aqueduct in 1984, and early in 1985, Welsh Water agreed to extend the appraisal to include the Foel Tower intake. Sir William Halcrow & Partners Ltd were commissioned to carry out the appraisal and the remedial works programme that followed, which are the subject of this paper.

7. The general arrangement of the Foel Tower is a circular masonry structure comprising a large wet well and a smaller circular dry well as shown on Figure 3. A detailed description of the original structure is given by Mansergh(1).

8. Three horizontal, carved granite bellmouths are constructed in the floor of the wet well at a level 1.2m below the crest of the submerged dam. Water can be drawn off at various levels by raising steel cylinders mounted above the bellmouths. Cast iron pipes built into the bellmouths pass beneath the dividing wall into the dry well, where the main control valves are situated and then outfall into the discharge chamber at the start of the Elan Aqueduct.

THE APPRAISAL

9. The objectives of the appraisal, broadly, were to assess the condition of the tower and its equipment; to review options to meet future service requirements; and to prepare a programme and budgets for refurbishment and future maintenance and the asset.

10. The appraisal team were asked to consider a number of improvements to the intake including:

- the prospect of raising the maximum capacity of the intake to 400 Ml/d, since abstraction at this rate might be possible at certain times of the year to maximise use of this cheap source of supply

- improving the control of abstraction. This is particularly important when the reservoir level is well down, and discharge is very sensitive to small changes in level. Remote operation of the intake was also to be considered

- retention of a degree of selectivity in taking water from the reservoir such that top water could be avoided, and water at a depth greater than 6 metres below the surface could be taken under most states of the reservoir

- improved ease of maintenance and enhanced safety.

11. The refurbishment of the tower was to be undertaken without disrupting the normal supply to the Elan Aqueduct. This meant that shut downs would be restricted to periods of 2.5 days every six months. In exceptional circumstances additional cut-offs of up to 6 hours duration could be arranged. At other times flow through the intake would be about 340 Ml/d.

CONDITION ASSESSMENT

12. With one or two minor exceptions, the structure and hydraulic equipment of the Foel Tower were original, prior to the current programme of works. However, the water in the Elan Valley is very acidic and the

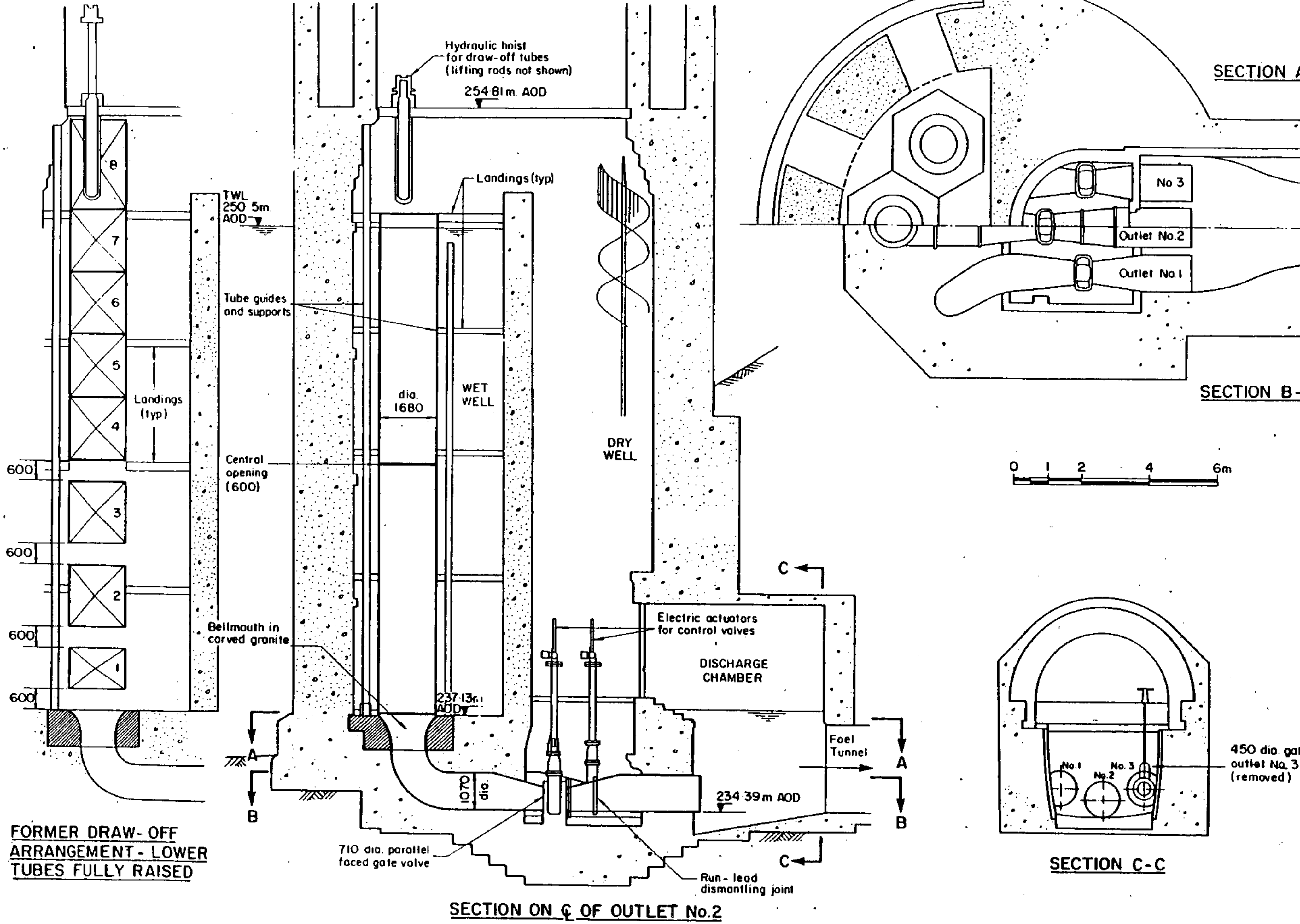

Figure 3: ...neral Arrangement of the Foel Towe...

steelwork had corroded over the years to such an extent that the residual strength of many of the components was no longer adequate. Severe operational demands related to the continuity of supply to Birmingham had prevented satisfactory repairs being carried out in the past.

13. The assessment of the operating equipment was carried out by reviewing records, discussions with operators, visual examinations and employing NDT techniques, such as the use of an ultrasonic thickness gauge and radiography.

Draw-off Tubes

14. The existing draw-off tubes are restrained by rollers running on vertical guide rails supported from the walls of the wet well at four intermediate landings. The steelwork forming these landings was severely corroded with holes penetrating the webs and ragged edged flanges in many places, particularly at sites within the "splash" zone.

15. The 8 steel tubes in each stack are 19mm thick, reinforced with a double thickness of plate at their top and bottom edges. The bearing surfaces are capped with a 6mm thick gunmetal strip intended to provide a water tight seal. Corrosion of the steel tube had, in most cases, led to distortion of the gunmetal and hence the loss of seal. This defeated an important function of the tubes, which should have been to provide an upstream guard facility against malfunction or failure of downstream control valves. Since for each stack there are over 40 metres of metal to metal seals it is unlikely however they were ever very efficient in this respect.

16. Each stack of tubes is lifted from a hydraulically operated crosshead by external lifting rods. The lifting gear was found to be in good condition.

17. Inspection of normally submerged items was carried out using divers during shut down periods. The divers were able to deploy a CCTV camera to offer visual inspection to engineers at ground level. The pictures of underwater steelwork were of reasonable quality though disturbance of deposited material was a problem.

Regulating Valves

18. Observations of the closed valves from the discharge chamber, revealed leakage suggestive of considerable wear. Calculations indicated that this could be the result of cavitation damage.

19. Internal access to inspect the valves was not possible due to the volume and force of leakage but radiographic examination was attempted from the outside of the valves and pipework, where these were exposed at the base of the dry well. This confirmed visual observations that only surface graphitisation 1-2mm deep had occurred, but gave no useful information on the loss of metal due to cavitation erosion.

20. To enable thorough internal examination, of the valves, a steel truncated conical plug was fabricated which could be lowered through the draw off tubes into the bellmouths. Mr F S Shaw of Plymouth Ocean Projects conceived the design for the plug and it was developed and fabricated in conjunction with Halcrow, as an urgent task prior to one of the bi-annual

shut downs. At a later stage, cofferdams to isolate the wet well were also designed and developed as a joint exercise.

21. The plug was constructed in two halves to enable access around the hydraulic lifting cylinders and to make handling easier. Two valves were fitted in the base of the plug to allow pressure equalisation and hence relifting of the plug; these were a 150mm 90° turn butterfly valve and a 150mm ball valve and 19mm hose connector for an airline.

22. During a shutdown, with the control valve closed, the plug was carefully lowered into one of the bellmouths by divers and sacking rammed around the outside to provide additional sealing. The control valve could then be opened to check for watertightness, and once this was considered acceptable, engineers were able to enter the pipework and valve from the discharge chamber, to carry out visual examinations. As expected serious cavitation damage was found, on the underside of the valve gate and the body with a maximum erosion depth of 40mm. The gunmetal sealing rings were, however, in reasonable condition.

REPLACEMENT CONTROL VALVES

23. The inspection indicated the need to replace the control valves at an early date. The replacement system would ideally meet the following requirements:

- o high capacity at low heads to meet the required aqueduct flow when the reservoir is drawn down
- o sensitive flow control and remote operational capability
- o no cavitation under normal operating conditions
- o rapid installation capability given the restriction on shut downs.
- o efficient energy dissipation, without excessive turbulence and air entrainment.

Options

24. Seven valve options were considered as replacements for the parallel faced gate valves; these included butterfly valves, terminal discharge (sleeve) valves and streamlined valves (Larner - Johnson). All alternatives however suffer some drawbacks including:

- o greater loss coefficient than a gate valve
- o more onerous cavitation characteristics than a gate valve
- o severe requirements from the civil works and programming viewpoint.

25. For these reasons replica parallel-faced gate valves were favoured, particularly with regard to ease of installation, and acknowledging that when used in pairs, damaging cavitation conditions could be avoided. The three new valves with electric actuators would increase the capacity of the intake at the lowest reservoir levels, and by extending the existing telemetry system, they could be operated remotely. The actuators and gearboxes could be mounted close to the valves on short extension spools.

26. The valves would be of the external rising spindle type, which lend themselves to the provision of accurate valve position indication, using direct coupled potentiometers; output from these can be converted to the standard 4-20mA signals required for control purposes.

Installation

27. The removal and replacement of each valve was effected in sequence. The bellmouth plug was installed upstream of the valve and a blank flange was fitted on the outlet pipework in the discharge chamber, so that the valve was isolated. A run lead dismantling joint on the downstream taper pipe offered the means of removal of the existing valve and jointing up of the new valve without disturbing any of the built in pipework. The new valves were ordered as size for size replacements for the existing valves.

28. Once a valve had been installed, a short shut down was arranged to allow removal of the blank flange and plug, and to carry out visual examinations for watertightness. Removal of the plug required divers to attach lifting slings to the outer edge, a rope to the handle of the butterfly valve and a compressor airline to the air inlet valve. The butterfly valve was then opened from ground level and the pipeline upstream of the control valve flooded; air was released through the air line. Finally, the plug was raised to the surface using hand operated winches. The airline also offered an alternative means of unseating the plug, were it to prove impossible to balance pressures using the butterfly valve alone.

29. Each control valve actuator was connected to the new control system immediately after installation and work was not sanctioned on another valve until the new one had been proved to be working satisfactorily.

WET WELL REFURBISHMENT

30. The requirements of the refurbished wet well arrangement are to allow a limited degree of flexibility in the draw off of water, to provide a secure upstream guard gate, and to be straightforward and relatively cheap to maintain.

31. Given the logistical problems associated with keeping the intake operational throughout the period of refurbishment, these requirements could only be met by a new system of draw-off tubes and guides, despite the high cost.

Logistics

32. Provided that water levels are at least one metre above the submerged dam the intake can operate on two of the three outlets. It would be possible therefore to dismantle and reinstate each draw off in sequence.

33. The bellmouth floor level of the wet well had never been exposed since original flooding of the tower. The existing tube guides are supported in cast iron 'shoes' at this level and in order to renew the guides it was decided that the wet well would have to be drained, and the reservoir retained at a low level for a period of several weeks.

34. Maintenance of low water levels in Caban Reservoir could be achieved by releasing excess water through the outlets in Caban Coch dam. This dam is provided with two 914mm compensation flow pipes and four 610mm scour pipes. Calculations and field trials indicated that this provision would be adequate to cope with conditions arising from excessive runoff and hence allow the reservoir level to be maintained within a narrow band. To give additional security, some spare storage could be left in the upper reservoirs without jeopardising supply strategy during the following winter refill period.

35. Before relying on this strategy, it was considered advisable to confirm that the scours were operable given that there was no record of their use previously nor certainty about the intake arrangement at the upstream base of the dam. A survey was therefore carried out using a remotely operated submersible vehicle (ROV) with on-board CCTV camera, to examine the scour intake pipework, followed by trial openings of the scour valves.

36. The means adopted for closure of the wet well was to install steel sliding gates (cofferdams) on the outer edge of the four masonry openings, using divers. Pulleys, winches and fixings installed above top water level formed the gate raising and lowering system.

37. During normal operation of the intake the gates would be raised about 1.5m above bellmouth level for water to pass freely beneath. At a shut down, and with a lowered reservoir level, the four gates could be lowered into position and water in the wet well drained out through the control valves into the aqueduct.

38. Lowering of the level of Caban Reservoir over an extended period could result in "loss" of water so far as supply to Birmingham is concerned and might also incur a risk of failure to meet supply in the future should refill not be achieved the following winter. These considerations led to the decision to extend refurbishment of the wet well over two consecutive summers during which Welsh Water would aim to achieve specified periods of low water. The contract for the work was thus based on two phases.

Refurbishment Design

39. The new arrangement retains the bellmouth intakes and their gunmetal seating rings as well as the hydraulic lifting devices. The tubes, lifting rods, guides and intermediate platforms are to be scrapped and replaced. Access will be provided by a hoist, and by ladders.

40. The new tube stacks are two lengths of tube each 7.35m long. Separate lifting rods will raise the upper and lower halves of the stack, thus allowing opening at mid height or at bellmouth level. Flexible seals are provided on the lower flanges of each tube length.

41. The three new guides and rails for each stack are offset by 60° in plan from the existing guides to allow installation ahead of dismantling the existing arrangement. Four intermediate supports fix the guides rigidly in position. Low friction, grooved sliding blocks attached to the tubes, positively locate them and prevent undue deflection by wave or hydraulic forces.

42. The tubes, lifting rods and all submerged steelwork are galvanised to a minimum coating thickness of 140 um giving an intended life of at least 20 years to first maintenance. The guide rails have a powder epoxy coating to give good corrosion protection and a low friction surface finish.

Contract Phasing

43. Phase one comprised the installation of the new guides, rails and intermediate supports. The work was completed by October 1987; it included an intensive 2.5 day operation during the aqueduct shut down in September when the new guide bases and lowest sections of steelwork were installed. This was the first time that the wet well had been drained in its 80 years of operation.

44. Work stopped at the end of September to allow the reservoir to refill over the winter. The next phase of the work cannot start until the reservoir can be lowered once again, in the middle of 1988.

45. During Phase two each stack of tubes will be replaced in sequence and then the old guides and intermediate steelwork landings will be dismantled and removed from the tower. A further draining of the wet well is anticipated in September 1988, to allow final adjustment of the tubes and to inspect steelwork installed the previous year under Phase one. Following the refilling of the reservoir, divers will be employed to remove the cofferdam gates and guides.

SUMMARY

46. Despite a long record of reliable service, the Foel Tower intake was found to be in a serious condition so far as its hydraulic equipment was concerned. An investment programme valued at over £300,000 was embarked upon to ensure the future of the asset and the security of supply to Birmingham. The work has been successfully carried out within the severe constraint of keeping the intake operable.

CONTRACTS

47. The principal contracts, contractors and value of work (excluding VAT) carried out, on the refurbishment of the Foel Tower is shown below; it includes some work, for example on the roof, which is not described in this paper.

Contract	Contractor	Value of Work (£)
1. Supply and installation of valves	Crane Service Division	59 000
2. Electrical and Control system	Chippenham Electronics	28 000
3. Diving services/Cofferdams	Plymouth Ocean Projects	23 000
4. Roof refurbishment	Norman and Underwood	21 000
5. Dry well refurbishment	Dawson (Structural) Engineering	8 000
6. Wet well refurbishment	Airload Engineering	175 000

ACKNOWLEDGEMENTS

48. This paper is presented by permission of Mr D A Jeffrey, Managing Director, Welsh Water. Particular thanks are due to Mr G R Patey of Welsh Water, Mr M D Bennett and his staff from Severn Trent and Mr J M Mather and Miss C M Thomas of Halcrow.

There were occasions, especially during the aqueduct shut down periods when a considerable amount of work was required in a short time. The authors would like to draw attention to the spirit of cooperation shown by contracting staff who worked on the Foel, even during these stressful periods.

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1. Mansergh, E L and Mansergh, W L (1912) The works for the supply of water to the City of Birmingham from mid-Wales. Proc. Instn. Civil Engrs CXC pp 5-90.

PROBLEMS WITH VALVES AT RESERVOIRS IN STRATHCLYDE REGION

P Gray MBA BSc CEng MICE MIWEM

SYNOPSIS

Three principal factors have contributed to the problems associated with valves at impounding reservoirs within Strathclyde Region. These are the age of the reservoirs, a lack of maintenance and a reluctance to operate valves, combined with concern at the consequences of such operation.

Since 1975, there has been a growing concern by both the Director of Water for Strathclyde Region and relevant Inspecting Engineers (and subsequently by Supervising Engineers) that draw off and scour valves should be readily operable.

This paper describes briefly the history of these problems, and gives examples of various repair contracts at impounding reservoir sites.

Problems have been encountered with old and new valves. . The types of valves involved are gate valves, butterfly valves, lifting bulkheads and sleeve valves.

INTRODUCTION

1. The problems associated with valves at reservoirs within Strathclyde Region stem from a number of factors.

a) The age of the reservoirs and their ancillary works.

b) A lack of maintenance which has occurred over the previous 25-30 years and which has led to particular problems.

c) The impact of the Reservoirs Act 1975.

2. The reservoirs which are the subject of this paper are those for which Strathclyde Regional Council are undertakers in terms of the 1975 Act and does not include those 100 or so reservoirs which are either privately owned or in the ownership of other public bodies and for which the Regional Council are the Enforcement Authority.

3. Table 1 shows a profile of the Region's reservoirs by age and type.

Type	Age of Reservoirs > 200 yr	150-200 yr	100-150 yr	50-100 yr	< 50 yr	Total
Earth Embank.	1	11	51	47	9	119
Conc. Grav.	-	-	1	6	8	15
Conc. Tank	-	-	-	1	6	7
Brick/ Masonry	-	-	2	-	1	3
Other	-	-	1	-	1	2
Total	1	11	55	54	25	146

Table 1: Reservoirs by Age and Type

With this number and range of reservoirs it is understandable that there is a wide diversity of valve types encountered. This in turn leads to a variety of problems arising with respect to ease of operation.

HISTORY OF UNDERTAKING.

4. In 1968, 199 local authority controlled water undertakings in Scotland combined to form 13 ad-hoc Water Boards. The original 199 water undertakings ranged in size from small burghs to the Glasgow Corporation Water Department and the amount of maintenance effort given to valve repair or replacement varied accordingly.

5. In 1975 as part of local government reorganisation in Scotland the 13 Water Boards were replaced by 9 Regional and 3 Island Councils, one of whose functions was water supply. Again during the period of Water Board control the amount of maintenance devoted to reservoirs and particularly valves was variable but relatively low.

6. Since 1975 Strathclyde Regional Council has recognised that a substantial sum of money was required to bring its 146 large raised reservoirs up to a reasonable standard. It was recognised that the eventual implementation of the Reservoirs Act 1975 would mean additional expenditure on most of the Region's reservoirs. To spread the impact of this it was decreed that the Department should operate, in so far as it was possible, as if the Act was on the statute book. A Reservoir Engineer was appointed and since 1978 the control imposed on operational engineers in respect of reservoir maintenance was greatly increased.

7. One recurring aspect highlighted in the Reservoir Engineer's annual reports was problems associated with valves. As part of his annual visit the Reservoir Engineer was required to witness the operation of every

critical valve at the reservoir site. The Director of Water became increasingly concerned with the great number of valves which were not in working order. Over the past ten years the situation has improved and the many valves at reservoirs now working are given an annual witnessed run up and down.

8. It does take a considerable amount of maintenance effort to keep abreast of such work. In some cases valves which worked at one annual inspection might be found to be inoperable at the next and vice versa.

9. Since 1975, significant among many varied jobs carried out on reservoir valve installations are the following:

KELLY RESERVOIR.

10. Kelly Reservoir, an earth embankment dam 10.5 m. high and 238,000 c.m. capacity was constructed in 1850 to serve Inverkip on the western edge of Renfrewshire.

11. In 1973, following an inspection under the Reservoirs (Safety Provisions) Act 1930, the Inspecting Engineer called for various works to be carried out. This work was agreed to be phased over several years.

12. Phase 1 involved the construction of an access road to give vehicular access for the first time. Under Phase 2 an existing 20" dia. mushroom valve on the outlet main, situated in the downstream toe within a brick and masonry chamber was to be replaced with an in-line gate valve clear of the embankment. The layout of the original pipework is shown in Fig.1.

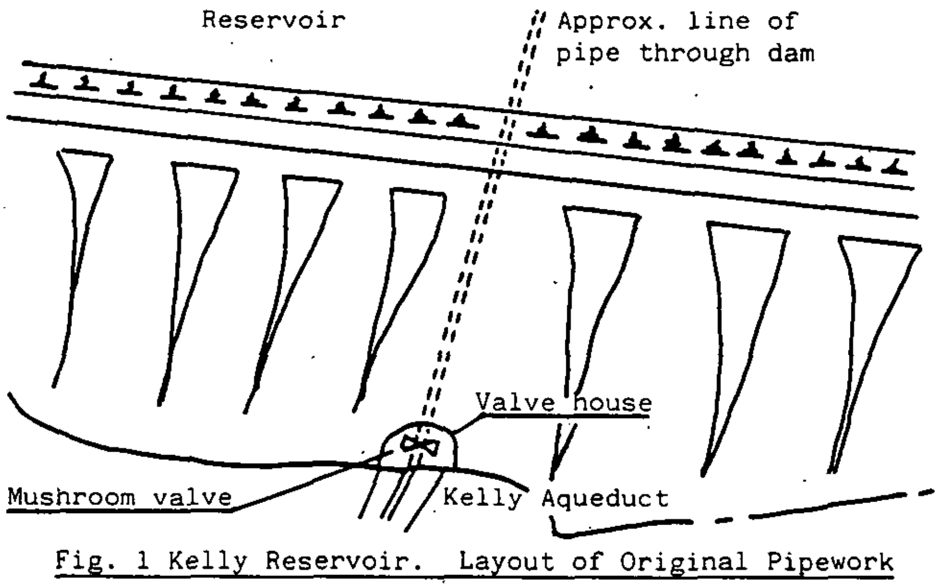

Fig. 1 Kelly Reservoir. Layout of Original Pipework

13. Scouring of the reservoir was achieved by opening the mushroom valve at the toe. This involved operating the spindle of the valve from a suspended timber floor above the valve. This action flooded the chamber, with water escaping by the front door of the chamber. In addition scour branches on the supply main could be opened. There was great reluctance

to routinely scour the reservoir because of the effect this had on the flow to the nearby treatment works.

14. The new works were designed to enable the reservoir to be scoured directly into the Kelly Burn through a new 450 pipe. In addition a new spillway and a wave wall along the crest were to be built. In the final phase, a valve tower was to be constructed together with the removal of silt from the floor of the reservoir.

15. There was a history of leakage from the pipe through the embankment thought to be from the vicinity of the valve. As part of the Phase 2 works divers were employed in the difficult task of sealing the upstream end of the pipe. This was achieved by inserting a neoprene coated, robust plastic plug in the bellmouth. This plug was provided with flexible fins around its circumference and incorporated a 100 orifice through it, controlled by a 100 sluice valve mounted on top. This latter enabled removal by equalising water pressure on each side. At this stage following a period of very heavy rain, a slip occurred on the downstream slope around the valve chamber. The bank, heavily saturated by the leakage from the valve and the steady rain, was presumably affected by vibration associated with the demolition of the roof of the valve house.

16. Emergency stablisation work was immediately undertaken utilising road stone from an as yet unused section of the new access road, to form a berm against the downstream toe. Access to the valve was maintained by installing a steel caisson around the valve together with a pipe placed vertically over the spindle.

17. Eventually the reservoir was drained by means of syphons and scouring from the supply pipe. A valve tower was built and a 2.4 m. dia. concrete culvert was laid through the dam incorporating a 300 supply pipe and 450 scour pipe.

KILMANNAN RESERVOIR

18. Kilmannan Reservoir was purchased by the Regional Council from the British Waterways Board in 1978 to supplement supplies to Clydebank. The earth embankment dam was built around 1770 to feed the Forth and Clyde Canal. At the time of purchase it was recognised that major remedial works were necessary to bring the dam up to an acceptable standard. Among other things the Inspecting Engineer asked that the outlet works be improved. The original outlet works were a timber built culvert through the dam. Sometime between 1930 and 1970 a 9" (225) pipe was threaded through and the annular space grouted. Its upstream end was controlled by a penstock with a sloping spindle running down the upstream face. This penstock resisted all attempts to move it. During the early attempts the concrete blocks anchoring the spindle in the face of the bank moved. Eventually divers were sent down to investigate and found the penstock covered with 2 metres of silt.

19. During the remedial works contract the reservoir was lowered by means of syphons and the existing outlet works were reinstated. In addition a new 450 pipe was laid, as an additional scour, through the embankment at a higher level, again controlled at its upstream end by a penstock with sloping spindle down the upstream face of the dam. Additionally gate valves were placed at the downstream end of each pipe. In subsequent reservoir inspections all valves have proved completely successful.

DAER RESERVOIR.

20. Daer Reservoir, completed in 1956 by the Daer Water Board is an earth embankment dam with articulated concrete core of 41 m. height and 25,500,000 c. metres capacity.

21. As part of a Statutory Inspection in 1978 the Inspecting Engineer asked for a test of the scouring capacity of the various scour outlets. At top water level the theoretical discharge capacity was 550 Ml/day. Unfortunately even with the inflow to the reservoir being well below average, and with outlets open, the reservoir level fell by only a small amount. This, together with the anticipated need to control the level of the reservoir during major remedial works contracts (1), confirmed the requirement to increase the scouring capacity.

22. In his 1978 report the Inspecting Engineer set out the existing pipework situation. "The main scour to the reservoir is a 36" (914) dia. pipe extending from the valve tower through the dam below the culvert. When closed it provides water to drive the turbines. It has a short length of 24" (600mm) dia. pipe and a 24" jet disperser at the outlet end. In addition there is a short 24" dia. scour branch off the 42" (1067) dia. supply main where it crosses the overflow channel." This was not included in the above test on account of possible supply difficulties to the adjacent treatment works.

23. With all inlets to the supply pipe open, this scour was capable of a greater discharge than the main scour, but even so the combined discharge was assessed to be somewhat light for emergency use in wet weather.

24. The Inspecting Engineer recommended substantial reconstruction works on both the upstream face protection and the overflow channel. Consequently he asked that a suitable means to control the reservoir level during these repairs be provided.

25. In January 1980 a further test of discharge capacity was carried out. This showed with 115 Ml/day being maintained to the treatment works, the main (36") scour could discharge 195 Ml/day while the 24" scour to the spillway channel was simultaneously discharging 235 Ml/day. During the 16 hours of this test the reservoir level dropped 200 mm. At the start of the test the reservoir level was 780 mm. below top water level.

26. It was determined that a new branch and valve should be taken from the 42" supply main at a position as far upstream as practicable in order to mininise adverse effects on the flow to the treatment works. The maximum size of the branch and valve was determined by the limiting size of the supply main and the space limitations associated with the options available. The choices were either to place the branch in the vertical cast iron supply pipe immediately above the duckfoot bend at the foot of the valve tower, or in the horizontal steel pipe immediately downstream of the bend.

27. In order to maintain access up the culvert the option of piping the scour discharge down the length of the culvert was rejected. Since the scour would discharge on to the floor of the culvert it was necessary to design some method to divert the water past the turbine house and into the spillway.

28. It was appreciated that it might be difficult to insert a branch into the vertical stack pipe and it was decided to weld a branch on to the horizontal steel pipe and subsequently to cut a hole through the pipe wall during a relatively short shut down period.

29. Then detailed consideration was given to the space limitations in the culvert and it was agreed that the largest size of branch which could be accommodated was 600 mm. The valve chosen was a 600 dia. wafer type butterfly valve both because of space limitations and ability to transport the valve up (and possibly back down) the culvert.

30. The 42" supply main runs along the length of the culvert on concrete stools. To reduce turbulence at the exit point and avoid possible damage to the supply main a short length of 800 dia. flanged pipework was laid downstream of the scour valve.

31. It was calculated that the discharge from the scour would be approximately 135 Ml/day. Under full head the throw of the jet would be 35 metres spreading to 20 metres under unrestricted conditions but at Daer the jet would be confined by the culvert walls.

32. Difficulties were encountered in the operation of the new valve shortly after installation and testing. Movement of the valve off its seat was difficult. Over the remainder of its travel operation was as expected.

33. To ascertain the nature of the problem the valve was inspected in-situ. No obvious distortions could be seen and the manufacturer was called in to advise on the likely cause. Meanwhile the valve was operated, as necessary, by hand.

34. The manufacturer's representative detected a slight distortion of the rubber seat. Initial possibilities were:

a) the valve was faulty.

b) The valve had been damaged by the contractor either in transit or installation.

c) operation of the valve may have caused damage.

35. Following the in-situ inspections it was agreed that the valve be removed and taken to the manufacturer's premises for detailed examination in order to determine the exact cause of the problem. At this time the manufacturer expressed reservations about the operating conditions of the valve. His concern centred on the calculated velocity of flow past the tongue in an open end discharge situation such as at Daer. The predicted flow velocity was 23m/s while the manufacturer stated, at that time, that the maximum recommended velocity should be no more than 5 m/s.

36. Fortunately the installation could, apart from access restrictions, be readily dismantled, with the new branch being blanked off and the supply restored during a six hour shut down.

37. During discussions with hydraulic engineers it was confirmed that the operating conditions for the valve were very onerous in that in the fully

opened position, with a differential head of 36.5 m., the flow velocity would be higher than it was prudent to operate for long periods. Apart from cavitation, excessive stresses could be experienced on the valve shaft, taper pins and gearbox.

38. The factory inspection of the valve determined that:

1. there was excessive seat distortion.

2. the shaft pins had sheared.

3. the actuator motor had burned out.

4. the actuator torque shaft had sheared.

The actuator torque shaft is located within the actuator body and should not be confused with the main valve spindle.

39. In a trial assembly, flange gaskets had been fitted and this could cause undue compression of the rubber seat of a wafer type valve. It was the manufacturer's opinion that the valve was either installed in the closed position or had been incorrectly aligned between the flanges. Either of these situations could cause a damaging overtorque condition.

40. It was agreed that the valve was undersized for its situation and that the flow velocity was excessive. This had not contributed materially to the problems to date but could lead to a reduced operating life for the valve. Accordingly, after the refurbished valve was reinstalled a 300 dia electrically operated sleeve valve was placed immediately downstream. With this done the sleeve valve became the scour valve and the butterfly valve is utilised as a guard valve.

41. The consequent reduction in capacity has been overcome by provision of a further 600 scour branch taken off a more recently laid bypass main to the treatment works.

GLENGAVEL RESERVOIR

42. Glengavel Reservoir is another earth embankment dam in Lanark Division built in 1898, 21 m. high and 2,027,000 c. metres capacity.

43. At Glengavel the scouring problem was different. Theoretically the capacity was adequate but the operating mechanism in the valve tower was immovable. It consisted of a counterbalanced carbon steel gate across the top end of the culvert through the dam. The means of raising the gate had long since fallen into disuse and some years ago the counterbalance was removed for fear that the chains would break and the large weight fall the height of the tower and smash the supply pipe. Any required lowering of the reservoir level was done by means of relatively small sized scour branches off the supply main.

44. Following submission of a report, under the Reservoirs (Safety Provisions) Act 1930, it was agreed with the Inspecting Engineer that the gate would be removed and replaced with a mass concrete plug incorporating a 600mm pipe and 600mm gate valve.

45. In addition thr Director of Water required that the draw off valves in the tower be refurbished. Access to the top and bottom of the tower was also to be improved. Access along the culvert through the dam was not easy given that the supply pipe supports spanned the culvert at a height of 1 metre above floor level. The access bridge to the top of the tower was showing signs of severe corrosion and required replacement.

46. For lifting of the scour gate various contractors were approached in order to draw up a select list of tenderers. Only one experienced contractor was willing to get involved because of the unknowns inherent in this unusual type of job which was let on a fixed price basis. To accommodate the work the reservoir was partially drained by removing a bend from the 27" (686) supply pipe downstream of the end of the culvert. With the reservoir at this level freeing and lifting of the gate was attempted by means of two ten tonne chain blocks. This was based on the contractor's previous experience but proved unsuccessful.

47. Next the bottom of each edge post was cut out to allow the addition of two fifty tonne hydraulic jacks. These along with the two ten tonne chain blocks succeeded in raising the gate about 50 millimetres. This was not enough to lower the reservoir level sufficiently quickly to meet the programme dates of the associated civil engineering contract, even if the inflow to the reservoir was zero.

48. The contractor next tried removal of the lintel bar which was seen to be in contact with the skin plate of the gate and it was thought that encrusted rust could be preventing further movement. Plates with lifting eyes were welded on to the roller trains and connected to the chain blocks. By this means it was hoped that the roller trains could be induced to go with the gate, again failure. Next the gate roller trains and the lintel were pressure jetted. At this point accumulating material behind the gate stopped water flow from beneath the gate. The opportunity was taken to replace the two 50 tonne jacks with two 200 tonne jacks. This succeeded in raising the gate by a further 100 mm. which was enough to allow the reservoir to empty. Thereafter the metalwork of the gate and guides was removed with oxy-acetylene cutting gear.

49. The culvert was then plugged with mass concrete incorporating a 600 dia. pipe with a 600 scour valve at the base of the tower. This valve was designed to be operated manually from the top of the tower by means of an extension spindle incorporating universal couplings and 'frictionless' bearings.

50. With the reservoir empty the opportunity was taken to renovate or replace as necessary the valves in the valve tower. On completion of this work, operation of all valves was checked and found to be satisfactory. The reservoir was then allowed to fill under controlled circumstances over the next winter period.

51. The following summer, remedial work on the spillway channel was started and the reservoir was drawn down by 2 metres to cater for expected levels of rainfall without any overflow occurring while contractors were working on the channel. At a time when sections of the channel floor and walls were broken out and ready for reconstruction a period of exceptionally heavy rain caused the reservoir to rise suddenly and threaten an overflow situation. The reservoir keeper attempted operation of the new scour valve

and found it very difficult to move. Additional men were called in and eventually managed to open the valve sufficiently to gain control of the reservoir before an overflow occurred.

52. The original specification stated that the valve should be capable of one man operation under an unbalanced pressure of 40 m. head. Difficulties were experienced with a differential head of around 16 metres.

53. On initial investigation, the varying degree of stiffness led to a suspicion of a slightly bent spindle. However in a subsequent test, carried out with a blank flange fitted downstream and pressure equalised both sides of the valve, operation was easy and this tended to discount this theory.

54. At this stage it was not known whether the difficulties were due to problems with the valve itself or with the long extension spindle running the height of the tower. The valve was disconnected from the spindle to allow an independent assessment of the performance of each.

55. It was later agreed with the valve supplier that given the inadequacies of an in-situ inspection, the valve be replaced and taken to the Water Department workshops for a more thorough examination.

56. The results of this examination indicated that the manufacturing tolerances of the valve were such as to give rise to the major part of the difficulties experienced. The manufacturer did not accept this and suggested that the flow velocity past the gate was causing the problems. He proposed the fitting of a diffuser plate with a relatively small orifice downstream of the valve. This was rejected as unacceptable since this would seriously reduce the discharge capacity of the reservoir scour.

57. The valve supplier was instructed to supply a new valve, fitted with 4:1 gearing, which it was considered would adequately overcome the difficulties experienced with the previous valve. It was concluded that the valve manufacturer had been somewhat remiss in originally offering an ungeared valve for one man operation against an unbalanced head of 40 metres. It was felt that because an integral 70 mm bypass had been specified, the supplier assumed an in line situation with much reduced head loss across the valve. Since fitting the new valve no difficulty has been experienced during the regular operation at the reservoir.

BURNCROOKS RESERVOIR.

58. Burncrooks Reservoir to the north west of Glasgow, supplying Clydebank, posed an interesting problem in the late 1970's. The reservoir was constructed in 1959 and the outlet works consist of a dry tower incorporating a vertical 24" cast iron pipe fed by four 18" draw offs and a 30" scour pipe passing through the base of the tower. The scour valve on this pipe was 24" diameter located at the base of the valve tower. Each of the four inlets to the tower and the scour pipe had a flap at its outer end, these flaps being raised manually by means of a windlass through a system of heavy chains and brass rods.

59. In 1977, a relatively dry year, problems were experienced at the treatment works served by the reservoir. Sludge was rising to the top of the first stage settlement tanks and this was found to be caused by air in the incoming water, like a crude flotation process. On investigation at

the reservoir it was found that the top of the second draw off was just above water level. The external flap valve and the internal draw off valve were both open and air was being drawn in intermittently through wave action. The lower draw offs were found to be closed. The operatives attempted to open the flap valves on both the third and fourth draw offs. This proved to be impossible and divers were called in to check these. The rod and chain mechanism was found to be dislocated and the divers were instructed to remove the heavy flaps from these two lower draw offs.

60. With the divers on site opening of the scour valve was attempted. The divers had reported that there was some silt around the base of the tower at the scour inlet. The rod and chain mechanism proved unable to raise the flap and was therefore replaced by a wire hawser which has operated satisfactorily ever since. The keeping of the flap in front of the scour has enabled test operation of this valve with no fear of damage downstream of the reservoir.

61. During the last statutory inspection of the reservoir it was indicated to the Inspecting Engineer that if he wished to witness the opening and closing of each valve in the tower this could take more than a day since each of the five valves is very highly geared. Accordingly the Inspecting Engineer indicated that this was not a satisfactory situation but he wished some assurance that each valve was operable over its whole range. In conjunction with the Supervising Engineer a recording system for valve operation was set out, such that over a period of 36 months every valve was opened and closed fully by stages.

ACKNOWLEDGMENTS

62. The author would like to thank the Director of Water for Strathclyde Regional Council for his permission to present this paper and is grateful for all of the help given by colleagues in its preparation.

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PROCEEDINGS: TECHNICAL SESSION 3

RENEWING AND UPDATING DRAWOFF WORKS

Session Chairman: M F Kennard

Good afternoon Gentlemen. This third session deals with the renovation of valves and outlets at dams. The Authors include Gordon Gregory and Ian Hay who are talking on renewing drawoff works at two dams in Yorkshire, Douglas Gallacher on remedial and improvement works to reservoir draw-off works of 3 dams in Scotland, Tim Ingham and Jack Logan on refurbishment of the Foel tower intake to the Elan aqueduct in Wales and Peter Gray on problems with valves at reservoirs in Strathclyde recently.

A D H CAMPBELL (Fairhurst & Partners)

With reference to Mr Gallacher's paper a common cause of cracking on valve shafts at the point where they come out of the embankment is ice pressure from the reservoir face.

D GALLACHER (R H Cuthbertson & Partners)

In the case of the first tower, Loch Cote, I think we do mention in the paper that ice is possibly a cause, but on inspection of the bridge there all the bearings were in fact seized up and things were twisted. We felt that other things could be contributing but that the major cause was in fact the bridge, but I take your point regarding ice pressures.

M F KENNARD (Session Chairman)

There are a few points that occurred to me in the paper that might be worthy of discussion. One has partly been raised already: what does the inspecting engineer do when the owner refuses to operate a valve. I have had the argument that 'we've got to be able to be sure we can shut them afterwards'. I would have thought that in a major reservoir, it is important that they are operated, and if the consequences are that they can't be completely closed then that has to be dealt with at the time.

I would also like to raise the point for discussion: how far is it essential to have bottom scour valves on minor reservoirs? My own personal view is that there are many cases where it does not matter. It is not worth interfering with the dams too much if there are no such bottom scours. In the papers there was mention of problems with butterfly valves and I wonder whether the engineers here have any views on whether butterfly valves should in fact be used as partially opened control valves.

W J CARLYLE (Binnie & Partners)

Talking about Mr Gray's paper on valves in the Strathclyde region, I would like to draw him out on what he regards as a satisfactory bottom outlet discharge capacity. I do not like the use of the word 'scour valve' - anyone who has tried to use them in that sense finds they just simply do not move any sediment from the bottom of the reservoir after the first five minutes. I take Michael Kennard's point, but in my view a satisfactory bottom outlet capacity is a necessity on any reservoir. The question is, what should the capacity be? I was a bit surprised at the suggestion that a thirty-six inch common outlet on Daer Reservoir was not adequate. There are a good many reservoirs with nothing like that capacity. Again there aren't any rules, certainly in the international scene, but I think the Swiss have given a lead on this, that you should be able to lower the reservoir capacity by half in a matter of two or three weeks. That could be satisfactory in low volume high head alpine reservoir, but certainly is not appropriate in much larger reservoirs in other parts of the world where the heads involved are much lower. So I would be grateful for Mr Gray's comments.

The second thing that comes out of all these papers is that some of the very old ironmongery has been very well thought out and has worked extremely well over many years. There is a tendency for modern replacement not to be anything like as good, in that people are often using waterworks valves in regulation mode. Although there are a number of variants of the ordinary sluice valve, by and large once these discs are off seat the guides are very crude so you have a loose gate used in a mode which it was not designed for. If heads are low you may get away with it but many of these old reservoir engineers had clearly thought out the need to have valves, even though the heads were not particularly high, where there was no freedom for cavitation to commence. One final point is the Gladhouse Reservoir draw-off. There was a replacement draw-off works on a 1400mm steel pipe and I think the valves were 1100m butterfly valves, and I noticed the installation of an air valve immediately downstream of the second valve - I presume the second valve is the control valve, and the upstream valve is the guard valve. I would be interested to know what the designer thought the air valve was doing. I believe it is a very good feature, in that if there is a tendency for cavitation to develop there and if the pressures there drop low that air valve could admit air, but I wondered, did he put it there to admit air or to let air out?

L JACK (Welsh Water)

On the subject of the Foel draw-off itself, we were worried at the beginning of the job as to whether we could control the water levels sufficiently. We stated there were severe operational constraints, and the position is that the supply can only be shut off for three days in September and three days in March. So all the work that requires drying out of the tower has to be done in that three day period. The water level has to be at a sufficiently high elevation to restore the supply to Birmingham immediately at the termination of that three-day shut off. So we had the arrangement of the sliding gates, which worked extremely well. At that point, perhaps I should commend the designers and the contractors for the successful completion of the job yesterday, and say that scour facilities were available in case of very wet weather: four 24 inch scour lines at the dam and two and a half of these are now operational. We had the usual worries: would they work, would they not work, and they had not been operated for a number of years. We now give them regular testing twice a year, and we have, not the full scour capacity, but enough to bring down the reservoir fairly quickly.

P GRAY (Strathclyde R C)

I would like to talk about the issue raised by Bill Carlyle, on the necessity to increase the capacity at Daer bottom outlet valve. Traditionally I use the term 'scour valve' but we very rarely use them as a scour mechanism; they are there principally to lower the level of the reservoir when needed. We weren't particularly concerned at Daer about the existing lowering capacity that we had. I indicated in the paper that we didn't do a test and the weather was relatively kind at the time and the level fell imperceptibly over a period of 8 hours.

The particular remedial measures were replacing the upstream protection and we felt that we could well be caught with areas of the concrete facing removed and be hit by a summer storm. We wanted to be sure that we could keep the level down.

D GALLACHER (R H Cuthbertson & Partners)

The butterfly valves at Gladhouse are really open and shut valves. They are not firstly control valves. The air valve downstream is, as Mr Carlyle said, to get air out of the pipe when you are actually charging the pipe, or if any collects there but, also, to let air into the pipe when the pipes downstream are drained. You fall away from the particular point so obviously you must let air into the pipe.

S M HAWES (Consultant)

My concern has mainly been with agricultural reservoirs of less than 50 million gallons and unless they are impounding we have avoided scour pipes like the plague; instead putting in syphon pipes which are primed at top water level on the principle that the only time one needs to use a scour valve is in the event of over-topping or cracking in the top. Therefore if one can drop the water level by about a metre in 24 hours one is going to get down to a much thicker section of embankment. The smaller sizes of reservoir can, in any rate, be pumped out in the present day and the disadvantages of pipes in embankments, in those circumstances are well worth avoiding.

In the case of impounding dams, then one has to put a pipe in to take the flow through while they are being constructed, but this enables a tower to be put in and a valve to be put in to let the water level down if you want to but, generally speaking, there doesn't seem to have been many cases when the scour valve was necessary.

A I BIELDERMAN (North West Water)

I'd like to take up Mr Kennard's query relating to submerged butterfly valves. North West Water installed submerged butterfly valves in at least two reservoir locations about five years ago. I was personally involved in the latest of these installations. The valves have been installed and have been operated regularly and have proved serviceable in duties where only open and shut positions are required. They were installed in guard valve locations without any upstream valve tower.

One or two points of note. These valves are operating up to about 25 metres head and in one location it proved that there was inadequate gearbox torque capacity and that had to be altered in order to get full operation under full reservoir head.

On the job that I was concerned with, the downstream supply valve was also put on to hydraulic operation, which proved not sufficiently sensitive to give the degree of flow control that was necessary for supply regulation: additionally the valves tended to close over a fairly lengthy period and the hydraulic operation again was not very suited to correction of the original position, so I would suggest that hydraulic operation be used just for open and shut positions for valves and not at this stage for flow regulation.

C J A BINNIE (W S Atkins)

I would add support to Mr Carlyle's view that scour valves are not scour valves effectively. I have seen several reservoirs where this has been tried and it's been absolutely useless apart from a small channel out into the reservoir with minimal storage. Emergency draw-down valves I am sure is a much more applicable title for these. Concerning the rate of draw down, I believe this is a function of the likely problems in the dam itself and what might occur to it and that will be a factor very much on its age and its condition and also the conditions downstream - whether the failure of the dam or serious breaches of the dam could affect lives downstream. You balance these factors together in order to try and assess what a suitable draw-down rate would be. I am sure that many panel engineers like myself have inspected many small dams which have got no emergency draw-down arrangements and it would be very expensive indeed to put in - more than the value of the dam and reservoir itself.

My personal view on that is that I accept pumps under those conditions. We do try and make sure that there is sufficient access to the reservoir to get the pumps there (and quite often farm tractors will do), and that in the reservoir book is a note of where the pumps could actually be obtained, and that the supervising engineer checks up on a regular basis that that organisation can still provide pumps.

G ROCKE (Babtie Shaw & Morton)

I would just like to talk about some very large butterfly valves at Keilder Reservoir. Some of these valves run up to six feet in diameter and have been giving good use since they were installed about 1979/80. One or two things did happen to them though. We found that during erection, when you were trying to connect across between two parallel lines on a right angle connection that sometimes the fit is not too good, and of course the fitters, rather than be beaten, tend to tighten up too much on one side than the other, and the result was distortion occurred in the body of the valve and caused leakages which were very troublesome and trying. They are very difficult to spot until eventually we de-watered and carried out precise surveys across the diameter to find there was distortion. Large valves of that size have got hydraulic seals which are filled by hydraulic pressure from outside the reservoir. This is a very delicate operation and we found that the solenoids which controlled the entry of water into these inflatable seals became jammed up. We could not understand why.

Dismantling them, we found that, with an iron rich water at Kielder there was a small deposit rather like a bentonite cake forming inside the solenoid valve. Eventually, we had to cure that by putting a head water tank in the valve tower of pure water to operate the rubber seal inflation. They are now working perfectly well. The sleeve valves which we use for discharge downstream have worked reasonably well, but we have encountered what I am sure other consultants have encountered, and that is splash back coming violently off any concrete surface on the trajectory of the jet. Splash back, I believe, has been encountered by Gibbs when they tried many experiments and is written up. We have encountered it at Kielder where we house the sleeve valves inside a concrete surround, and this has of course caused stripping of the grease on the side spindles on the sleeve valves. We have recently, by consulting the airlines on what they do in the extreme conditions up there, found a grease which resists and it is now staying on and making a great difference to the operation of the side pinions.

In our firm, we tend to use bulkhead gates ahead of in line valves, especially the big ones, so that the bulkhead gate, which is kept in the valve house, and can be maintained and looked after, will be dropped down during an emergency to shut off the main inlet and allow the removal of the valves downstream, and this ensures there is no last minute panic, because you can maintain that bulkhead gate to a high standard.

We have used pot valves, where the valve discharges through perforated vertical cylinders. These have caused two troubles: one: excessive wear, which generally turns out to be somebody dropping stones down, or a bolt being left in the annulus around the pot valve and that bolt swivels round and causes colossal erosion. So these cylindrical valves have to be very thoroughly cleaned out and kept cleaned out.

On the size of valves for removal of water from a reservoir, we have tended to go for something similar to the bank-full condition of the incoming river, which in Scotland can be five or six times the average daily flow. That is a general guideline, but it can be larger than that. I should say that I have encountered conditions where you are in danger of making your scour too large, because unless you examine the mile or two downstream of your dam, you could be causing riparian owners considerable trouble by opening up your valves too much. So you should not size them too large, or you could be getting into fresh trouble.

R M ARAH (Binnie & Partners)

Following Mr Rocke's contribution, I think there must be emergency situations when you would much prefer riparian damage downstream to an overtopped embankment. If you provide a large scour it does not have to be used, but somebody has the option of taking the decision to use it and drop the level quickly. I believe that the norm is to have a large bottom draw-off which could drop the level quickly in emergencies. If you have not got it, I think you have to have a very good back-up system and contingency planning, or something like syphons if you are really going to face up to the prospects of emergency draw downs.

M F KENNARD (Session Chairman)

If you haven't got such a large bottom drawoff, would you consider it necessary to install it?

R M ARAH (Binnie & Partners)

Certainly in some circumstances it would depend, as another contributor said, on the age of the dam, the state of the river downstream, and what was at risk. Yes, I believe you must certainly think of that option. I do believe we ought to all be saying what we think are reasonable rates of draw-down, at which we do not have to take any further action. Let me suggest a metre in a day as the level at which you could hardly ask for more in a normal reservoir.

M F KENNARD (Session Chairman)

Do you mean a metre a day at full reservoir level?

R M ARAH (Binnie & Partners)

At full reservoir level and disregarding any inflow, talking of a rate of draw down in the simplest way.

M F KENNARD (Session Chairman)

Representatives of dam owners, do you have anything to say on that?

L JACK (Welsh Water)

I am not sure that you should be asking the dam owners, or whether you should ask the inspecting engineer who inspects the dams for us and sets these standards. For what it is worth, as I said earlier, we have a problem with a particular dam, in that we cannot get it down if it rains at all hard, and we are in the process of making decisions on this. We have priced getting a scour operational, and we are rather frightened about the price, and we are looking at other solutions to the problem in terms of the possible mechanisms of failure and looking to minimise the risks associated with the other considerations as an alternative to spending rather a large amount on getting a scour operational.

T KINGHAM (Sir William Halcrow & Partners)

Briefly taking up the point Mr Carlyle mentioned on valve types in relation to the Foel. I fully support his view of the original designers of these valves who put a great deal of thought into their construction and how they were going to work and the ones at the Foel were probably no exception either. These were parallel gate valves, so that there was support all the way down for operation in this position as a control valve, and we were anxious to provide something at least as good as that when we were looking at options.

In the event, we went for the same type of valve, largely because of the civil engineering constraints, the restriction on space and the fact that we were looking for a valve with a very low loss coefficient in the fully open position and you really cannot beat a fully open gate valve for that. So we were able to persuade Glenfields to dig out their old patterns and cast another gate valve exactly along the lines of the old ones.

J D HUMPHREYS (MRM Partnership)

In 1975, the hydrologists got together and made sure that panel engineers would be busy for the next 10 years. I think that Mr Arah, if his criterion is accepted for what I agree with Mr Carlyle shouldn't be called scour valves, bottom draw-off valves should be the capacity to draw down the reservoir at 1 metre a day, I think that we can look forward to another 10 years of full engagement, and I thank him for it.

R M ARAH (Binnie & Partners)

I am not sure that I would go along with John Humphreys in welcoming 10 years of such work. I think there will be considerable problems and I believe in many cases leave well alone: but I was suggesting a metre a day as a remarkable provision for dealing with drawdowns by simple discharges. If that facility is not available the alternatives should be considered.

WRITTEN CONTRIBUTIONS

R Y GIBSON (Gibson Civil Engineering Ltd)

It has been with considerable interest that we have heard the papers and discussion on draw off works and on scour valves in particular. Lest we get carried away in our enthusiasm to run scour values up and down on the annual visit, we supervising engineers should first take account of the risks and responsibilities involved in such an operation.

1 Whose valve is it? Who is operating it, under whose instruction?

2 What will be the effect of sudden increased flow downstream? Will the stream carry the flow within bank top? Is it a fishing river? Campers? Is it school holiday time? Has anyone walked the first couple of miles looking for obstructions or works in progress? Have the Police been notified?

3 What is the quality of water being released?

4 What effect will silt and turbidity have on stream culture?

5 What if the valve jams open? Cavitation? Silt? Cost of Repairs? Loss of Revenue? Rapid draw down effect?

By all means let us get Britain's reservoir controls operational. At the same time however, 'festina lente'.

L THAKUR (North West Water)

In 1975/76 I was a project engineer representing T & C Hawksley, one of the joint consultants with R K & L and Binnie & Partners, involved on Shap Aqueduct scheme for Manchester corporation Water works.

The existing 36" scour pipes within Haweswater dam were to be converted into new suction pipes for the proposed Haweswater Pumping station being built downstream of the dam.

One of the main operations was to replace an existing 36" leaking sluice valve downstream of the guard valve by a butterfly valve.

To my knowledge this valve has been operated satisfactorily for control purposes. I feel that there is a place for butterfly valves in dams.

B A HUTCHINSON (City of Bradford Metropolitan Council)

Mr Gregory and Mr Hey in their paper 3.1 used one slide to illustrate the paper which showed the supervising engineer inspecting a valve seating in a confined space.

Could the authors outline the method used for entry into this confined space? Was a written 'permit to enter' issued? What was the back-up/rescue system, and what safety/rescue equipment was available on site?

J HAY & C G GREGORY (Rofe Kennard & Lapworth)

The Engineer's inspection was made under the Authority's normal written Safe Working Procedure for access to the valves at Dale Dike as follows:-

1 <u>Prior to leaving Bradfield Filter Station</u>

(a) The Supervisor was notified of the proposed time of entry into the tunnel and an estimated time for completion of the inspection.

(b) Safety equipment, including 2 Oxygen Deficiency Meters, 2 Breathing Apparatus Sets and sufficient torches for the party and standby personnel, were checked.

(c) To provide standby, the engineers inspecting the tunnel were accompanied by 2 personnel qualified in the use of breathing apparatus.

2 <u>At Dale Dike Reservoir</u>

(a) Following the closure of the upstream valve, one of the standby personnel equipped with breathing apparatus and an oxygen deficiency meter checked the tunnel for sufficient oxygen.

(b) On his return, the engineers entered the tunnel with an oxygen deficiency meter and carried out their inspection. The two standby personnel remained at the tunnel entrance with the breathing apparatus and maintained verbal communication with the party inside.

(c) On completion of the inspection, the valves were reopened and the Supervisor notified accordingly.

The tunnel at the valve shaft is vented into the shaft via ducts and through the vertical standpipe. To date there has been no recorded problem of oxygen deficiency at Dale Dike.

DR A K HUGHES (North West Water)

As an undertaker NWW has been involved with a large number of projects involving replacement of valves, provision of guard valves and refurbishment of valves.

In the case of provision of new valves in addition to existing valves within valve shafts of limited size 'thin' valves has been often used, some of which have caused engineering and operational problems.

In general I would suggest that the purchase of 'cheap' valves is false economy and that the adoption of simple and tried and tested designs should be adopted where possible.

PROCEEDINGS: TECHNICAL SESSION 4

OVERFLOW REPAIRS AND EXTENSIONS

PAPERS:

DISCUSSION:

HIMLEY HALL GREAT POOL OVERFLOW AND STABILISATION WORKS

P F Johnson CEng MICE
PWD Dudley Metropolitan Borough Council

E A Jackson CEng MICE MIWEM
Sir Alexander Gibb & Partners

SYNOPSIS

1. The 400 metre long Great Pool dam was built 200 years ago by "Capability" Brown and is a classified listed building within a conservation area. The lake was, until April 1988, jointly owned by two Local Authorities and within a third Local Authority District.

2. Following the 1970/71 statutory inspection, a new spillway of 3.5 m^3/s capacity was constructed and the crest free-board increased. A further inspection in 1980 concluded that the dam should be classified under category 'A' and that the overflow capacity was insufficient for the expected PMF of 35 m^3/s.

3. Following consideration of many options, remedial works were completed in 1987 at a total cost of £450,000 which included an emergency spillway that discharges across a trunk road.

OBJECT

4. This paper illustrates the procedural and technical problems that have arisen concerning an ornamental lake which is required to comply with the Reservoirs Act. Compliance with the Act, from the initial appointment of the Panel Engineer to undertake a periodical inspection to final certification following implementation of remedial measures, took a period of 8 years.

BACKGROUND

5. Himley Hall and its grounds were formerly the home of the Earl of Dudley, whose family had developed the grounds over several hundred years. Whilst the Great Pool dates back to pre-18th Century, it was of a very different size and shape to that of its present form. The grounds of the present Hall were landscaped by "Capability" Brown during the mid-18th Century at which time it is most likely that the dam to the Great Pool was constructed. Great Pool is the lowest in a series of four artificial pools, all of a similar construction, and formed by the impounding of a stream known as Himley Brook. The catchment area draining to the Great Pool is about 3.4 km^2, mainly parkland, and the lake has an area of some 6.6 ha.

6. Part of the estate now known as Himley Park, including two pools - Great Pool and Rock Pool - was sold by Lord Dudley to the Coal Board upon nationalisation. In 1966 the park was sold to the Boroughs of Dudley and Wolverhampton and Dudley M.B.C. became the sole owner in 1988.

7. Great Pool is used extensively by fishermen and is one of the best coarse fishing waters within the area. It is leased to a local sailing club for dinghy sailing and training.

8. The dam which retains the Pool is about 400 metres long with a maximum height of about 6 metres. It is constructed of a silty sand and has an upstream puddle clay seal. The lake has a capacity of about 97,000 m^3 and a surface water area of approximately 66,000 m^2. As originally constructed, the Great Pool Dam was provided with an overflow spillway of about 1 m^3/s capacity.

9. 450 m. upstream of Great Pool is Rock Pool. Constructed at the same time as Great Pool and of a similar construction, the 6.5 metre high dam spans some 50 m. across a valley. Other than being a very pleasant small ornamental lake (of about 22,000 m^3 capacity), the only amenity was trout fishing. Upstream, but owned by the local District Council and within an area known and now developed as Baggeridge Country Park, is Island Pool, containing about 11,000 m^3, and Spring Pool which is of smaller capacity. Both of these pools are formed by similarly constructed dams and were probably all constructed during the same period.

10. Figure No.1 shows the Great Pool and surrounding land together with the sequence of pools upstream.

11. During 1970, a Panel Engineer was appointed to undertake a statutory inspection (under the Reservoirs (Safety Provisions) Act, 1930) of the Great Pool, together with the Rock Pool.

12. Following the Engineer's inspection, works at a total cost of £50,000 were undertaken upon both pools during 1974/75 as follows:-

(i) Great Pool

* The dam crest was raised to give 0.6 metres of free-board.
* The existing overflow was abandoned and sealed and a 17.5 m. length box spillway weir provided to convey a flow of 3.5 m^3/s (the calculated run-off from the catchment produced by 1 in 100 year storm intensity).
* The lake drain culvert passing through the dam was grouted up.
* Certain trees, deemed to be dangerous, were felled.

(ii) Rock Pool

* The existing overflow was abandoned and a new 17.5 m. box spillway weir constructed to convey 3.5 m^3/s. The weir was set at a level to ensure that the reservoir did not qualify as a large Reservoir within the meaning of the Reservoirs Act.
* Certain trees were felled.

13. Following completion of the works, the Panel Engineer certified that the Great Pool complied with the requirements of the Reservoirs (Safety

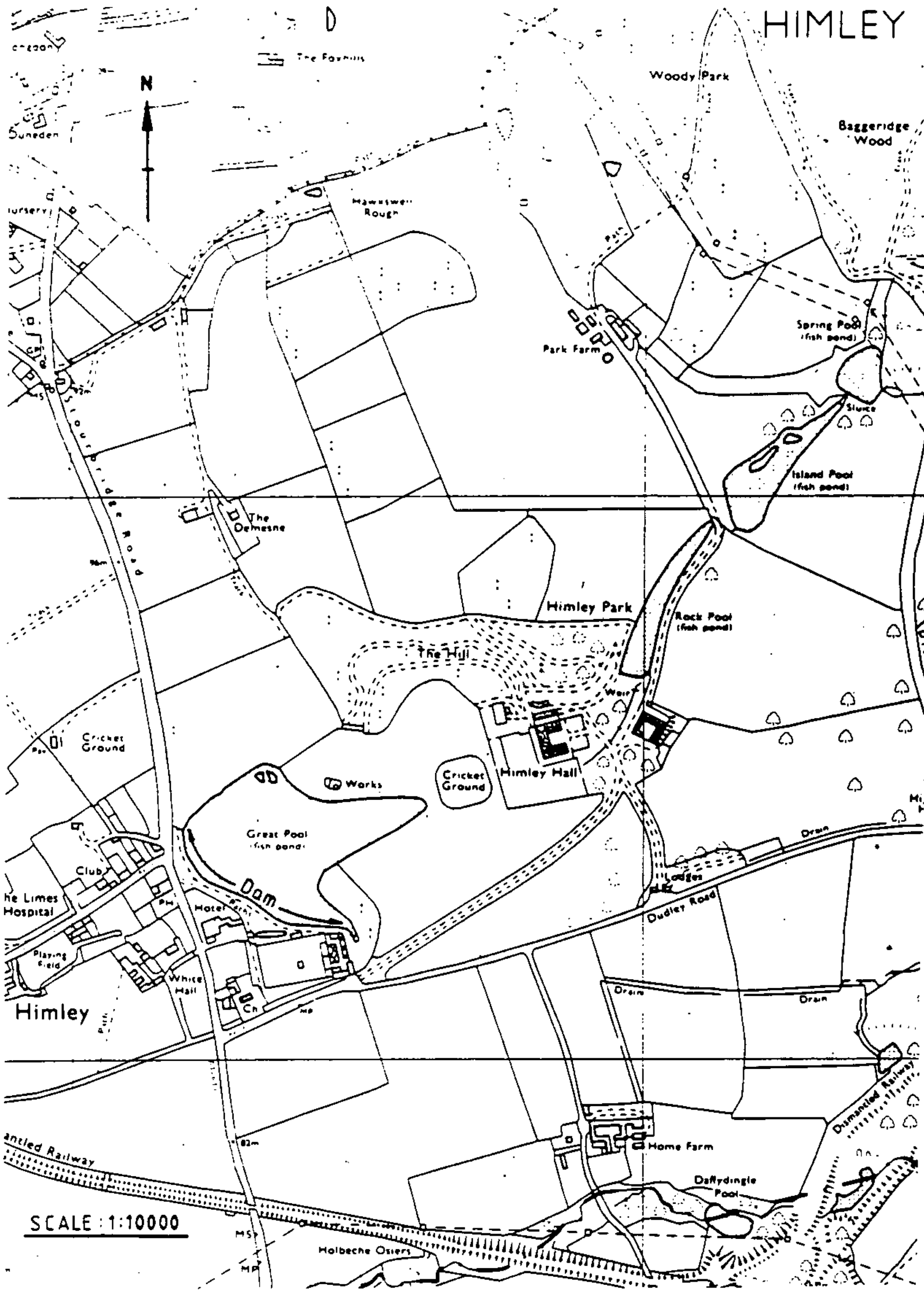

Figure 1 : Great Pool and surrounding land.

Provisions) Act 1930, but stipulated that a further inspection should be carried out within five years i.e. 1980.

THE 1980/82 INSPECTIONS

14. When, in 1980, the Panel Engineer who, in 1975, certified the works to the Great Pool, declined an invitation to undertake a further statutory inspection of the Great Pool, a new Panel Engineer was appointed. In view of the previous works, the inspection was expected to be a straightforward matter, with a certificate being issued following minor maintenance works which were known to be required.

15. The Engineer undertook a preliminary inspection and produced a Draft Report. He took into account the new recommendations contained within the then newly published "Floods and Reservoir Safety: An Engineering Guide" (1978). The presence of Himley Village which includes a Hotel and Public House only a short distance downstream of the dam, obviously placed the reservoir in Category 'A'. In carrying out his inspection, the Engineer also included the Rock Pool together with the two further pools upstream having an aggregate storage capacity of about 40,000 m^3/s, since he considered the domino effect of a possible failure of one of these reservoirs could have a detrimental effect upon the Great Pool itself.

16. During his preliminary inspection a small damp patch was observed on the downstream face of Rock Pool dam, together with a wet area on the downstream bank of Spring Pool dam. As the Great Pool Dam was thought to be of generally similar construction to those upstream, it was considered that this occurrence was indicative of possible weakness at the Great Pool Dam also, and sufficiently serious to warrant investigation, even though its outward appearance gave no cause for concern. A number of shell and auger boreholes together with a trial pit and some laboratory testing of samples was therefore undertaken at both the Great Pool and Rock Pool embankments.

17. Using data from the investigations, stability analysis of the Great Pool embankment was undertaken by a specialist firm of consultants. Their report indicated that the lowest factor of safety for the embankment was 1.37, which is less than the commonly accepted minimum for normal operating conditions of 1.5.

18. The investigations at the Rock Pool Dam indicated a water table almost at the surface of the embankment, the whole structure appearing to be composed of running sand bound together with tree roots and stabilised with surface grasses.

19. Whilst it had not been demonstrated conclusively that the damp patch was caused by seepage through the puddle clay seal, the Engineer recommended that removal of the embankment should be considered most urgently in the interest of safety. Consequently, the decision was made to drain Rock Pool immediately and it was emptied in stages over a period of several weeks having first removed the stocked trout.

20. Following his final inspection in 1982, the Engineer produced a report of his inspections which concluded that:-

(a) "The existing embankment to Great Pool is not a safe structure within the meaning of the Act", and

(b) "Overflow arrangements are not capable of coping with the maximum flood to be expected."

IMPLICATIONS OF THE 1982 INSPECTION REPORT

General.

21. During the Engineer's initial inspections and subsequent meetings, the implications of the impending report became clear. The initial reaction of officers and elected Council members was total disbelief at the tenfold increase in spillway flows that the reservoir should now accommodate. The unacceptable conclusion was that a dam which had been certified as safe in 1975 could now be certified as unsafe following a change in guidelines. Of immediate concern was that if extensive works were again undertaken, could they again be deemed inadequate following implementation of the 1975 Reservoirs Act or following yet a further review of design criteria?

Financial.

22. An initial appraisal into providing emergency overflow facilities together with improvement to the stability of the dam embankment, indicated that the cost would be in the order of £1.0 m. Providing an overflow over the dam crest would have meant the removal of all trees within the area of the works and would have provided a strictly functional engineering solution not in keeping with the character of the area.

23. A simpler option considered, at an estimated cost of £30,000, was the provision of an ancillary emergency overflow weir at the northern end of the lake. This would discharge into a grass channel which would spill onto the adjoining Trunk Road with overland flow along the carriageway surface to the natural low point before discharging into the water course.

24. This concept was not proceeded with following legal advice that objections would arise from the Department of Transport and County Council, as agents for the Trunk Road, to water being introduced onto the carriageway surface at an artificial point above the natural low point.

25. The initial response was, therefore, to consider either abandoning the reservoir, by forming a permanent breach in the dam, or reducing its capacity to less than 25,000 m^3. The estimated cost of these works was between £50,000 and £180,000 depending upon the degree of final landscaping undertaken.

26. In addition to the considerations required to undertake capital expenditure upon the pools, another concern was the condition of Himley Hall itself. Whilst having been used as an educational centre by Wolverhampton Borough Council, the condition of the Hall was deteriorating rapidly and it was assessed that a sum in the order of £1m. was needed to renovate the structure. It was also indicated by Wolverhampton that they wished to cease use of the Hall and terminate their joint ownership of the Hall and grounds. Consequently, reviews were undertaken to assess possible future uses for the Hall and grounds and on what future policies

the single or both owners should operate.

27. Enquiries were made concerning possible grants towards the cost of retaining the lake including the Countryside Commission and Historic Buildings Council. It was also hoped that minor contributions may have been provided by the Department of Transport, the County Council or the Ministry of Agriculture, Fisheries and Food, in consideration of the degree of 'balancing' that the lake provided on the downstream catchment. However, all indications were that no contributions would be forthcoming.

Planning Constraints.

28. Himley Hall and grounds, including the dam embankments, are Grade 2 Listed Buildings. Numerous discussions were held with the Local Planning Authority concerning the overall future use of the Hall and its grounds including the water features.

29. Following the urgent draining down of Rock Pool in 1982, a Planning and Listed Building Application was made to demolish, or part demolish, the dam together with the subsequent abandonment of the pool. The application was made because the dam was no longer considered safe against structural failure. The option to strengthen the dam was likely to cost in the order of £350,000 against the cost of £40,000 to £50,000 of breaching the dam together with subsequent landscaping of the pool area.

30. Following extensive deliberations by the Planning Authority, the application was refused. They considered that the removal of the pool would result in a loss of a significant feature in the landscape attributed to "Capability" Brown together with the loss of an amenity within Himley Park and Baggeridge Country Park, to the north, which are major recreational attractions in the area.

31. The Planning Authority indicated that they wished to consider a joint application for both the alterations to the Great Pool and to the Rock Pool urging the retention of the stretches of water together with the amenities of fishing, sailing and general enjoyment which they provided.

32. However, the need for safety was accepted for the Rock Pool dam and it was indicated that a much reduced water feature, together with associated landscaping works, may be acceptable.

33. Shortly after the planning refusal, Himley Country Park and its surrounding area was declared a conservation area to reinforce the powers already available under the Planning and Listed Building legislation.

34. It was eventually realised that the cheapest course of action involving the abandonment of Great Pool (together with Rock Pool) was not a viable solution because:-

* The water feature(s) contributed significantly to the value of the Country Park but, without the legislative safety certification, the value of the grounds would have been minimal, being unsaleable and remaining a liability.

* The Authorities' Insurance Company was showing concern about the potential liability that existed for damage to property and loss of

life. They concluded that these seemed to outweigh all other factors and suggested that immediate action was necessary.

* The impending 1975 Reservoirs Act would make non-compliance with the Act a criminal offence, and give the Enforcement Authority powers to undertake works and recharge the owners.

* To proceed without Planning/Listed Building Approvals, enforcement notices would have been served to reinstate both the dams and the co-owners as Planning Authorities had to be seen to be complying with the standards they themselves would enforce.

35. It was therefore decided early in 1984 to retain a small water feature at Rock Pool and to appoint a new Panel Engineer, Mr. J.B. Bowcock, of Sir Alexander Gibb & Partners, to advise on appropriate remedial works to bring the Great Pool Dam to an acceptable standard of safety.

GREAT POOL EMBANKMENT - INVESTIGATION

Investigations

36. The previous investigations (1982) comprised four boreholes in the embankment, with borehole water levels recorded during and immediately after boring. Laboratory tests were carried out on six samples from the boreholes, comprising grading and moisture content tests and consolidated drained triaxial tests.

37. To supplement the information obtained from this work, further investigation was undertaken during the winter of 1984/85 comprising:-

* 12 No. static cone penetrometer soundings in the crest and downstream shoulder,

* 7 No. soft ground boreholes, with in-situ permeability testing and installation of standpipes piezometers (2 No. per borehole in the deeper holes),

* trial pits and hand auger holes in the downstream foundations,

* classification and index testing of soil samples from boreholes.

Condition of the embankment.

38. The results of the investigations confirmed that the embankment is constructed of a generally non-plastic loose to medium dense silty sand of fairly high permeability, with occasional bands of denser material, gravel and peat. Foundations comprise weak and poorly cemented red-brown weathered sandstone, with thin bands of more clayey material. The maximum height of fill over foundation rock was measured as 7.5m.

39. The investigations indicated that the embankment was well drained and that the clay lining appeared to be providing an effective seal to the upstream face and the bottom of the lake. No signs of seepage from the downstream face of the embankment or from the foundations, or other signs of distress, could be seen.

Internal Water Levels.

40. Water level measurements made during drilling and in standpipe piezometers suggest that the fill is horizontally stratified, and that there is a very significant downward seepage flow into the comparatively high permeability foundation material, resulting in low excess pore pressures in the embankment. Piezometers near the downstream toe of the embankment are dry.

41. Measured permeabilities were consistent with the materials encountered, with average values as follows:-

* embankment fill material : 10^{-5}m/s

* surface layer of foundation rock : 5×10^{-4}m/s

Shear strength parameters

42. Analysis of results of both series of laboratory tests indicated that appropriate shear strength parameters for stability analysis are as follows (in terms of effective stress):-

* cohesion (c') = 0.
* angle of shearing resistance (ϕ') = 35 degrees

Similar values were assumed for the upper layers of the weathered sandstone foundations.

EMBANKMENT - STABILITY

43. The stability of the embankment was investigated by means of circular slip analysis, using the strength parameters described above.

Original profile.

44. Assuming the embankment to be saturated below a static phreatic line joining the lake water level and the downstream toe, the minimum factor of safety of the highest sector of the embankment was 1.37, which is less than the commonly accepted minimum for normal operating conditions of 1.5.

45. Although the assumption with regard to the phreatic surface is pessimistic when compared with actual measurements in the piezometers, and ignores the effect of the downward flow which appears, it was considered nevertheless that it was not an altogether unreasonable assumption and could possibly occur during winter when transpiration from trees was at a minimum and saturation by rain and snow at a maximum.

46. It was therefore concluded that some work to improve the stability of the embankment was justified.

Modified profile.

47. Works to improve the embankment stability comprised a small downstream drainage/stability berm incorporationg a gravel drainage layer, together with a lined toe drain.

48. Making similar pessimistic assumptions for the internal water

pressures, this resulted in an increase in the minimum factor of safety to 1.53 under conditions of normal lake level, and 1.31 under extreme flood conditions with lake at embankment crest level.

49. Figure 2 shows the embankment profile at its highest section, the assumed and measured internal water levels, and the critical failure surfaces for the normal lake level.

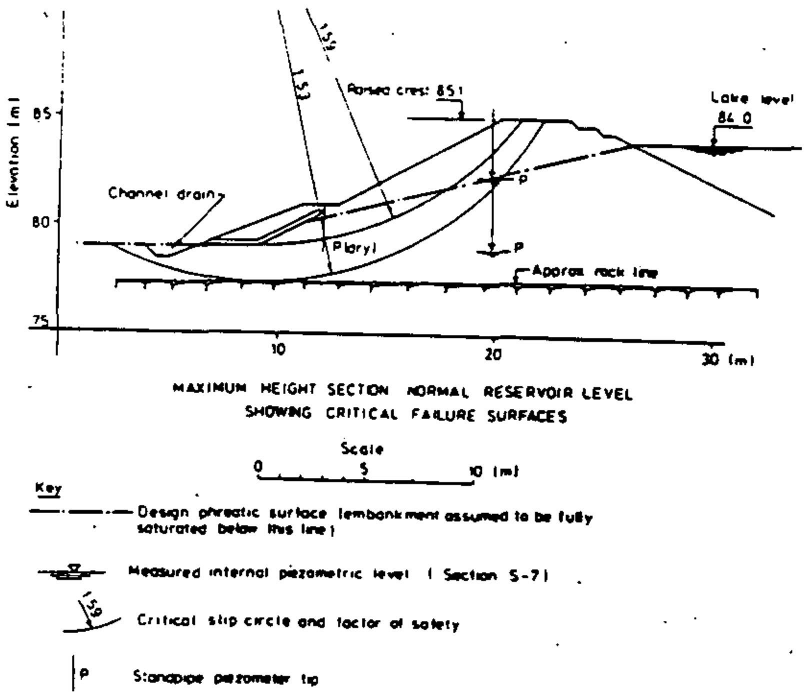

Figure 2 : Embankment Profile.

50. Factors of safety based on pore pressures derived from a flow net consistent with measured internal water levels are 1.75 and 1.32 at normal and extreme flood lake levels respectively.

AUXILIARY SPILLWAY

Flood Estimate

51. An estimate of the probable maximum flood, based on the Flood Studies Report gave a peak inflow of about 35 m^3/s with a time to peak of 4 hours, and a volume of some 400,000 m^3.

52. The existing spillway box weir discharges into a surface water sewer pipe of about 1,000 mm. diameter running beneath the verge at the A449 Trunk Road and discharging into the original stream channel. This limited the spillway outflow to about 3 m^3/s, with very little scope for increasing the capacity. However, even though the dam had originally been

constructed with a spillway capacity of only about 1 m^3/s there appeared to be no evidence that the overflow capacity had been seriously exceeded during the life of the dam. It was decided, therefore, to retain the existing system unchanged and provide an independant auxiliary spillway to deal with flows in excess of the 3 m^3/s, up to that arising from the P.M.F. inflow to the lake.

53. The design of the auxiliary spillway is complicated by the fact that the boundary wall with the A449 trunk road is close to the downstream toe of this embankment, giving very little space for either the discharge channel or an energy dissipating device. Figure No. 3 shows the arrangement finally adopted which comprises a 60 m. long reinforced grass-lined secondary side channel with overflow weir set at 0.3 m. above the sill of the existing spillway. The flow is conveyed along the downstream toe of the embankment in buried twin box culverts to a narrow hydraulic jump stilling basin situated below the highest point of the embankment. Ports are incorporated in the stilling basin walls to discharge low flows on to the A449. At high flows, water would overflow a low section in the stilling basin walls adjacent to the road. The reservoir provides a small amount of attenaution to the incoming flood, and the total spillway capacity provided by the new and existing works is about 32 m^3/s.

54. The arrangement by which spillway discharges having return periods in excess of around 100 years flow across a main road may appear somewhat unconventional. However, the argument that this would have happened anyway if the dam did not exist was eventually accepted by Authorities concerned.

55. The alternative of constructing a large culvert, sufficient to discharge the maximum flood flow without flooding the road was considered briefly. However, avoidance of flooding, desirable though it may be, would have also necessitated enlarging and improving the existing channel downstream of the road crossing to prevent backing up, and possibly also enlarging a small culvert through a railway embankment a short distance downstream, all costly works having no bearing on the safety of the dam.

ADDITIONAL WORKS

57. The rise in lake level necessary to discharge the P.M.F. inflow is about 1 m. This necessitated raising the embankment crest by about 0.3 m. and constructing a wave wall to give a freeboard for waves of 0.6 m. under extreme flood conditions (the minimum recommended under the ICE Floods and Reservoir Safety guidelines). A short length of flood bund was also needed at the south end of the embankment to prevent flooding around the flank.

58. Considerable attention was given to the appearance of the raised crest and wave wall. The wave wall is of reinforced concrete faced with brickwork of a striking pink, and the upstream section of the crest is generally "Enkamat" reinforced grass, incorporating a series of fishing bays. The crest road is of crushed pink limestone. The works have been complemented by extensive planting by the Council's Leisure Services Department and, overall, the effect is extremely pleasing.

59. The other item in the programme of remedial works was the construction

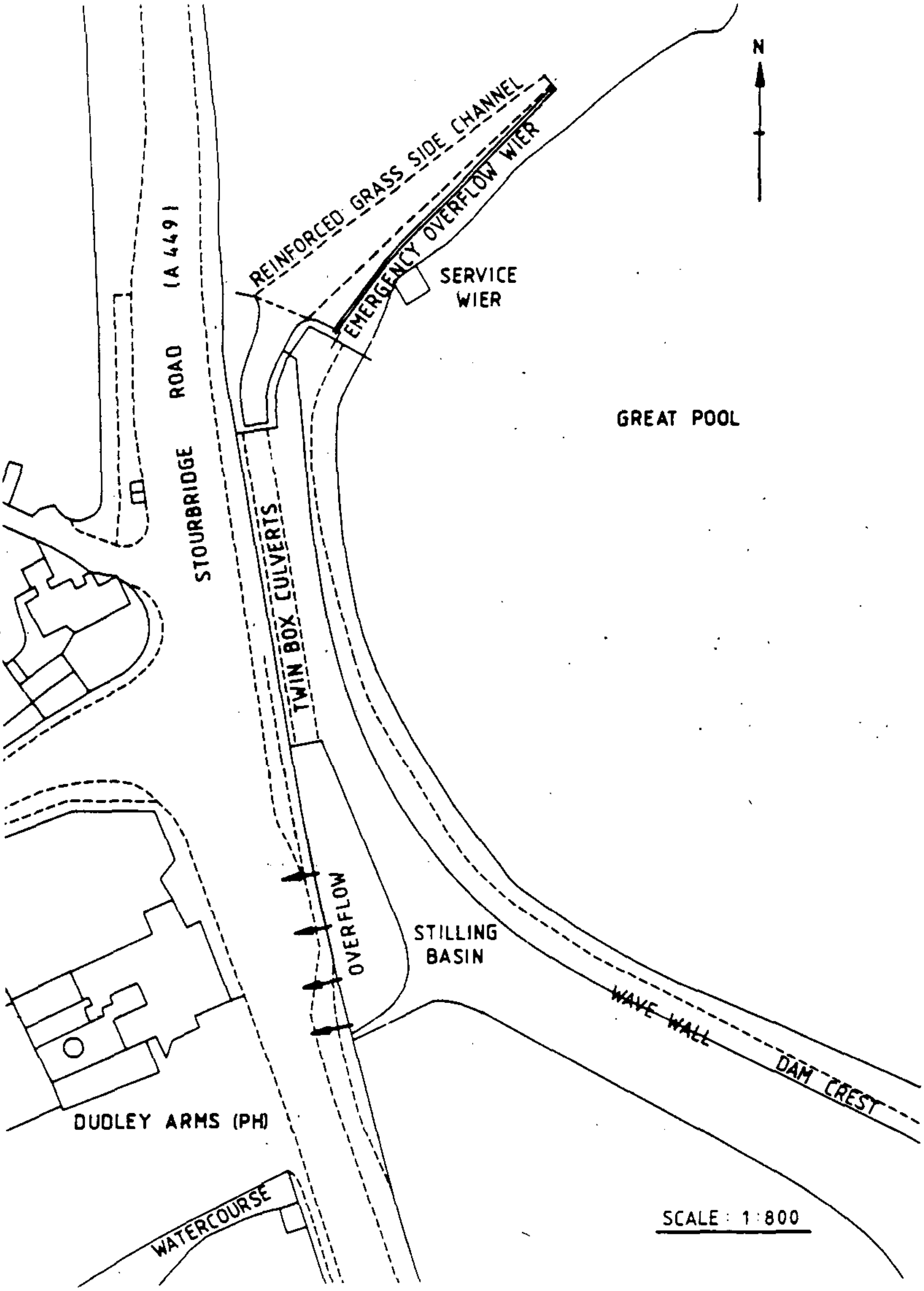

Figure 3: Emergency overflow arrangement.

of a 450 mm. diameter drain to enable the lake level to be lowered by about one metre in case of emergency. An added complication in this respect was the fact that the lake had been stocked with grass carp (Ctenopharyngoden idella Val) to control the growth of weed. These fish are classified as 'exotic species' and are covered by the Wild Life and Countryside Act 1981. Special screening arrangements subject to approval of the Severn-Trent Water Authority were necessary to prevent their escaping into the downstream watercourse.

CONSTRUCTION

60. The construction contract was awarded to Droitwich Construction Company Limited in January 1987 and construction was completed in October 1987.

61. No significant problems were encountered during the course of the construction work which was completed on programme and without incurring unforeseen costs.

CONCLUSION

62. It is considered that following the extremely lengthy procedures involved prior to implementing the works, the adopted solution has been most satisfactory in that it has both retained and enhanced the character of the lake.

ACKNOWLEDGMENTS

63. The authors are grateful to Mr. J. Eastwood, O.B.E., C.Eng., F.I.C.E., Borough Engineer, Dudley Metropolitan Borough Council and Mr. J.B. Bowcock, M.A., FI.C.E., F.A.S.C.E., Panel Engineer, Sir Alexander Gibb & Partners, for permission to present this paper.

MODIFICATIONS TO RESERVOIR OVERFLOW SYSTEMS

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SYNOPSIS

In this paper reference is made to a number of reservoirs, which, having been inspected under the Reservoirs (Safety Provisions) Act 1930 or under the Reservoirs Act 1975, have been found to have inadequate overflow discharge capacities. The reservoirs vary from those having fairly large dams and owned by public authorities to very small ones owned by individuals. In all cases it is required to find the most cost effective safe solution to the provision of additional overflow capacity, whilst bearing in mind aesthetic considerations. Whereas the method to be used in some cases is self evident, others can have a number of possible answers. In many instances the use of a model may lead to savings in construction costs.

INTRODUCTION

Most of the dams referred to were constructed in the latter part of the nineteenth century or in the early part of the present century. Since then the design standards for the capacity of overflow discharge have changed, on more than one occasion.

Following the Reservoirs (Safety Provisions) Act 1930 an Interim Report of the Committee on Floods appointed by the Council of the Institution of Civil Engineers was produced in 1933 and in 1960 there was a reprint with additional data on floods recorded in the British Isles between 1932 and 1957.

The terms of reference of the Committee were :-

"To examine the present state of knowledge in regard to the magnitude of floods in relation to reservoir practice in Great Britain and to make recommendations on the best methods of dealing with them in that connection".

The report led to the realisation that many existing reservoirs had inadequate overflow systems, and in many cases reports from Panel Engineers were received, but no action was taken.

Following the 1976 Flood Studies Report, the I.C.E. Publication "Floods and Reservoir Safety" was produced, in which various categories of dam are tabulated with further description appearing in the text. Again many reservoirs were found to fall below the required standard, but following the implementation of the Reservoirs Act 1975, the reports by Panel Engineers were taken more seriously by owners who had previously, perhaps, not appreciated the risks associated with reservoirs having inadequate overflow discharge capacities.

A few dams of which the author has personal knowledge are briefly described below, some having already been altered, others still requiring attention. For the most part the author has not identified the individual dams to avoid any embarrassment which might be caused to owners.

RESERVOIRS HAVING INADEQUATE OVERFLOW CAPACITY

1. A dam in Yorkshire having a class A road across the crest, and a catchment of 22 km^2, has on the downstream side a couple of restaurants and housing. Its construction is uncertain, but appears to be rubble fill between masonry retaining walls. The type of water retaining membrane is unknown. The minimum freeboard is only 0.1 m and the overflow structure is at one end of the dam in the form of an arc of a circle.

Water passing over the weir is led into a culvert through the dam and thence to a natural watercourse. It was the author's opinion that this dam should be in category A minimum standard. Although there seemed little likelihood that the dam would fail even under P.M.F. conditions, there was, in the author's view, a risk to life in the event of a massive overflow over the middle of the crest, even without dam collapse. Due to the configuration of the crest, the flow would be concentrated in the middle.

The P.M.F. outflow was calculated at 121 m^3/s and the discharge capacity at about 28 m^3/s before overtopping would occur. The culvert was also restrictive.

It was clear that additional discharge capacity would be required, but there were various possible solutions, the most obvious being an additional culvert together with either a straight sideweir or the replacement of the weir by a new arc having a larger diameter than the original. It was also apparent that a safe but possibly unnecessarily large, overflow system could be designed using mathematical calculation, but, having regard to economics, it was felt that alternatives should be examined.

One alternative which might be proved to be more economical was that of lowering the invert of the existing culvert coupled with a new overflow weir. The hydraulics being somewhat complex, in respect of the facts that the weir would drown under severe flood and that the discharge conditions of the culvert could not be appreciated fully, it was recommended that a model should be constructed in order to try a number of variations.

A decision regarding the next step to be taken has not yet been reached.

2. Another dam in Yorkshire about 14 m high, the lower of a cascade of two, has a bypass channel capable of conveying 25 m^3/s. The overflow is a side weir discharging into a channel leading through a culvert just below the dam crest.

The area of catchment is some 48 km^2 producing a P.M.F. outflow of 331 m^3/s. The dam is situated above a small community and there was no doubt in the author's mind that there would be risk to life in the event of a dam breach. The freeboard is sufficient to permit a discharge capacity calculated at 90 m^3/s whilst leaving an additional margin of 0.6 m for waves.

Looking back over records it was seen that, due to the presence of the bypass, only on two occasions had the flow over the weir exceeded 150 mm, but it was beyond doubt that, in the event of a flow from the catchment exceeding 25 m^3/s, the excess would pass straight into the reservoir. To increase the length of the weir appeared at first sight to be the ideal solution, but unfortunately the culvert and the depth of the existing channel impose an additional restriction.

A survey of the submerged valley side upstream of the existing weir showed steep sides which would present foundation problems, and it was evident that to increase the carrying capacity of the culvert by increasing its depth or width would be difficult and therefore costly.

An alternative would be to construct a secondary weir along part of the dam crest with a wide, shallow waste watercourse conveying the water into the stream passing across the toe of the embankment.

Some investigation of ground conditions and a model test would be required to establish the feasibility of any solution, other than that of the secondary overflow weir. It was therefore necessary to convince the owner that by agreeing to some early expenditure, cash savings may be made. However there are still doubts in the owner's mind.

3. An earthwork embankment dam in Lancashire constructed during the latter part of the last century, having a height of 22 metres and a crest length of 110 metres is situated in a steep sided valley. Its original purpose was to ensure a water supply to an Industrial Estate.

It was constructed with an overflow 9 metres long at the northern end of the dam discharging into a masonry lined waste watercourse. A bypass channel also discharged into the same watercourse.

In the 1950's a second weir and waste water channel were constructed at the southern end, doubling the discharge capacity, but following the change in design standards, in order to provide for a "Probable Maximum Flood" as defined in the 1976 Flood Studies Report, the capacity was again found to be inadequate.

A cost exercise was carried out, comparing an extension of each of the existing side weirs with the provision of a bellmouth overflow having a short drop shaft and a pipe through the dam at each end. The existing weirs were to be retained and the discharge pipes were to connect into the existing waste water channels. It was found that, due to the proximity of the rock head to the face of the steep valley sides, the extension of existing weirs and deepening of channels would be more expensive than cutting through the dam to install new discharge pipes. The bellmouth solution was therefore selected, although it was appreciated that an experienced contractor and careful supervision would be required to ensure satisfactory restoration of the puddle core.

The method of construction was to excavate a slot thorugh the core for each new discharge pipe and to rebuild the puddle core around the pipes. Although puddle clay is nowadays little used in substantial quantities, it did not prove to be too difficult for the contractor to procure the necessary quality of material and to achieve the standard of compaction.

Provision was made in the contract for some grouting of the valley sides where there would be disturbance of the areas where the dam structure abutted undisturbed ground or fissured rock. It was also intended that any small leakage which might occur after completion could be sealed by grouting.

In the event, the grouting of the rock resulted in minimal acceptance of grout, but some very slight leakage did appear when the reservoir was allowed to refill. This was sealed successfully by the use of a small quantity of bentonite spread on the water surface.

No further trickle of water has been apparent since that time.

4. A small mill dam, the lowest in a cascade of reservoirs, this reservoir in Lancashire has a catchment of 2.2 km^2 additional to that of the upper reservoirs which, being owned by the Water Authority, are well documented. The existing overflow is a semi-circular structure close to the centre of the dam and leading into a steep waste water channel. It enters a stream flowing past the toe and conveying water from another catchment, almost at right angles.

The total outflow from a P.M.F. is likely to be of the order of 163 m^3/s. The existing overflow capacity of this category A reservoir is 17 m^3/s whilst still retaining 0.6 m freeboard for waves. It was felt that there could be several solutions, the most likely appearing to be a long sideweir constructed on the eastern side. After a detailed survey it became clear that the most economical solution would be to provide a new, secondary weir along the crest, to one side of the existing overflow, together with a curving waste water channel leading into the stream.

There is a further restriction downstream in the form of a culverted section of the stream. The first length has a cross sectional area of 12 m^2 but an extension has been added with a cross sectional area of 3.2 m^2. In the event of a P.M.F. this would cause a back-up at the dam toe of some 4 m depth.

The ownership of the culvert is not the same as that of the reservoir and a solution to this problem has not yet been found.

5. Another instance of inadequate overflow capacity is that of a reservoir in Nottinghamshire which has been subject to mining subsidence.

A recently mined coal seam heading longitudinally under the dam caused the whole structure to settle progressively but not quite uniformly. It was considered necessary to remove the existing main weir almost down to the bed of the waste water channel as a temporary safety measure but its discharge capacity was still well below that required for even the outflow from 0.3 of a P.M.F.

The dam is on a main river and has a catchment of some 97 km^2. The overflow has ultimately to be restored to a level close to original, to prevent flooding problems in other stretches of the river.

The recommended course of action is a composite solution, the embankment to be raised and the existing main weir length to remain unchanged. In addition the embankment would be armoured to resist the effects of rare overtopping and an existing secondary weir would be doubled in length.

The reservoir was considered to be in category C with a design flood of 35 m^3/s.

6. A small dam in Lancashire having a catchment of 3.85 km^2, is perched above a village. That it should be in category A was not in any doubt. A bypass channel had been constructed along one side and had been extended as a concrete channel supported by columns from the dam crest to a point slightly downstream of the toe. The overflow had been a straight weir spilling into the bypass channel but the flow from a P.M.F. was calculated to be far beyond the carrying capacity of the channel and would result in a flow into the reservoir with no exit except over the crest of the embankment.

At the time of inspection the overflow had been reduced in level a little but had been left without stone or concrete protection over the weir. Natural sub soil had been left exposed.

The only solution other than complete removal of the dam was to provide a weir along its full length and to armour its downstream face.

A recommendation to remove the dam was considered but, in view of its small capacity, it was felt that a dam breach would not materially increase the risk to life which would have been present in the event of a probable maximum flood, even if the structure had never been erected.

7. A dam also in Lancashire having similar characteristics and situation was the subject of considerable expense by the owner, adding material to the downstream side of the embankment and raising the level of the surrounding land. He did not, however, consult a qualified engineer before or during this construction work and is now faced with the problem of an inadequate overflow, spilling into an inadequate bypass.

It will be possible in this case to provide a secondary overflow and discharge facility where there is no problem of reversing the flow back into the reservoir.

8. A dam in Yorkshire had in the past been subjected to mining subsidence and had been restored to its original height by the addition of fill material and repair of the face protection. For many years the level had been kept down by keeping low level draw-off culverts open.

This had become such an accepted practice that a neighbour had built a chicken run in the overflow channel, which channel was totally inadequate to discharge the design flood. The catchment area is 10.3 km^2 and the reservoir in category A.

The owner required that a lower level of overflow sill should be provided and it was thought that this could be achieved by deepening and/or widening the existing channel. It was however, steeply sloping and the masonry ruinous in parts, and a desk study led to the conclusion that a less costly and aesthetically more acceptable solution would be to provide a new central overflow with a wide, fairly shallow, waste watercourse down the embankment face. At the same time a new draw down pipe and valves were installed and the work completed about 2 years ago. Figure 1 depicts a plan of this structure.

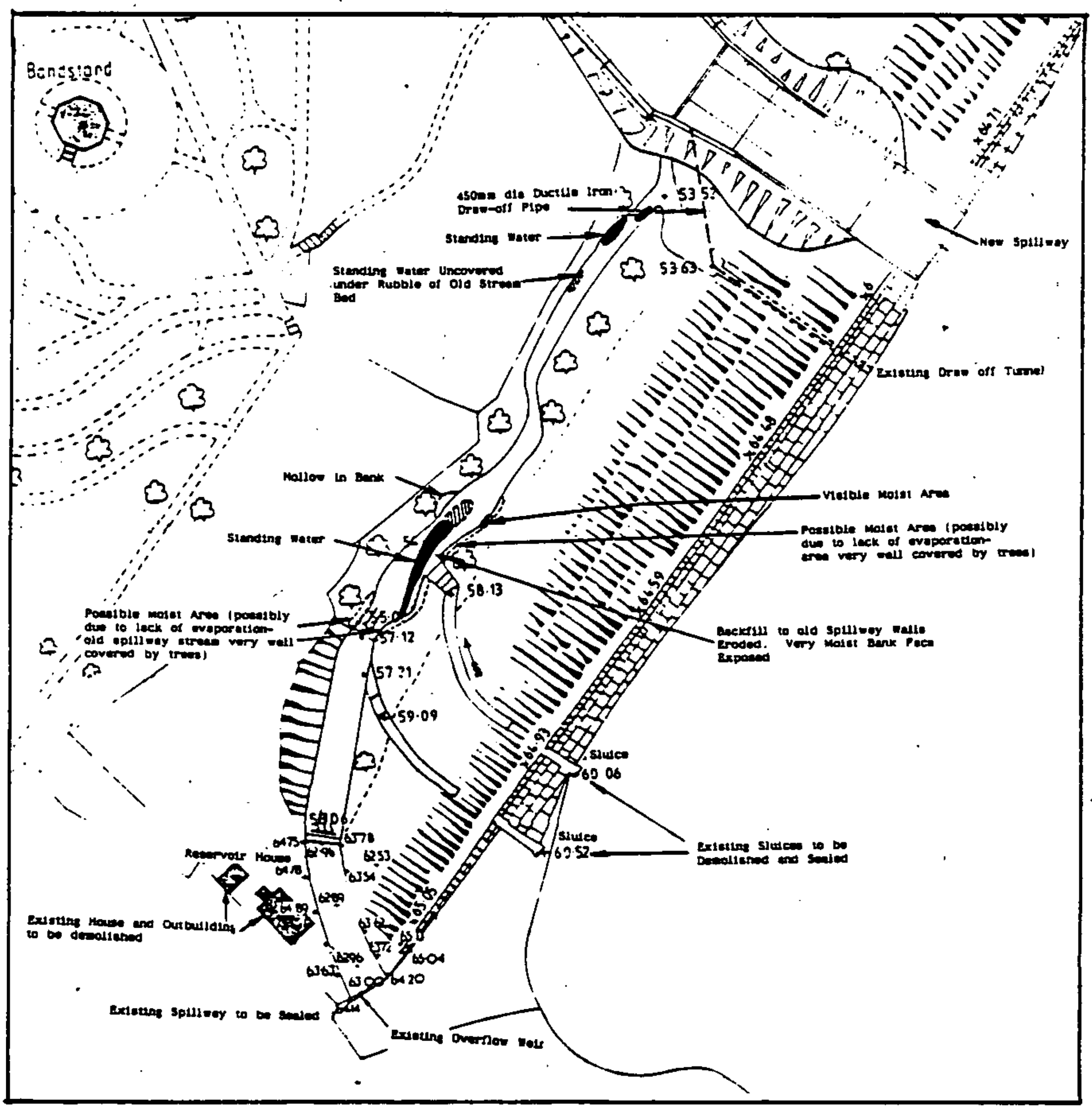

Fig. 1. Typical New Overflow Cill and Wastewater Channel

9. Another old dam had two existing overflow structures, a side weir and small central weir. Although it was placed in category A by the inspecting engineer, the owner felt that a second opinion was required and the category was confirmed.

Downstream of the dam are some habitations which would be affected by the wave from a breach, but even more importantly there are mineshafts which would be flooded, thereby putting the lives of many underground workers at risk.

The method chosen to provide sufficient overflow capacity was the construction of a long concrete sill along the dam crest complete with concrete waste water channels down the dam face.

The finished structure, designed by staff of the reservoir owner, although originally opposed by many, has since proved to be a local feature which is regarded with pride by this responsible Local Authority.

10. Two dams in cascade in West Yorkshire have a catchment of 0.87 km^2 and are positioned above a large community.

The only possible category is A, leading, in the case of the upper reservoir to a design flood of 19.5 m^3/s. A freeboard of only 0.37 m gives a discharge capacity, without any allowance for waves, of 3.0 m^3/s.

There is a minimal bypass and no possibility of a sideweir due to the steepness of the valley sides. Some leakage which occurred on the downstream face and an apparent movement of a retaining wall supporting the toe of the upper embankment led to a recommendation that it should be investigated and that the water level should be maintained at a low level until a decision concerning the dam'sfuture could be made.

Ultimately it became clear that an additional weight block would be required on each side of the dam and that it might also be necessary to effect a seal between the core of the dam and the original ground upon which it was founded. It seemed possible that the dam had been founded in part on top of the gravel of the original stream bed.

It was apparent that additional overflow capacity would be required and the only satisfactory arrangement would be a new weir on the crest of the dam.

Various forms of channel were considered due to the importance of producing an overflow which was as unobtrusive as possible, since these reservoirs are in an area of notable beauty.

Because of the dam height, 14 m, it was considered that, although some form of block through which grass would grow would be aesthetically acceptable, such material could not be regarded as being sufficiently tested in similar conditions of velocity of water to be used without some doubt about its ability to withstand such conditions indefinitely.

A composite structure with concrete on the lower part of the embankment was felt might be a safe comprise. However a serious problem on this site is access, and it was ultimately decided that a reinforced concrete structure would lead to fewer difficulties with logistics than concrete blocks.

After the results of the site investigation had been obtained, it was felt that a structural stability investigation should also be carried out at the lower dam, the inadequacy of its overflow having already been established.

In these cases models were not required to ascertain preferred solutions, and work on construction is expected to be started by the time of publication of this paper.

11. A reservoir in Lancashire in category A has a catchment area of 7.5 km^2, a weir length of 21.6 m and a freeboard of 1.5 m. The design flood is 98 m^3/s and the capacity of overflow leaving additional freeboard of 0.6 m is 33 m^3/s. It was clear that the additional overflow requirement could be provided in various ways.

A model was constructed and the initial tests showed that with the existing spillway the dam first overtopped at a flow rate of 45 m^3/s, and the head over the dam at 100 m^3/s was 0.44 m.

Further tests were carried out which eventually led to the decision to construct a secondary spillway near to the centre of the dam. The model was invaluable in determing the best arrangement and size for the dragons teeth set in the spillway channel.

12. A small reservoir near Nottingham with a catchment of 3.3 km^2 and a freeboard of 0.48 m is in category D, the design flood being 7.8 m^3/s and the combined overflow capacity being 2.3 m^3/s. There are two overflow systems, one a short weir with a pipe leading to a delapidated tailbay structure, the other a culvert through the dam leading to a D shaped shaft through which the water rises and flows over the straight side.

The reservoir being privately owned, it was with difficulty that a suitable method of providing sufficient capacity was decided upon. The supervising engineer, who was also engaged to design a suitable overflow, ultimately suggested a tertiary overflow over a section of the crest road together with the armouring of the face downstream. In view of the very low height embankment this solution was considered to be acceptable.

13. A small reservoir in Hampshire with a catchment of 1.25 km^2 a weir length of 2.4 m and a freeboard of 0.39 m, has a design flood of 5.1 m^3/s. The necessary overflow discharge capacity was provided by means of a secondary weir over a very low part of the embankment. The crest was raised along the remaining length of embankment to produce sufficient freeboard.

14. A dam near Sheffield has a sideweir leading into a channel close to the mitre of the dam. In addition there is a circular overflow with a short drop shaft. The combined discharge capacity is inadequate to deal with the design flood.

There were also two unintentional escape routes for overflowing water, one at each end of the dam where the very substantially constructed wave wall had been stopped short of being tied in with the existing ground.

In this case the simple solution was to seal each end by extending the wall and armouring the downstream face of the embankment to permit rare overtopping.

15. A relatively new reservoir in Lancashire is an illustration of coping with a short dam with a large catchment, this reservoir was to be situated in a large catchment below two existing reservoirs. If it had been constructed at the turn of the century it would probably have had an inadequate overflow unless the whole length of the dam had been sufficiently armoured to withstand overflow along its whole length.

The solution in more recent times was to provide a siphon overflow with priming levels of the siphons staggered to avoid excessive overflow.

For a relatively small reservoir in a large catchment this is a method worth serious consideration. However it is essential that a model should be constructed to avoid the problem of dangerous vibration of the structure both on priming and de-priming.

CONCLUSION

It is hoped that the examples given will go some way towards indicating the many different overflow problems which may be encountered by an inspecting engineer, all interesting and most requiring careful thought to establish the most suitable solution from the aspects of safety, cost and aesthetics.

In the author's view it is clear that the provision of a model in instances, where the expense involved in constructing an extension is likely to be high, is a far more satisfactory method of achieving the required form for producing additional capacity for the least cost, than relying upon calculation. It is therefore well worth the relatively modest outlay involved, although this philosophy is not always accepted by owners. In fact it is sometimes difficult to persuade an owner who has never experienced a severe flood that such disasters can happen.

IMPROVEMENTS FOR OVERFLOW WORKS AT SOME BRITISH DAMS

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SYNOPSIS

At several embankment dams, the overflow capacity of the existing spillweirs and spillway channels has been found to be inadequate to cater for P.M.F. outflows.

The range of options available range from new or enlarged works to accommodate P.M.F.; a channel to partially contain the flow; increased flood storage by heightening the dam and wave wall; use of flap gates; use of siphons; and fuse plugs.

Some of the different options are discussed with examples of solutions adopted.

INTRODUCTION

1. "Floods and reservoir safety: an engineering guide" published by the Institution of Civil Engineers in 1978 sets out recommended standards for reservoirs according to the category of reservoirs listed in Table 1 of the guide. Although it is stated that these standards are not mandatory it is almost impossible to adopt other than the recommended standard for a Category A reservoir.(1) This calls for the reservoir to cater for a "Probable Maximum Flood" or "PMF" with no overtopping whilst a margin is left between maximum flood level and the crest of the dam to provide for a wave allowance. A minimum standard of 0.5 PMF or 10,000 year flood can be used if rare overtopping is tolerable. In practice, this is hardly used for a major dam. The PMF is considered so rare an event that a great deal of thought and effort has been expended to achieve solutions which meet the standards at minimum cost but which would never be considered for a new reservoir.

EXISTING INADEQUACIES

2. The need for additional overflow capacity may come about from several different situations or combinations of restrictions which can be summarised as follows:-

(i) overflow capacity adequate but wave allowance insufficient
(ii) overflow capacity inadequate but each section of equal capacity for a lesser flood.
(iii) overflow capacity inadequate due to one section being under sized, e.g. throttling in a bellmouth overflow or in a narrow section of spillway channel downstream of the weir.

3. The need for available capacities under items (ii) and (iii) above

may or may not be achieved with the required wave allowance.

POSSIBLE IMPROVEMENT WORKS

4. The direct engineering solutions to the deficiences listed above are relatively straight forward but where these are to be constructed in National Parks or similarly scenic areas the embellishments necessary to meeting planning requirements greatly exceed the basic cost of the works. For instance a simple wave wall to remedy item (i) above becomes prohibitive if it has to be carried out in natural stone. Undertakers are reluctant to meet such costs when they see no obvious benefit and this had led to the investigation of alternative solutions.

5. With adequate overflow capacity the required wave allowance can be achieved by lowering the sill level or by raising the dam. However it is largely the provision of additional spillway capacity that has received special attention and this can provide a solution for all the inadequacies listed. An additional overflow to give a total capacity for a P.M.F. may be quite costly, especially if stone facing is required and forms of emergency overflow which can be covered or concealed by grass, such as syphons or reinforced grass, have offered satisfactory results in certain areas.

6. Where only a section of the overflow system may be the restriction to the full capacity it may be that work at this point alone may give the required results. However breaking out may be costly and undesirable if the reservoir is to be kept in operation whilst the work is carried out. Two alternatives to avoid this have been considered: raising of the dam crest or wave wall to provide additional flood storage or raising of the dam and overflow weir to provide extra depth of flow at the restriction to enable the PMF to be safely passed.

7. Of course just as inadequacies are found in combinations so solutions may also be adopted in combinations. There are no clear cut schemes which offer general solutions and each case must be considered on its own merits.

8. Auxiliary spillways involving flap gates or fuse plugs giving increased discharges for a particular reservoir flood level have been considered for certain reservoirs, but are not described further in this paper.

EXAMPLES

9. The following examples with which the authors have been associated received consideration of many of the options discussed above.

<u>Dowdeswell Dam, Gloucestershire</u>

10. The reservoir is situated on the River Chelt, 5 km south east of Cheltenham, and the dam was constructed in 1886 with a side spillweir of 18.3 m long leading to a channel around the right end of the dam. Assessed flood flows for a P.M.F condition gave an inflow of 48 cumecs, and a 1 in 25 scale model test showed the existing channel to have a capacity of only 25 cumecs. The owners, Severn-Trent Water Authority decided to proceed with the construction of a new spillway channel to

contain an outflow of 44 cumecs within its walls, as overspilling of the channel could damage the toe of the dam, and nearby houses.

11. In 1985 a new reinforced concrete channel 6 m wide and up to 6m deep and including a stilling basin was constructed, following hydraulic model testing carried out at the University of Birmingham.[2]

Ladybower Reservoir, Derbyshire

12. This reservoir was completed in 1945. It lies in a typical Pennine Valley to which it was believed the term 'upland' could be properly applied. The dam is an earth embankment with puddle clay core and two bellmouth overflows were provided one near each end of the dam which were the subject of a hydraulic model test. The design flood, the catastrophic flood, which was twice NMF could be discharged with a head of 1.83m (6 ft) over the bellmouth sills.

13. In 1974 the reservoir was taken over by the Severn-Trent Water Authority on its formation. When the Flood Studies Report was published followed by the Engineering Guide, their hydrologists carried out flood assessments on their reservoirs and it was discovered that the overflows at Ladybower were not able to pass the PMF of about 1100 cumecs. The flood assessment and description of the works form the subject of a paper to the 1988 ICOLD Congress by Mackey.[3] A solution, strongly favoured on engineering grounds was an additional spillway but this proved too costly. Other options considered were syphons, raising the dam, and lowering the bellmouth sills. The problem was solved by making provision to store excess flood water above crest level by the construction of a high 'wave' wall across the dam and armouring the east mitre which may to accept water escaping along the main road past the east end of the dam, whilst maintaining the existing tkop water level.

14. The wave wall is some 2.5m high and designed as a reinforced concrete cantilever wall faced with masonry similar to adjoining walls on both faces. The base slab is linked to the clay core with interlocking sheet piling.

Taf Fawr Reservoir, Wales

15. These three reservoirs (Beacons, Cantref and Llwyn-On) are owned by the Welsh Water Authority forming part of the water supply to Cardiff and are at the head of the Taf Fawr Valley in what might be regarded as typically upland country. The impounding reservoirs are formed by earth embankments with a puddle clay core. The overflows for each reservoir comprise a side weir and spillway channel passing round the end of the dam.

16. An inspection under the 1930 Act found that the overflows for each reservoir were not adequate. This was subsequently confirmed after the FSR and guide were published. In each case the problem was throttling of the discharge in the spillway channel as it passed the end of the dam. The means of improving overflow capacities has been the subject of careful study and debate. The remedial works on the upstream reservoirs affect the required capacity of those downstream and final decisions have only recently been made.

Beacons Reservoir

17. This reservoir lies at the upper end of the cascade where the valley is relatively wide with moderate sloping sides. The A470 trunk road passes close to the back of the overflow at the east end of the dam. If the weir could be given a free discharge the PMF could be passed without overtopping the dam but with inadequate wave allowance. The crest was unusually wide and raising both dam and sill would have been possible were it not for the A470. An additional overflow at the west end of the dam was considered but rejected on cost brought about by the excessive length or channel required to return the water to the stream channel.

18. Remedial works have been carried out which comprise a lowering of the invert of the channel at the critical section to enable the flood to be discharged with a water level in the reservoir which did not exceed the old crest level and a raising of the crest to provide the wave allowance. The length of channel which needed to be lowered was relatively short and narrow which enabled uplift to be resistricted by the wide walls.

19. This solution also had the benefit of attenuating the flood and thus reducing peak inflow to the lower reservoirs.

Cantref and Llwyn-On Reservoirs

20. The middle reservoir, Cantref, was constructed where the valley is narrow and the sides quite steep. The existing overflow is at the west end of the dam where sandstone outcrops. Because of the rock and steep slopes any works on this channel would be very costly. Alternatives considered and rejected were raising the dam but not the overflow and providing a new overflow at the east end where there is very little room with the A470 road close by.

21. The scheme proposed was a series of syphons to be located at the centre of the dam. By adopting syphons it was possible to arrange for the whole of the construction to cross the clay core above the normal top water level and to keep the discharge within the width of the valley bottom.

22. The lowest reservoir, Llwyn-On, was constructed where the valley had hillside slopes similar to Cantref but a much wider bottom. Here again syphons were originally proposed as the most economical solution.

23. The details of the syphons (Fig. No. 1) were developed as a result of hydraulic model testing which was carried out for the Welsh Water Authority by Wimpey Laboratories Ltd.(4) The syphons were air regulated with a near straight line rating curve until blackwater flow (Fig. No. 2). Good priming characteristics were achieved with a sealing pool at about the level of the upper berm when the negative pressures in the syphon, at worst condition, were acceptable. Vibration which was thought might be troublesome was found to be quite modest and appear to be a result of the long nearly horizontal inlet leg.

24. Despite the success of the model test the Water Authority were still looking for better value for money. A further study was carried out to cover the raising of the dam and top water level to provide an additional source of water which could produce revenue to pay for at least part of

the cost of the works. It was found that at both Cantref and Llwyn-on a raising of top water level by 4m and a slighly greater increase in the height of the dam would enable the exsitng overflows to pass the design floods. Some works to prevent uplift behind the weir would be required but this was little extra cost compared with an additional overflow. In both cases diversion of roads would be necessary and the excessive cost of these eventually ruled out both raisings.

25. During these deliberatons testing of water flowing down reinforced grass slopes had been carried out by CIRIA.[5] This demonstrated that for short periods earth dams with slopes protected in this way could be overtopped by nearly 1m provided that the works could be confined to a section in the embankment away from the mitres. A further study was carried out for Cantref and by allowing overtopping in the centre of the dam with the face protected in this way the number of syphons originally proposed can be reduced and related and a reduction in cost achieved. A small embankment on the downstream side of the crst to 0.6m above max. flood level will prevent overtopping, except where desired.

26. This scheme has been approved and on final routing of the flood showed a marked reduction in peak outflow. This is due to the increased flood storage above crest level which is brought into plan when overtopping takes place after the syphons have gone to blackwater flow.

27. In the course of these investigations it became clear that the cost of syphons was 10 times the cost of reinforced grass covering the same area. Thus since the capacity of syphons is about 10 times that of reinforced grass per m width of structure the cost of ancillary works becomes the deciding factor. Llwyn-On is so long that the works required to confine overtopping to the centre became excessive, even when further flood attenuation is taken into account. Syphons alone are proposed for Llwyn-On.

ACKNOWLEDGEMENTS

The authors wish to thank Severn-Trent Water Authority for permission to publish descriptions of Dowdeswell and Ladybower Improvement Works; and Welsh Water Authority for Taf Fawr Reservoirs.

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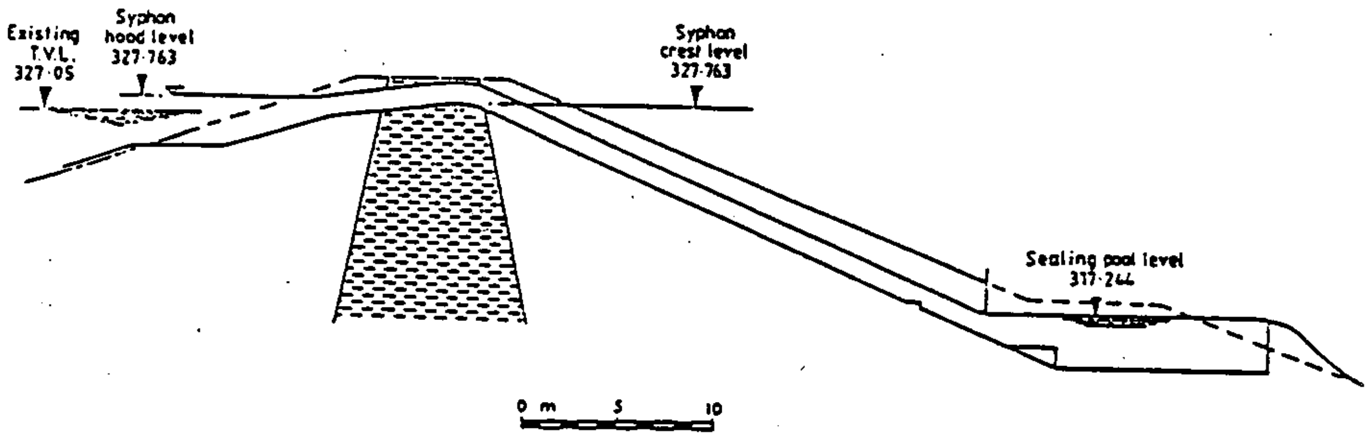

Figure No.1: Syphon for Cantref Dam

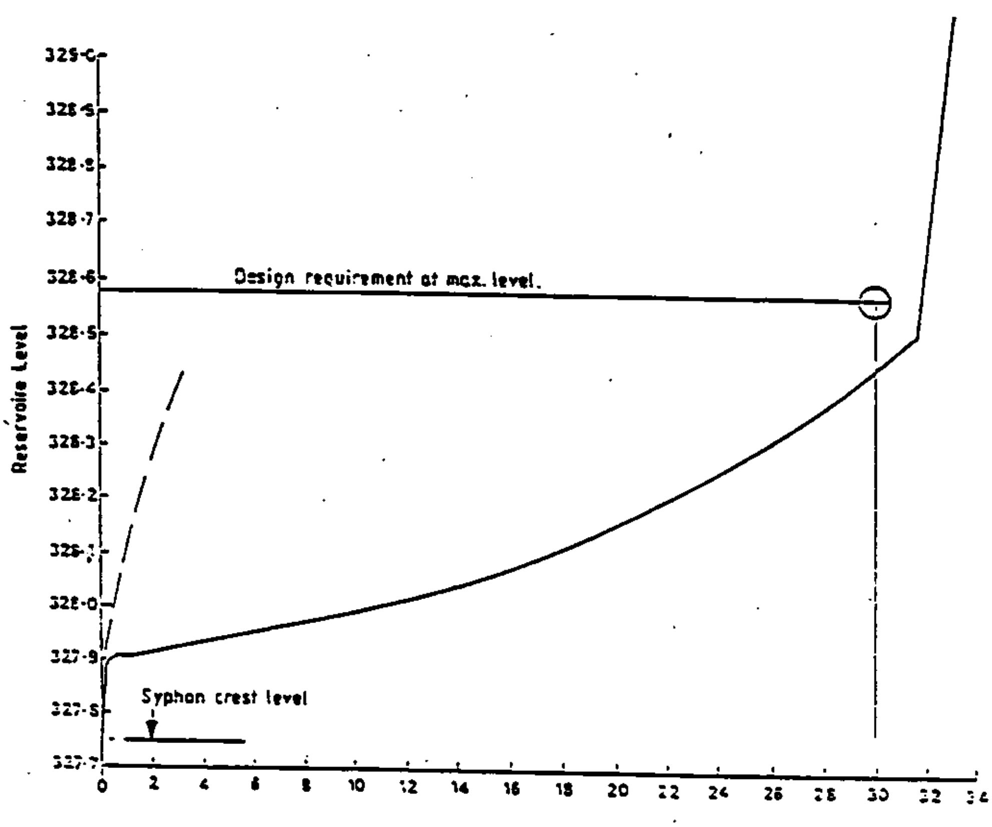

Figure No 2: Flow per Syphon Unit

REINFORCED GRASS SPILLWAYS AND EMBANKMENT PROTECTION

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SYNOPSIS

Various proprietary geotextile and cellular concrete products can be used as reinforcement to enhance the erosion resistance of grass. Its environmental attractiveness and low cost make reinforced grass a possible option on earth dams for auxiliary spillways or as protection against overtopping in extreme flood events. The paper describes key aspects of the research carried out by CIRIA to confirm the effectiveness of reinforced grass, and develop the guidelines on its use by the dam engineer which have recently been published in CIRIA Report 116 - Design of reinforced grass waterways. Effectiveness is particularly dependent on achieving good contact between the reinforcement and the subsoil, as well as maintaining good overall grass cover. Failure generally results from a localised weak point and the importance of careful attention to details, such as the crest and toe, is emphasised.

POTENTIAL FOR REINFORCED GRASS

1. The increase in the number of earth dams requiring additional spillway capacity, which has followed the implementation of the 1975 Flood Studies Report and the Reservoirs Act 1975, has coincided with the availability of a wide range of proprietary materials with potential to improve the resistance of grassed surfaces to erosion or wear. Most of these materials were developed for purposes other than erosion protection against high velocity flow, however engineers involved with earth dams and flood embankments soon appreciated that the low cost of reinforced grass in relation to conventional materials and its environmental attractiveness made it worth considering as an engineering system. The CIRIA project was developed with the principal manufacturers of these materials and major UK owners of earth dams within the DoE Reservoir Safety research programme to evaluate the effectiveness of different proprietary systems and to provide guidance on the design of reinforced grass waterways.

2. Figure 1 shows typical sections through geotextile - and concrete-reinforced waterways. **Geotextile materials** currently available can be subdivided into (a) two-dimensional woven fabrics, (b) two-dimensional synthetic meshes and (c) three-dimensional synthetic mats. The latter are typically 20mm thick and may be either an open fibrous structure or asphalt-filled. Advantages of geotextile-reinforced grass over plain grass can be any or all of the following:-

- Improvement of ground cover and protection of the soil surface from erosion.
- Assistance to the root structure in restraining surface soil particles from erosion by flowing water.
- Improvement in lateral continuity between grass plants and a reduction in the risk of localised failure.

3. **Concrete reinforcement systems** all provide an 80 to 150mm thick surface layer of concrete containing cells which are filled with soil and grassed. They may be subdivided into (a) non-tied interlocking blocks, (b) cable-tied interlocking blocks and (c) in situ concrete. These systems are usually used in conjunction with an underlying woven geotextile, the functions of which are to prevent erosion of the subsoil formation and to assist grass root development. It is important that a concrete system is sufficiently flexible to accommodate minor differential movement with the subsoil.

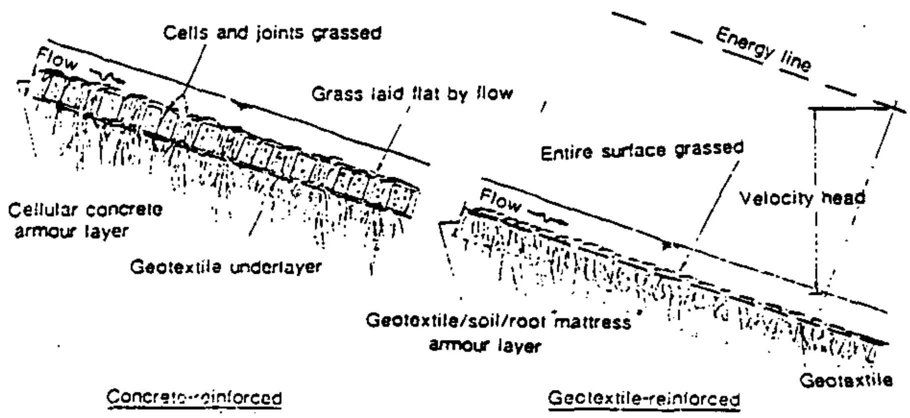

Fig. 1: Cross-section through typical reinforced-grass waterways

BASIS FOR DESIGN RECOMMENDATIONS

General Approach

4. The design recommendations have sought to provide an advisory framework within which the dam engineer develops an appropriate site-specific solution and exercises his own experience and judgement. Considerable effort was put into obtaining a good understanding of the present perceptions and requirements of UK dam engineers and owners. The study has built on the earlier CIRIA Technical Note 71 - Guide to the Use of Grass in Hydraulic Engineering Practice and advice given in the ICE Guide on Floods and Reservoir Safety on acceptable overtopping and use of low-cost auxiliary spillways.

Phase 1 programme

5. The work programme carried out under the Phase 1 study in 1984-85 comprised a State-of-the-Art review which included a literature review; collation of information on existing reinforced grass installations; survey and discussions with UK dam engineers and owners; and recommendations for further short-term research.

6. It demonstrated considerable interest and some use of reinforced grass. However since no basic principles governing use nor experience in service were available, engineering practice was based almost entirely on the judgement of the user and this varied widely. The Phase 1 preliminary design recommendations were published in 1985 as CIRIA Technical Note 120 (now superseded).

Experience of plain grassed waterways

7. The review confirmed the general acceptance and effectiveness in UK of plain grass as protection to auxiliary spillways on small earth dams (particularly for farm reservoirs and flood storage ponds) and to river flood embankments subject to occasional overtopping. Such embankments are typically constructed of moderately cohesive soil with downstream slope of between 1:2 and 1:3. Experience suggests that well-managed grass slopes have withstood short-duration overtopping at heads of up to 0.3m (say discharge intensity, q, $0.25m^3/s/m$ and terminal flow velocity on slope, V_t, 4m/s).

8. However, experience had also shown that unacceptable erosion and failures had occurred under less severe overtopping conditions due to:

- selective erosion at local irregularities and localised areas of flow concentration or poor grass cover
- gulley erosion (with related headcutting) working backwards from the downstream toe
- progressive loss of grass plants due to gradual erosion of soil from root zone, and
- widespread stripping of the soil/root mat, initiated by high localised drag forces at an upstream edge

The unpredictability of subsoil erosion once the protective cover of grass is lost has caused most UK engineers to regard any erosion of embankment subsoil as unacceptable.

Phase 2 programme

9. It was clear that further research was necessary to overcome the gaps in theoretical knowledge and experience in service which constrained the use of reinforced grass protection. The Phase 2 programme carried out in 1985-87 comprised:

- The prototype field trials installation on the 1:2.5 upstream face of North West Water's 10m high disused Jackhouse dam. The objectives of the field trials were to evaluate and compare (a) the installation and management, and (b) the hydraulic and geotechnical performance of nine different reinforced grass systems together with a plain grass control channel. 135 hours of flow tests were carried out in the ten 1.0m wide channels at discharge intensities up to $0.9m^3/s/m$ (terminal flow velocity on slope 8.0m/s) for periods up to 5½ hours at a time.
- Physical model studies to investigate the hydrodynamic performance and failure modes in high velocity flow of (a) concrete block protection systems, at University of Salford and (b) geotextile reinforcement, at Imperial College, London.
- Production of revised recommendations. This included discussion meetings arranged in conjunction with BNCOLD and comment from overseas, in particular from the ASCE Task Committee on the Mechanics of Overflow Erosion and Overtopping of Dams.

10. Due to the complexity of the physical processes which determine the performance of any reinforced grass system, the design approach in the new report necessarily remains empirical. Particular attention has therefore been given to providing the engineer with the necessary appreciation of its context (e.g. related need for effective grass management) and limitations (e.g. subsoil type).

CONCLUSIONS OF CIRIA RESEARCH PROGRAMME

Hydraulic roughness

11. On steep slopes such as dam and flood embankments, grass will be laid relatively flat by the flow throughout the range of discharge intensity, q, likely to be encountered in engineering design. The grass is relatively smooth hydraulically and roughness does not vary greatly. Calculated values of Mannings n for 17 separate tests covering the range of $0.05 < q < 0.1$ $m^3/s/m$ are shown in Figure 2: 80% of these are in the range of $0.018 < n < 0.024$.

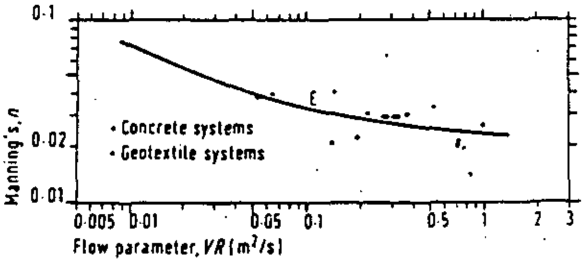

Fig. 2: Calculated values of Manning's n, field trials

Performance of different types of protection

12. In the field trials, failure thresholds (defined as the onset of uncontrolled erosion of the subsoil) were reached for plain grass, geotextile-reinforced grass, and for non-tied interlocking concrete blocks laid without a geotextile underlayer. Recovery of the grass after each test run was excellent, and the individual grass plants within the reinforcement (sward length 50 to 200mm) resisted erosion much better than expected, with substantial cover remaining after all tests. Where grass cover was lost, this was generally from areas where the cover was substandard at the outset.

13. The **plain grass channel** failed at 3.7m/s (equivalent $q = 0.2m^3/s/m$ after 25 minutes testing). In one location the soil/root mat was lifted from the underlying subsoil, and in another a local scour hole developed into an active gulley once the surface protection had been lost. The gulley proceeded to cut downwards into the stony clay subsoil at a rate of 10mm/minute until the test was terminated.

14. All four **geotextile-reinforced channels** failed within the range 5.5 to 7.0m/s ($q = 0.35$ to $0.6m^3/s/m$) by localised uplift of the geotextile/ grass armour layer. The failure scenario is illustrated in Figure 3. Failure was observed to occur at locations where (a) local conditions of substandard grass cover and poor geotextile/soil contact encouraged inflow below the geotextile, and (b) conditions of good cover immediately downstream discouraged pressure relief.

15. The subsequent model study at Imperial College confirmed that the failure mechanism is due to the inherent instability across the geotextile when it acts as a finite-permeability interface between two separate flow fields. The failure potential is exacerbated both by high seepage flow below the geotextile and by surface irregularities - the latter causing high localised lift and drag forces.

16. The channel protected by **non-tied interlocking blocks** failed at 5.0m/s ($q = 0.3m^3/s/m$) after 2.5 hours with an erosion tunnel forming in the subsoil surface below the blocks, which remained interlocked and bridged over the void below. This confirmed that erosion of the subsoil below concrete systems can occur if the subsoil formation is not protected comprehensively by a geotextile underlayer or root growth.

17. Four **concrete-reinforced channels** (three protected by cable-tied interlocking blocks and one by in situ concrete) survived the maximum test conditions of 8.0m/s ($q = 0.9m^3/s/m$) without erosion of the subsoil (this was confirmed during demolition). All these protection systems were laid on a geotextile underlayer. The flow conditions were equivalent to an overtopping head of 0.7m at the top of the channel or dam crest. With all concrete block reinforcement, no appreciable block movement was observed. Following the main test programme, one channel was systematically "vandalised". Only after removing one complete row of blocks across the channel bed, thus forming a 350mm gap in the protection, and loosening the next row downstream was failure eventually induced at 8.0m/s.

18. The model study at Salford University demonstrated that the failure mechanism for concrete block protection is initiated by block vibration and/or lift-off due either to fluctuating pressures caused by flow turbulence or, where seepage flow exists, to pressure difference between the flow fields on either side of the block. The failure scenario for untied blocks, including the effects of misalignment, is illustrated in Figure 4. Wedging between the joints was shown to raise the failure threshold of 25mm thick solid concrete blocks (100mm prototype) laid on an impermeable bed from 4.0m/s (8.0m/s prototype) to 6.5m/s (13.0m/s prototype) with wedging.

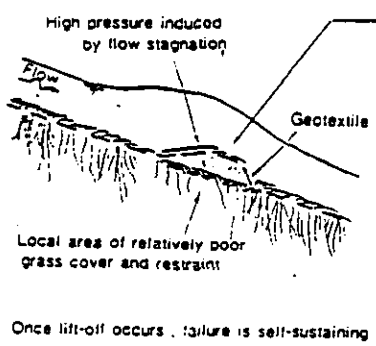

Fig. 3: Failure scenario, geotextile reinforced channels

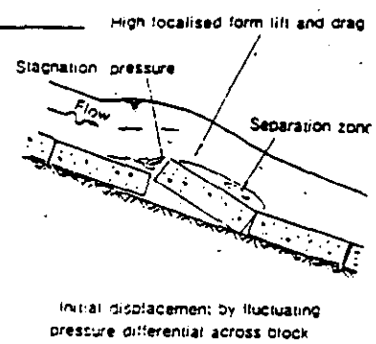

Fig. 4: Failure scenario, concrete block model

Geotechnical/structural tests

19. In situ shear tests carried out after the hydraulic tests with a special block-sized shear box demonstrated that grass roots can make a significant contribution to the shear strength on potential shallow failure planes below the armour layer. Roots contributed about 4 kN/m^2 to the average shear strength of 11 kN/m^2 measured at the block/geotextile interface.

20. Physical lift-off tests were also carried out on blocks. A single 15 kg block - totally isolated from any surrounding blocks - required a peak force of 19 kg to break the grass root anchorage and lift it off the

slope. In an assessment of the effects of additional restraint gained from interblock friction and soil/root wedging, a similar block required a peak force of 140 kg to withdraw it out of the armoured slope.

Key principles and limitations

21. Failure is almost invariably initiated by some local - as opposed to general - inadequacy of erosion protection (e.g. poor cover; irregularity in geometry) which exploits locally the kinetic energy of the flow. The effectiveness of either plain or reinforced grass in preventing subsoil erosion depends on four key requirements being achieved:

- full and intimate cover of the subsoil surface
- discouragement of seepage flow in direction of slope
- good integration of the armour layer with the underlying subsoil
- avoidance of surface irregularities which cause high localised drag

22. Limiting values for erosion resistance of different reinforcing systems recommended in the new report are shown in Figure 5. These are based on the earlier recommendations in CIRIA TN 71 amended in the light of the research programme which includes the corporate judgement of the CIRIA Steering Group. It is emphasised that all values for reinforced grass assume good, well-established grass cover.

23. Experience gathered by the ASCE Task Committee on performance of embankments under overtopping clearly indicates the differing performance of cohesive and non-cohesive soils (the latter having lower erosion thresholds and appreciable seepage flows). The recommendations in the new report are applicable only to grassed waterways with relatively low permeability subsoil and subject to unidirectional flow.

DESIGN, SPECIFICATION AND CONSTRUCTION

24. Because of the importance of constructing and maintaining a high quality of both civil works and grass cover to achieve the potential erosion resistance of reinforced grass, the design procedure in the new report is presented within a framework which encompasses feasibility, specification, construction, grass establishment and management. This was considered particularly important because of the need to improve on the relatively poor standard of planning and establishment of vegetation which is generally associated with civil engineering contracts.

Planning

25. The initial stage in the design procedure must establish the feasibility of using a reinforced grass waterway, together with the basic design parameters. Points to be considered at this stage will include the following:

- Frequency, duration and quantity of flow. Grass-lined waterways are not recommended for flow duration longer than 48 hours, and a period of recovery of (say) 2 weeks must be possible after discharge.
- Type of subsoil. Reinforced grass waterways should only be constructed on a relatively cohesive subsoil with low permeability (typically less than 10^{-5}m/s).
- Management capability. The designer and client must be satisfied that an effective grass management progamme can be carried out to maintain the waterway in a serviceable condition.

- Access to site may determine the type of construction plant and materials. In one situation, consideration was given to using a helicopter to deliver cable-tied concrete blocks to the site.
- The type of organisation who will construct the works. Smaller owners may wish to use direct labour instead of a contractor, and the designer should be satisfied that such an organisation is capable of constructing the works to the required standard.

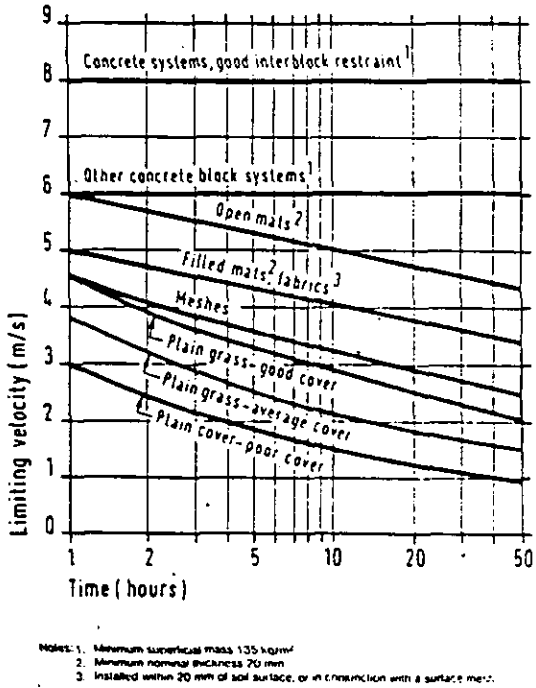

Fig. 5: Recommended limiting values for erosion resistance of plain and reinforced grass

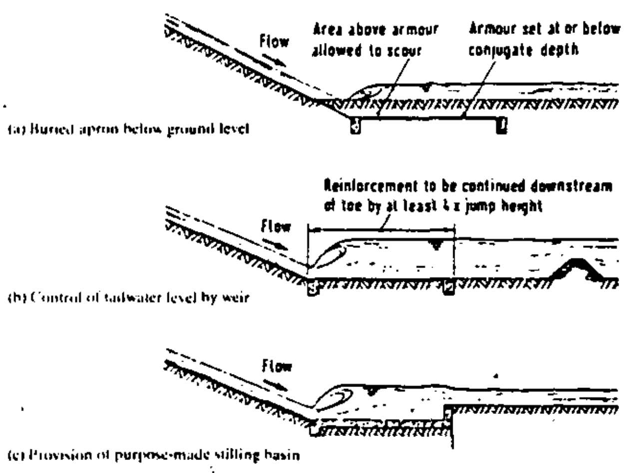

Fig. 6: Typical toe details

Detailed Design

26. **Hydraulic** aspects to be considered include the determination of the type of reinforcement system to be used. It is necessary to estimate the hydraulic loading in terms of velocity and duration down the waterway during the design flood (using Manning's equation for peak discharge) and decide on the most suitable type of reinforcement system using Figure 5. The velocity can be changed by varying such parameters as waterway width and slope, and reducing the velocity may result in a cheaper type of reinforcement system being required. In many situations, however, the waterway dimensions are constrained by the topography of the site. Panel engineers would generally prefer to use concrete systems for high discharge intensity and for Category A dams where it is of paramount importance that there is no failure. Cheaper and less erosion-resistant geotextile systems are more likely to be used for lower discharge intensity and on lower categories of dam.

27. From the **geotechnical** viewpoint, a ground investigation should be undertaken to determine the general nature of the subsoil. Trial pits should be inspected for consistency of material and any discontinuities such as cracks, animal burrows and root holes which could allow water to penetrate the subsoil both rapidly and deeply. Non-homogenity and/or

discontinuities can radically effect the saturation and consequent stability of the subsoil under flow conditions. Information from the ground investigation should be used to consider (a) the groundwater and seepage flow conditions within the subsoil and (b) the shallow and deep slope stability prior to, during and after operation of the waterway. Consideration should also be given to the absolute and differential settlement of the waterway.

28. The **toe** detail must provide for satisfactory transition from supercritical flow on the slope to the tailwater condition, which will generally be subcritical. Depending on the tailwater rating and discharge, a hydraulic jump may occur either on the slope or at the toe, or supercritical flow may continue downstream. One reason for limiting the maximum design velocity to 8m/s was to limit the energy dissipation at the flow transition. The least satisfactory situation is when the possible range of tailwater levels causes a potential hydraulic jump over a wide range of locations on the reinforced grass slope. In such situations, additional mechanical anchorage of the reinforcement may be needed. The reinforcement system should be terminated in a way that will minimise the possibility of undermining. Enhanced protection, such as gabion mattresses may be required in the area downstream of the toe. It may be necessary to control a hydraulic jump and methods by which this may be done are shown in Figure 6. The detail in (a) would only be acceptable if scouring was expected to occur infrequently.

29. The **crest** detail must minimise the risk of erosion of the waterway from the area upstream. The crest detail should prevent any direct flow path through to the underside of the reinforcement from upstream so as to discourage excessive seepage flow. Examples of typical crest details are given in Figure 7. With limited space and for large discharge, designers may wish to allow overtopping along the entire crest length but this can concentrate flow at the mitres and lead to rapid erosion. Any changes in waterway width should be gradual.

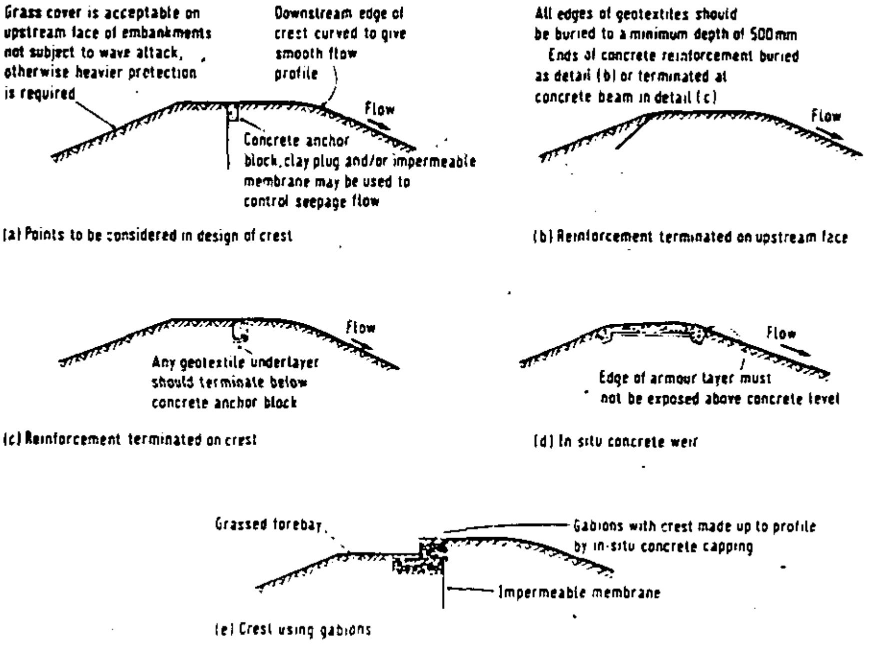

Fig. 7: Typical crest details

Specification

30. Materials, workmanship, and any other requirements of the designer not shown on the drawings must be specified. This applies especially to constructional tolerances and establishment of vegetation. The recommended procedure is (a) to specify the required properties or performance of the components, and (b) to indicate general methods of working and acceptance criteria for each stage of construction.

31. Particular specification requirements relating to establishing satisfactory grass growth include:

- The seed mix should take into account the local soil and climate, and seeding should only be carried out at certain times of the year.
- The maintenance or "aftercare" period should span two growing seasons and should make provision not only for all necessary cutting, fertilising and weedkilling but for revegetation of areas of poor grass establishment.
- Geotextile opening size must balance the apparent conflicting requirements of (a) allowing root penetration and slope drainage, and (b) preventing erosion of the subsoil.

Construction

32. It is recommended that a method statement should be obtained from the contractor for approval before the works are constructed. Particular points requiring attention include:

- A firm, even formation must be achieved with specified tolerances consistent with the type of reinforcement.
- Vehicles, particularly construction plant, should be prevented from passing over the waterway after the reinforcement system has been laid unless it has been designed to withstand traffic loading.
- Correct jointing methods should be adopted between adjacent sheets of geotextile to ensure continuity. This is normally done by overlapping.
- Establishment of grass can be difficult on steep slopes where run-off can wash grass seed and soil down the slope. Artificial establishment aids are available to protect the soil surface and seeds. Hydroseeding has been found to be particularly successful on steep slopes.
- Careful trimming and preparation of the subsoil formation is necessary in order to ensure that no voids exist between the subsoil and reinforcement. In order to achieve deep root penetration, any topsoil layer must be worked into the upper subsoil horizons.

MAINTENANCE OF GRASSED WATERWAYS

33. The ability of the owner to provide the necessary maintenance will have been considered at the planning stage, and design should include the provision of an appropriate maintenance plan. The objective should be to maintain an even sward between 50 and 150mm, in height. Typical examples of the maintenance abilities of owners are as follows:

- Owners of strategic dams, such as Water Authorities, usually have sufficient machinery and staff to carry out regular maintenance.

- Owners of amenity lakes, where a high standard of horticulture is often undertaken, can use their existing resources to maintain a reinforced grass waterway at little additional cost to themselves. This low capital and maintenance cost together with its visual form makes reinforced grass particularly attractive in such locations.
- Owners of small farm dams can maintain grassed waterways by grazing sheep. Grazing by cattle is not recommended since they can cause considerable damage to the surface.

34. It may be necessary to apply dressings of fertilizers, particularly in the early years of growth. Limited weed growth can be tolerated but weeds which threaten to compete with the grass should be controlled with suitable chemicals.

35. Grassed waterways should be regularly monitored to ensure that they are in satisfactory condition and immediate steps should be taken to repair any damage. In situations where the waterway comes under the Reservoirs Act 1975 the supervising engineer will be responsible for inspections and the frequency of his or her visits will be recommended by the inspecting engineer. An inspection should also be carried out immediately after the waterway is known, or thought, to have operated. Points to be considered when inspecting a grassed waterway include the following:

- Quality and uniformity of the grass cover, especially the presence of any bare patches.
- Localised settlement of the armour layer, and/or subsoil, particularly of the crest.
- Any damage to the reinforcement system caused by animals, mowing machinery, vandalism or any other means.

CONCLUDING REMARKS

36. The CIRIA project has resulted in practical guidelines for the planning, design, construction and management of reinforced grass waterways. Feedback from the engineering profession has been positive both in the UK and overseas and there has been an increase both in the understanding and use of reinforced grass in the UK as a result of the project. Gaps in knowledge still remain, and further long-term research could usefully investigate in more detail the stability of the reinforcement and the behaviour of the armour layer/soil interface under high velocity flow.

ACKNOWLEDGEMENTS

37. The authors gratefully acknowledge the permission of the Director of CIRIA and the Partners of Rofe, Kennard & Lapworth for permission to publish this paper, and the support of the many individuals and organisations which have participated in and supported the research. Particular acknowledgement is given to the Department of the Environment as principal funder, North West Water for making available the Jackhouse Reservoir site, and Salford University Civil Engineering Ltd who carried out the field trials.

TIPPING GATES FOR AUXILIARY SPILLWAY CONTROL

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SYNOPSIS

Auxiliary spillways have been constructed (1980-82) at Greenfield and Yeoman Hey reservoirs, operated and owned by North West Water. These spillways are intended to operate only during very rare floods, to take flows in excess of the main spillway capacities, corresponding to return periods of 10 000 years or more. Flows to the spillways are controlled by tipping gates, which comprise inclined concrete slabs designed to overturn when the moments due to water pressures overcome the moments of weight.

This paper describes the principle of operation of tipping gates, the reasons for their selection, the design criteria, including wave considerations, and gives the results of tests on a simplified prototype gate and on production gates.

BACKGROUND

1. Greenfield and Yeoman Hey reservoirs are impounding reservoirs for water supply, situated in a tributary valley of the River Tame about 10km east of Oldham, Greater Manchester. Yeoman Hey reservoir is situated just below Greenfield and discharges into Dove Stone reservoir.

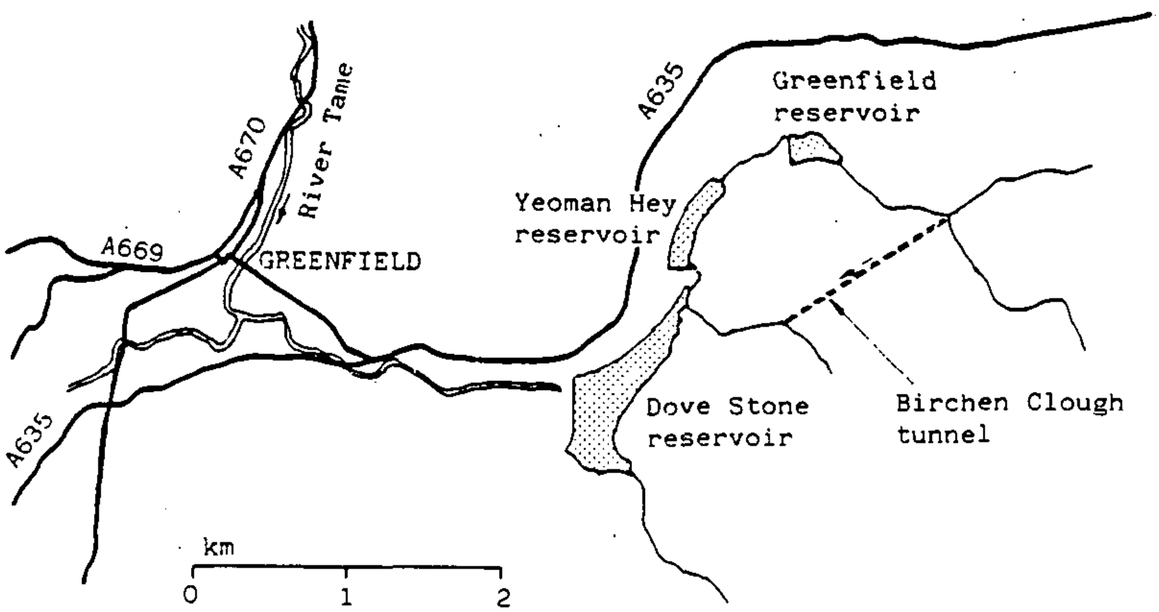


Figure 1: Location of Greenfield valley reservoirs

Name of dam	Built	H (m)	Capacity (Ml)
Greenfield	1897-1902	19	462
Yeoman Hey	1876-1880	20	936
Dove Stone	1960-1967	31	5048

2. Statutory inspections were carried out under the Reservoirs (Safety Provisions) Act 1930 in 1979. Breaching of either of the two upstream dams would be likely to result in failure of Dove Stone dam, and would have a catastrophic effect on the populated areas downstream. The dams are of earthfill with puddle core construction and were considered to be in Category A, as defined in **Floods and reservoir safety: an engineering guide** (ICE, 1978), and required to be capable of passing the probable maximum flood (PMF).

3. Model tests of the pre-existing spillways at Greenfield and Yeoman Hey indicated that the structures could be uprated to pass the 10 000-year floods, but could not reasonably be improved to pass the PMFs.

4. Various options for providing for the PMF event were considered including, individually or in combination:
(a) auxiliary spillways at each dam;
(b) extensive improvements to Birchen Clough tunnel which provides a bypass around the reservoirs; and
(c) lowering the retention level of Dove Stone reservoir to absorb the floodwater resulting from the possible failure of the upstream dams.
It was decided that the existing spillways should be uprated to pass the 10 000-year floods and that auxiliary spillways should be provided for the extra discharges up to the probable maximum floods.

5. The dams sit in the highly confined Greenfield valley, where space for construction was severely limited: auxiliary spillways operating only beyond the 10 000-year event, protecting the dams but permitting some damage resulting from their use, might thus represent an economical solution. To achieve this each auxiliary spillway would have to:
(a) pass no flow until the reservoir reached the peak level of a 10 000-year flood; and
(b) pass the additional flow above the 10 000-year flood discharge with a minimal increase in reservoir flood level.

6. The above criteria indicated the choice of some form of spillway crest control using automatic gates or a fuse plug. Tipping gates, operated by hydrostatic forces, were chosen for the following reasons:-
(a) Operation would be automatic, reliable and predictable by theory within close tolerances.
(b) The gates could be designed to tip rapidly once they became unstable.
(c) Operation of the gates would not be significantly affected by floating debris.
(d) Operation need not depend on electrical or mechanical components, nor on a power supply.
(e) Gates operating on the same principle had been successfully employed in an auxiliary spillway at Fontburn reservoir, Northumberland.

PRINCIPLE OF OPERATION

7. In its basic form (Figure 2) the tipping gate comprises a thin rectangular plate of weight W, hinged along its lower edge and resting against upstream piers of negligible width inclined at an angle σ to the horizontal. The disturbing moment of the hydrostatic pressures on the upstream face is given by:

$$M_D = \frac{bh}{2\sin\sigma} \cdot hw \cdot \frac{h}{3\sin\sigma} = \frac{bh^3w}{6\sin^2\sigma}$$

where b = width of gate and
w = unit weight of water.

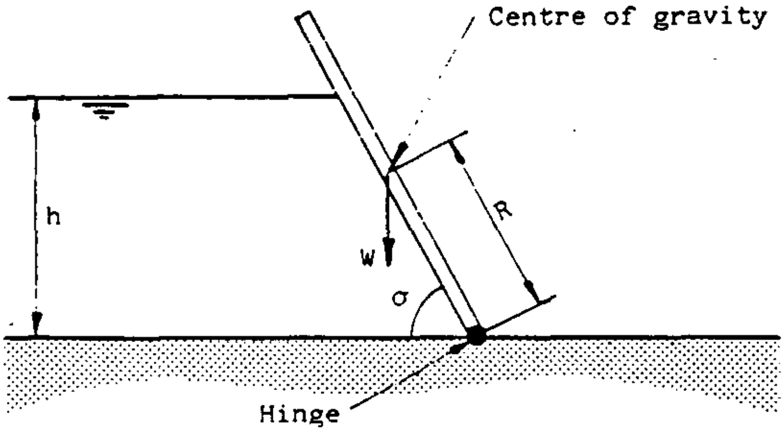


Figure 2: Basic cross section of tipping gate

8. The restoring moment of the weight about the hinge is given by:

$$M_R = WR\cos\sigma$$

The gate will move away from the supporting piers if the disturbing moment exceeds the restoring moment, so on the point of tipping:

$$\frac{bh^3w}{6\sin^2\sigma} = WR\cos\sigma \text{ and hence } h = \sqrt[3]{\frac{6\sin^2\sigma WR\cos\sigma}{bw}}$$

9. If W, R, b and w are fixed, then this function for h reaches a maximum at σ = 55°. This means that if the gate is supported at an angle of less than 55° to the horizontal it will start to move away from the pier at a water level lower than the level required to tip it right over. As the water level increases, the angle of inclination will increase until the gate reaches unstable equilibrium at an angle of 55°. If the gate is supported at an angle of more than 55° then it will not move until the water level reaches the level required for tipping, at which state the gate will immediately be in unstable equilibrium and so tip rapidly.

FLOOD STUDY

10. A flood study of the Greenfield and Yeoman Hey catchments was carried out based on preliminary performance characteristics for the proposed auxiliary spillway arrangements. The principal data and results are summarised in the table below.

Reservoir	Greenfield	Yeoman Hey
Main spillway crest level	271.58	234.09
Auxiliary spillway crest level	271.58	234.09
Tipping gate sill level	271.58	233.45
Gate tipping level	273.10	235.40
10 000-yr flood:		
Maximum reservoir flood level	273.07	235.40
Probable maximum flood:		
Main spillway discharge	68	70
Auxiliary spillway discharge	39	70
Maximum reservoir flood level	273.14	235.41

Levels are in metres above Ordnance Datum Newlyn and discharges are in m^3/s.

DESIGN CONSIDERATIONS

11. In addition to the criteria in paragraph 6, the following factors were taken into account in the design of the gates:-

(a) There should be minimum maintenance requirements.

(b) The construction materials should be able to withstand severe weather conditions.

(c) The gates should not be significantly damaged in operation and should be capable of being reset without difficulty.

(d) Facilities for the verification of the tipping level by testing should be incorporated in the design if possible.

12. The gates are inclined at an angle of 65° to the horizontal, to ensure unstable equilibrium at the point of first movement. They were designed to tip into recesses in the floor of their spillways, so as not to obstruct the flow, with the recesses filling with water as the gates start to tip to provide a cushion to break their fall and reduce the risk of damage in operation. Because of the recess and the practical requirement for a gate of finite thickness, the pivot had to be positioned below the spillway floor and comprised a stainless steel nosing along the lower downstream edge of the gate, resting against a stainless steel angle set into the spillway floor.

13. Concrete was chosen for the construction of the gates because of the weight required, which is of the order of 1000 kg/m^2 for a gate inclination of 65° and water depths at tipping of 1.5 to 2m.

14. Although the tipping level of a single gate can be determined with relative precision, there were fears that, in a group of several adjacent gates, the operation of the first gate would slightly delay the tipping of the others, due to the upstream drawdown in the vicinity of the tipped gate and the hydrostatic pressures from the tailwater which would result from its operation. The gates were therefore linked together in groups by means of stainless steel connecting bars, inserted into sockets in the adjacent gate edges. These and other design features are shown on Figure 3. In other situations, where the approaches and tailwaters of the gates could be effectively separated, it would probably be advantageous to design the gates to tip at successive increments of level, so that the rate of increase in reservoir outflows would not be so severe.

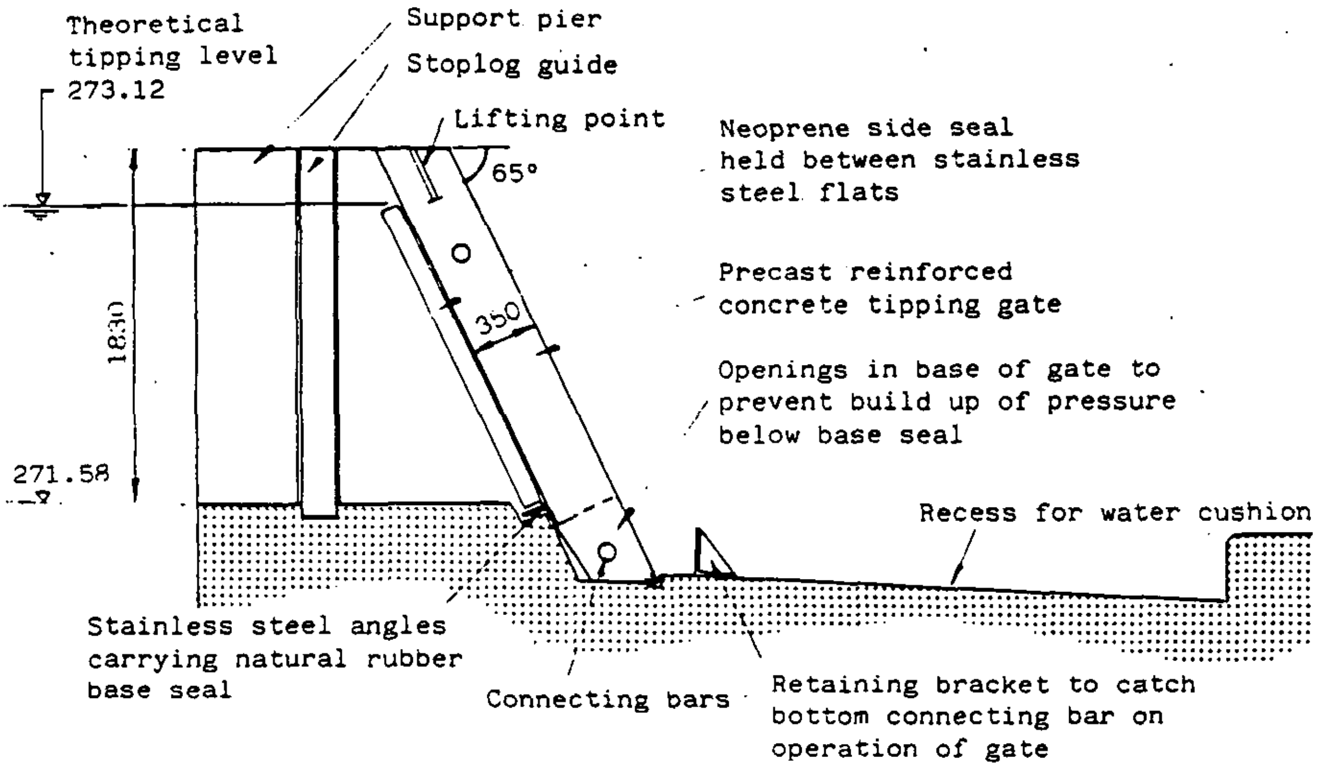


Figure 3: Cross section of Greenfield reservoir tipping gate

15. Checks were also made of the sensitivity of the tipping levels to various errors or tolerances in the construction. The most significant variable is the unit weight of reinforced concrete, with a difference of 4% affecting tipping levels by about 30mm. The combined effect of various allowable dimensional tolerances would be about the same, suggesting an expected variability of tipping levels of $\pm$50mm. This is likely to be no more significant than errors in the flood predictions or spillway discharge ratings.

EFFECTS OF WAVES

16. Like all shoreline structures, tipping gates are subject to intermittent pressures caused by wave action, which would make them vulnerable to premature tipping. Theoretical studies were carried out in order to estimate the relationship between wave magnitude and the amount by which the tipping level would be reduced.

17. Considerations of the approach depths to the gates and the relative effects on the disturbing moments of breaking, broken and non-breaking waves led to the conclusion that the gates would be most vulnerable to non-breaking waves. Three approaches were considered in the studies:

(a) evaluations of the peak disturbing moments during the wave cycle to check whether the sum of the dynamic moment and the hydrostatic moment would exceed the restoring moment due to the weight of the gate, thereby initiating movement;

(b) comparisons of the amount of energy which has to be applied to the gate to take it from rest to the point of tipping, with the energy content of the waves; and

(c) dynamic analyses of the movement of the gate, taking account of its inertia, the variation with time of the dynamic wave moment and the variation of the hydrostatic and restoring moments during the movement of the gate.

18. The results of the studies for Yeoman Hey reservoir are plotted on Figure 4, with curves A, B and C corresponding to approaches (a), (b) and (c) above. All these approaches involved simplifying assumptions, the most significant of which tend to cause the vulnerability of the gates to move or tip to be overestimated. In approach (b), not all of the energy content of the waves is likely to be applied to the gates, as they have to move before any work can be done, so a family of relationships was obtained, representing 50%, 75% and 100% of the wave energy being converted into work done on moving the gates.

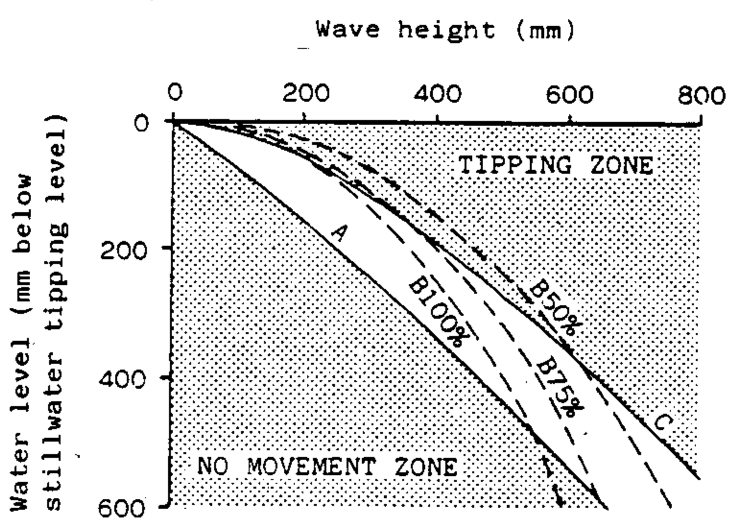


Figure 4: Effect of waves on tipping level of Yeoman Hey gates

19. Curve C is plotted from dynamic analyses based on wave heights of 400 and 600mm. The dynamic wave moment was assumed to increase from zero to the peak value in the first quarter of the wave cycle and then reduce to zero in the second quarter, according to the sine function. Negative dynamic pressures during the third and fourth quarters of the wave cycle, which would have restoring effect on the gate, were ignored. Other simplifications were that there would be no loss in hydrostatic pressure or dynamic pressure due to leakage of water past the sides of the gate, and that there would be no reduction in pressures due to the acceleration of the gate. The analyses consisted of step-by-step calculations of the angular acceleration of the gate, until it reached the point of equilibrium and tipped, or stopped moving and then started to move back towards the piers.

20. The estimated relationships between wave heights and the corresponding reductions in tipping level are tabulated below for both Greenfield and Yeoman Hey reservoirs. These relationships are based on approach (c) and are almost certainly conservative, for the reasons already given and also because the linking of the gates and any obliquity of the waves would further reduce their tendency to tip prematurely due to wave action. However, it should be borne in mind that it only takes one wave to tip a gate, and that the biggest wave in an event lasting for 2000 waves will probably have a height of about twice the significant wave height.

Wave height (mm)	Reduction in tipping level (mm)	
	Greenfield	Yeoman Hey
100	20	20
200	70	60
300	140	120
400	220	190
500	310	270
600	420	350

TESTING

21. It was decided to provide for the testing of a simplified prototype gate to obtain an indication of the degree to which construction tolerances and initial leakage might influence the tipping level and in order to gain practical experience of any problems which could be overcome in the design of the production gates. Temporary concrete piers and a sill were cast in the reservoir bywash channel to support the prototype gate and water admitted to the bywash, where it ponded up behind the gate. The gate was successfully tipped twice at the predicted level, with a video film being made of the test.

22. Most of the production gates were also tested individually (with their linking bars removed). The procedure comprised:
(a) fixing restraining brackets to the tops of the piers to catch the gate after a top movement of 200mm;
(b) placing stoplogs in grooves provided for the purpose just upstream of the gate;

(c) filling the intervening space by pumping; and
(d) noting the level at which the gate began to move.

23. The production gate tests gave a poorer demonstration of tipping gate performance, as there was insufficient pumping capacity to maintain the upstream water level in the face of the initial leakage past the gates as they started to move. The leakage was reduced by using tape as a temporary seal, but insufficiently to tip any gate as far as its temporary restraining brackets. On some of the Yeoman Hey gates there was an initial movement at a water level up to 270mm below the theoretical tipping level and up to 200mm below the level at which more substantial movement occurred (breaking the seals and causing the test to end). This was attributed to the gate being initially supported at a point a short distance back along its base, with the initial movement being caused by the gate rocking slightly as the pivot point moved to the downstream edge. The test results are given in the table below.

GREENFIELD	Theoretical tipping depth = 1.54m	
Gate N°	Depth at initial movement (m)	Depth at major leakage (m)
1	1.53	1.53
3	1.48	1.48
5	1.53	1.53
YEOMAN HEY	Theoretical tipping depth = 1.95m	
Gate N°	Depth at initial movement (m)	Depth at major leakage (m)
2	1.71	1.82
3	1.73	1.76
4	1.68	1.86
5	1.71	1.79
6	1.73	1.75
7	1.70	1.78
8	1.73	1.93
9	1.78	1.87
10	1.73	1.83
11	1.90	1.93
12	1.95	1.95

ACKNOWLEDGEMENTS

24. The authors thank Mr P L Birtwistle, Regional Water Supply Manager, North West Water, for permission to include details of the improvements to the Greenfield valley reservoirs. Mr W A Foreman of Binnie & Partners was responsible for the design of the tipping gates, under the direction of Mr K K Law (partner, now retired). Dr Hughes was a member of the design team and was Assistant Resident Engineer for the improvement works. Mr Ackers carried out a design review, including the study of wave effects.

DISCUSSION: TECHNICAL SESSION 4

OVERFLOW REPAIRS AND EXTENSIONS

Session Chairman: H N Jones (Deputy Managing Director,
M J Gleeson Southern Ltd)

The first paper is on the Himley Hall Great Pool Overflow and Stabilisation Works and we have 2 authors: Mr Johnson of the Dudley Metropolitan Borough Council and Mr Jackson of Sir Alexander Gibb & Partners.

Our second paper is 'Modifications to Reservoir Overflow Systems' by Mr Ormerod of Sir Murdock MacDonald and Partners. Then we have a paper to be introduced by Michael Kennard, on improvements to overflow works at some British dams. The fourth paper on reinforced grass is given by Mr Bramley of CIRIA and Mr Hewlett of RKL. Finally we have a paper on tipping gates to be presented by Dr Hughes.

J K HOPKINS (North West Water)

One of the reservoirs mentioned in Mr Ormerod's paper is Walverton reservoir which is situated near Nelson in East Lancashire. It's a small compensation reservoir. It has a nominal capacity of about 90 megalitres, but due to siltation its effective capacity now is only about 55 Ml.

The design flood which is the PMF for the reservoir is about 100 cumecs. The weir to bank full level has been model tested and it can pass about 45 cumecs. It is interesting that the capacity of the stream as it passes through Nelson is a great deal less than that.

The spillway passes into a drop weir structure and the draw-off culvert passes out into that same weir.

To accommodate the full PMF flow, we considered two options seriously: one was to lower the existing weir level by half a metre and to protect the surface of the dam with grass-reinforced blocks; the other option was to provide an additional weir over the centre of the dam, which is in fact the chosen design and under construction at present.

From model-testing quite a high tailwater depth is developed at the toe of the dam and, to accommodate that and splashing and over-topping of the side walls, the lower parts of the embankment are to be protected by a geotextile. We have provided chute blocks as energy dissipators. Ground conditions, which are quite poor at the toe, effectively prevented us putting in a stilling basin, as below about a metre of clay there are sands and silts with artesian water pressures in them.

The design of the channel in cable-tied blocks. They are actually solid, but it will allow for settlement and movement and dissipation of any pore pressures.

The top two rows of chute blocks are slightly smaller than the rest. The first row is a third height and the next row is two-thirds height. This modification we made during the model tests, as we had quite heavy splashing from the first two rows of blocks. The blocks are designed for half-PMF flood and they performed quite adequately at full-PMF, but with just a certain amount of splashing over the side walls, so a geotextile protection adjacent to the walls has been used to protect against erosion.

D E EVANS (Binnie & Partners)

I endorse the view that in undertaking the costly process of upgrading spillways to existing reservoirs to meet current flood standards it is justifiable in some cases to economise by accepting that parts of the spillway channel and tailworks could be overloaded and possibly damaged in an extreme event, provided the dam would not be affected.

A case in point was Lower Carno reservoir where Binnie & Partners had designed works for Welsh Water. At Lower Carno a discharge capacity of 140 cumecs was required for PMF conditions but the original side channel spillway could only handle 40 cumec, controlled by the restricted width and depth of the channel leading to the head of the chute at the left mitre of the dam embankment. The topography ruled out the possibility of heightening the embankment to provide greater flood routing. A detailed study had been made of providing a second spillway of 100 cumecs capacity at the other end of the dam, with a chute following the right hand mitre and terminating in a stilling basin near the embankment toe. The stilling basin was essential to dissipate energy and turn the flow through 70° in plan to line up with the streamcourse. This arrangement had been rejected in favour of enlarging the existing spillway which, although difficult to do, would be cheaper because a new stilling basin could be dispensed with on the grounds that the existing chute discharged directly in line with the streamcourse well clear of the dam embankment.

In the works under construction at Lower Carno the original masonry weir, the wide channel and the upper half of the chute were removed and replaced by reinforced concrete structures comprising a slightly longer weir; a side channel 4m wider and 4m deeper than the original; and a tapered transition sloping down to meet the retained portion of the original masonry chute. The walls of the masonry chute were raised 1.5m to contain the 140 cumecs flow and a bridge across the chute was rebuilt to provide an adequate waterway beneath.

It was realised the curved side channel and asymmetrical taper needed to fit the site could generate unacceptable cross waves in the chute flows. Therefore the end of the taper was designed to act as the hydraulic control on conditions in the curved channel and prevent acceleration before the direction of flow was aligned with the chute. The geometry of the channel and taper was designed with the aid of a mathematical model before going on to simple model tests to check performance and wave generation. A minor change of the taper geometry was found to almost eliminate cross waves and confirmed the planned raising of the chute walls would be adequate.

No modifications are being made to the downstream end of the chute where the existing shallow stilling basin can contain a hydraulic jump at flows up to 10 cumecs. At greater discharge the stilling basin will merely act as a deflector directing flow downstream.

J B BOWCOCK (Sir Alexander Gibb & Partners)

I wanted to add very briefly to some of the points that were made in the paper on Himley Hall, for which I was appointed panel engineer in 1984.

This case history illustrates both points made this morning; the need to protect old dams and the need for cheap solutions. There was a definite need here for appropriate solutions as in the case of all old dams and the principles of preserving the old dam and finding the cheapest solution were applied. This was particularly the case for the spillway; a number of solutions had been considered over the years, including some fairly massive concrete solutions. I think the grass-crete solution illustrates how well that has blended in to the surrounding land and it has not been necessary to cut down some of the attractive trees on the crest of the dam.

J ELLIS (University of Strathclyde)

Many of our older spillways are of fairly unusual shape involving combinations of bends, steps, tapers, bridge piers, all of which make it rather difficult for the engineer to assess the capacity of the spillway channel and the risks of walls being overtopped. This is especially so in the case of steeper channels, where the flow is super-critical and strong cross-waves are likely to arise.

Over the past two years, CIRIA has been supporting a project which aims to try and develop a numerical model which will allow prediction of flow patterns in some of these more complicated forms.

In the model each point is defined by a set of co-ordinates along each wall of the channel. Likewise, if the crest is of a regular form, it too would be defined by a set of co-ordinates, as would be wall heights and other features such as bridge piers.

Typical information which you might obtain from this model includes contours of water depth, and the velocity magnitudes and directions can also be found. A set of representative cross-sections may also be produced.

The simplest and probably the most common form of presentation would be simply depths of flow, perhaps along the centre line of the channel, but more likely, along the walls.

Some empirical data for cascades of steps have been incorporated into the model and it is now able to represent a range of channel features.

A C MORRISON (Sir William Halcrow & Partners)

A few years ago I was asked to look at the reservoirs in different ownership upstream of the Himley Hall reservoir and I am pleased to see that the effects of their failure has been considered by the panel engineer.

Spring Fall might have been built 200 years ago by Capability Brown with an upstream clay membrane, but during the formation of the Country Park, which was carried out by landscape architects, it was breached so that a bulldozer could work within the reservoir and the breach was filled with, and I quote, 'impermeable colliery shale', which might account for the seepage on the downstream face.

Another interesting point in the valley is upstream of Spring Fall. There are the remnants of the outer ends of an embankment, which was clearly breached in the past.

P JOHNSON (Thames Water)

I have a question to the speakers for the Himley Hall paper. I note that clearly the scale of the works that were required for spillway modifications were very much dependant upon the quality of the hydrological analysis that was done. The FSR techniques were used for a catchment of only 3½ square kilometres upstream. Were the authors confident that the sensitivity of that analysis was satisfactory and that therefore the peak flows they had to design to were acceptable and couldn't in fact have been reduced legitimately?

P F JOHNSON (Dudley MBC)

There were several studies of the run-off produced on the catchment area. Basic Flood Studies Report checks were first carried out and Dr Woods at Severn Trent also carried out a hydrological study. That confirmed a PMF of around 35 cumecs. We now have something like 10-fold the capacity of the spillway that was present.

C D ROUTH (MRM Partnership)

I have two questions for the authors of the Himley Hall paper. The paper mentions that the clay seal of the Great Pool was performing well, there was no seepage from the downstream face, and there were no signs of distress.

The piezometers were reporting very low pressures but the dam appears to have been condemned on the basis of an assumed phreatic surface. The summer piezometric levels only were mentioned and the assumption was made that the phreatic surface would have gone up in the winter. Was it not possible to monitor or delay designing the works until winter levels had ben measured? In view of the cost of the works, could this delay not have taken place?

The second point is that the dam is in the not infrequent situation where inundation would occur whether or not the dam was in place. The paper mentions the use of Enkamat and also the fact that crest works were undertaken to provide a small crest wall, and I would be interested if the authors could give an indication of whether they assessed the duration of overtopping and whether in fact it would have been possible to provide a reinforced downstream embankment for quite a substantial part of the crest?

E A JACKSON (Sir Alexander Gibb & Partners)

The piezometers showed that there was a lot of downward drainage in this embankment so wherever you put a piezometer you get that phreatic surface. The situation which we have actually got is much safer than we actually designed for. We designed for a fairly high phreatic surface which is somewhat conservative, but nevertheless we did look what would happen if the bottom piezometer caught up with the top piezometer, which showed that maybe a little berm would help to stabilise the toe and to control seepage if it ever occurred at the toe. At the moment, there is absolutely no seepage whatever coming up anywhere.

As far as the overflow was concerned, we certainly thought of overflowing over the top of the dam, but to do that would have meant cutting down all the trees on the face of that overflow section, and there is no way that the conservation and the environmental people would have allowed us to do that.

C D ROUTH (MRM Partnership)

Is the berm required?

E A JACKSON (Sir Alexander Gibb & Partners)

Under normal circumstances no, but there is always a possibility that it might and piezometric levels may rise in the future, or in very severe rainfall conditions, and it's not a very expensive appendage. It was 1% of the cost.

C DE SOUZA (Anglian Water)

My question is directed at the authors of the Himley Hall paper. Is it unacceptable to have trees on a reservoir crest as far as stability or long-term strength goes?

E A JACKSON (Sir Alexander Gibb & Partners)

As far as trees on the downstream surface are concerned, we did say that if there were any big trees they should be pollarded, but it would be very unwise to cut them down because the roots would then rot and you would have ready-made holes deep into the dam. As far as I am aware, there was no damage caused. Trees on the face of the dam will help with the drainage providing they are well looked after and maintained by people who know about trees.

C J A BINNIE (W S Atkins)

I have experience with tree preservation orders on dams. The panel engineer has the power to have trees cut down and the tree preservation order has to be amended.

H N JONES (Session Chairman)

I would like to direct a question to Dr Hughes.

Was there any problem with frost and the possibility of the gate seizing up?

DR A K HUGHES (North West Water)

There has not been a problem of frost. The gates will not move until 6 feet (1.83m) of water is against the gate, i.e. it is immersed, and secondly the downstream side of the gate is drained.

W McLEISH (R H Cuthbertson & Partners)

I would like to make two points. One on the paper by Kennard and Bass, to commend the solution of increased flood storage as one of the options to be considered when dealing with inadequate spillways. So often development downstream has tended to make communities much more vulnerable to large discharges from reservoirs. To lag the flood is a very good idea if the circumstances are suitable.

Secondly, on the tipping gate: I was involved in the design of Fontburn gates. We used a semi-circular bottom in the gate and were somewhat concerned that it might choke with silt. I note the present designers used a square edge which on balance solves that problem rather well and is probably better. However, when considering another tipping gate at Barnoldswick we found that there had been no problem at Fontburn so we adopted a semi-circular base there as well. Each gate was tested by cofferdamming which was a tricky exercise.

C TUXFORD (Ardon International)

Did I understand Mr Bramley to say that reinforced grass systems are of a short term nature and did I also understand him to say by implication that the Russian wedge shape blocks have superior performance to any block system being currently used in this country? On the American tests you commented that the reinforced grass systems were unsuitable with sand underlayers. As far as I am aware the American tests showed no preference for impermeable or permeable underlayers if precautions were taken.

M E BRAMLEY (CIRIA)

Firstly the concept of reinforced grass is essentially short duration and is separate from the concept of cable tied blocks without grass. Secondly I did not necessarily imply that the Russian blocks are better. The Russian concept is capable of being expanded up to unit discharges of 100 cubic metres/second/metre. They have been tested up to 50 cubic metres/second/metre in prototype situations.

With regard to the American trials of blocks without grass there were indications that the underlayer concept can be impermeable or permeable and I don't think any conclusions have been made yet.

R W HEMPHILL (Lewin & Fryer)

My own experience of tipping gates is that they have been subject to vandalism. Has the possibility been considered that some of our modern vandals may tip the gates down?

DR A K HUGHES (North West Water)

We have no incidents of people trying to push the gates open. We have had stones thrown into the plunge pool downstream of them. The gates weight 7.6 tonnes each and are inclined at 65 degrees to the horizontal. The HSE did suggest we put signs on the downstream side to warn people they are about to tip over!

W J CARLYLE (Binnie & Partners)

I would like to point out that you get little benefit from increasing spillway capacity but by increasing freeboard you get benefit at all levels of flood by virtue of the attenuation gained. Thus increasing freeboard should always be considered as a means, if possible, of dealing with flood deficiency.

WRITTEN CONTRIBUTIONS

M F KENNARD (Rofe, Kennard & Lapworth)

In the paper by Messrs Johnson and Jackson (4.1) reference is made to a stability analysis, with pessimistic phreatic surface assumptions and C' = 0, giving a factor of safety of 1.37, and as this is less that 'the commonly accepted minimum for normal operating conditions of 1.5' works to improve the factor of safety were carried out. Where an embankment has been standing for 200 years, and hence the fill and foundation have been fully tested, the use of a factor of safety of 1.5, which would be used for a new embankment dam, appears to be high. It would be of interest to know of precedents for this, and the author's reasons for the adoption of a high factor when actual in-situ soil properties and pore pressures are known.

E A JACKSON (Sir Alexander Gibb & Partners)

In a paper under the heading of overflow extensions matters relating to embankment stability were of necessity treated only briefly. The question permits this aspect of the design to be enlarged upon.

The fact that the embankment had been standing for some 200 years without showing signs of distress was not overlooked, and if other major works had not been required the decision as to whether the comparatively minor stabilizing works should be constructed would have been much more difficult to make. As it was, the work formed a logical extension of the other much larger work on the downstream side of the embankment, at a cost which amounted to 1.6% of the total bill for remedial works.

In view of the increased design lake level arising out of the uprating of the spillway, the increased load due to the raising of the embankment crest, and the serious consequences of failure due to the location of the dam, it was considered that this small expenditure involved in providing a modest improvement in stability was fully justified.

On a more general level, however, the authors would like to question the statement which implies that on the basis of information obtained from ground investigations it can be stated that 'soil properties and pore pressures are known', and that lower factors of safety can be accepted than would be the case for a new dam. The authors suggest that in the absence of any construction records, and with what appears to be a rather variable fill material, foundation conditions and soil properties are in fact known with considerably less confidence than would be the case with a new dam. Similarly, pore pressure is only known at the points where it is measured, and accepting the answers assumes that the rather disruptive process of installing the measuring device has not significantly modified the pressure measured.

Such considerations could be used to argue for the use of a higher factor of safety (or factor of uncertainty) than the value of 1.5 quoted. In general, the authors would be uneasy about accepting a factor of safety of less than 1.5 for a dam in similar circumstances, and share the view stated in a recent IWEM Paper 'The Reservoir Safety Programme in Northern Ireland' by G A Cooper, August 1987, that remedial work should be considered if the factor of safety is less than 1.5, or if the embankment has deformed, has shown significant seepage, or has a high phreatic surface.

DR A K HUGHES (North West Water)

I fully endorse Mr Evans' comments about spillway capacity. I am of the opinion that damage on very rare design events such as PMF can be allowed to occur, and moreover will occur as long as the damage can be limited. It is not always necessary for a spillway channel to have design capacity for all of its length, and we must always keep in mind the rarity of these events. This means we must have due regard to less expensive methods of providing spillway capacity, protecting and armouring fills, and I also believe under controlled conditions limited overtopping can be allowed.

PROCEEDINGS: TECHNICAL SESSION 5

INSTRUMENTATION AND DRAINAGE OF EMBANKMENTS

PAPERS:

DISCUSSION:

Index 5 (cont)

DETECTION AND INVESTIGATION OF PROBLEMS AT GORPLEY AND RAMSDEN DAMS

P Tedd (Building Research Establishment)
J R Claydon (Yorkshire Water)
J A Charles (Building Research Establishment)

SYNOPSIS

The presence of wet areas on the downstream slope of Gorpley dam and signs of significant settlement at Ramsden dam prompted investigations into the possibility of internal erosion of the puddle clay core occurring at these two embankment dams. From measurements of horizontal earth pressure and pore pressure made in the cores no evidence was found of hydraulic fracture occurring. Examination of the filter properties of the fills immediately downstream of the cores suggests that they would halt any erosion of the core at both dams. Investigations at Gorpley dam have shown that the wet areas are related to rainfall. The continuing settlement at Ramsden dam is related to reservoir fluctuations and is still being investigated.

INTRODUCTION

1. When old embankment dams are inspected two conditions are quite commonly detected that may give cause for concern. Wet areas are found on the downstream slope and the crest and upper parts of the embankment show signs of significant settlement. There are several different possible causes of these two conditions. It is of particular concern that either of these conditions could indicate leakage through the core and associated internal erosion. This situation could also result in high pore water pressures in the downstream fill leading to a reduction in slope stability.

2. Investigations have been carried out to determine the causes of these common conditions at Gorpley and Ramsden dams. Both reservoirs are owned and operated by the Western Division of Yorkshire Water. Gorpley reservoir is an isolated reservoir supplying drinking water to Todmorden in the upper Calder valley. Ramsden reservoir is one of a chain of four in the Holme valley supplying water to Devsbury.

GORPLEY DAM

3. Gorpley dam was built between 1900 and 1904, the engineer being G F Deacon. The construction of the embankment is illustrated in Fig 1 and is described in detail by Parkin[1]. The site of the dam was in a narrow part of the valley where the stream went over a 7m high waterfall. The maximum height of the embankment from the head of the waterfall is 27m. The upstream slope is 1 in 3 and the downstream slope for the upper part 1 in 2.5. At the deepest section in the steep sided valley, dry stone rubble was placed on the downstream side with the outer slope being 1 in 1 and faced with stone. Parkin[1] states " there is much in favour in this method of construction with a large mass of rubble being placed in the lower usually narrow part in and about the stream bed. It reduces the risk of slipping of the earthwork above and it acts as a drain to the lower part of

the embankment". The fill was placed in layers of 0.23m at a downward slope of about 1 in 10 towards the puddle core with each layer being rammed. More clayey material was used next to the core. The clay core is 1.8m wide at the crest and increases in width with depth with both faces having batters of 12 in 1.

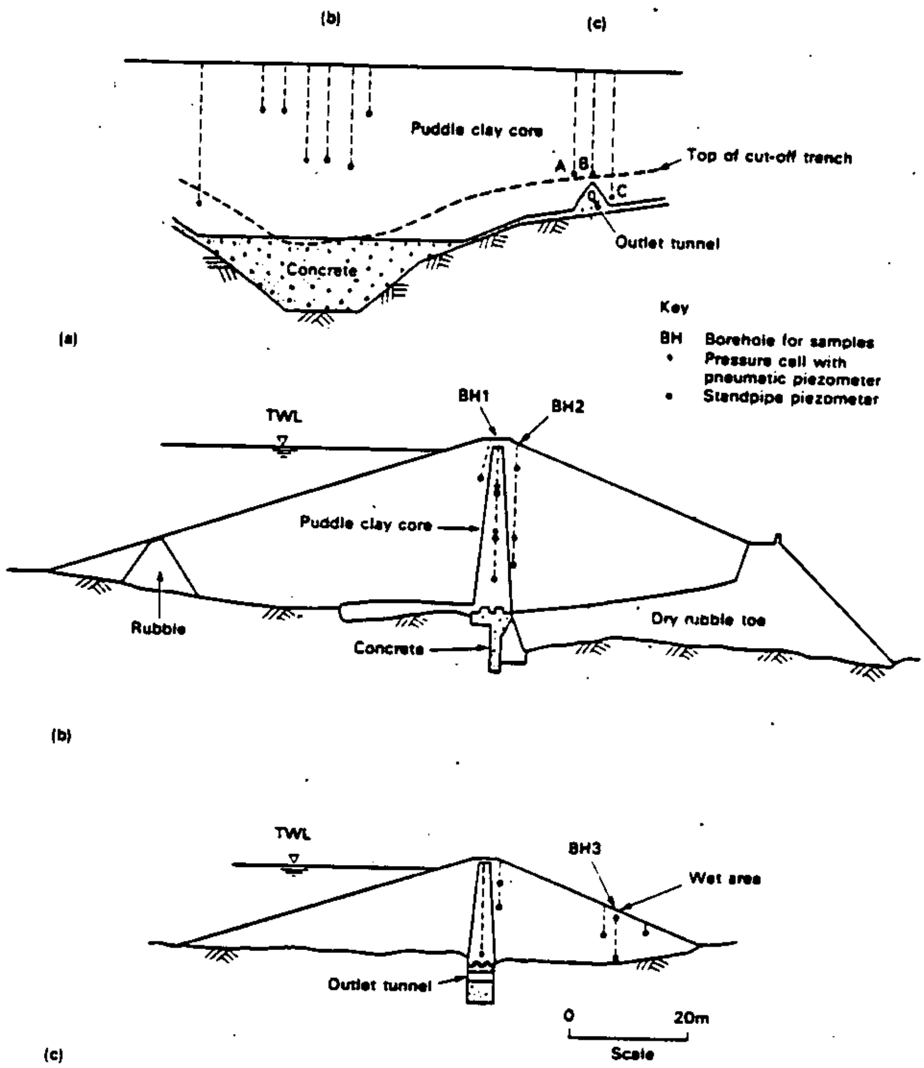


Figure 1: Gorpley dam: (a) longitudinal section; (b) transverse section at centre of dam; (c) transverse section at outlet tunnel

4. The cut-off trench was free from water during its construction except at the lowest part below the stream bed. The central deepest part of the cut-off trench below the stream bed and adjacent to the waterfall was filled with concrete. On either side of the central section the cut-off trench was taken down to a good watertight shale wherever practicable. Except for a short length towards the ends of the trench a layer of concrete having one or two projecting keys was placed at the bottom of the trench and on this the puddle clay core was founded. The maximum depth of the clay filled trench was approximately 7m.

5. The outlet tunnel was constructed of concrete in an open cutting passing through beds of strong shale with some bands of hard grit. Where it passes through the cut-off trench, the concrete keys at the base of the trench were continued across the top of the tunnel. The depth of the cut-off trench here is approximately 6m.

6. When Gorpley dam was inspected under the Reservoirs (Safety Provisions) Act in 1983 the Inspecting Engineer recommended inter alia:

- the construction of a low wave wall
- replacement of the valves in the draw-off tower
- observation and investigation of the wet areas on the downstream face

7. The wave wall has been completed. In replacing the draw-off valves, the tower has been converted from a permanently flooded design to a dry shaft with ventilation through the draw-off tunnel. Investigation of the embankment was carried out by the Building Research Establishment (BRE) as part of their research programme into the behaviour of old earth dams.

RAMSDEN DAM

8. Ramsden dam was constructed between 1879 and 1883 for Batley Corporation Waterworks and the engineer was G H Hill. The reservoir did not come into service until 1892[2] because the cut-off had to be extended to prevent water escaping round the eastern end of the embankment trench. The maximum height of the dam is about 25m. The upstream slope is 1 in 3 and the

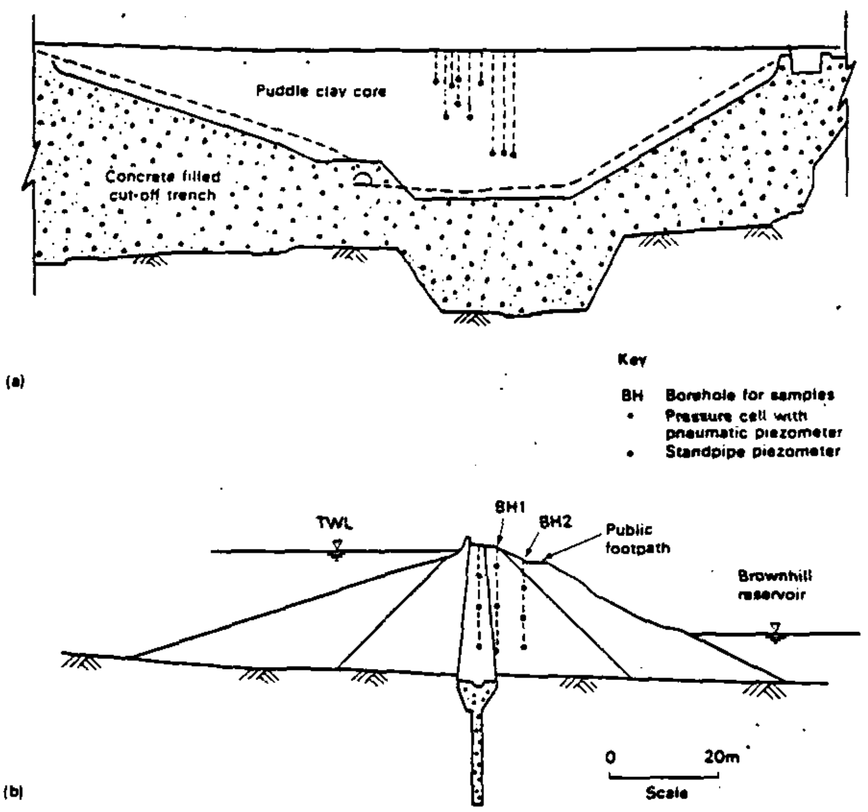


Figure 2: Ramsden dam; (a) longitudinal section; (b) transverse section

original downstream slope was constructed at 1 in 2, but during the construction of Brownhill dam in 1928 the upper part of the downstream slope was steepened to accommodate a public footpath, see Fig 2. The original construction drawings indicate that selected fill was placed either side of the core. The puddle core is founded on a concrete filled cut-off trench that has a maximum depth of 22m in the centre of the dam. The clay core is indicated to be 3m wide at the crest and increases in width with depth with both faces having batters of 12 in 1.

9. Ramsden dam was last inspected under the Reservoirs (Safety Provisions) Act in 1985. At the same time as the inspection, hydrological studies were being carried out on all four reservoirs in the valley. It was concluded that improvements to overflow capacity were required at all four dams. Consideration of spillway geometry and freeboard led to the conclusion that the most economic solution would be to enlarge the spillway at the two uppermost dams, Yateholme and Riding Wood, and to construct substantial water retaining crest walls on the lowest two, Ramsden and Brownhill. It was not possible for operational reasons to carry out work at all four reservoirs at the same time. The crest wall at Brownhill was completed in 1986. The enlarged overflows at Riding Wood and Yateholme were completed in 1987.

10. It was originally intended to construct the Ramsden wave wall in 1987, but this was deferred following consideration of the detailed design with the Panel Engineer. Due allowance could have been made for the present rate of settlement, but uncertainty over the cause of the settlement raised the possibility that adding a substantial wall could affect the stability of the embankment. It was decided to delay construction pending investigation of the embankment, which is being carried out by BRE on contract to Yorkshire Water.

INVESTIGATIONS

Wet areas at Gorpley dam

11. At Gorpley dam wet areas on the downstream slope of the dam have been known to exist since 1969. One wet area was particularly boggy and very localised being only about 0.6m across. Immediately above this area, there is a larger wet area where reeds are growing and some very minor surface slipping has occurred. The wet areas could indicate leakage through the core which could cause internal erosion. Leakage through the core may also cause high pore water pressures in the downstream fill leading to a reduction in slope stability. Various methods have been used to determine the source of the water causing the wet areas.

(a) Temperature measurement
12. Infra-red thermography and temperature measurement were used as a method of locating wet areas and identifying the source of the water[3]. The method depends on the wet areas being at a different surface temperature to the surrounding ground. Two infra-red surveys were carried out at Gorpley, one in the summer and one in the winter. The wet areas were clearly identified using infra-red thermography by being colder in summer and warmer in winter than the surrounding ground.

13. To investigate the source of the water, subsurface temperatures within the embankment and reservoir temperatures have been measured on several

occasions. Two sets of measurements are shown in Table 1. The measurements within the embankment were made by lowering a thermocouple down the access tube of standpipe piezometers in the core and the downstream fill. The reservoir temperature was measured close to the valve tower down to a depth of 12m.

Table 1: Temperature measurements at Gorpley dam

Location	Temperature °C	
	Winter	Summer
Reservoir	2.0	13.5
Clay core at 14m	8.0	7.9
Downstream fill at 8m depth	8.1	7.0
Downstream fill at 1.8m depth	5.0	10.5
Water at wet area	2.5	12.0
Ground 10cm deep away from wet area	2.0	13.0
Air	1.0	12.5

14. As the temperature of the water emerging from the wet area was similar to that of the near ground surface it is possible that the water is from rainfall on the downstream slope of the dam. Had the water emerging at the wet patch come through the embankment then its initial low temperature of 2°C in winter would soon reach the temperature of the dam, 8°C. The temperature of the emerging water would depend upon the flow rate, route of flow and the thermal gradient near the surface of the dam amongst other factors. If water were coming directly through the dam by the shortest route then the temperature of the emerging water would probably be larger than the 2.5°C actually measured. Alternatively the water could be coming through the dam at a higher level than the wet area and then flowing close to the surface of the dam before emerging. The temperature of the water would then be close to that of the near ground surface. The measurements in Table 1 cannot be considered to be conclusive in identifying the source of the water.

(b) Chemical analysis of soluble ions

15. Samples of water from the wet area, the reservoir, the river above the reservoir and a spring close to the dam were analysed for quantities of soluble ions. Filtered samples of the waters were analysed for anions by ion chromatography and for cations by flame emission or atomic absorption spectrophotometry. It was hoped that the sample from the wet area could be matched to the reservoir water and that the spring and river water would be sufficiently different. Unfortunately there were insufficient differences in the samples to draw firm conclusions.

(c) Flow rate, rainfall and reservoir level measurements.

16. Excavation 0.3m below the surface at the wet areas revealed water flowing through the ground. At two locations this has been channelled into pipes and the rate of flow measured by timing how long it took to fill a 500ml container. Measurements of flow made at weekly intervals are presented in Fig 3 together with weekly rainfall records and reservoir levels for a 6 month period. There is a clear correlation between rainfall

and flow rate from the wet patches which suggests that the water from the wet patches has come from rainfall on the downstream side of the dam and not leakage through the dam. Where flow rate observations have been taken on a number of consecutive days it was found that there was a rapid increase in flow rate following heavy rain with a gradual decrease if no further rain fell. It seems unlikely that the reservoir level is influencing the flow rates as many of the rapid changes have occurred when the reservoir was full.

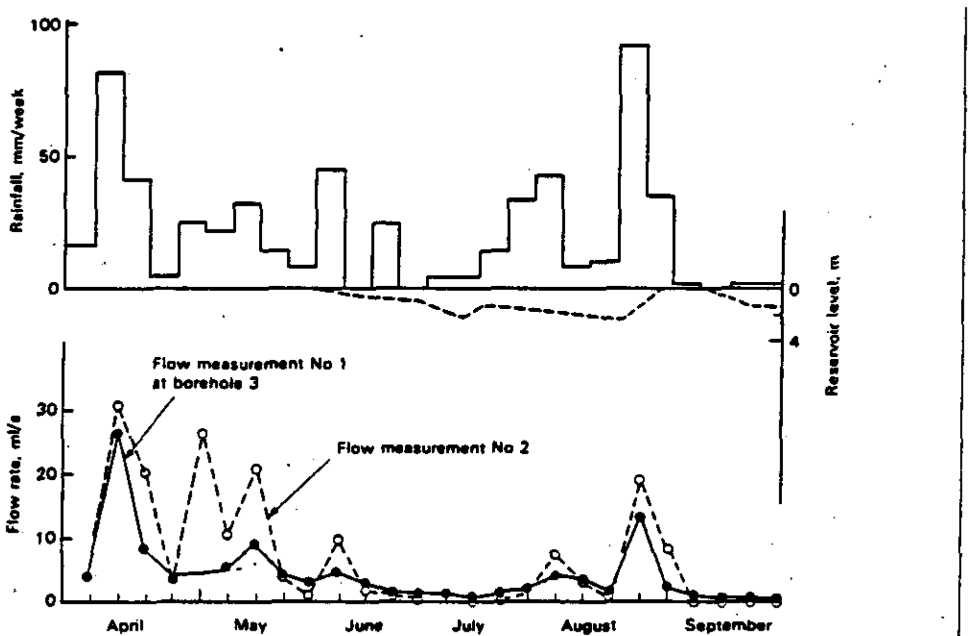


Figure 3: Records of rainfall, reservoir level and flow rate from wet areas at Gorpley dam

17. The water collected from the wet areas was very clear showing no sign of any eroded material.

Settlement at Ramsden dam

18. Gorpley dam shows little evidence of significant settlement having occurred. There is a slight general depression in the upstream pitching between the spillway and the valve tower. In contrast the embankment at Ramsden dam shows obvious signs that considerable settlement has occurred. The wave wall has deformed and tilted significantly. Examination of the present geometry, and construction and remedial work drawings give an indication of the magnitude of the settlement that has occurred since construction. The upper part of the upstream slope is now approximately 1 in 3.3 which is flatter than the 1 in 3 indicated on the construction drawings, see Fig 2. A survey of the present upstream slope indicates a total settlement since construction of approximately 0.6m near the crest assuming the upstream slope was originally built at 1 in 3.

19. Settlement and possibly some downstream movement are also indicated by the significant curvature of the interface between the reservoir water and upstream face of the dam as seen in plan. Assuming this interface would have been a straight line shortly after first impounding, the maximum horizontal offset from a line of the present interface is 2.5 metres. If

this horizontal offset were entirely due to settlement, it would amount to approximately 0.8m of settlement in 104 years, assuming an initial upstream slope of 1 in 3 and that the pitching had not been built up in this period. However as it was common practice to over build embankments by a proportion of their depth to allow for future settlement the actual settlement over the 104 years may be greater than that estimated above.

20. Drawings of the restoration of the embankment in 1927 show that a new wave wall was constructed and that the embankment was raised to 236.52m AOD giving a freeboard of 1.52m. The top of the wave wall was 1.2m above crest level. In October 1949 the embankment crest was raised to a uniform level of 236.75m AOD, but there is no evidence of any alterations to the wave wall. The minimum height of the present wave wall above the crest is 0.75m which indicates the wave wall settled 0.22m between 1927 and 1949 giving an average rate of 10mm per year. This assumes the crest was not built up between 1949 and 1986. In August 1986 the lowest point on the crest was approximately 236.32m AOD giving a total settlement of 0.43 m in 37 years and an average rate of 13mm per year between 1949 and 1986.

21. Since 1977 precise surveying of the level and horizontal alignment of stations on the crest have shown a continuing and varying rate of settlement and downstream movement. The maximum settlement has occurred at the central section of the dam that has the greatest depth. The average rate of settlement between 1977 and 1985 is approximately 8mm per year and the average rate of downstream movement is 3mm per year.

22. Since construction the records and observations indicate that the crest of Ramsden dam has settled somewhere between 8 and 13mm per year. Settlement of a dam may be attributable to a number of processes. These include:

a. Primary consolidation of the puddle clay
b. Volume reduction of the upstream fill on reservoir first filling
c. Secondary compression of puddle clay and shoulder fill
d. Stress changes due to fluctuations in reservoir level
e. Internal erosion
f. Slope stability

23. Settlement due to causes (a) and (b) should be completed during the early years of the dam's life. To determine whether settlements measured many years after the completion of a dam can be attributed to secondary compression of puddle clay core and creep of the shoulder fill or erosion or slope stability, Charles[4] has proposed the following settlement index (S_I).

$$S_I = \frac{s}{1000\ H \log(t_2/t_1)} \qquad (1)$$

where s is the crest settlement measured in mm between times t_1 and t_2 since the completion of the embankment at a section of the dam H metres high. Where values of S_I are greater than 0.02 it is suggested that some mechanism other than creep may be causing the settlement and the situation should be seriously examined.

24. Settlement measurements at Ramsden dam[5] give a settlement index of 0.077 between 1977 and 1985, and 0.12 between 1983 and 1985. The measurements have shown that settlement and downstream movement are

affected by reservoir level. Approximately 10mm of settlement occurred when Ramsden reservoir level was drawn down by 7m, but not all of this was recovered when the reservoir was refilled. Measurements to investigate the effect of reservoir level in Ramsden and Brownhill and the rate of drawdown are continuing. To investigate the depth at which the movements are occurring magnet settlement gauges and inclinometers were installed in the core of the dam during 1987. Since taking the zero readings on these instruments in September 1987 the reservoir has remained full and crest movements have amounted to less than 1mm. It is intended to lower Ramsden and Brownhill reservoirs during 1988 for remedial work to the Ramsden valves and movements will be closely monitored during this drawdown. The possibility of (e) internal erosion and (f) slope stability contributing to the settlement are examined in the following sections.

INTERNAL EROSION

25. The possibility of internal erosion has been investigated at both dams. Hydraulic fracture of the clay core by the reservoir water pressure may be a mechanism that leads to leakage through the core and to its subsequent erosion. Measurements of horizontal earth pressure have been made in the clay cores of both dams to assess their susceptibility to hydraulic fracture. If leakage through the core started to erode the clay, a serious situation could still be avoided by an effective filter downstream of the core. The filter properties of the fills immediately downstream of the cores have been investigated to assess their ability to halt any erosion of the core.

Susceptibility of the clay cores to hydraulic fracture

26. The susceptibility of the clay cores to hydraulic fracture may be assessed by measuring the horizontal earth pressures[6]. Measurements of total horizontal stress and pore water pressures using push-in spade-shaped pressure cells with associated pneumatic piezometers have been measured at both dams[7]. At Ramsden dam measurements of horizontal stress at 12 m and 18 m below crest level were considerably larger than the reservoir head. However at 6m depth, the horizontal earth pressures were only marginally above the reservoir head as shown in Fig 4b.

27. At Gorpley dam measurements were made at the central section of the dam and adjacent to the outlet tunnel as shown in Fig 1. The measurement at position C is about 0.8m horizontally away from the outlet tunnel and 3m below the top of the 5.5m deep cut-off trench. Examination of the measurements in Fig 4a shows the total horizontal stresses in the main body of the core to be larger than the reservoir head, but at location C close to the outlet tunnel the stress is significantly less than the reservoir head, thus indicating a susceptibility to hydraulic fracture. However measurements of pore water pressure at position C were less than the total horizontal stress indicating that there was a positive effective stress in the clay and therefore hydraulic fracture was not occurring at the particular point at which the measurements were made.

28. The total horizontal stresses measured at 14m and 21m in the main body of the core at Gorpley were significantly smaller than those measured at Ramsden at comparable depths. These low stresses at Gorpley are probably due to the relatively thin core and hence the greater reduction in stress due to arching between the fill and the core as the core consolidated[6].

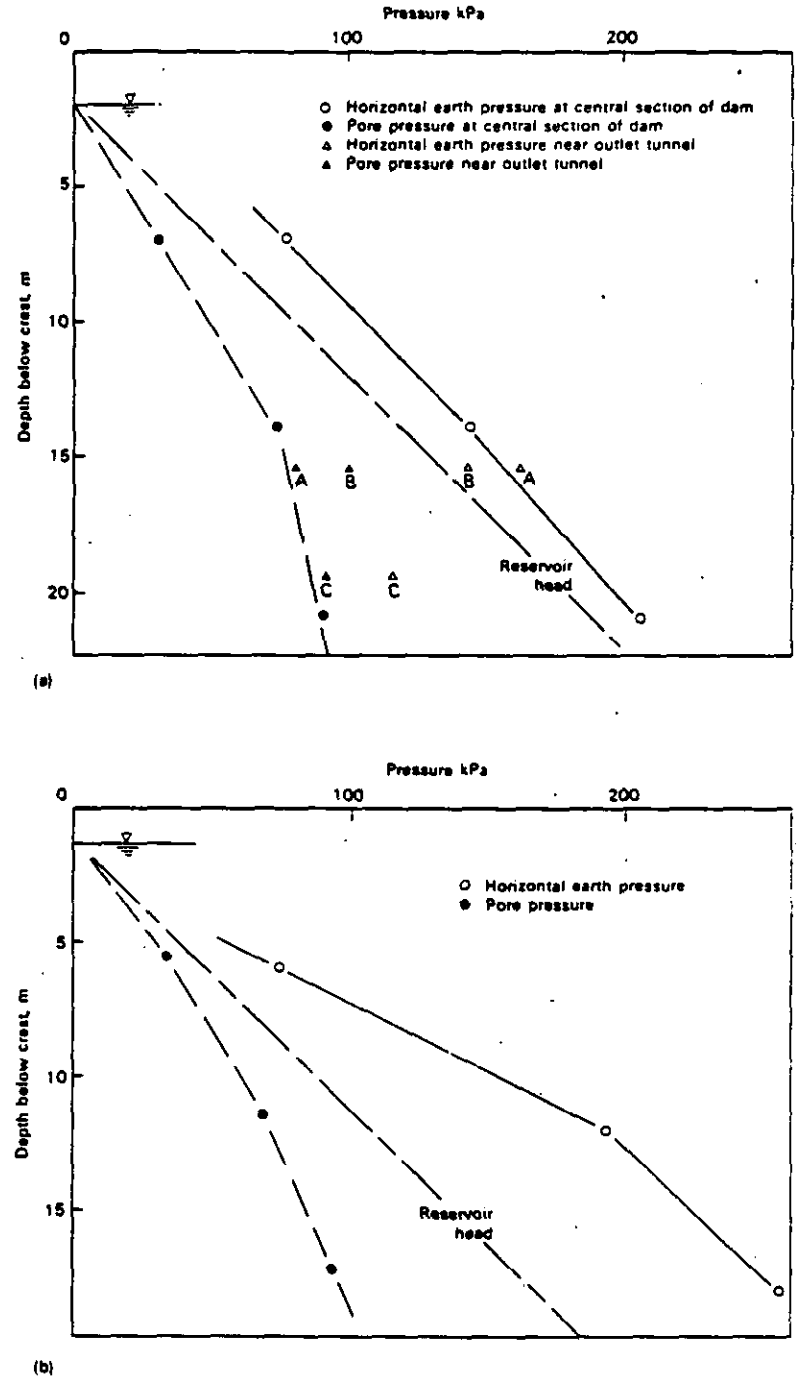


Figure 4: Pressure measurements on centre-line of the clay cores: (a) Gorpley; (b) Ramsden dam

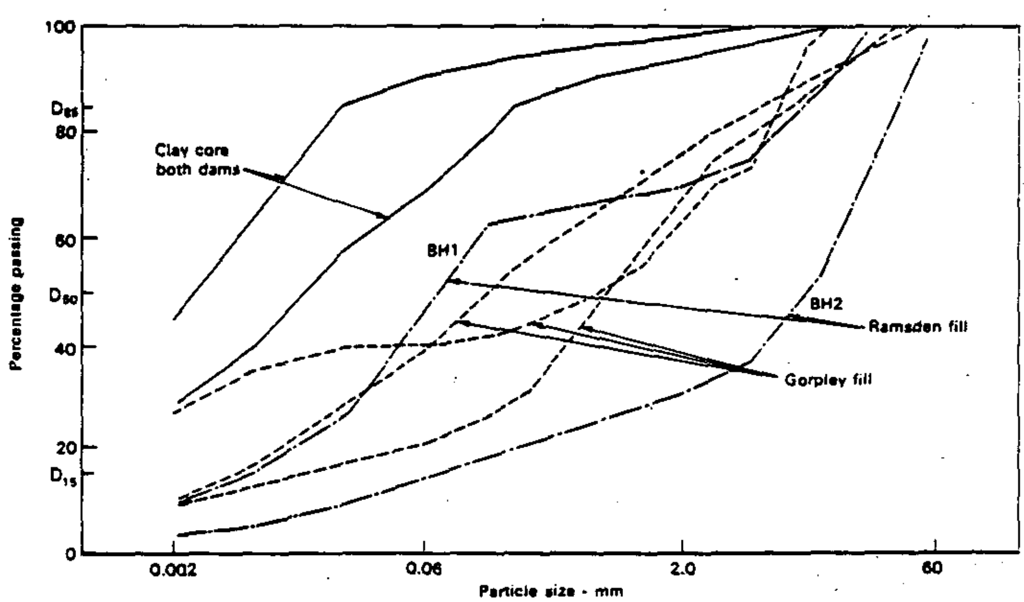
Filter properties of downstream fill

29. Although the fill immediately downstream of the clay core was not designed to act as a filter in old embankment dams, it became good practice to place cohesive selected fill immediately upstream and downstream of the core. Records and construction drawings indicate this was done at both Gorpley and Ramsden dams.

30. An investigation of the filter properties of the fill immediately downstream of the clay core has been carried out at Gorpley and Ramsden dams. Two approaches have been investigated. Firstly the particle size distributions for the puddle clay and downstream fill have been examined to see if they conform to empirical filter design rules for non-cohesive soils. Secondly the permeability of the downstream fill has been measured as a means of quantifying the effectiveness of the fill as a filter to prevent erosion.

(a) Particle size rules

31. Figure 5 shows a range of particle size distributions for the clay core and downstream fill for both Gorpley and Ramsden. Table 2 summarises the empirical filter rules[8], (the maximum recommended particle size ratio) that were used in this investigation. At Gorpley dam the samples came from two boreholes 19m deep, one in the centre of the clay core and the other in the fill 2.8m downstream of the first borehole. Eighteen samples were graded from each borehole. At Ramsden dam samples from the centre of the core have been compared with samples from boreholes 3.6m and 8.5m downstream of the core. Both boreholes downstream of the core were 19m deep with over 20 samples being graded from each borehole. Examination of the particle size ratios in Table 1 for both dams shows the $(D_{15})_f/(D_{85})_c$ rule is more than adequately met, but the $(D_{50})_f/(D_{50})_c$ rule is not. It was not possible to examine the $(D_{15})_f/(D_{15})_c$ ratio since the $(D_{15})_c$ is smaller than the minimum size determined in the grading analysis. From laboratory studies, Sherard et al[9] concluded that for silts and clays, filter criteria using the ratios $(D_{15})_f/(D_{15})_c$ and $(D_{50})_f/(D_{50})_c$ are not meaningful. They also concluded that for for fine grained clays, with $(D_{85})_c$ equal to 0.03 - 0.10mm, a medium sand filter with $(D_{15})_f$ equal to 0.5mm is conservative.



Clay	F	M	C	F	M	C	F	M	C	Cobbles
	Silt			Sand			Gravel			

Figure 5: Range of particle size distributions for the clay core and downstream fill from Gorpley and Ramsden dams

Table 2: Measured particle size ratios in relation to filter design rules

Particle size ratio	Maximum recommended	Gorpley dam		Ramsden dam			
		2.8m from core		3.6m from core		8.5m from core	
		Highest	Mean	Highest	Mean	Highest	Mean
$\frac{(D_{15})_f}{(D_{85})_c}$	5	0.55	0.10	0.20	0.03	2.6	0.036
$\frac{(D_{50})_f}{(D_{50})_c}$	25	321	57.6	550	12.5	6,500	218
$\frac{(D_{15})_f}{(D_{15})_c}$	20	-	-	-	-	-	-

where $(D_{15})_f$ is the 15% size for the filter (downstream fill) and $(D_{85})_c$ is the 85% size for the puddle clay and similarly for $(D_{50})_f$ and $(D_{50})_c$.

(b) Permeability

32. Following the investigations into the hydraulic fracture of the clay core at Balderhead dam and its subsequent erosion and failure of the downstream filter, Vaughan and Soares[10] concluded that filter permeability might provide a better basis for quantifying the effectiveness of a filter than a particle size ratio. Filtering of eroded material is dependent on the pore sizes in the filter material and permeability is a direct function of pore size.

33. From a series of laboratory filter experiments, Vaughan and Soares[10] obtained the following empirical relationship between k, the coefficient of permeability in m/s of the filter and δ, the average diameter in μm of an eroded particle, that will just pass through that filter. Above this size clogging will occur.

$$k = 6.7 \times 10^{-6} \times \delta^{1.52} \qquad (2)$$

Using this relationship and the permeability measurements made in standpipe piezometers in the downstream fill at both dams, the sizes of particle that will just pass through the fill material have been calculated and are presented in Table 3. At Gorpley dam the values of δ appear very small. The largest value measured was in a piezometer installed beneath the very wet area 18m away from the core, but even here the size is equivalent to a small silt size. Values of δ close to the core at Ramsden dam were generally higher than at Gorpley. The large values of δ or permeability tended to be where water had been encountered during drilling.

34. If erosion of the core were taking place there is some uncertainty as to the size of particle that could be eroded. If clay size particles, less than 2μm, were being eroded then according to equation (2) they would pass through the fill locally around the piezometer 11.75m below crest level and 3.6 m from the core centre at Ramsden dam and erosion would not be halted.

Table 3: Filter properties of downstream fill as function of permeability

Depth of piezometer below crest, m	Distance downstream from core, m	Measured permeability, m/s	Particle size, δ, μm
Gorpley dam			
20.2	2.8	4.4×10^{-7}	0.166
15.5	2.8	1.2×10^{-9}	0.003
4.3	2.8	1.5×10^{-8}	0.018
3.8	2.8	5.3×10^{-9}	0.009
6.8	2.8	2.4×10^{-9}	0.110
12.0	17.3	4.1×10^{-10}	0.002
9.6	18.8	2.5×10^{-5}	2.380
15.8	18.8	1.2×10^{-6}	0.320
11.9	23.1	1.9×10^{-6}	0.420
Ramsden dam			
4.45	3.6	2.6×10^{-6}	0.536
11.75	3.6	2.0×10^{-5}	2.053
20.70	3.6	7.4×10^{-8}	0.051
8.20	8.5	1.8×10^{-5}	1.916
13.40	8.5	8.1×10^{-6}	1.133
20.60	8.5	1.0×10^{-6}	0.286

Note: particle size has been calculated from equation (2) and represents the maximum size that would pass through a soil of the measured permeability

However Vaughan and Soares[10] suggest that flocculating conditions are likely to exist and that the smallest size of eroded material would not be less than 6μm. If 6μm were the smallest floc size that could be eroded then the downstream fill would only need a permeability less than 1.02×10^{-4}m/s to filter the eroded material. This is equivalent to a medium sand and is in broad agreement with the conclusion of Sherard et al[9] stated earlier. The maximum permeability that can be measured using standpipe piezometers depends upon the permeability of the porous element and of the surrounding sand cell. To investigate the effect of the piezometer permeability on the in-situ permeability measurements made in the dams a series of laboratory experiments was carried out to determine the permeability of the piezometers. The maximum permeability measured in the fill close to the core at either dam was about 2×10^{-5}m/s. The laboratory investigations indicated that measured values of k below 2×10^{-5}m/s in the dams would not have been significantly affected by the piezometer permeability, however measured permeabilities much above this value simply depend on the piezometer permeability. The maximum reliable permeability that can be measured in the piezometer is therefore less than the permeability that would indicate an unsatisfactory filter, ($k = 1.0 \times 10^{-4}$m/s) using the above method. However the method can be and has been used to show that the downstream fill is satisfactory as a filter to halt erosion.

SLOPE STABILITY

35. There is no evidence to indicate that problems with embankment stability have occurred at either of the dams, during or subsequent to construction except for the very minor surface slipping at Gorpley. It seems it is unlikely that either embankment will have stability problems having survived the most severe conditions in terms of construction pore pressures, first filling and normal drawdown rates, unless caused by some exceptional circumstances such as high pore pressures due to leakage through or around the core or much faster drawdown. Measured pore pressures are very small in both dams (see Table 4) giving values of r_u (ratio of pore pressure to overburden pressure) of 0.1 at Gorpley and 0.08 at Ramsden. However the lower part of Ramsden dam is submerged by Brownhill reservoir and this is reflected in the pore pressure measurements in the deepest piezometers.

Table 4: Downstream fill pore pressures and shear strength parameters

	Depth below surface m	Piezometric head m	r_u	Depth below surface m	ϕ'
Gorpley dam					
Borehole 2	3.75	0.47	0.06	4.70	33.8
2.8m from	15.0	1.80	0.06	8.5	35.0
core centre	-	-	-	9.2	32.9
	19.3	2.40	0.06	19.3	32.5
Borehole 3	1.8	0.36	0.1	1.4	37.3
18.8m from	-	-	-	2.0	34.5
core centre	8.0	0.30	0.02	8.0	34.3
Ramsden dam					
Borehole 1	3.80	0.67	0.08	6.3	32.4
3.6m from	11.10	0.55	0.02	8.7	33.8
core centre	20.05	2.26	0.05	11.2	27.5
	-	-	-	19.8	36.7
Borehole 2	6.4	0	0	6.5	38.3
8.5m from	10.6	0.70	0.03	9.0	40.2
core centre	17.8	2.18	0.06	11.7	37.3
	-	-	-	18.0	35.4

36. At both dams the angle of shearing resistance, ϕ', has been measured by triaxial consolidated undrained tests with pore pressure measurement on U100 samples taken from the downstream fill. The confining pressure in each test was equivalent to the overburden pressure. Values of ϕ' for each test are shown in Table 4. Generally, values of ϕ' are larger further away from the core indicating the use of coarser material away from the core. Combining the results from each borehole gives small values of c' between 6

and 11 kPa and ϕ' of 31° at Gorpley and 34° at Ramsden. Shear strength parameters for the dry rubble toe at Ramsden are not available. Material from both dams was very variable and this makes calculation of reliable factors of safety very difficult.

CONCLUSIONS

37. The investigations at Gorpley dam have shown that the wet areas are related to rainfall. Correlation of flow measurements from wet areas with rainfall and reservoir level was the most conclusive method of determining the source of the water.

38. Low earth pressures measured in the clay core adjacent to the outlet tunnel in Gorpley dam indicated a susceptibility to hydraulic fracture, however a positive effective stress measured in the clay showed that hydraulic fracture was not occurring at the particular point at which the measurements were made.

39. At both dams it was shown that if the core were eroding, the fill immediately downstream of the core should filter any eroded material. Use of the permeability criterion(10) to determine the performance of the downstream fill as a filter to halt erosion can demonstrate that it is satisfactory when using the permeability measurements that are made in typical Casagrande standpipe piezometers. However the permeability of the piezometer limits the maximum permeability that can be measured and this must be taken into account in interpreting the results.

40. Records indicate that the crest of Ramsden dam is settling at an average rate of approximately 8mm per year. Examination of the possible causes of settlement and levelling records indicates that the settlement is related to changes in reservoir level.

41. The investigations at Gorpley dam have not indicated a need for any remedial work to the embankment. Work is continuing to investigate the depth at which the movements at Ramsden dam are occurring and the reason for their occurrence.

ACKNOWLEDGEMENTS

42. The work described in this paper forms part of the research programme of the Building Research Establishment and is published by permission of the Director. The main customer for the work is the Water Directorate of the Department of the Environment. The assistance of I R Holton in the work is gratefully acknowledged. The assistance of the General Manager of Yorkshire Water Western Division, Mr J.R. Layfield, in permitting use of the dams for research purposes and encouraging publication is gratefully acknowledged.

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MATTERS CONCERNING DRAINAGE OF EXISTING DAMS

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SYNOPSIS

The use of filters and drains is an integral part of the present-day practice of good embankment dam design. The same principles need to be applied to the design of improvement works, involving drainage, to existing dams.

Various general matters regarding filters, drainage and geotextiles are briefly referred to, in order to stimulate discussion on the subject. Four cases of remedial works are briefly described.

INTRODUCTION

1. "It is sometimes thought that if seepage control measures of some kind are installed in dams seepage surely will be controlled. The actual success of any seepage control system depends on how well it conforms to actual existing conditions. Measures that attempt to control seepage by keeping water out depend for their success on a high degree of perfection. Slight lapses in blankets, cutoffs, or grout curtains can drastically reduce their effectiveness. Unknown porous joints or strata of high permeability can allow water to bypass cutoffs. Likewise nonuniformities and lapses in embankment construction can reduce the effectiveness of certain kinds of drains. Insufficient permeability in drains can also reduce their effectiveness.

2. Success in drainage control in dams depends on designing and building systems capable of coping with conditions as they really exist. Adequate exploration and testing programmes, rational design methods, and the observation of completed works are the cornerstones of successful engineering projects."

3. Cedergren in 1967[1] summarises the importance and effectiveness of drainage control measures for embankment dams in the above extracts from "Seepage, drainage and flow nets". The fact that a substantial text book was published over 20 years ago on this important topic, and many relevant papers have been published since then, shows the importance of drainage, yet in some cases it does not receive adequate attention. In tables of possible improvement or remedial measures to existing embankment dams, drainage is not always included.

4. However, in many cases drainage is the simplest and most economical method of dealing with seepage problems, and with associated matters such as erosion, displacements and settlements, potential slips and defective road foundations.

Drainage

5. Cedergren [1] categorises the effects of uncontrolled seepage into two categories. These categories are:-

1 - "failures caused by migration of particles to free exits or into coarse openings"

and 2 - "failures caused by uncontrolled saturation and seepage forces".

6. Tables listing the failures under these categories are also given in the U.S.B.R. Design Standards for Embankment Dams[2] and are reproduced here.

Category 1
Failures caused by migration of particles
to free exits or into coarse openings

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1. Piping failures of dams and reservoirs caused by:

   a. Lack of filter protection
   b. Poor compaction along conduits,in foundation trenches, etc.
   c. Gopher holes, rotted roots, rotted wood, etc.
   d. Filters or drains with pores so large soil can wash through
   e. Open seams or joints in rocks in dam foundations or abutments
   f. Open-work gravel and other coarse strata in foundation and abutments
   g. Cracks in rigid drains, reservoir linings, dam cores, etc., caused by earth movements or other causes
   h. Miscellaneous manmade or natural imperfections

2. Clogging of coarse drains, including French drains

Category 2
Failures caused by uncontrolled
saturation and seepage forces

---

1. Most landslips, including those in highway or other cut slopes, reservoir slopes, etc
2. Deterioration and failure of roadbeds caused by insufficient structural drainage
3. Highway and other fill foundation failures caused by trapped ground water
4. Earth embankment and foundation failures caused by excess pore pressures
5. Retaining wall failures caused by unrelieved hydrostatic pressures
6. Canal linings, basement and spillway slabs uplifted by unrelieved pressures.
7. Drydock failures caused by unrelieved uplift pressures.
8. Dam and slope failures caused by excessive seepage forces or uplift pressures.
9. Most liquefaction failures of dams and slopes caused by earthquake shocks.

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7. It can be noted that drainage is referred to in these tables, and that where not mentioned, drainage can be the means of ensuring that the described modes of failure can be safely avoided.

8. Following from these considerations, the primary design requirements for filters and drains are well stated by Cedergren [1] as :

> "Many of the problems associated with the design of adequate filters and drains stem from the needs for satisfying two conflicting requirements.
>
> 1. Piping control requirements. The pore spaces in drains and filters that are in contact with erodible soils and rocks must be small enough to prevent particles from being washed in or through them.
>
> 2. Permeability requirements. The pore spaces in drains and filters must be large enough to impart sufficient permeability to permit seepage to escape freely and thus provide a high degree of control over seepage forces and hydrostatic pressures."

9. It is the reconciliation of the conflicting requirements that has often led to problems of drains in embankment design, construction and performance. For example, a drain if too coarse may permit adequate flow of water, yet permit passage of fine particles, whilst conversely a fine filter may have too low a mass permeability for the flow conditions. Combining the requirements for controlling piping and having sufficient permeability has led to multi-stage drainage systems and the inclusion of pipes. The pipes need to withstand the loading conditions especially in the base of a chimney drain, and cases are known where the pipes have failed.

10. Filter design criteria for systems where filtering is the prime need are based on the original work by Terzaghi and subsequently supplemented by laboratory tests and studies of others including US Corps of Engineers Waterways Experiment Station, USBR, Sherard, Vaughan, etc. The latter work of Vaughan & Soares in 1982[3] relates to the conditions resulting from hydraulic fracturing and seepage through cracks, whereas the original work was based on flow through a soil mass.

11. There is the need to distinguish between material interfaces that can be subject to the critical condition of a continuous flow from a full reservoir through a cracked core and those which can only be subject to transient flows. Interfaces subject to critical conditions are those on the downstream side of the core and such interfaces would normally be assessed using the principles set out by Vaughan and Soares[3] with a fine filter immediately adjacent to the core. Elsewhere where there is not a continuous flow through a mass, the conditions are less rigid and the conventional criteria for granular filters may be used. Such situations would be the design of drainage blankets to aid pore pressure dissipation or transition layers under rip-rap.

12. This paper does not detail the filter criteria but reference can be made to many published works, including Terzaghi & Peck [4], TRRL Report No. LR 346[5], and other manuals and papers where these are set out and discussed.

13. In 1967, at Balderhead Dam described by Vaughan et al,[6] a large sink-hole appeared in the crest of the dam. Investigations showed that the partial failure was due to cracking and internal erosion of the clay core. The filter drain provided on the downstream face of the core was ineffective in preventing this erosion. At the time of design in 1959 there was no agreed upon method for the design of filters to protect cohesive cores of embankment dams. There are other cases of filters being too coarse to prevent internal erosion of a clay core, yet there were many cases of filters being effective.

14. As a result of the Balderhead partial failure a design method was developed from this case and the method was immediately applied to the Cow Green Dam, then under construction. The design of these filters is described by Vaughan & Soares,[3] and the use at Cow Green is described by Vaughan et al,[7].

15. The Cow Green design was based on the fact that fine particles from the face of a crack may be transported with the coarser particles still deposited in the crack. Thus plugging of the crack as a result of the self-filtering of eroded debris cannot be relied on. The Balderhead filter had been designed using the D_{85} size of the core material, which for a crack results in using too high a particle size and therefore too coarse a material for the filter.

16. The method of Vaughan and Soares is the conservative concept of a "perfect filter" which is designed to retain the smallest particle which can arise during erosion. It does not rely on the self-filtering process of eroded debris, but should operate independently of the amount of segregation when flocculating conditions exist. The smallest particle is a clay floc, and it was shown that a non-cohesive sand filter can be provided which will retain flocs of the size encountered.

17. The method was based on the finding that there is a relationship between the size of particle retained by a filter and its permeability. Satisfactory filters with sufficient drainage capability have been designed and placed on this basis.

18. Other references such as Sherard [8]consider that because of the silt sized particles which comprise a substantial fraction of all fine-ground clayey soils, as used for cores, and which must be available to seal concentrated leaks, it is not necessary to provide a "perfect filter" to catch the clay flocs of 10-20 microns size. Sherard[9] reports that since 1955 piping occurred in about 15 dams with cores of similar material and coarse filters, with behaviour very similar to Balderhead. This confirmed the need for finer filters, but he considered that satisfactory performance of many dams shows that sand or gravelly sand filters of reasonable widths with D_{15} size of the order of 0.5-1.0mm would have prevented the troubles in that group of dams.

19. For non-critical situations, such as on the upstream side of clay cores, or under rip-rap, a concentrated leak cannot take place. Sherard et al [8] consider that the growing practice to use relatively coarse gravels or small size quarried rock for these transition zones, with no reference to filter criteria is safe and reasonable, and often leads to a considerable saving in cost. Multi-layer graded filters in such non-

critical situations may represent over-design and involve additional cost with no safety advantages.

20. When considering introducing filter drains into existing dams to deal with leaking cores, the above aspects need to be considered.

Geotextiles

21. Geotextile materials are available in a wide range of synthetic fibres and in many forms either reinforced or non-reinforced and with specially formed openings or weaves. Composite materials are available incorporating drainage voids to enable the geotextile to perform several functions.

22. The purposes required of geotextiles in earthwork construction are:

- separation of dissimilar materials or zones
- filtration
- drainage

and • reinforcement

23. In the context of this discussion the aspects of filtration and drainage are of more interest. The geotextile, as a filter, arrests the movement of finer soil particles from the protected layer of lower permeability to one of greater permeability. Drainage allows the controlled passage of water from one zone to another across the geotextile (or along it with composite drainage geotextile) without an increase in hydraulic head.

24. Because of the wide range of materials available, it is necessary to select the material with the right properties for the particular application. Several text books are now available covering geotextiles.

25. The use of geotextiles can be combined with sand and gravels to create the necessary filters and drains, so as to use less natural materials, which may be in short supply and which may be expensive, and to enable coarser drainage material to be separated without the need for a finer material. Stone filled trenches, where the sides of the trench are covered with a suitable geotextile are typical examples of the continued use of manufactured and natural materials.

26. In the CIRIA research on the use of reinforced grass (10) a geotextile is recommended under the concrete blocks as a separation layer, which enables filtration and drainage to occur. The correctly chosen geotextile does not hinder the growth of grass roots.

27. In improvement works to existing earth dams, there are many possible applications of geotextiles. There is adequate performance data to show sufficient long term reliability of the materials for such uses. It is not considered advisable for any geotextile or geomembrane to be left exposed to sunlight or other atmospheric conditions.

Examples of improvement works

28. The following case histories are presented as typical recent examples of improvement or remedial works to existing earth dams, where

drainage was required.

A. Increasing freeboard with additional fill on the crest and the downstream slope

The need to raise the crest by 2.25m of a subsidiary earth fill dam to give adequate freeboard, required the placing of an additional width on the downstream face of the 8 m high embankment. The dam was raised by extending the existing upstream slope and moving the crest track downstream with the fill placed partly on the downstream slope and partly on the foundation beyond. The area beyond the toe was very wet and marshy and a ground investigation had indicated that this was due to groundwater diverted as a result of the reservoir rather than seepage, and moderate flows could be expected. Consequently drainage measures were necessary and a conventional ground drainage blanket of a sandy gravel material was initially proposed. However preliminary enquiries to suppliers indicated that the remote location of the reservoir and the approach through narrow country lanes would result in a high supply price and other drainage options were investigated. Recent developments in the field of synthetic membranes were studied and it was considered that the use of 'Filtram' as a base drainage blanket could prove attractive. This product combines a double layer of 'Terram' separated by a semi-rigid plastic mesh whose laminar construction utilised the filtering characteristics of the 'Terram' membrane whilst allowing flow within the core of the combined system. Although initially developed as a drainage/filter system for use against vertical or near vertical concrete faces, studies indicated that sufficient flow could be achieved when used in a near horizontal position under the relatively low loadings from the outer portion of the extended downstream shoulder.

Consequently tenders were invited on the basis of both the conventional ground drainage blanket and the 'Filtram' alternative, and the latter was shown to be cheaper and was subsequently adopted for the remedial works.

Placement on site was carried out in 1982 after the cutting of vegetation with the geotextile merely rolled out across the ground surface above the very soft surface deposits. Under the wet conditions, jointing by tape was not considered feasible and adjacent lengths were simply lapped and joined by use of an industrial staple gun. An initial layer of fill, nominally a half metre thick, was end dozed over the membrane and by careful workmanship, rucking of the membrane was avoided. Flow from the membrane was collected by a conventional toe drain and regularly monitored throughout the remedial works and subsequently, and has proved highly satisfactory to date.

B. Drainage to cater for leakage and instability

At a 190 yr old canal dam, now used as part of a county council country park, there has been a long history of settlement and downstream slope instability. The embankment appears to have been constructed from weathered London Clay with no distinct core or drainage features, but past investigations have shown numerous inclusions of gravel, topsoil and decayed vegetation. The latest major slips occurred in 1975 (11) when failure of both shoulders occurred following the removal of vegetation on the downstream face and the addition of material to the crest to increase the freeboard of the dam. Remedial works comprising execavation and replacement of the slipped materials and reconstruction of the crest track were carried out at that time and the embankment subsequently appeared stable other than continuing slow settlement of the heavy upstream wave wall.

Subsequently a number of wet areas developed on the downstream face in 1978 and cracking and minor deformation of the tarmaced crest track were noted, indicating signs of incipient instability of the downstream shoulder. The problems were considered to be due to high water pressures in the shoulder which had increased somewhat as a result of the removal of vegetation, and also local leakage close to T.W.L.

Remedial works comprised the lowering of the top water level by 600 millimetres and the installation of a series of deep counterfort drains running normal to the slope over the affected length. The drains were designed using a method subsequently published by Bromhead (12) and comprised 600mm wide trenches to five metres nominal depth at five metres spacing, discharging via a collector drain to the spillway channel beyond the downstream toe. The trenches were excavated in short lengths to maintain stability and filled with 19mm single sized aggregate to Table 1 of BS 882 with a surrounding filter fabric of 'Terram' 3000. A 150mm uPVC pipe was installed in the collector drain but not included elsewhere. Following the installation of the drains, the affected area was regraded and crest track reconstructed and no further movements have since been noted.

C. Improving stability of downstream slope under heavy rainfall conditions

At a 115 year old earth dam with a puddle clay core and decomposed granite shoulders it was found that pore pressures in the downstream shoulder, consequent upon heavy rainfall, could lead to a low factor of safety and instability. The dam is situated in a tropical area where a Probable Maximum Precipitation of 1000mm/day is possible.

A number of boreholes were sunk and piezometers demonstrated that the foundation was permeable in some areas, allowing dissipation of pore water pressures, but that other areas were underlain by more impervious materials which prevented the escape of water. In these latter areas, pore water pressures were observed to rise during periods of rainfall, with consequent detriment to the stability.

The remedial measures proposed and subsequently installed, comprised a series of 1.8m deep trench drains in the downstream shoulder. These were provided with porous collector drains to discharge clear of the downstream toe and backfilled with graded sand. The trenches were capped off with clay to exclude surface water run off. Relief wells were also incorporated at the downstream toe to limit water pressures under the toe.

D. Drainage relief holes in downstream shoulder to deal with leakage and high pore pressures

Two adjacent reservoirs in the Pennines were constructed in the mid nineteenth century, on ground containing a coal seam that was being actively mined at the time of construction.

There has been a continuing history of settlement and survey points were installed in 1980. Observations showed that differential settlement was occurring and together with wet patches on one of the embankments led to a recommendation for a full investigation of the foundation and embankment fill. This investigation and piezometric observations showed that the weathered rockhead was acting as a drainage layer to the downstream shoulders.

Improvement works have been carried out in 1986 comprising drainage relief holes at 3.3m centres and drilled at $7\frac{1}{2}^{o}$ to the vertical from the downstream edge of the embankment crest so as to give an increasing horizontal distance from the vertical core zone with depth. The relief holes comprised a slotted steel pipe, wrapped with a geotextile and the annular space filled with fine sand.

The performance of the system was originally monitored by weekly observations of water levels in the relief drains and after a review the frequency was considerably reduced.

Since the drains have been installed settlement has ceased. It has been shown that downward drainage of the fill is occurring and the wet areas, probably due to perched water conditions, have dried out. Overall, it is considered that the dams have been improved by the works, and that future observations of the performance of the dams will enable the improvement of the safety of the dams to be shown.

Conclusions

This paper is not intended to cover the wide-ranging and most important topic of filters and drainage. It has only discussed a few topics in a non-comprehensive way in order to stimulate discussion, within the context of remedial and improvement works to existing embankment dams. The four cases are only given as examples where drainage, rather than other more expensive remedial and seepage control measures, have been used.

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INSPECTION AND BEHAVIOUR OF DRAINAGE INSTALLATIONS IN SLOPES ON EMBANKMENTS

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SYNOPSIS

1. In designing drainage systems it is most important to obtain an exact as possible knowledge of the hydrogeological surroundings, in order to be able to take constructional measures in good time to prevent, reduce or delay damage due to ageing. These damages result from mechanical, biological, hydraulic and chemical influences and may reduce the hydraulic discharge capacity of the whole installation.

2. As proved by laboratory tests, the resistance of the pipe against the influx grows overproportionally with the degree of ageing. This fact confirms that it is necessary on the one hand to plan carefully , and on the other hand to check the drainage system periodically and just as carefully, for example with a television probe.

INTRODUCTION

3. Drainage systems in embankment dams have to collect the seepage water and to drain it off reliably. These systems often consist of permeable gravel layers with drain pipes to raise the discharge capacity. The main purpose of such installations is to keep the seepage line at a sufficient distance from the embankment surface, which influences the stability of the construction to a large degree. This function can only be guaranteed, if the drainage installations operate well over the entire lifetime of the structure in question.

4. To asure this, periodical controlling of the seepage water (quantity of water per time unit and hydrogeological boundary conditions, infiltrated fine constituents and the chemical compound of the seepage water, etc.) should be performed to obtain information about the usability or, at worst, to be able to detect a possible endangering of the stability of the whole construction in due time. Therefore all parts of the embankment construction involved in drainage and seepage water control have to be watched intensely.

5. As many field investigations have shown, in practice the opposite is often the case: drainage pipes are laid without regard to filter criteria; the pipe materials are chosen

irrespective of the chemical compound of the water (e.g. POROSIT-concrete-pipes were used in spite of water agressive to lime); the pipe diameters are too small for either flushing or inspection with a television probe; in the same way, branchings in the pipe system do not allow any inspection with available probes.

6. Independent of this, the location and the surroundings may affect the discharge capacity of drainage systems, a fact, which is commonly termed as ageing. There are various forms of ageing, which can be distinguished as follows ((1) and (2)):

* Ageing due to mechanical influences:

- Rupture or displacement of the pipe section as a result of soil movements in the area of the pipe,
- Rupture or compression of the pipe section as a result of static overload,
- Rupture of the pipe during installation (e.g. damage caused by compactors),
- Compression of the pipe due to creeping effects of the pipe material, which greatly depend on the lokal temperature.

* Ageing due to hydraulic influences:

- Clogging of the drain pipe openings (cf. paragraph 5),
- Siltation of the pipe section,
- Damming up and sedimentation from the tailwater.

* Ageing due to biological influences:

- Clogging with biological phlegm,
- Ingrowing roots.

* Ageing due to chemical influences:

- Corrosion,
- Iron hydroxide deposit,
- Sinter deposit.

7. Numerous field investigations with a television probe between 1986 and 1988 have shown ageing symptoms due to one or more of the influences mentioned above in almost every drain pipe that has been explored optically. As a result of these field investigations and of additional laboratory tests, instructions for the design of drainage systems, taking the hydrogeological conditions and the chemical compound of the water into consideration, are given. It should be pointed out, that it is of great importance to install the pipes carefully in order to preserve the hydraulic discharge capacity and to reduce the damage due to ageing.

BASIC RULES FOR THE DESIGN OF DRAIN SYSTEMS

Recognition of possible causes of damage

8. Mechanical reasons:
Drain pipes are to be regarded as embedded structures and, therefore the calculated loads at the vertex of the pipes may increase due to different degrees of rigidity in the surroundings of the pipe. Additional loads can result from soil displacement. Moreover, as the same durability as the whole structure is often demanded from drain pipes, reduced material characteristics are used for the calculation. Characteristic values can be taken from the "ATV Arbeitsblatt 127" (3).

9. Hydraulic reasons:
The pipe openings must be adjusted to the surrounding material according to the well known filter rules. If necessary graded filters are required. Any entry of material should be prevented. It is necessary to control and clean the shafts, as often the entry of material into the pipe takes place from here. Careful designing and installing of the pipes must ensure a sufficient hydraulic gradient in the pipe line over the whole lifetime of the structure.

10. Biological reasons:
Any possible activity of plants or very small animals has to be prevented by a suitable covering. If there is any danger of ingrowing roots (short distance to the surface), pipes without connection joints have to be used. In some case, if necessary, no plants should be placed in the area surrounding the drain system.

11. Chemical reasons:
By choosing the pipe material according to the water conditions expected, corrosion influences can be eliminated. Limiting values of the corrosive reaction of the water are given in (4). If there is danger of iron hydroxide deposits, an attempt should be made to avoid these by airtight insulation, e.g. with siphon installations. Sinter deposits cannot be avoided with these measures because of the complicated chemical correlations. With the current level of technology the only possibility often is high-pressure-flushing. Here early recognition of damage is necessary, consequently requiring regular periodical checks.

Registration of the surrounding conditions

12. Systematic recording of data is necessary for planning, in order to create conditions which delay the ageing process. This means firstly an exact registration of the surrounding conditions:

- soil samples
- drill logs (indications of ground-water levels)
- ground-water level progress lines

- precipitation progress lines
- chemical compound of the groundwater and reservoir water
- static loads
- soil movements
- planting of the planned line of the pipes

13. The chemical analyse of the water should at least comprise the following data:

- pH-value - oxygen content - electric conductivity - temperature	to be measured on site immediately after sampling
- calcium - magnesium - total hardness - iron - sulfate - free carbon dioxide - ammonium - chloride	to be measured in laboratory

14. The pipe materials and diameters must be chosen according to these data. The pipe diameters should not be less than 150 mm, for reasons explained below. The ground-plan of the drainage system should be as simple as possible, in order to be able to clearly relate the discharge measurements to the other hydrological and hydrogeological data.

15. The distance between two shafts should not exceed 50 m, since longer distances make the use of television probes or flush appliances difficult. If the pipe diameters are less than 150 mm, even small deposits can make probing impossible. At every branching there should be an inspection shaft, from where the servicing can be carried out.

16. It should be made possible to measure the discharge in suitable shafts regularly and without any problems (e.g. by means of discharge pipes, etc).

17. Discharge measurements should at least be carried out from time to time. These measurements are to be compared with precipitation progress lines or storage lines. In this way qualitative and continuous information about the discharge capacity of a drainage system is obtained. At certain intervals there should also be a chemical analyse of the water, which for example includes data that is easy to obtain such as the pH-value, electric conductivity and oxygen content.

<u>Proposal for an inspection plan</u>

18. At regular intervals, or when irregularities occur (e.g. detected by discharge measurements), television probings should be carried out. An inspection plan should be made containing all important data:

1. date of the probing
2. leader of the probing
3. object of the probing (e.g. inspection, damage, etc)
4. short description of the drainage
4.1 number of drain pipe or shaft
4.2 depth of the shaft
4.3 diameter of the shaft
4.4 distance between the shafts
4.5 type of drain pipe
4.6 diameter of drain pipe
4.7 age of the drainage system
5. first or repeated probing
6. date of the last probing
7. date of the last flushing
8. drain working or dry
9. surrounding conditions
9.1 water (analyse)
9.2 filter
9.3 surrounding soil
10. stated ageing symptoms
10.1 kind of ageing
10.2 degree of ageing
10.3 distance of the detected damage to the inspection shaft
11. other (photographs, video tapes, problems during the inspection)
12. general assessment of the system

19. If the damage results from chemical or hydraulic ageing the only solution often is high pressure flushing. However, even these measures can only be successful, if the deposits are not yet totally solidified. Therefore it is necessary that the damage be detected in due time, which again demands regular inspections of the system. Indeed, the discharge pipe diameters are usually larger than needed, but the discharge capacity of the pipe decreases even with slight changes in the effective section area (e.g. by means of deposits), as the following paragraphs will show.

LABORATORY TESTS WITH THE PRESSURE POT

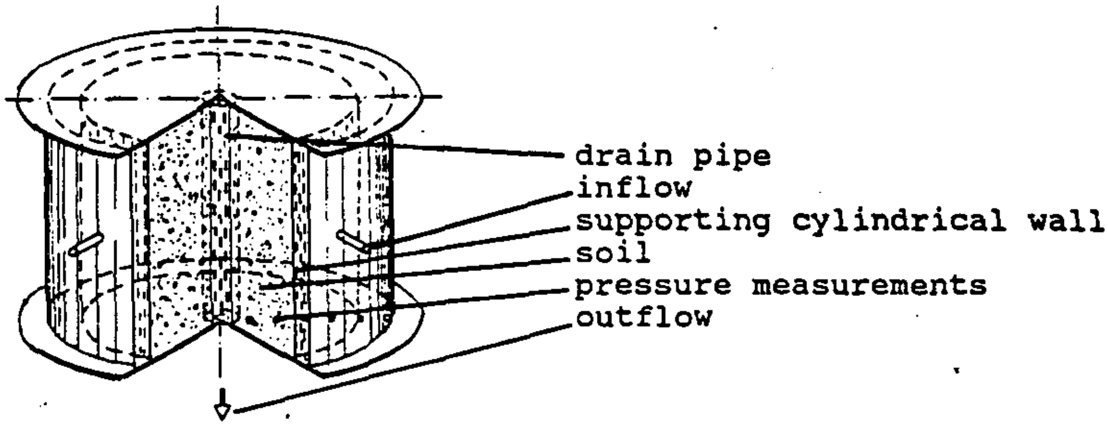
General

20. Drain pipes in a layer of permeable gravel are real structures with a finite number of openings and a certain thickness of the pipe wall. Therefore drain pipes put up resistance against the influx, which results in a loss of pressure (see (5)). This loss of pressure is composed of a number of individual losses, which in each case depend on the soil conditions and the geometry of the drain pipe. The hydraulic conditions are so complex, that an analytical examination of the problem could only provide useful solutions after time consuming procedures and with the help of computers. The problem becomes still more complex due to the fact that a three-dimensional influx is involved here.

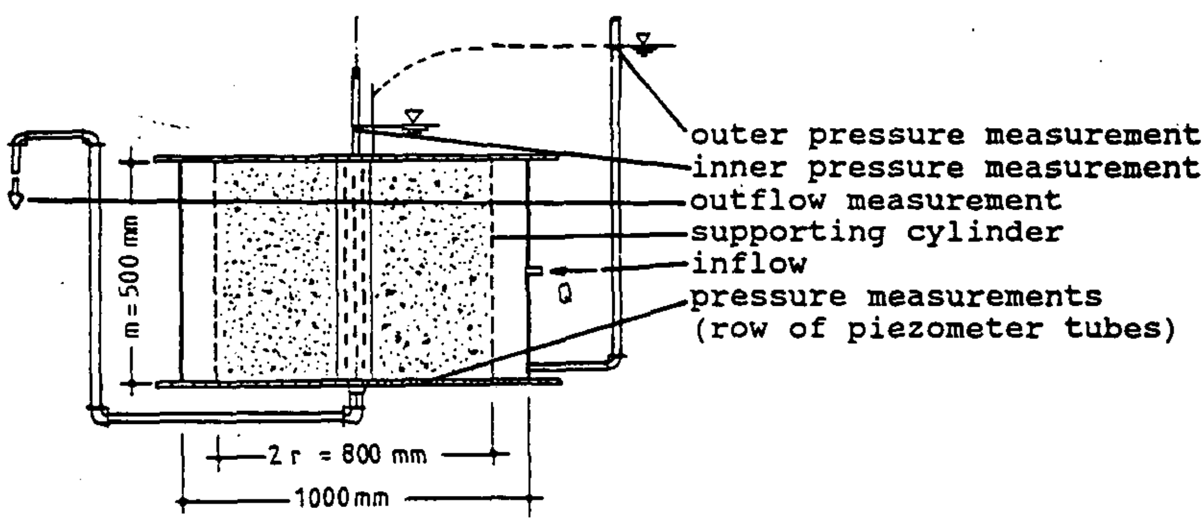
21. Consequently, it was decided to perform laboratory tests. With the help of suitable equipment the total pressure losses were to be measured, which arise in the regarded pipe depending on the various soil-pipe combinations. In this way characteristic loss values are obtained, which can be used as a criterion for the hydraulic capacity of a soil-pipe combination.

<u>Experimental set-up</u>

22. The problem was to build equipment, so that the inflow conditions could be defined exactly and capillary effects could be eliminated. The equipment was set up in a pressure pot, as shown in fig. 1.



isometric diagram



section

Figure 1: diagram of pressure pot

23. The filter pipe to be examined is fit in the pot vertically and the space between it and the supporting cylinder is filled with the soil to be tested. The pressure pot is closed with an acrylic glass lid, ensuring that no air

bubbles form under the lid.

24. Piezometer connections in the base of the pot allows to measure the pressure in the drain pipe and in the soil between the drain pipe and the outer supporting cylinder and thus show the pressure conditions in the soil-pipe combination by means of a row of piezometer tubes.

Realization of an experiment

25. During an experiment water flows through the soil-pipe combination from the outside to the inside. The flow through the drain pipe is measured with a Thompson weir. The pressure distribution in the soil, as well as the outer pressure and the inner pressure in the pipe can be read from piezometer tubes. The evaluation of an experiment is shown in fig. 2.

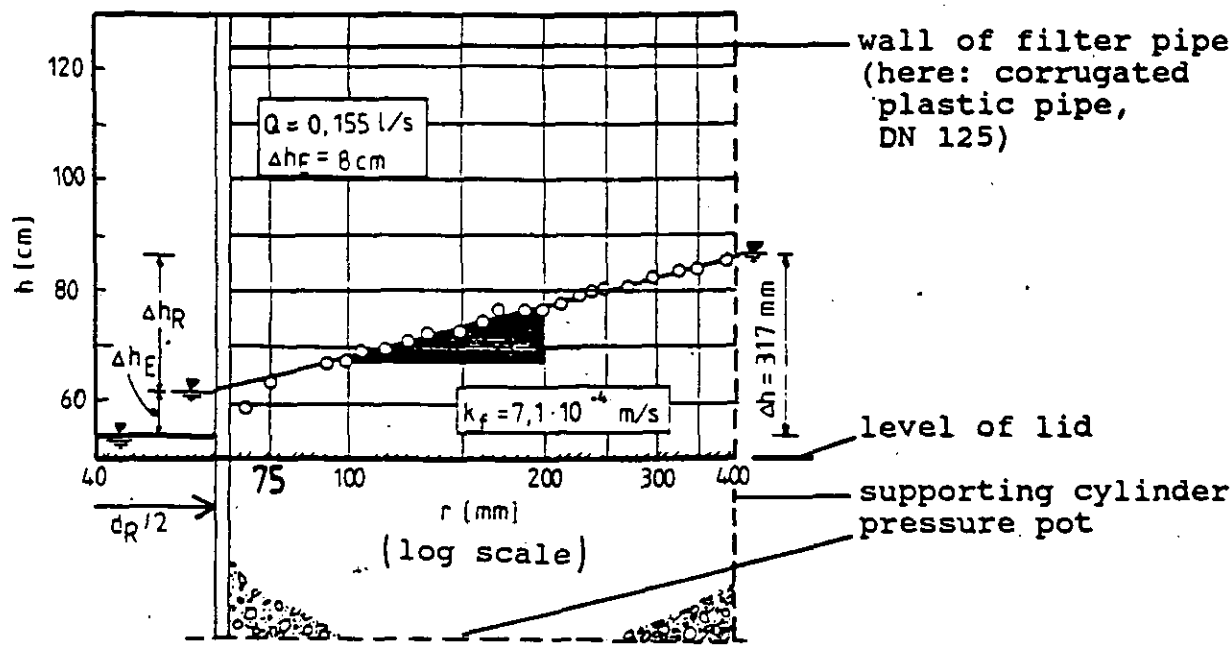


Figure 2: Evaluation of an experiment with the pressure pot

26. Δh indicates the difference between the outer and inner pressure. Δh_E is the total entry loss, which, as is usual in hydraulics, is expressed with the dimension of a heigth.

27. By variing the outer pressure, different pairs of values (Q, Δh_E) are obtained. By division with the inner surface of the pipe A_{in}, Q can be expressed by a heigth of the form $v^2/2 \cdot g$, where $v = Q/A_{in}$.

28. With the help of the well known formula $\Delta h_E = \xi \cdot v^2/2 \cdot g$, the values ($Q$, Δh_E) can be used in a diagram to obtain a loss value ξ, which is characteristic for the regarded soil-pipe combination and which defines the entry losses numerically.

29. The aim of the investigations was to determine how much the discharge capacity depends on the degree of ageing. For

this purpose the pipes were intentionally blocked to a varying degree. The degree of ageing was defined as the relation of the actual open area to the full open area of an intact pipe.

30. For every degree of ageing the experiments were carried out as described in paragraphs 27 and 28.

Results

31. The type of soil, diameter of the drain pipe openings and the degree of perforation, i.e. the open area per meter of pipe, were varied. This led to the following results:

- With the same degree of perforation, pipes with numerous small holes show a better hydraulical behaviour than that of pipes with a few large holes.
- The relation between intake capacity and the degree of perforation is overproportional.
- The change (loss) in intake capacity of a drain pipe due to ageing (in form of partial blocking of the openings of the pipe) proved to be as shown in fig. 3, where

  Q_{ref} = discharge into intact pipe at a given Δh_E,
  Q = discharge into "aged" pipe (with partially blocked openings) at same Δh_E,
  A_{ref} = degree of perforation of intact pipe
  A = portion of perforation

  All soil-pipe combinations tested in the study showed the behaviour presented in fig. 3.

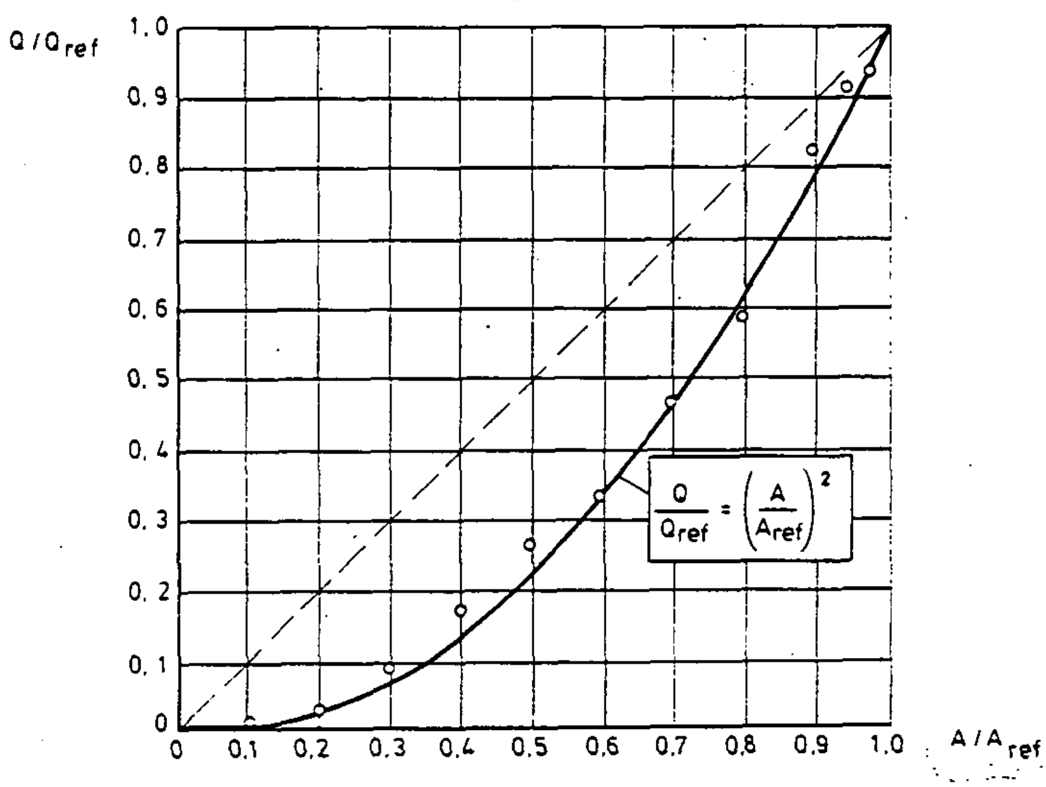


Figure 3: Diagram of the intake capacity depending on the degree of ageing (here: stoneware pipe)

Consequences for practice

32. The results show that, as far as the pipe statics allow, a large degree of perforation should be provided; besides this, it was discovered that the hydraulic capacity decreases overproportionally with the degree of ageing (if only a third of the holes are closed, the intake capacity only amounts to a ninth of the quantity in an intact pipe). For this reason large reserves must be provided for the pipe cross sections and the openings. With the above results, in which small openings were given preference from a hydraulic point of view, this means that a compromise should be made regarding the geometry of the holes.

CONCLUSION

33. Drain pipes can be damaged in several ways. Extensive and careful planning can help to reduce the possible damages to a minimum. The aim of this paper is to provide aids for the planning of drainage systems. The pressure pot provides a simple testing method, which makes it possible to experimentally check the efficiency of a drainage pipe in a given soil.

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DISCUSSION: TECHNICAL SESSION 5

INSTRUMENTATION AND DRAINAGE OF EMBANKMENTS

Session Chairman: Dr A D M Penman (Consultant)

Gentlemen we have papers, Gorpley and Ramsden dams drainage in existing dams, and the inspection and behaviour of drainage installations in slopes of embankments, and we are going to include Technical Note 3.

D J KNIGHT (Sir Alexander Gibb & Partners)

I wish to synthesise three separate elements contributed by others at this conference. The three elements are:

1 From Messrs Millmore and Charles' Technical Note 1, "A survey of UK Embankment Dams", it is shown on Table 1 that 69% of dams for which information was supplied by owners, principally the large water authorities, have puddle clay cores, representing 447 dams. Of those, 294 had unprotected outlet pipes in the embankment.

2 Mr Kennard's paper 5.2 gives interesting case histories of recent examples of the improvement of or remedial works to existing earth dams, each of which deals with what could be called "general dam drainage improvement" along its length.

(Incidentally, I should like to draw general attention to the two papers by Sherrard, Dunigan and Talbot in Proc. ASCE Geotech. Division, June 1984, entitled "Basic properties of sand and gravel filters", and "Filters for silts and clays", which represent a sound and practical statement of well-tested filter design rules. These are a good basis for dam designers. I also agree that upstream "filters" are nothing like as critical as downstream filters, and should not be subject to the same stringent grading requirements).

3 Dr Charles and his BRE colleagues have been concentrating recently on hydraulic fracture and the erosion of puddle clay cores.

These three elements focus on two aspects of embankment dam safety, namely:

(a) the general adequacy of the dam along its length, and

(b) its local safety, especially at the position of outlet culverts through the embankments.

Dealing with each in turn:

1 <u>General dam improvement</u>

Ways of effecting a general improvement to a dam's safety in terms of general seepage control and thus stability include installing:

(a) A discontinuous vertical drainage curtain, located downstream of the core, and comprising a curtain of sand-filled vertical drains.

(b) A continuous vertical drainage curtain, located as (a), but comprising a continuous diaphragm wall of sand, installed perhaps with the assistance of a degradable polymer to give temporary support.

(c) A horizontal outlet linking (a) and (b) to the downstream toe drain. Such an outlet could comprise a series of horizontally drilled drainage holes.

2 Particular critical locations

One of, and possibly the most critical places in an embankment dam is where the outlet culvert passes through the core and the downstream shell. This can be the most dangerous place in respect of dam safety, as I saw most dramatically and tragically demonstrated with the failure of the Kantalai sluice near Trincomalee in Sri Lanka in April 1986 (Proc. ICE, Part 1, December 1987, Vol. 82, pp 1261 - 1265). This highlighted the need for very careful inspection or investigation of the inside of old outlet culverts for cracks, gaps, deteriorating joints or any other feature whereby core material could escape downstream, generally unnoticed. Investigation would indicate an appropriate treatment of the condition, whether it be repair, abandonment or, in certain cases, the installation of filter or geotextile protection.

Finally, yesterday we saw the unusual sight, for a BNCOLD meeting, of a dam being dismantled. In installing additional drainage we also have to do things in reverse, but let me suggest a forward looking alphabetical mnemonic to remember: Add Blanket and Curtain Drains to the Embankment, and Filters (Geotextiles?).

Now we surely can all remember the first six letters of our own English alphabet.

M KENNARD (Rofe, Kennard & Lapworth)

I would like to thank Mr Knight for those comments. He referred to the work of Sherrard and compared it with Vaughan and Soares : I entirely agree. I was trying just to bring in the British development that took place based on what had occurred at Balderhead dam, which led to the research of Vaughan and Soares methods which have been used at Cow Green dam and elsewhere, and overseas. They may be more conservative than Sherrard's but nevertheless I entirely agree that they are in the same direction, but Vaughan deals more specifically with the known effects of hydraulic fractures, that have occurred and been investigated very thoroughly.

I would also agree about the critical aspect of culverts, and I would say that where there has been known leakage outside culverts it's far better to start off work by putting in filters and drainage before considering the sealing methods and other works. We know of cases where people have rushed in to do some grouting, but it still leaves the internal erosion problem in the dam.

DR J A CHARLES (Building Research Establishment)

Mr Knight referred to the BRE research on hydraulic fracture and I think the distinction he made between the uniform conditions along the length of the dam and the particular critical locations is a very valid one. Our measurements of stress in puddle clay cores have shown that cores with a particular slenderness ratio, that is thickness to height, the thinner they are the more susceptible they are to hydraulic fracture, and one can define a slenderness ratio at which hydraulic fracture becomes probable and we published that in a paper in the proceedings of the Institution.

In the discussion to the paper, Professor Skempton contributed some historical data which did substantiate that in looking at the dams that actually have undergone distress, probably due to internal erosion, but I think the point that Mr Knight was making about critical locations is very valid and our measurements in our paper on Gorpley dam show a critical location near to where the culvert went through the puddlecore.

Coming back to Technical Note 1, where we summarised some information from a survey of dams, and the point that of course Mr Knight raised about the unprotected pipes there, I think that is quite a surprising proportion of dams in Britain which have these. One thing one has to remember is that most of our dams which come within the reservoir legislation are quite small dams.

The data from that survey suggested that situations where people knew what sort of core there was in a dam, 65% were puddle cores, but we had a very large percentage where it wasn't known whether there was a puddle core or any sort of core and I think that does illustrate some of the difficulties which the panel engineers have in inspecting some of these structures.

J M McKENNA (Consultant)

Mr Kennard refers in his paper to the design of filters and refers to the 1984 paper of Sherrard Dunigan and Talbot. Sherrard and Dunigan published two more papers before Sherrard's death last year, but I would like to draw the attention of delegates in particular to a 1986 publication of Dunigan which gives latest design rules for filters. This has been published by the Soil Conservation Service of the United States Department of Agriculture and is their Soil Mechanics Note N° 1. This is particularly good as the design rules are followed by examples of the application of these rules to four different soil types. I think it is an excellent paper, strangely little known in this country.

Mr Kennard also raises the question of the need for a perfect filter. For more degraded soils, such as glacial tills, the filter has to be designed using the D85 size of the soil fraction finer than 4.75 mm. If this is done, there is no evidence that a perfect filter, with its inherent low permeability, is needed. I know of no case where a fine filter with more than 5% passing .63 micron sieve has been needed. If there is any concern about the ability of a filter to protect a soil, then Sherrard's no-erosion filter test should be done. This involves passing water at a pressure of 400 k Pa through a 1 mm diameter hole formed in the soil, supported on the proposed filter. Under this hydraulic gradient, all soils erode and there is a very sharp division between those filters which fail and those which prevent erosion. Thank you.

W J CARLYLE (Binnie & Partners)

I wanted to make a contribution on Mr Kennard's paper and strangely enough was going to refer to the USBA soil conservation service paper. It really relates to the problem that often confronts designers when considering the filter downstream of a core and the need, of otherwise, to have a 2-stage filter drain. Some valuable guidance is given in that particular paper and the design criteria is really that the filter, when properly designed, will take care of any erosion from any crack by blocking, and the requirement for permeability is that the drainage system as a whole should take care of the flow through the mass of the core rather than the potential high flow through a crack when it initially forms. I think that, in the case of the sorts of cores we normally use, particularly in this country with clay cores, really means that an appropriately designed sand filter with an effective size of .5 mm to 1 mm, will give both appropriate permeability and doesn't need to have a gravel drain behind it. The exception I think is the base drainage mattress, where one is frequently dealing with really unknown conditions of foundation, seepage and drainage, and in that case it's certainly our practice normally to have a filter, drain, filter sandwich.

K SWETTENHAM (North West Water)

Dr Penman raised the subject of the solid gates in Haweswater. I would like to point out that the original gates were, as you described them, prison gates and the solid gates were placed there in 1969/1970 following the upsurge in problems across the water. It was believed that they would be more secure. I would also like to make the point, when the original 1982 trials were done there was a difference in the quality of workmanship, which was very apparent, and the two panels that were not successful, one of the reasons would be definitely down to this quality of the workmanship.

I would like to just make the point that the decision wasn't made by water engineers, it was made from London.

DR A D M PENMAN (Session Chairman)

Thank you very much. This brings us to this problem of security and I have always thought that if only we could all be honest citizens and only take advantage of the contractor what a glorious world it would be! The shutting off the drainage for a dam for security reasons can cause the face to fall off! So, I think that our masters in London, who ordered the solid doors to be put on, should be brought up here and made to see what damage they have caused by a thing like that, but it could be said that we were doing this in ignorance because I am sure that the people who fitted the solid doors never believed for a moment it was going to cause damage to the face of the dam. They didn't think the two things could be related and they felt sure that, in securing the dam against a worse fate of being blown to smithereens by explosives pushed in through the gates. What I should like to discuss more, is who amongst you, who design the concrete dams, allows for drainage in the form of ventilation through the galleries? Indeed, has it been given due consideration in the past?

F G JOHNSON (North of Scotland Hydroelectric Board)

I was very interested to hear the problem at Haweswater. It's very similar to two problems we've had on two of our dams; they are concrete dams, one at Loch Dubh near Ullapool and the other at Dunalastair in the Tummel Valley near Pitlochry, where the downstream face has spalled and we have lost a similar amount of 3 or 4 inches of concrete.

We experienced this or noticed it in the late 60s - early 70s. We did quite a lot of tests and found that there was free water within the body of the concrete. The first dam we looked at was Loch Dubh and we've recently done Dunalastair and we felt the solution to this was very good drainage of the concrete. On Loch Dubh dam we actually painted yellow panels on the downstream face and we asked the waterman every week to go along and note where the bits were dropping off, drawing it on sketches. We did this for a few months before we drilled and from then onwards we've had little or no spalling of that dam, and that was done in the very early 70s. We therefore believe that the solution to this is first class drainage.

Again, it's interesting because we have quite a number of buttress dams, and on the spillway sections, they are plated over and we get contained and can get very damp conditions there, but we make sure that we've got first class drainage and air ventilation. I just wonder whether an alternative solution, which I believe would be successful, could be to put first class drainage in each buttress and maybe put a port in, say, between 2 or 3 feet in diameter, at the bottom and the top, say, so you've got an air current up thorough the elephant stalls, as we call them, and to keep the concrete dry, and thereby to stop the freeze/thaw action, which we believe is the cause of the spalling.

On another matter I was very interested to see the infra red survey done at Ramsden dam and the very wet conditions on the downstream face. We had a case, Lower Shira, this is an embankment dam, with a concrete core, and we had it inspected by Bill McLeish who went to see it. He inspected it in December, in a very wet period, he found water issuing out of the downstream face in a number of points. We were concerned when we heard this and put in V-notches and measured the water and, sure enough, the water was issuing from the downstream face. There was a line running across the whole length of the embankment and eventually, when we looked at all this what we found was that it wasn't a leak, but the lower portion of the downstream embankment had been constructed of earth, fairly impermeable, but they had run out of earth, and the upstream portion, about the top third of it, had been constructed out of quarry rubble and waste, which was very permeable and the water was rainfall falling on the downstream shell, passing out of the downstream shell and then coming out. We have monitored that since and in dry weather it stops and in wet weather it runs.

G ROCKE (Babtie Shaw & Morton)

I was resident engineer on the dam with the 'leak'. It was one of the exceptional dams with a central concrete core of highly reinforced concrete on a spherical cradle. There was no way water was going to get past that central concrete core, in fact, piezometers we installed in the downstream shell dried up immediately, and never gave a reading.

What really happened was that the atrocious weather we have in Scotland was driving the contractor crazy and, at that point, the agent and I discussed the matter, and we weren't going to finish that dam unless we brought in quarry spoil and that was put in for the top 3 or 4 metres of the dam and we finished on time.

T A JOHNSTON (Babtie Shaw & Morton)

I was particularly interested in the paper on Gorpley dam because I had the pleasure of inspecting it earlier this year and have never been presented with such an erudite voluminous display of data on a dam, which certainly made life very interesting and made my task very easy, in fact, the only thing that I could suggest that they hadn't tried in trying to find the source of the water in the downstream face was a technique which I know that some other water authorities use of a bacteriological survey.

Michael Kennard mentioned a very interesting example of catering for a metre of rainfall on the downstream face and I wonder whether in this country now that we are so concerned about the PMF rainfall in respect of floods, whether we should be considering what is the effect of PMF rainfall on the upstream face and I think of this particularly in cases where a number of dams have been steepened near the crest to provide additional freeboard. While generally they have a satisfactory downstream slope of 1 in 2½ or 1 in 3, but get near the top and the slope is 1 to 1½ and I wondered whether there was a possibility of a surface slip in very heavy rainfall which could endanger the crest and I wonder if BRE have any information on that.

I think that Mr Knight's comments about the unprotected pipes through the puddle clay cores is one which is very relevant and is worrying a lot of us. I was very concerned while looking at Gorpley and reading BRE's report talking about the susceptibility of hydraulic fracture in the vicinity of the pipe.

I think really that pipes are a topic that is under-researched - we just don't know enough about the pipes under the dams, but now we can put CCTV cameras underneath them, I've asked various contractors why they can't tell us a bit more about the state of the pipes, about the extent of corrosion and graphisation, and the answer to this is it is just a matter of economics. There is a tremendous demand from the water industry to provide information about sewer pipes, water mains, but nobody has really asked the industry to provide the techniques to investigate these pipes, we're sure they can do it if someone would just ask them properly to do it. So perhaps that is another topic for investigation.

The topic of damp areas inside concrete dams does arise also in embankment dams where we have increasingly either galleries at the upstream toe or galleries through dams and it is very noticeable that the seasonal effect of summer/winter and rainfall affects just how damp the conditions are. Very often the saving grace is either the chimney effect of the draw-off tower so that the air is drawn through from the downstream toe right through the gallery, up the draw-off tower, or the forced extraction that we put in due to concern brought about by Health and Safety legislation concerned about collections of noxious gases, and this has been given as a side benefit in respect of drainage and avoidance of dampness.

DR J A CHARLES (Building Research Establishment)

Mr Johnston raised the question of stability and rainfall. I think on most of our old dams, particularly the sort of construction that is used on the Pennines, the slope stability is often likely to be critical at fairly shallow levels and therefore the stability is critical on the pore pressures and any cohesion that the soil might have. So I think it's quite true that something like very heavy rainfall could upset the balance. It's very difficult on these slopes, I think, to accurately calculate a factor of safety, simply because the calculation is so dependent on the assumption of pore pressure and any cohesion.

M F KENNARD (Rofe, Kennard & Lapworth)

First, I'd like to thank Mr McKenna and Mr Carlyle for the additional comments they made on drainage. I didn't know of the 1986 reference, but obviously it's worthwhile studying. It does show that there is an on-going consideration of filters and drainage. It didn't stop with Terzaghi's rules 40 years ago.

A question that Mr Johnston has just raised: I didn't wish to give the impression that the works on that particular dam I mentioned in Hong Kong were solely designed to deal with 40 inches of rain; it was to deal with a smaller amount, which is much more frequent. I don't think it would be right to check the shallow slope stability of some of our dams for a probable maximum precipitation. I think it is such a rare event and if it's only a surface slip, a shallow slip, I wouldn't be too worried myself because we have good grass cover, you see, so the infiltration is probably quite different from a dam built of decomposed granite.

R M ARAH (Binnie & Partners)

I have recently inspected Roundhill reservoir for Yorkshire Water Authority, which is a concrete gravity section. It has galleries, which are very well ventilated, and the secondary concrete in those galleries, things like steps and pipe plinths, is decomposing due to frost action on the concrete caused by ventilation in the chimney effect within the dam. It also has under-drains, which have produced gas problems; you have also to think of that in ventilation.

In the early days of the Brenig design, which is a rock-fill embankment with a very large concrete culvert under the foundations, we compromised with the security authorities on a curious sort of louvre of z-shaped slats, which left them happy because explosives could not be pushed through and left us happy because there was considerable ventilation. After all this effort, the last time I saw Brenig the architect had put on some extra self-operating louvres on the back, but of course it was too cold within the culvert.

We have to remember that the shoulders of old embankment dams.are rarely homogeneous, often anything from boulders down to clay, and with no particular rational distribution. I believe this often causes the flows from the downstream slopes, which we do see. I can think of three where this has happened quite prominantly even when the reservoir has been emptied, for years in one case. Deanhead is still empty and there are damp patches on the downstream shoulder which are only explicable in terms of rain being collected and distributed within the shoulder and appearing in concentrated locations.

This leads on to a second point, which is that erosion within that sort of upstream shoulder can lead to spectacular local deformations of pitching and settlements of the order which Mr Claydon has been mentioning, often happening when the reservoir has been drawn down. I think the important question is what can we do about it, and I think the answer is: very little. If it is sufficiently serious to justify filtering and putting a whole new slope protection system in, you might then pin the fines down and stop them moving out of the shoulder, but you still can't stop them moving within the shoulder and without major grouting programmes I think it is something you have to live with. It may enforce consideration of discontinuing the dam, if it is serious.

A final point on filters. As often, there are two lines of approach - the practical and the theoretical - in our time. The practice of filters in dams was developed in this country in the 19th century and up to about the end of the last war ,to the stage where, with most of the common materials, we had rules of thumb and were taught how to do it. We now have good theoretical insights into the particle sizing of the various elements of filters for various duties, but I think there's very little on the effective life of a filter. It seems obvious that, if the purpose of a filter is to stop fines from moving past it, it's going to accumulate those fines somewhere. It is odd that we engineers don't ask ourselves about the life of the filters that we use in various circumstances. I would suggest that the thickness of the layers of the different sizes is important because it offers more space for fines to be accumulated, if they can accumulate in such a way that the water can still move through. I'd be very interested to know if any work has been done on the theory of the effective life of filters in dams. It could perhaps come from the theory of water filtration and it may be that the sort of experiments that Dr Schultz and Professor Brown have done could be extended to take in the lives of filters.

MR C C D KU (Hong Kong Water Supplies Dept)

Since Mr Kennard mentioned in his paper the dam in Hong Kong having this problem, I would like to provide up-to-date information. At present, the drainage system is working very well. We monitored the piezometric readings and we find that pore pressures have been lowered since the introduction of this drainage system, particularly after heavy rainfall. The penetration and all the problems caused by penetration from the surface have been greatly reduced. On the other hand, we are still not sure of the seepage through the dam core itself and eventually, after further inspection, we have decided to lower the spillway level in order to reduce the seepage through the core. We found a serious seepage point, at about 1 metre below the top water level, so we lowered the spillway in order to ensure that the seepage through the core was reduced.

M F KENNARD (Rofe, Kennard & Lapworth)

I'd just like to thank Mr Ku for that last comment. The question of seepage through the core is slightly different from the effect of rainfall, but they do combine of course in increased pore pressures in the downstream shoulder, so it's very interesting to know that the drainage aspects are satisfactory.

D B WICKHAM (North West Water)

I would just like to make two further points. The first one on the question of drainage: at Haweswater there is no free drainage, all the seepage water is pumped out by electric pumps running on a level control system, so there is always free water standing in the body of the dam. It is perhaps significant, I think, that one of the reasons the louvres were put on the doors was that condensation was playing havoc with the electric controls, as much as for the security of the dam structure.

The other point is the question of the trials that we carried out. In 1982, the intention was to test three different manufacturers' materials and carry out comparative tests. What actually happened was that because we used the individual manufacturers as contractors, we found ourselves testing the contractors' workmanship, not the materials. It wasn't until 1985 when we carried out tests on materials using the same contractor to apply them that we were able to make sensible tests of the materials.

WRITTEN CONTRIBUTIONS

B A HUTCHINSON (City of Bradford Metropolitan Council)

In Paper 5.1 by Mr Tedd, Mr Claydon and Mr Charles, the supervising engineer inspected the addit found after the dam had been drawn down. Could you please comment on precautions taken and system for entry into this confined space. What equipment was used/available on site for safety and rescue purposes?

MR J CLAYDON (Yorkshire Water)

The Supervising Engineer inspected the adit with one other YWA employee. The initial perception of risk was that the roof might be dislodged, as broken wooden props could be seen from the entrance. For this reason breathing apparatus was not used as it could have been an encumbrance. Both people had protective clothing and a digital readout gas detector was carried. Oxygen measurements were noted at frequent intervals and they would have left immediately if the oxygen level had reduced. It did not do so. No explosive gases were registered. Other YWA staff remained at the adit entrance to provide assistance if required.

PROCEEDINGS: TECHNICAL SESSION 6

EMBANKMENT DETERIORATION

PAPERS:

DISCUSSION:

INVESTIGATION AND REMEDIAL WORKS TO THREE EMBANKMENT DAMS

D Gallacher MSc DIC CEng FICE FIWEM (Partner)

Robert H Cuthbertson & Partners

SYNOPSIS

Problems arising from deterioration of elements of three embankment dams with puddle clay cores and remedial measures to one of them are described. The dams are Coulter (Strathclyde Region, Scotland), March Ghyll (West Yorkshire) and Lower Oakdale (North East Yorkshire). Details of investigations carried out and results of monitoring are given.

INTRODUCTION

1. A survey of unsatisfactory performance of U.K. earth dams (1) indicated that the primary cause of in-service failures and serious incidents, excluding construction failures, arose from internal erosion. In a recent survey carried out to assist with the preparation of an Engineering Guide on the Safety of Dams, internal erosion, seepage and leakage were confirmed as primary causes of significant deterioration or major incidents in dams.

2. This Paper describes investigations at three embankment dams with puddle clay cores where core problems were disclosed due principally to internal erosion. In two cases (Coulter and March Ghyll) investigations were recommended in reports of statutory inspections. In the third case (Lower Oakdale) the investigation arose as the result of an incident at the dam requiring the reservoir to be drawn down.

3. Remedial works to the core at Coulter Dam, fissure grouting of bedrock below the core and the raising of the top section of the core are described. Results of instrumentation to monitor performance are also given. Remedial measures to the other two dams (March Ghyll and Lower Oakdale) are discussed.

COULTER DAM

BACKGROUND

History

4. Coulter Reservoir is in Strathclyde Region, Scotland about 7 km south of the village of Coulter. The dam is an earthen embankment, 24 m high and 225 m long, with a central puddle clay core, and upstream and downstream slopes of 3:1, and 2.25:1 and 2:1 respectively (Fig.1).

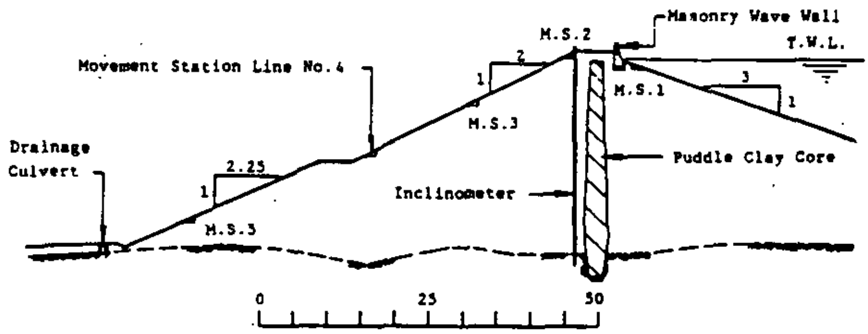


Figure 1: Typical Embankment Section

5. The dam was completed in 1907 and as early as 1912 it was reported that the extent of make up of the dam, most likely the puddle clay core, indicated that something was seriously wrong in the puddle wall or trench. References to sudden subsidence and issue of dirty water at the tail of the bank were also made. There is no record of the remedial work carried out but grouting is referred to in correspondence.

6. The next record of investigatory work is on a drawing dated about 1930 which shows trial pits across the embankment crest presumably to locate the puddle clay core. Survey in 1968 referenced to the 1930 trial pit records showed that the embankment had been made up but not the wave wall. Further references refer to grouting within the culvert to seal leaks (1930) and make up of a length of the core with 280 tons of puddle clay (1936).

7. In the 1968 Statutory Report, 10 years of movement records were reviewed and it was concluded that embankment movement was continuing to such extent that there was a risk of sudden failure particularly in view of the narrowness of the core and the apparently unyielding nature of the embankment shoulders. Crest markers were settling and moving back downstream horizontally with the largest movements occurring where the total subsidence was greatest in the wave wall. That report recommended a programme of investigation including sinking of boreholes and trial pits to be followed by implementation of measures to stabilise and safeguard the dam.

8. Recommendations were also made to augment the capacity of the overflow works and to maintain the reservoir at a level at least 2.3 m below overflow sill level until remedial and improvement works had been carried out.

Further Investigations

9. Further investigations of the embankment were carried out in 1969/70. Field work included 7 trial pits on the embankment crest to ascertain the position of the core and the probable extent of settlement, and 17 boreholes in the embankment core and shoulders to determine the strength of the embankment material and pore water pressures within the embankment. Flows from drains were also measured and analysed.

10. One trial pit was excavated to a depth of 4.6 m in the core and, the others only to top of core level. In the deep pit the core was 1.07 m below crest level and there was evidence that it had been made up over a further depth of about 2 m, and that the make up depth was offset towards the reservoir in relation to the lower section of core. The deduced vertical and horizontal movements of the core are indicated on Fig.2.

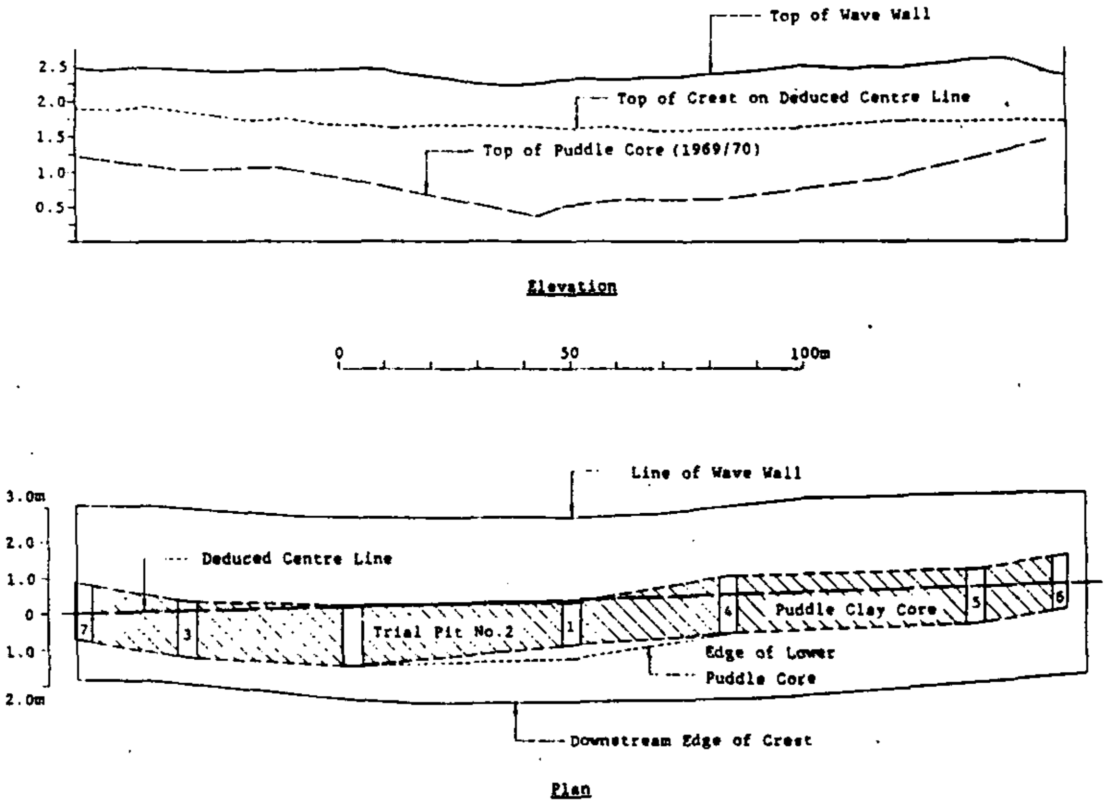


Figure 2: Movements of Core

11. Continuous U4 samples were taken in the early holes in the core but this was superseded by piston sampling to achieve better core recovery in weak areas. Zones of weak material with high moisture content were found at intervals throughout the length of the core. Changes in water levels in core boreholes also indicated that there were leakage zones but their continuity was not known.

12. The embankment shoulder boreholes disclosed a loose to medium dense gravel with cobbles and boulders. The underlying overburden to rock head was also mainly gravel without any extensive zones of soft clay or silt.

13. It was concluded that the embankment shoulders were stable and that the vertical deformation at the crest was due to core settlement and also possibly settlement in the fill material immediately downstream. The leakage quantity through or under the core could not be determined due to collection difficulties and the influence of ground water, but it was considered that this leakage combined with removal of core and embankment material could present a serious risk to the dam safety by erosion.

14. Grouting remedial works to the core and rock below were recommended in conjunction with the raising of the embankment core and crest, reconstruction of the wave wall which retains the upper part of the dam, and collection of leakage/drainage at the downstream toe. Reconstruction of the overflow works was also recommended. The alternative option to carrying out the above works was breaching of the reservoir but this was not an economic solution.

REMEDIAL GROUTING WORKS

Preliminary Works

15. Remedial grouting works were carried out between March 1976 and July 1977 at a cost of about £320,000 following completion of reconstruction of the overflow works. The major objectives of the grouting works were to reduce leakage through and under the puddle clay core and to strengthen the core where necessary. A drainage collection culvert founded on bedrock parallel to the downstream embankment toe had been constructed under the overflow works contract and readings of drainage, reservoir level and rainfall taken over a period of about 9 months to establish the pre-grout conditions.

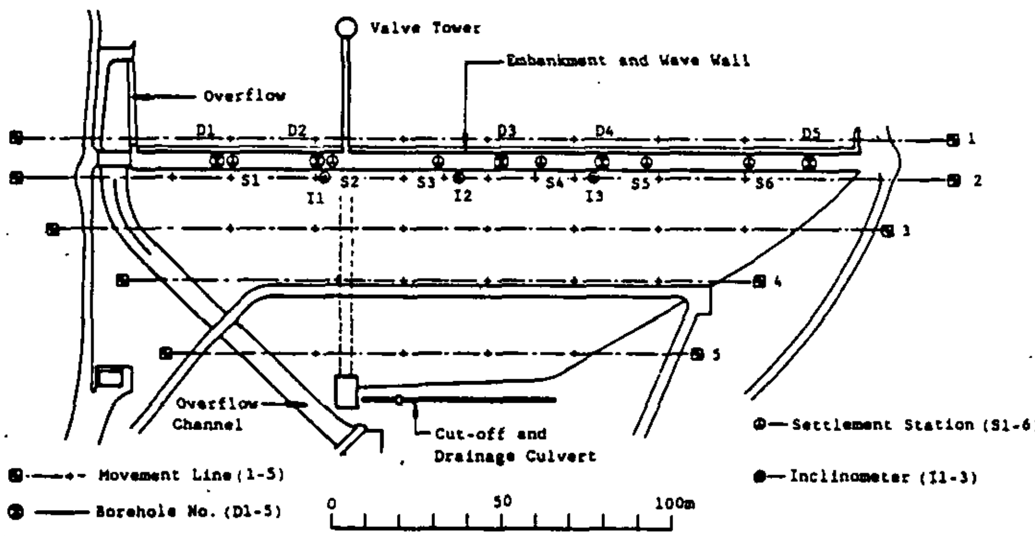


Figure 3: Plan of Dam

16. Before grouting commenced 5 further boreholes (D1-5, Fig.3) were put down in the core to bedrock with continuous undisturbed sampling to supplement information from the previous investigation. Samples were extruded on site and tested for undrained shear strength (Vane Test) and moisture content. Other samples were used for off-site testing (shear strength (Undrained Triaxial), moisture content, and Atterberg limits). This investigation also disclosed zones of very soft core material and there were areas where no recovery was possible. The very soft material was found at discrete horizons both above and below the original ground levels between zones of firmer clay. An average of 23% of vane test samples were very soft with the maximum percentage of samples in hole D1(47%) and the minimum percentage in hole D5(6%). A summary of the pre-grout test results is given below.

Shear strength:	Vane		Undrained triaxial		
	Av	28 kN/m^2	Av	33.6 kN/m^2	
	Min	4.8 "	Min	3.5 "	
	Max	74 "	Max	86 "	
Moisture Content:	Av	35%	Liquid limit		63%
	Min	28%	Plastic limit		24%
	Max	69%			

17. Instrumentation (inclinometers, settlement stations, and lines of movement stations) were installed to monitor horizontal and vertical movements (Fig.3).

Grouting

18. The puddle clay core was grouted using the tube-a-manchette (T.A.M.) method as this permits grout to be injected at selected discrete horizons. The bedrock below the core was grouted by normal fissure grouting except in the weathered top zone where the tube-a-manchette method was used. Control boreholes were put down in the core as T.A.M. grouting proceeded to check its effectiveness and the need for further injection. Rock fissure grouting followed T.A.M. grouting and on completion of all grouting the top level of the core was raised with a bentonite/cement slurry to give the design freeboard above spill level, and 5 boreholes (3 in core and 2 in downstream shoulder just clear of crest) were sunk to bedrock and piezometers installed for future pore pressure monitoring.

19. The core was divided vertically into 4 sections and into three horizontally for grouting reference (Fig.4). T.A.M. 75 mm PVC grout tubes were installed in bentonite/cement at 3 and 1.5 m centres in sections 1 and 4, and 2 and 3 respectively with provision for grout injection at 0.3 m vertical intervals. T.A.M. grouting was carried out in stages by injecting at a sleeve a computed volume of grout (about 7.5% of the core volume for holes at 1.5 m centres); grouting was stopped if the specified maximum pressure was reached. The grout mix was 1:1.09:8.26 parts bentonite:cement: water by weight which had a 28 day shear strength of about 50 kN/m^2.

20. T.A.M. grouting started with primary holes (6 m centres) in section 2A working upwards from the puddle clay/rock interface followed by secondary holes (3 m centres) with every sleeve being tested for grout. Some breakages of PVC grout tubes (Class B) occurred which required their replacement with Class D pipe. Breakages occurred when the packers were raised after grouting of a sleeve; the most likely cause was pressure reduction within the pipe below the packers when they were raised. No breakages occurred with the stronger pipe.

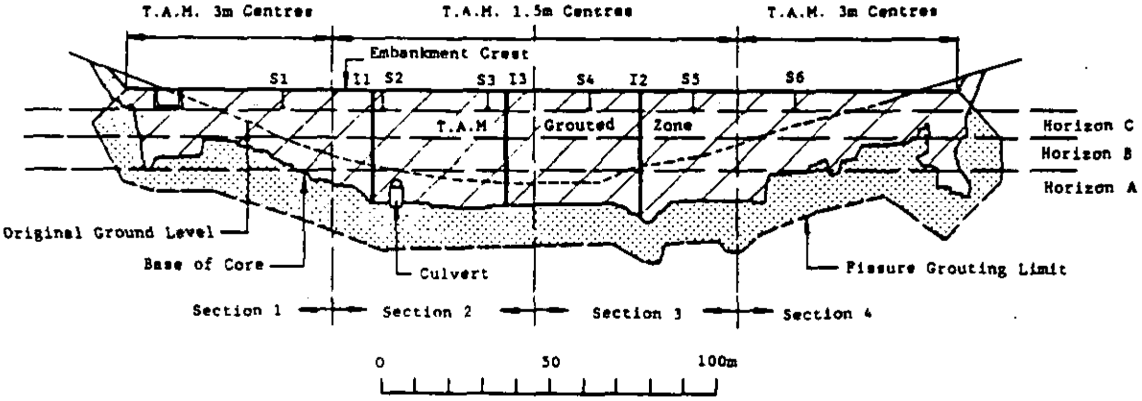


Figure 4: Elevation of Cut-off

21. A longitudinal crack and a depression were observed at the crest in Section 2 when second stage grouting was in progress in Section 2A and first stage in Section 3A. Grouting in Section 2 was stopped but continued in Sections 3 and 4 following a modified grouting procedure to reduce and give improved control of embankment and core movements. With the latter procedure grout was injected more evenly into the core and it was used to complete the T.A.M. grouting in Section 2 and for Section 1. The total volume of grout injected by T.A.M. was 852 cu m which represents about 6% of the estimated volume of the clay core; some of this grout may have permeated into the embankment shoulders.

22. T.A.M. grouting was extended up to 4 m from the embankment crest. It was not possible to grout above this level with the low overburden cover but it was felt that leakage through this zone would be minimal with the low reservoir head. The core was raised with a bentonite/cement slurry over the full length of the embankment to give a minimum freeboard of 1.18 m above reservoir spill level. A trench was dug to a minimum depth of 0.3 m into the core and filled in 20 m sections with slurry. The slurry bentonite: cement ratio was 1:3 and the water:cement ratio 1:3.7 all by weight. The slurry was covered by geofabric filter sheet with a minimum 0.2 m gravel protection layer above.

23. Rock fissure grouting was carried out on completion of T.A.M. grouting using the same grout pipes. Stages of 3 m were drilled, water tested and grouted where necessary and re-water tested; each stage was grouted and tested before the next stage was drilled. Grouting was carried out to an average of 5 stages below rock level. Grout mixes were 5:1, 2:1 and 1:1 water:cement all by volume with 5% bentonite to weight of cement added as a lubricant to improve grout penetration. A total of 363 stages were drilled of which 269 required grouting. 87.3 tonnes of cement were used in the fissure grouting giving an average take of 0.324 tonne per 3 m stage grouted. The maximum weight of cement used in any one stage was 0.8 tonne. The effectiveness of the grouting was checked by tertiary holes and very little further grouting was required.

Control Boreholes and Instrumentation

24. As T.A.M. grouting proceeded control boreholes were put down in the core to make a before and after grouting comparison. A lower average shear strength was obtained from these samples than from samples from the pre-grout holes but the results were more consistent. Very soft areas were still found and further T.A.M. grouting was carried out at selected locations. Bentonite/cement grout was evident in many samples where it had displaced puddle clay or filled voids.

25. On completion of all grouting 3 further boreholes were sunk in the core by continuous piston sampling and 2 in the embankment shoulder just downstream of the crest by shell and auger drilling. Piezometers were installed in all final holes for future monitoring. The final holes in the puddle clay still disclosed very soft zones. The average permeability of the lengths tested after grouting were about 50-60% of the pre-grout permeability.

26. Inclinometer access tubes (I1, I2, and I3) were installed in the downstream embankment shoulder near the crest and extended about 2 m into bedrock. Deflection readings were taken during and after grouting at 1 m intervals over the full depth of each tube. The horizontal deflections at a depth of 1 m below ground level are shown in Fig.5 covering the grouting period. Deflection was in direct proportion to the amount of T.A.M. grout injected but were more or less unaffected by the rock fissure grouting. The maximum deflections of I1, I2 and I3 were 95, 113, and 103 mm respectively. Good correlation was obtained with the downstream crest movement stations.

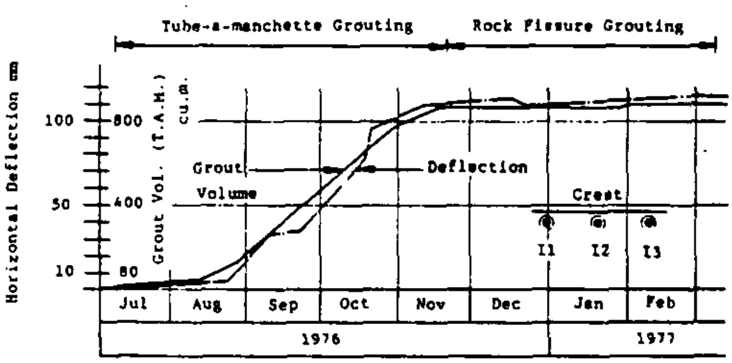


Figure 5: Horizontal Deflection and Grout Volume

27. Horizontal and vertical movements were measured at 5 lines of surface movement stations. The maximum horizontal movements of the upstream and downstream crest stations were 40 and 125 mm respectively (lines 1 and 2). The maximum horizontal deflection half way down the downstream embankment shoulder was about 25 mm and with no deflection at the toe. Maximum vertical settlements of 13 and 15 mm occurred on lines 1 and 2 local to the crest but elsewhere vertical movements were very small.

28. Daily measurements of drainage at various points, and rainfall were taken before, during and after grouting. A formula was established to relate drainage, rainfall and reservoir level in the form:

$$Q = K_1R + K_2H$$

where Q = Drainage (m^3), R = Rainfall (mm), H = Av Reservoir Level (m)

and K_1 and K_2 are constants

Analysis of the pre-grout and post-grout conditions gave the following results:

Pre-grout $Q = 0.29R + 17.27H$
Post-grout $Q = 0.29R + 3.97H$

The post-grout to pre-grout percentage leakage from the reservoir is the ratio of the reservoir constants i.e. 3.97/17.27 = 0.23 or 23%

29. Piezometers were read weekly before, during and after grouting works to monitor pore pressures within the embankment and rock foundation. The estimated free water surface in the downstream embankment was lower after grouting indicating that leakage from the reservoir had been reduced.

MARCH GHYLL DAM

HISTORY

30. March Ghyll Reservoir is in West Yorkshire to the north of Ilkley. The dam is an earthen embankment L shaped in plan with a central puddle clay core, 20 m high and 450 m long, and with upstream and downstream slopes of 3:1 and 2:1 respectively. The dam was completed in 1906 and in 1971 the dam crest including the wave wall was made up by about 1.05 m to reinstate settlement which had occurred since construction. The reservoir overflow was reconstructed at the same time.

31. In the 1986 Statutory Report a recommendation was made that an investigation of the embankment puddle clay core and fill material should be carried out local to an area in the embankment crest where continuing and differential settlement was evident from levelling records. Investigation of a slip in the natural ground downstream of the embankment and continued monitoring of level and movement stations were also recommended.

INVESTIGATIONS

General

32. Levelling monitoring pins were established in the clay core in November 1984 (Fig.6).

33. The embankment investigation was concentrated at the area of maximum measured settlement (Pin 3) of the main embankment crest and was extended to include a borehole on the flank embankment between Pins 11 and 12 opposite a wet area at the downstream embankment toe. Three trial pits were also dug on the embankment crest to check the detail of the core raising carried out in 1971. Further visual inspections of the embankments and overflow works were made and the results of monitoring of movements and piezometers were reviewed.

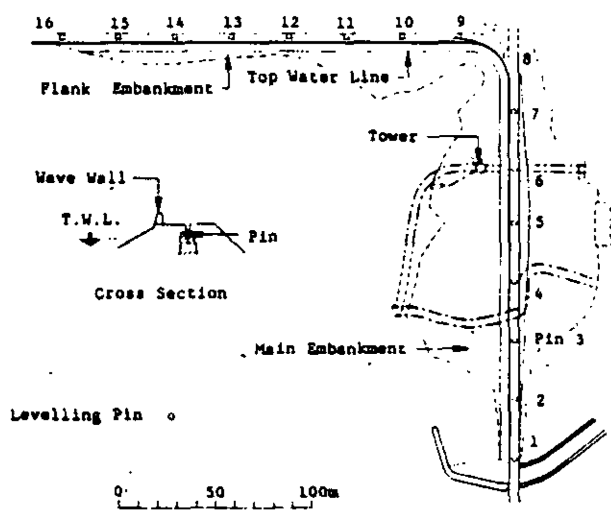


Figure 6: Plan of Dam

Boreholes and Trial Pits

34. A borehole was put down in the core local to the area of maximum settlement and continuous piston samples were taken. The undrained shear strength was determined on site by Vane Test where this was possible. At a depth of 2.5 m the core material contained sand and gravel with some free-water and between 2.5-6 m the material was variable with organic inclusions. Between 9.5-14 m the core was very soft with evidence of free-water and between 14-18 m the core was soft to firm with signs of disturbance but no free-water. Below 18 m the core quality improved becoming firm to stiff. The conditions found confirmed that there is a weak zone in the core in the area of maximum continuing settlement without there being complete breaks indicating hydraulic fracture as in the Coulter core. Similar weak conditions were not found in the other core boreholes.

35. Boreholes in the embankment shoulders upstream and downstream of the weak core zone disclosed variable fill material with very soft areas where in some cases U4 samples could not be obtained. There were pore pressures in the downstream embankment shoulder at about the level of the doubtful core zone indicating leakage/seepage from the reservoir and/or possibly water ingress from the adjacent hillside where a slip had occurred.

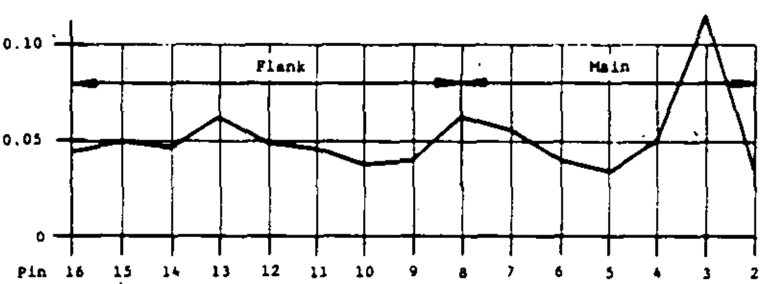
36. A trial pit and borehole on the flank embankment showed that the puddle clay core had not been made up at these points with the same material as the main embankment core in 1971. The make up core material on the flank embankment is of much inferior quality and therefore it was possible that the effective top of the core was low over most of the length of this embankment. There were several wet areas at the toe of the flank embankment which could be leakage/seepage from the reservoir.

37. The embankment conditions at the area of maximum settlement on the main embankment and at the point of investigation on the flank embankment gave cause for concern regarding embankment stability. The limited testing of the embankment fill showed that only small pore pressures could be permitted in the downstream shoulder without reducing its factor of safety to an unacceptable level. The extent of water penetration into the downstream shoulder was not known except at the small areas investigated. Overtopping of the core of the flank embankment by high reservoir levels appeared to be possible which could raise pore pressures in its downstream shoulder thereby increasing the failure risk.

38. Sandy clay overburden about 3.4 m deep to rock head was found at the slip area on the hillside at the right abutment of the main embankment. Moisture content of samples increased with depth to a maximum of 34% where the undrained shear strength was only 8 kN/m^2. The most likely cause of the slip is water from the hillside lubricating the soil/rock interface and softening the soil above. The source of water could either be from the reservoir or rainfall with the latter the more likely contributary cause of the slip as the incident was reported after heavy rainfall.

<u>Instrumentation</u>

39. Settlement records were available for a period of about 2.3 years at the time of the investigation. Settlement Indices (SI) for each levelling pin are given in Fig.7.



<u>Figure 7: Settlement Indices (SI) (Nov '84-Feb'87)</u>

40. The maximum heights of the main and flank embankments are about 20 and 6.15 m at pins 4 and 14 respectively and there is a deep puddle clay cut-off trench over the full length of both embankments varying in depth from 13.5 to 29 m. The core in the cut-off trench is therefore a large proportion of its total height to the embankment crest and consequently, Settlement Indices (SI), as defined by Charles (2) have been calculated using the full core height, i.e. cut-off trench plus embankment.

41. Settlement Index (SI) is analogous to the coefficient of secondary compression (C) for clay soils in one dimensional compression to represent soils in the field. Charles (2) suggests that where measurements of crest settlement give Settlement Index values greater >0.02 the possibility of some other mechanism causing settlement besides secondary consolidation by creep should be considered. In this case all Settlement Index values exceed the above amount (0.02) with the maximum value (0.115) occurring at the point of maximum settlement (Pin 3).

42. The conditions found in the core and downstream embankment boreholes local to Pin 3 and the Settlement Index at this point support the view that some other mechanism is causing the magnitude of the continuing settlement in this area. The most likely cause is erosion, and softening of the core and downstream embankment shoulder. The phreatic line through the embankment at Pin 3 (Fig.8) indicates a seepage flow through the core rather than from the hillside or rock below.

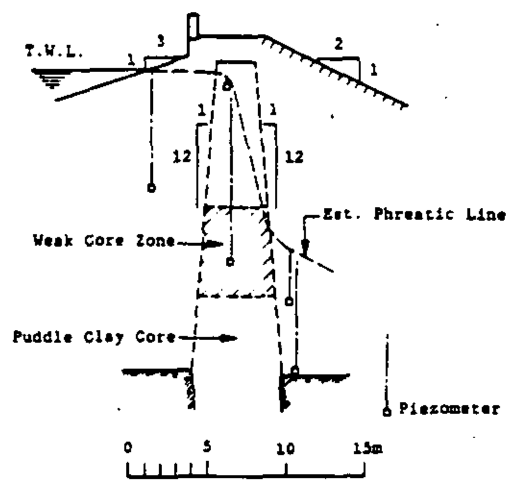


Figure 8: Pore Pressures at Maximum Settlement Area

Recommendations

43. Recommendations were made to investigate the core on the flank embankment by further trial pits, and, by boreholes, the downstream shoulder of both embankments and the natural ground behind the slip area. It was also recommended that settlement pins and piezometers continue to be monitored and that the reservoir water level be controlled to a maximum elevatopm of 1 m below overflow level until the core on the flank embankment has been raised and any remedial measures arising from the further investigations are complete. Field work is complete and laboratory testing is in progress.

LOWER OAKDALE RESERVOIR

HISTORY

44. Lower Oakdale Reservoir is in North East Yorkshire near Osmotherly. The dam is an earthen embankment about 11 m high and 50 m long with a central puddle clay core. In November 1986 a major sink appeared in the central area of the embankment crest just downstream of the core (Fig.9). The reservoir was drawn down immediately to safeguard the dam and an investigation was carried out.

INVESTIGATIONS

Preliminary Investigation

45. Investigation using a flow through sampler showed that the major sink had been caused by erosion through the core at high level below a very substantial wave wall constructed in 1970 when the overflow works were enlarged. Depressions on the upstream and downstream embankment slopes in line with the sink were evident and voids were found below the downstream slope to a depth of about 3 m. The small draw-off/scour culvert was inspected internally and some leakage into it was found. The culvert is constructed from cast iron segments which are sound apart from corrosion of fixings and gaps between units at bends.

46. A much smaller sink was also evident on the embankment crest adjacent to the overflow channel wall (Fig.9) and its most likely cause was leakage at the core/wall junction.

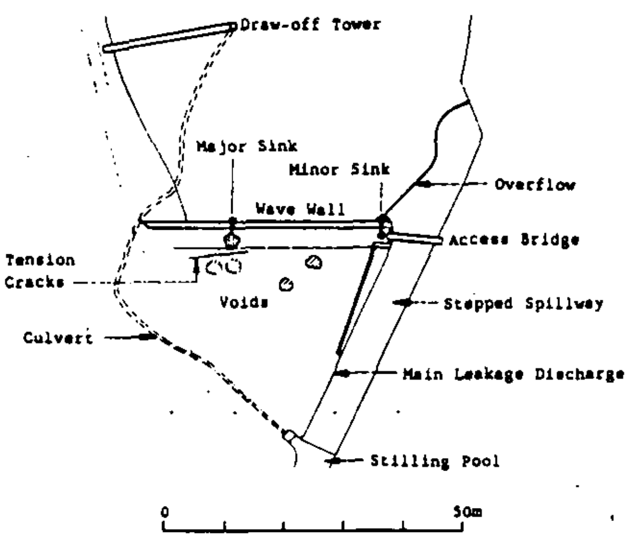


Figure 9: Plan of Dam

47. Further investigation works were recommended to determine the extent of remedial works to put the reservoir back into service. Pending this work the water levels of the Lower and Upper Oakdale Reservoirs, which are in series, were lowered to 2.5 and 1 m below overflow level respectively.

Further Investigation

48. The embankment core and shoulders were investigated by boreholes with continuous sampling in the core. In addition the flow through sampler was used in the embankment core and shoulders to locate voids but this was not fully conclusive and therefore a controlled impounding test was carried out to confirm leakage paths and monitor piezometers.

49. The clay core had significant voids particularly near the top and with other very permeable areas throughout its extent. The core material is a very weak soft grey clay with a high organic content and with moisture contents at or approaching its liquid limit in several areas. The core is founded on mudstone over its length except at the north end adjacent to the culvert where it appears to terminate in loose sand.

50. The embankment shoulders generally comprise medium-coarse sand downstream and a more clayey material upstream. Voids were found in both shoulders which contained weak areas; the downstream shoulder became progressively wetter with depth and the impounding test showed that it is subject to high pore pressures due to leakage.

51. The further investigation revealed that voiding and areas of high permeability in the core were more widespread than was at first envisaged. It is not considered feasible to repair the core by grouting and the more positive solution of providing a steel sheet pile cut-off within the clay core has been recommended. Voiding around the culvert is less significant than at first envisaged.

52. Estimated capital expenditure costs for repair of the embankment to put the reservoir back into service and for abandonment are £166,000 and £80,000 respectively. Since preparation of the estimates a landslip has occurred just upstream of the embankment which will require stabilisation in the long term particularly if the reservoir remains in service and therefore the former estimate will require to be increased. The reservoir is used for compensation purposes and its future is under review by the Authority.

CONCLUSIONS

53. Extensive investigation at Coulter, both before and during remedial grouting works, disclosed weak zones in the core with evidence of internal erosion probably initiated by core fracture arising from settlement of the core within very stiff embankment shoulders. The sandy gravel downstream embankment shoulder presents a large holding area for migration of fines from the core which makes detection of internal erosion difficult. Repair of the core at Coulter presented problems despite the refined grouting technique adopted and the success of the remedial work is not easy to assess. Migration of grout from the core permeated into the embankment shoulders which will have widened the effective width of the core in weak areas. Assessment of drainage measurements indicates that core and bedrock grouting has significantly reduced leakage from the reservoir thereby reducing risk of internal erosion. Monitoring of leakage covering a longer period is necessary to determine long term performance subsequent to repair.

54. Investigation of the core at March Ghyll, in the area of maximum continuing settlement, has shown that there is a weak zone about 4-5 m deep and 20-30 m wide. Very soft conditions were found in the core but no complete breaks as at Coulter and Lower Oakdale. The core condition does not give cause for immediate concern but investigation works and monitoring are continuing. Grouting as at Coulter would not be a solution in this case; complete replacement of the core function is required. The Settlement Indices for the core settlement stations, excluding the point of maximum continuing settlement and based on the full depth of the core including the section in trench, are about 2-3 times greater than the limit suggested by Charles (2) beyond which the possibility of some other mechanism causing settlement besides secondary consolidation may exist. The investigations to date have disclosed nothing as to what such other causes may be and further monitoring of long term settlement is recommended.

55. Lower Oakdale dam suffered a complete internal erosion failure at least in one area and possibly in others. The strength of the core was consistently weakest of the three dams considered; all triaxial samples, where tests were possible, had shear strengths of less than 7 kN/m^2. Complete renewal of the embankment core is required to put the reservoir back into servic; as at March Ghyll grouting is not a suitable solution.

56. The cases demonstrate the different character and consequences of internal erosion which are not susceptible to the same form of treatment. The value of continual monitoring of settlement and leakage is underlined if deterioration is to be arrested at an appropriate time and effective remedial measures are to be undertaken. The cases draw attention to the importance of maintaining good records of performance to create a base for interpretation of when deterioration reaches the stage which demands remedial action.

ACKNOWLEDGEMENTS

The cases described in this paper are published with the permission of Strathclyde Regional Council and Yorkshire Water Authority and their co-operation is gratefully acknowledged. The assistance provided by and the discussions with the current Inspecting Engineer for March Ghyll and Lower Oakdale Reservoirs, W P McLeish, are also gratefully acknowledged.

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RECONSTRUCTION OF OLD MINING DAMS IN THE HARZ MOUNTAIN AREA, FRG

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SYNOPSIS

Between 1530 and 1810 more than 100 earth- and rockfill dams were built in the Harz Mountains for mining purposes. The up to 16 m high dams have grass sod sealings and wooden outlet facilities. Today 75 dams are still in order and declared as historic monuments. A number of these dams have undergone reconstruction. Leakage detection of the grass sod sealing was done by application of gas injection techniques. The wooden outlet pipes were inspected by video equipement. The repair works are described.

RECONSTRUCTION OF OLD MINING DAMS IN THE HARZ MOUNTAIN AREA, FRG

GENERAL

1. Between 1530 and 1810 more than hundred earth and rockfilles dams were built in the Harz Mountains for mining purposes. About 75 of these dams are still in existence and declared as historic monuments. Some are used for local drinking water supply.

2. The dams causing trouble are of three different types. All of them have grass sod sealings and wooden outlet facilities (figure 1a + b). A couple of dams are of the type according to figure 1c with the wooden control shaft in the centre of the dam and the sod sealing at the upstream face. In order to provide the necessary water for the ever deepening mining shafts it was quite common to heighten the dam. The method of heightening is most probably according to figure 1d. Quite a number of the dams have been heightened several times.

3. In general, the dams are in good condition. The grass sod sealings proved to be very durable provided they are well protected against wave actions and sealed against air. The same is valid for the oak-wooden outlet works as long as they are well under water.

MINOR REPAIRS NECESSARY

4. Due to their age at most of the dams minor repairs were necessary, especially at the masonry parts, i. e. the spillway and the upstream protection wall at the crest of the dam. The old masonry normally consisted of dry masonry without any mortar, sometimes with moss. In mountainous region of the Harz with severe frost conditions it is advisable to reconstruct the masonry in the old manner but without moss which is difficult to collect today. The kind of repair using mortar as executed during the last 50 - 80 years shows a considerable decay of the mortar layers. Therefore the much more expensive reconstruction without mortar is preferred.

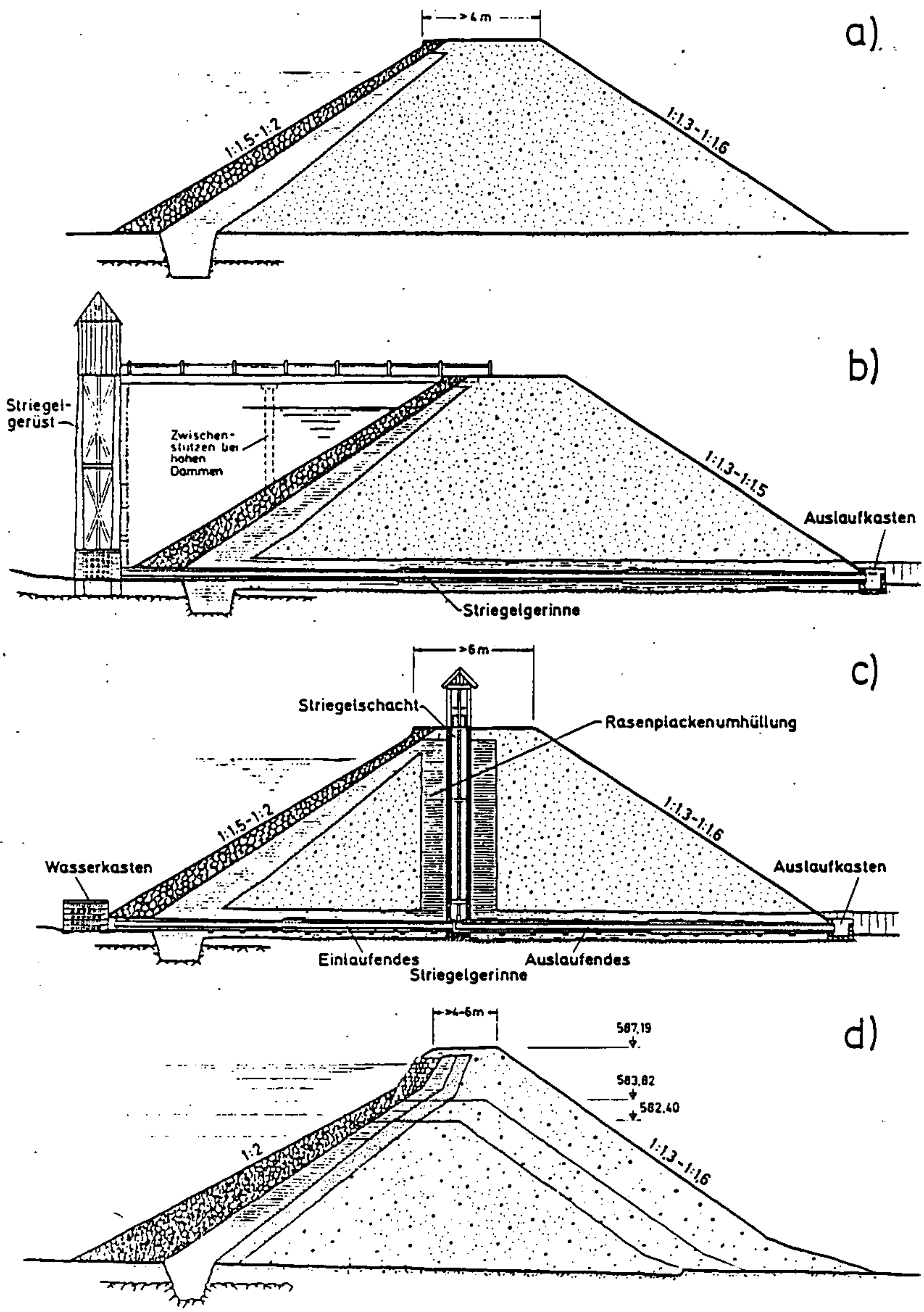


Figure 1: Typical cross sections of mining dams in the Harz mountain area

5. The wooden parts of the spillway structures and of the outlets have to be renewed from time to time. Today oakwood of the North American Rubiniewood (rubinia pseudo acazia) is used.

6. At a number of dams the original spillways were too small. In the past, they had to be enlarged to prevent overtopping. For several dams they had to be enlarged still further in order to meet the standards of present day hydrological knowledge. Though the dams are more than 300 years old, settlement is still taking place, because the dams were not compacted during construction. In order to keep the spillway capacity it is necessary to add further material on the crest of the dams.

7. Of the existing dams none is showing foundation problems. It is known that there were problems of this kind, but those dams were abandoned or replaced by construction of a new dam. The ruins of the abandoned dams are still standing.

MAJOR REPAIRS NECESSARY

8. There is a small number of dams showing growing leakages. Quite expensive reconstruction had to be executed in order to prevent accelerated erosion and depletion during summer time.

9. There are more or less three different types of damages:
a) growing leaks in the impervious layer
b) decay of the wooden outlet works
c) blockage of the outlet works

Leaks in the impervious layer

10. For all the dams their general type is known, but it is not known exactly how and how often the particular dam was heightened. This is of importance since the joints between the original dam structure and the added part are prefered zones of leakage. The first and main problem is to determine where the leakage occurs. In order to avoid too much digging up, a special gas injection technique is applied to detect the weak part of the sealing layer.

11. For this purpose carbondioxid (CO_2) gas is employed because it is harmless. It is delivered to the dam site in a deep cold, i. e. liquid condition by special trucks. Gas injection is executed from the upstream and/or downstream slope of the dam by short injection probes (l = 0,60 m, Ø = 30 mm) placed inside of drilled holes of about 0,8 m length. The pressure applied depends upon the permeability and moisture of the earth material and is normally between 0,5 to 1,0 bar. Cracking of the earth structure has to be avoided. In soils of good permeability gas pressures as low as 0,2 bar may be used.

12. The injected gas moves preferably along horizontal layers. As soon as there is no pressure potential anymore, the gas diffuses in vertical direction. The injected gas disappears along the zones of higher permeability in which gas pressure and gas concentration increases. The gas propagation is measured by a grid of probes pushed 0,5 m into the surface using gas test tubes or gas interferometers or by determining the geoelectrical potential. The lines of equal gas concentrations are plotted to show the zones of higher permeability.

13. The gas is injected into the dam in intervals of about one hour about three times. It spreads through the fill according the direction of the injection and flows preferably along horizontal interfaces or along leakage areas, breaches or fissures. Thus the weakened area of the sealing layer can be traced by an increase of the gas concentretion which is measured by another set of pipes placed along the upstream face of the dam. The measurements at the upstream face are done in intervals of 3 hours until there are practically no changes in the gas concentration anymore.

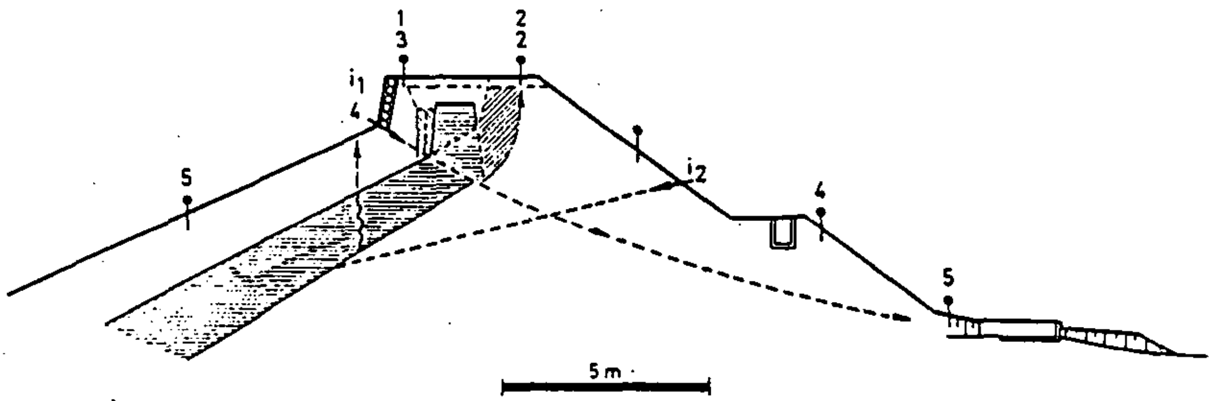


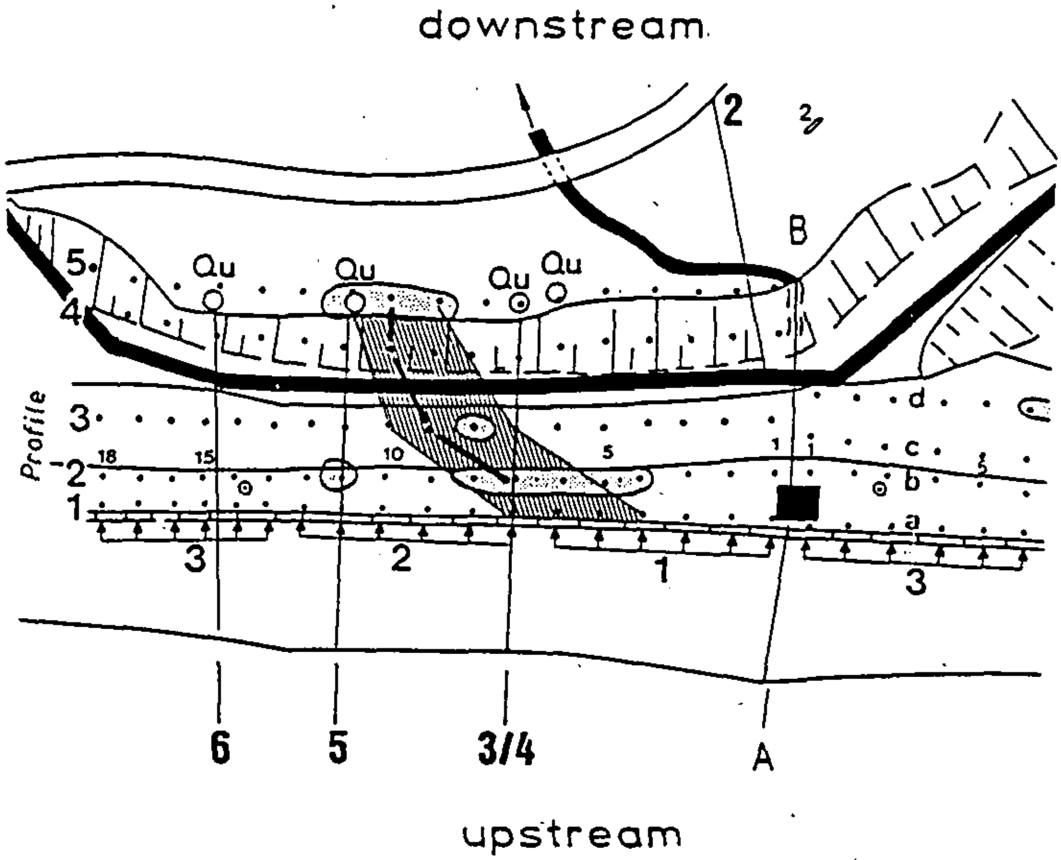
Figure 2: Arrangement of the injection and measuring pipes at the Schwarzenbacher Dam

14. The arrangement of the injection and measuring pipes at the Schwarzenbacher Dam are shown in figure 2. At each of the probes recordings were read at least three times. Per hour 40 m³ of gas = 80 kg were used. For the injection-time of 35 hours 1.400 m³ = 2,8 t of CO_2 gas was pumped into the dam structure. The gas migrates out of the dam within a few months. The results of the investigation at the Schwarzenbacher and the Upper Flambacher Dam are plotted in figure 3. As a matter of fact the applied gas injection method can not show exact results but gives an idea where to try first the repair of the sealing layer.

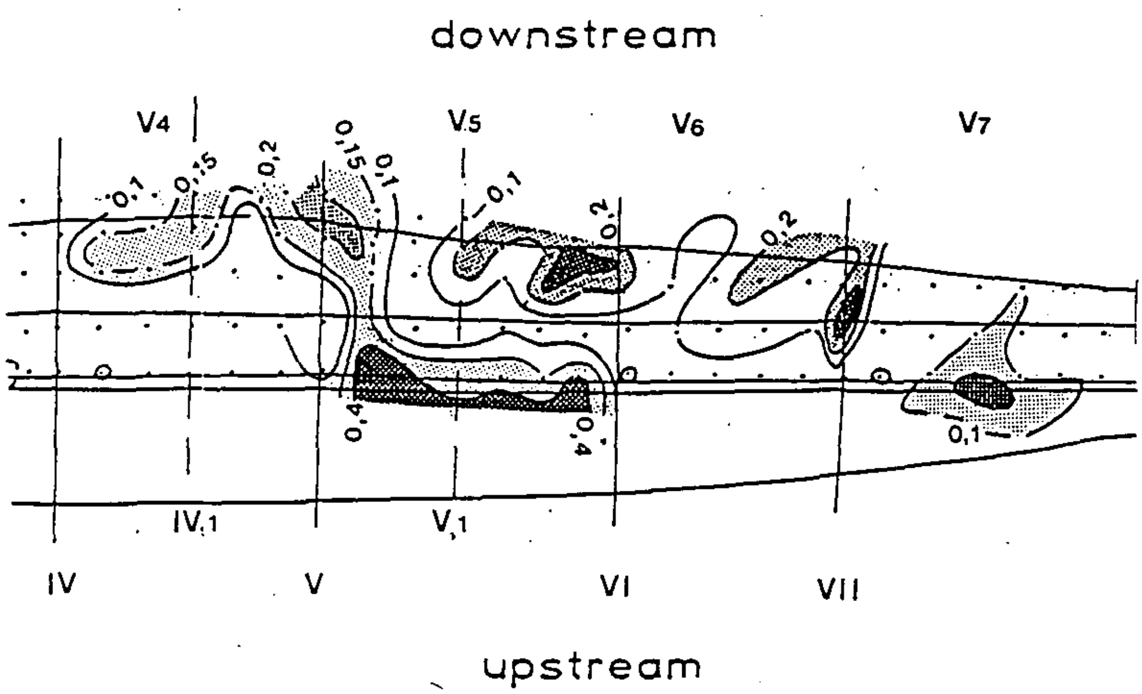
15. At the Prinzen Dam the leakages were observed 2 - 3 m below the crest, so it was quite sure that the weak zone was between the original dam structure and the section added later.

Repair of the impervious layer

16. At the Schwarzenbacher Dam, where the weak areas concentrate at two places, only a few meter below the crest the reconstruction is done by removing the protection layer and sealing the weak parts of the grass sod with natural clay and a non woven filter layer under the protection layer. We had a discussion with the officials of the Historical Monument Department who wanted the repair done by using grass sods in the old manner. But our argument was, that today there are no craftsmen having the experience in selecting the suitable grass sods and having the skill to place this type of sealing material, which was, in former times, a real "science" and well supervised by experienced mining engineers. Furthermore, the German regulations for dams prohibit the use of organic materials for dam construction.



a) Schwarzenbacher



b) Upper Flambacher

Figure 3: Gas concentration pattern at
a) Schwarzenbacher Dam, b) Upper Flambacher Dam

17. At the Prinzen Dam a trench along the whole crest was dug and filled by a special earth concrete, which was protected against the upstream and downstream embankment by a non-woven filter layer.

18. At the Upper Flambacher Dam we were able to convince the officials of the Historical Monument Department that it would be very difficult and expensive to repair the widespread leakages in the upper zone of the dam. So they finally agreed to lower the spillway crest in order to prevent further leakage except during bigger floods of very low probability.

Reconstruction of the wooden outlet works

19. To reduce the works of maintenance at most of the dams, the wooden inlet structure standing originally in front of the dam in the lake (figure 1b) has been replaced by cast-iron inlet valves driven by pneumatical equipment. None of the original structure of this type is still in order or working today. The dams with wooden shafts placed in the centre of the dam have still the original wooden installation which shuts off the opening in the wooden outlet pipe.

20. During the last hundred years or so in some cases some cast-iron pieces were installed but the friction between iron and wood showed negative results. There is no reconstruction or repair of this type any more.

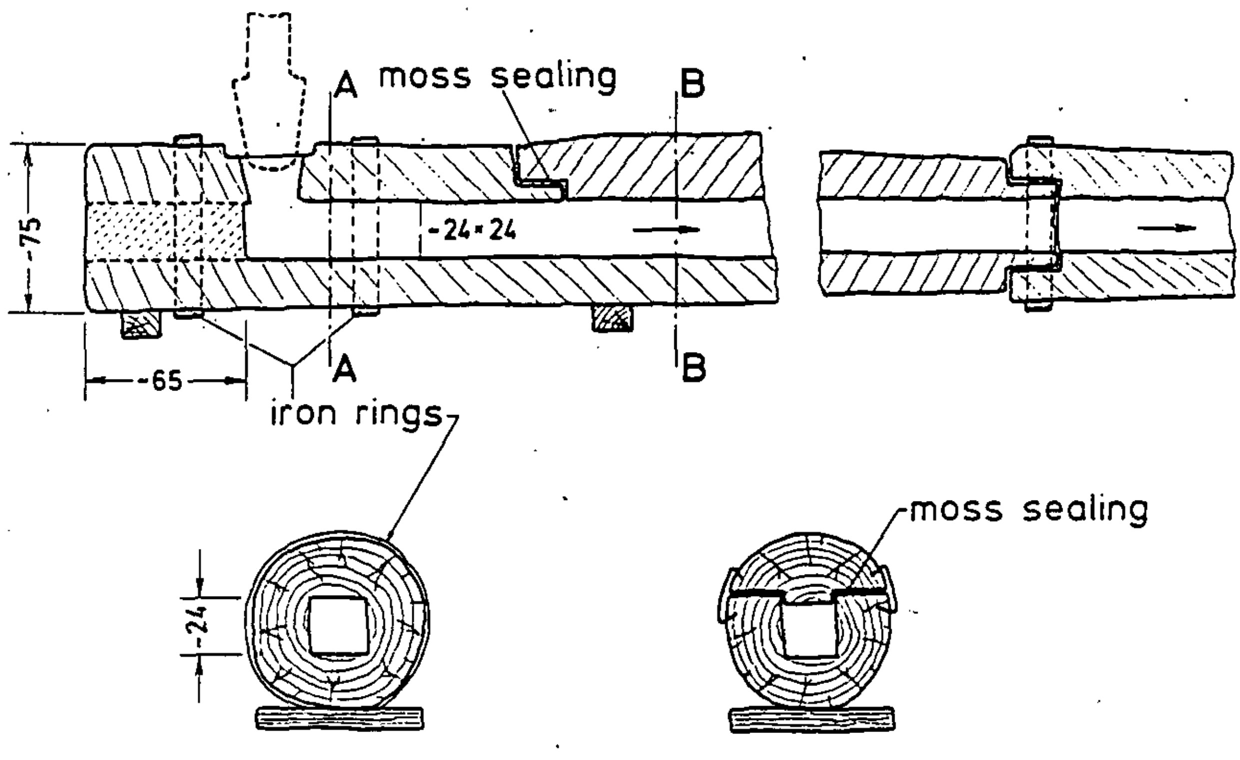


Figure 4: Wooden outlet pipes of the old mining dams

21. The main problem with the wooden pipe, which is constructed according to figure 4, too, is the detection of the leak or destruction. Where possible, small movable video equipement is used. The TV camera of a size of 19,0 x 17,3 cm is mounted on a vehicle which is able to climb obstacles up to 2 cm.

22. Normally the joints between different wooden sections, including the wooden cover-beam of the pipe, are fairly tight. In case of decayed parts of the wooden pipes the lake was emptied and shafts were sunk to repair the broken pipe. In former times the miners built small tunnels or made a cut in the dam to recover the outlet pipe and place a new one.

23. This could not be employed today due to lack of experiences in handling the grass sod sealings. In some dams, where the outlet pipes are totally blocked, it was the suggestion of the officials of the Historical Monument Department to replace the blocked pipe by a new wooden pipe, using the pipe jacking method as exercised in modern pipe laying construction. As any experienced dam engineer knows how dangerous it might be to cross the impervious layer without safe control we objected to employ this method. We convinced the officials of the Historical Monument Department that only the old outlet pipes had to be closed by injections. In order to allow the controlled depletion of the dam a syphon had to be installed. To avoid any future replacement and maintenance the syphons had to be of stainless steel. The syphon installed at the Prinzen Dam is shown in figure 5. Its discharge capacity is about 3 times as high as the capacity of the former outlet.

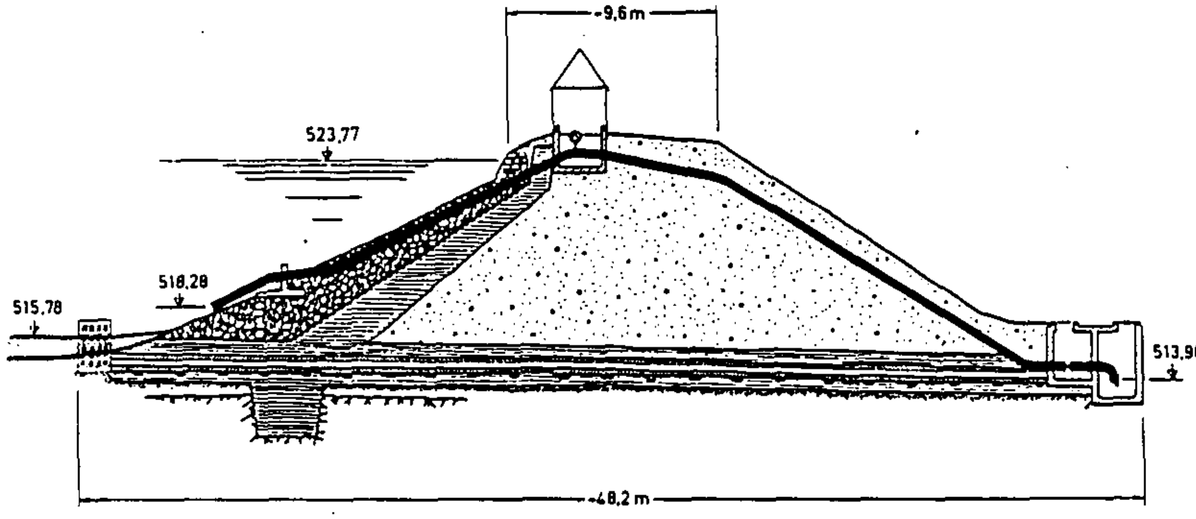


Figure 5: Cross section of Prinzen Dam with new syphon

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WAVE DAMAGE TO UPSTREAM SLOPE PROTECTION OF RESERVOIRS IN THE UK

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SYNOPSIS

The author outlines the commonly accepted methods for selecting appropriate storm wind speeds and deriving wave heights for the design of revetments for embankment dams. He gives examples of damage to recently constructed UK dams and gives details of the analyses and repair in three important cases. In each case the incidence of damage could have been predicted by the commonly accepted techniques.

INTRODUCTION

1. Significant wave damage has been recorded at a number of UK reservoirs of comparatively recent construction. Theoretical techniques exist to enable wave heights to be predicted with reasonable certainty for a given wind speed and duration. Doubts exist about the influence of reservoir shape in the determination of fetch. The appropriate probability of exceedence for design wind speeds may have been under-estimated. Wind and wave research is under way as part of the DoE funded research programme on reservoir safety.

2. The damage referred to in the paper has resulted from persistent wave attack at or near top water level, in certain cases culminating in severe damage as a result of storm force winds. In none of these cases has the damage coincided with severe flood encroachment of the freeboard.

DESIGN GUIDES AND STANDARDS

3. The ICE engineering guide "Floods and Reservoir Safety (1978)" is frequently referred to as a guide to the design of slope protection. <u>It is not</u>. The authoritative sections on wind and waves on reservoirs were intended to give guidance as to the appropriate freeboard above the flood capacity of the spillways for dams posing various degrees of hazard to life downstream. The factors relating to good engineering design are specifically excluded in Section 1.2 of the guide.

4. The sections on wave prediction were based on Saville's method and wind standards were set in a rather arbitrary way with a choice of frequency for the wind speed depending on hazard posed by the reservoir with a recommended minimum wave surcharge in each case. It was surely never intended, for example, that the 10 year max hourly wind speed was adequate for the design of the upstream revetment concrete or rip-rap for a Category A dam. Even if the design wave Hd had been Hs the significant wave height factored by 1.3 (the maximum value given in table 2, p22) the frequency of exceedence would be 4% of all waves. This would be enough to

trigger frequent blockwork revetment failure if the design was marginal. Nonetheless the first step in the design of a revetment must be to establish a wave spectrum of suitable severity.

WAVE PREDICTION IN RESERVOIRS

Present methods

5. The method given in the guide is that developed by Saville for the US Corps of Engineers, and published in the Journal of the Waterways and Harbours Division of the ASCE in May 1972.

6. The work was based on observations and data of winds and waves on a number of lakes and reservoirs in the States and differs from the Sverdrup-Munk-Bretschneider (SMB) formulae which are based on open-sea data and conditions. Firstly Saville found that his 'best fit' relationship for the significant wave height, as a function of fetch and wind speed, gave slightly lower values of wave height than the SMB formula for short fetch and high wind speed conditions. Secondly he introduced the concept of effective fetch for enclosed or sheltered bodies of water. This is a way of taking account of the irregular shape of the coast line and the resulting variation in fetch in different directions from the point of interest.

7. Saville's method is thus directly applicable to reservoirs and, until the recent research work by HRL at Meggat and Loch Glascarnoch, in northern Scotland, was the only available method which was based on data from short-fetch conditions. It was looked at in some detail by Frank Law when the ICE guide was being put together and considered the most appropriate method. Strictly it requires the use both of Saville's method for estimating wave height and his method of estimating effective fetch. The latter can of course be used with other wave prediction methods such as that of SMB but such results have not been validated. Generally, for a given fetch (or effective fetch), the SMB method gives wave heights about 15% greater for the same design wind speed in a reservoir.

8. HRL have proposed the use of the JONSWAP relationship. This was derived from data from the North Sea and is thus again primarily applicable to long fetch conditions. It has the advantage that it provides information on the spectral properties of the predicted wave climate, however it appears to under-estimate significantly the heights of waves in the short fetch, high-windspeed conditions applicable to reservoir design. (For the Foremark studies we estimated the 'JONSWAP' wave heights as being typically about 70% of the 'Saville' heights). Attempts have been made to adjust for this under-prediction by applying a different method for calculating the effective fetch. This method, developed by Seymour (though not from reservoir data), requires a computer based approach but in essence, where Saville considers the average length of fetch 'rays' by weighting each ray by $\cos^2 A$ (where A is the angle between the ray and the chosen wind direction, usually taken as the longest direct fetch), Seymour uses a term equivalent to $\cos^6 A$. This results in the rays closest to the chosen direction, generally the longest, having a greater influence. Hence the effective fetch is longer to counterbalance the smaller waves predicted by JONSWAP for high wind conditions in reservoirs.

9. It is worth noting that the Shore Protection Manual (SPM) originally proposed the use of the SMB method for forecasting deepwater waves in the open sea. In the most recent edition the JONSWAP relationships have been adopted, and in addition a 'windstress' factor has been included. This increases the actual or predicted wind speed, to an effective windspeed for use with the formulae, by a factor which increases with wind speed. For short fetches, the windstress factor increases the wind speed enough to offset the difference between JONSWAP and SMB predictions. Thus for short fetches the SPM predictions are little changed and slightly greater than those given by Saville's method.

10. There are other methods of predicting wave heights in short fetch conditions and these were compared in a recent literature review by HRL under a DoE research project.

The recent HRL work

11. Following the damage to the riprap protection at Meggat, HRL have been carrying out wind and wave measurements at that site and more recently at Loch Glascarnoch. The results of these observations are not yet available but HRL have already made some useful observations.

12. At Meggat they found that there were two main areas in which existing methodology was under-estimating predictions. Firstly the effect of the long narrow valley in funnelling the winds was greater than had previously been thought. Thus measured wind speeds at the dam site were higher than would have been predicted from the published wind speed maps, and, as a corollary, the winds were more strongly orientated along the valley and the maximum fetch of the reservoir. Secondly for a given wind speed the measured wave heights were higher than predicted using Saville's relationships.

13. Using the JONSWAP methodology HRL have found it necessary to further modify the Seymour method of calculating effective fetches to account for the increased wave heights. In effect a required weighting factor of $\cos^{30} A$ is required. This further weights the effective fetch length towards that of the maximum fetch - rays outside a central 45° arc have insignificant effect (compared to Saville's method where a 90° arc is considered).

Conclusions on wave prediction methods

14. The HRL work suggests that the use of Saville's method under-predicts wave heights. Since this method is the one recommended by the ICE guide 'Floods and Reservoir Safety', this is an apparent cause for concern. However the results from Meggat, and from Loch Glascarnoch, which we understand confirm the trends of the Meggat work, must be treated with caution when considering the bulk of the population of UK reservoirs.

15. Both these two reservoirs are long and narrow - the latter even more than Meggat - and confined in steep-sided narrow valleys. The HRL work certainly gives cause for concern in such cases both in estimating the appropriate design wind speeds and in extending those to the prediction of wave heights at the dam site. One possible reason for the higher than expected wave heights is that, with the wind direction so controlled by

the valley, there is less directional fluctuation during a storm event and hence less interference between waves generated. In effect there is greater coherence of the waves and less energy lost. If this is the case then any conclusions drawn from the Meggat work would have only limited applicability. Certainly the under-estimate of wave heights and the resulting overtopping occurrences had not been generally reported as a significant problem in the UK. In terms of freeboard, 15% is not particularly significant anyway when compared to the uncertainties involved in the estimation of maximum water levels and the run-up over different types of surface. Overall therefore, there is no evidence yet that the use of Saville's method results in under-design in most conditions. It will need the HRL work to be extended to reservoirs with different configurations and in different topographies before the present methodology could be shown to be generally incorrect. There is, however, a need to consider carefully the design approach for long, narrow reservoirs particularly if they are confined in steep-sided valleys.

WIND PREDICTION

16. The derivation of wind speeds for specific reservoir sites depends of course on the length of reliable measurements at the site. These are rarely available and reliance must be placed on the use of generalised wind data from the following sources:

- Maximum gust and wind speed and probability factors for UK regions - CP3 Chap V Part II 1972 and Jan 1986 Amendment;

or

- ICE guide "Floods and Reservoir Safety 1978". Maps of hourly wind speed for 1 year and 10 year return periods.

17. A typical analysis for a UK reservoir in Northumbria is given in Table 1. The wind values derived here for 10 year frequency are slightly greater than those given in the ICE guide.

Table 1: Mean wind speed over land in Selset reservoir area

Wind duration	Hourly wind speed ratio (1)	3 sec wind speed ratio	Mean wind speed-m/s (1 m/s = 2.23 MPH) 1 in 50 yr (2)	1 in 10 yr (3)	1 in 2 yr (4)
3 sec	1.51	1.00	46	40.48	35.42
10 min	1.04	0.69	31.74	27.93	24.44
20 min	1.02	0.68	31.28	27.53	24.09
30 min	1.01	0.67	30.82	27.12	23.73
40 min	1.005	0.666	30.64	26.96	23.59
60 min	1.00	0.662	30.45	26.80	23.45
2 hr	0.98	0.65	29.90	26.31	23.02
3 hr	0.96	0.636	29.26	25.74	22.53
6 hr	0.93	0.626	28.80	25.34	22.17

Notes:

(1) Based on Shore Protection Manual, Fig 3-13 and BS6235:1982.

(2) Based on basic 3 sec gust wind speed of 46 m/s from CP3: Chapter 5.

(3) Probability factor S3 = 0.88 (CP3: ch V)

(4) Probability factor S3 = 0.77 (CP3: ch V)

The above overland wind velocities are further adjusted to obtain the over water wind speeds using the relationship given by Saville. For an effective fetch of 0.9 km for Selset reservoir (= 0.56 miles) the factor is

$$\frac{\text{Velocity over water}}{\text{Velocity over land}} = 1.09$$

18. For the design of revetment it seems sensible to consider a wind/wave condition with a return period appropriate to the lifetime of the dam. Thus a return period of 50 years for wind speed is now considered appropriate.

19. For typical upland northern reservoirs in Britain overwater wind velocities for storm duration of 1 hour would be 33.5 m/s and for 6 hours would be 32 m/s (for 100 year return period these values would increase by less than 5%).

20. For typical UK reservoirs where fetch is generally of the order of 3 km maximum or less

Kielder	max	4.65	effective 2.2 km
Selset		2.70	
Foremark		1.79	1.0 km

the growth of waves will be limited by the fetch after about 20 minutes but the maximum wave height will continue to increase with storm duration and may be at the worst during the central six hours of the storm. Based on these wind conditions, ie 50 year return period 6 hour storm duration, the following wave heights were determined for three reservoirs where revetment damage had occurred:

	Wind speed m/s	Max wave height (m)
A. Kielder	33.9	2.5
B. Selset	32	1.7
C. Foremark	28	1.2

REVETMENT DAMAGE

21. No attempt has been made to make a comprehensive survey of revetment damage of UK dams. The author has been concerned in the inspection and repair of the three comparatively modern dams listed above and these will be described in some detail. Other cases have been reported but no information in available.

22. Two classes of damage may occur, either

Class 1 abrupt wave damage leading to loss of function of revetment posing a hazard by erosion of the fill leading to loss of freeboard

or

Class 2 progressive wave damage requiring repair but allowing this to be achieved without loss of function of the revetment.

This simple classification is by no means easy because storm wave damage has been experienced at below top water level which would have had much more serious consequences at high reservoir level. Table 2 sets out some details of 8 cases known to the author.

Table 2: Cases of damage

	Date of construction	Dam[1] type	Revetment[2] type	Damage	Repair[3]
Kielder	1980	E/SF	CB	Class 1	Re/Re
Selset	1960	E	MP	Class 1	Re/Re
Foremark	1976	E/SF	RR	Class 2	Re/Re
Silent Valley	1930	E/SF	MP	Class 1	Re/Re
Daer	1952	E/SF	CS	Class 2	Re/Re
Bewl Bridge	1975	E/CF	CS	Class 2	LR
Llyn Celyn	1965	E/CF	RR	Class 2	LR
Meggat	1986	E/SF	RR	Class 2	LR

1	Earth dam	E
	Clayfill	CF
	Sand/silt fill	SF
2	Masonry pitching	MP
	Concrete blockwork	CB
	Concrete slabs	CS
	Rip-rap	RR
3	Local repair	LR
	Redesign/Replace completely	Re/Re

Kielder

23. Built at elevation of 185m asl Kielder dam is an earth embankment some 52m in height. The original upstream slope revetment was 300mm precast concrete block work on a slope of 1 on 2.5. Blocks were of two sizes, 1065 x 400 x 310 kg and 705 x 500 x 260 kg. The blocks were laid on two layers of gravel filter each 300mm thick; the coarse filter bedding for the blocks had a D15 size of 9-12mm and D50 of 20mm. The blocks were laid with horizontal joints 15mm wide and vertical joints varying between 12 and 20mm as a result of the radius of the curved face. Initially filled with pea gravel the joints were washed out by wave action during the early life of the reservoir allowing some down slope movement of the blocks. This in turn appears to have permitted loss of supporting filter and uplift forces on the blocks from the breaking waves. Early damage not resulting in block ejection was repaired in October 1983.

24. Storm winds during December 1983 and January 1984 resulted in continual heavy waves breaking on the revetment with the reservoir at or above top water level. A significant number of blocks were ejected above water level and there was some risk of damage to the wave wall foundation. No actual measurements of wind speed or wave height were made. Strong winds were recorded at Newcastle weather station on 11 days between 25 December 1983 and 16 January 1984 with hourly speeds up to 20 m/s. This is some 60 km from the dam and only 85m asl; winds at Kielder may have

reached a maximum hourly value of 26 m/s. Wave heights were reported as 1.5 and 2.0m. The estimated return period of winds of this magnitude is about 10 years.

25. A re-appraisal of the exposure at Kielder led to reconstruction of the blockwork into larger panels set in concrete insitu giving a total thickness of 500mm and panel dimension of 7500 x 5780mm to withstand maximum waves of 2.5m.

Selset Dam

26. Built at elevation 330m asl Selset is an earthfill dam some 36m in height. The slope revetment consists of 200mm thick sawn masonry blocks set on 330mm of gravel 37mm-75mm. The blocks are of random size each with a specified minimum area of 2 ft^2. The maximum joint width is not specified or given on the record drawings, but most of the joints are filled with no-fines concrete or pea gravel.

27. The dam was built during 1955-1960 and the records show that there has been repeated damage to the revetment with ejection of the blocks and serious erosion of the supporting gravel. Repairs have been made locally using mass concrete. The most serious damage occurred during 1961 when emergency measures were required by the construction crew from Balderhead dam. Their prompt action in filling a large damage area near the crest may well have prevented a breach.

28. Analysis of the exposure of Selset leads to the conclusion that the existing revetment can only be expected to withstand waves of 1 to 1.2m height. This would correspond to an overwater wind velocity of 23 m/s which could be expected to occur every year at Selset. This is consistent with the frequency of damage.

29. The revetment should be reconstructed to withstand waves of 1.7m maximum height. This can be achieved by thickening the revetment to 300mm as a continuous concrete slab or by replacing it by 1.3m of rip-rap of D50 = 650mm.

Foremark dam

30. Foremark is a 39m high earth dam built between 1972-1977 of bunter sandstone fill with a clay core. The upstream slope has a slope of 1 on 3 and was protected by rip-rap 450mm thick with a two layer filter and support layer. The specified average rip-rap size was 300mm graded from 450-50mm.

31. During an inspection in September 1985 it was noted that the rip-rap had been damaged, the support layers had been washed out in places and the sandfill was exposed. Movement of the rip-rap had damaged the stone pitching protection to the top of the slope above normal water level. Damage was first reported after a storm in 1979 and progressed gradually until remedial work was executed in 1987. Analysis of the exposure led to a predicted significant wave height of 0.68m derived from 50 year return period hourly wind speed of 29 m/s.

32. CIRIA Report 61 provides criteria for the grading of rip-rap related to the slope angle and the significant wave height. For "no damage" criteria in the 50 year event, the D50 rip-rap must be equal or greater than Hs the significant wave height derived from the 50 year short period wind event.

33. In the event limited damage was tolerable and the 50 year hourly wind speed was used to derive Hs, giving a D50 rip-rap of 0.68m. A layer thickness of 2.0 x D50 size, ie 2 x 0.68 = 1.40m was adopted.

PERSPECTIVE

34. The success of slope revetment is dependent more on good design of the revetment than on the precise determination of the appropriate wave climate. Of 48 embankment dams built in the period 1963-1983 in the UK the revetment was:

Rip-rap	20
Concrete slabs	12
Blockwork	7
Asphalt	5
Stone pitching	3
Not known	1
	48

35. Rip-rap damage Class 2 has been noted and dealt with on 3 dams. Damage has definitely been "limited". In two cases the steepness of the slope may have been the cause of under-performance compared with CIRIA field trials on flatter slopes.

36. After rip-rap, concrete slabs are the most commonly adopted type. They are frequently surprisingly thin for their exposure. They rely chiefly for their stablity upon the slab dimension usually 2 x 2m minimum relative to the wave height (period 2 to 3 secs) and the tightness of the panel joints usually cast insitu with slots to relieve seepage water pressure.

37. No cases of Class 1 damage have been reported. Although built in the 1950s and therefore not included in this survey, it is worth mentioning the slabbing on Daer dam which was replaced after some 25 years service because of deterioration of the concrete with severe cracking resulting from swelling aggregate. Many cases with slabs as thin as 125mm have given long service. Isolated local repair has been needed where some dislevellment and consequent cracking has occurred. This may be associated with continuous breaking wave impact at or near top water level causing some pumping of the support layer. Early slabs were usually unreinforced but later the tendency was to introduce some crack control reinforcement and to be somewhat more generous with thickness.

38. Of the seven dams with concrete block facing, brief details are given in Table 3.

Table 3: Dams with concrete block revetment

Dam	Built years	Designer	Upstream slope	Blockwork thickness	Reservoir vol x $10^6 m^3$	Approx effective fetch km	Joint width	Joint filler
Turret	61-64	BSM	1 on 2.5	381mm (15")	18.2	1.0	12mm ($\frac{1}{2}$")	Slates
Derwent	60-66	owner	1 on 3	381mm (15")	50.1	1.5	16mm ($\frac{5}{8}$")	NFC
Dovestone	60-66	G H Hill	1 on 3	381mm (15")	5.1	0.5	12mm ($\frac{1}{2}$")	PG
Errwood	64-68	G H Hill	1 on 3	381mm (15")	4.2	0.6	12mm ($\frac{1}{2}$")	NIL
Backwater	64-69	BSM	1 on 2.5	300mm	25	1.1	12mm ($\frac{1}{2}$")	PG
Jumbles	68-72	G H Hill	1 on 3	381mm (15")	2.2	0.4	12mm ($\frac{1}{2}$")	PG
Kielder	76-82	BSM	1 on 2.5	300mm	188	2.2	12mm-19$\frac{1}{2}$mm	PG

PG = Pea gravel   NFC = No fines concrete

Of these seven, only Kielder which has much the greatest exposure, has suffered damage although the joint filling at Derwent has required regular attention as the no-fines concrete becomes eroded.

39. The only guide to blockwork revetment design is the empirical relationship which suggests that blockwork with tight joints (ie blocks not free to move on wave impact) will withstand wave heights up to six times block thickness, provided that the permeability of the underlayer is sufficiently low to prevent dynamic uplift on the blocks resulting from the breaking waves.

CONCLUSION

40. The experience of the few cases of damage reported suggests that if precast blockwork is to be used block dimension should be as large as is reasonable and interlock should be positive. The support layer should provide good positive support but should not be particularly permeable.

ACKNOWLEDGEMENT

41. The author is grateful to Northumbrian Water and Severn Trent Water for permission to publish.

EMBANKMENT DAMS & CONCEPTS OF RESERVOIR HAZARD ANALYSIS

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SYNOPSIS

The paper draws attention to the latent hazard represented by reservoirs, identifying the functions which a satisfactory expression for reservoir hazard level might serve in dam surveillance. Developments in hazard analysis are summarised in a brief critical review of approaches selected as being illustrative of the diversity in methodology, complexity and utility. The latter part of the paper introduces the concept of a Reservoir Hazard Rating. A structure for the proposed Rating is discussed, and its component dimensions of risk and consequence examined in the context of British experience of the embankment dam. Concluding sections suggest and briefly outline first and second level application of the Reservoir Hazard Rating.

INTRODUCTION

1. Reservoirs constitute a potential hazard to downstream life and property. The dam-break floodplain dominated by British reservoirs is frequently well-populated and of considerable economic importance. Dam failure with catastrophic breaching will in those instances result in unacceptable loss of life and extensive damage.

2. The legislative framework for surveillance of some 2450 reservoirs in Britain is provided by the Reservoirs Act (1975). Supervision and statutory inspections are directed to the timely identification of circumstances and events conducive to possible dam failure. In that context it is appropriate to analyse reservoir hazard and to examine its role as a parameter in surveillance.

3. Reservoir hazard cannot be quantified on any unique and absolute scale. Hazard level may be described or characterised in terms of its two dimensions; probability of catastrophic breaching and, secondly, the human and economic loss to be anticipated in consequence. Both dimensions are time-dependent functions of a series of variables; the field values attaching to either dimension range over many orders of magnitude.

4. Nominal hazard level can be expressed simplistically but adequately using estimated risk of breaching and/or predicted loss (or cost) as the determinants. Levels are readily compared or ranked in relation to either risk or loss provided either one is regarded as the sole or dominant parameter. Problems in the determination of nominal hazard level can be compounded by the intrusion of non-technical socio-political factors.

5. Qualitative categorisation of reservoir hazard is practiced as part of inspection and surveillance procedures in several countries. Hazard level is rated subjectively and simply, e.g. 'low', 'significant' or 'high'. Simplistic assessments of this form have little real merit beyond implying that correspondingly graded levels of surveillance may be applicable. British practice makes no formal recognition of differing hazard levels other than in relation to design flood standard.(1)

6. Hazard evaluation is a firmly established procedure in several areas of engineering, notably the nuclear power and chemical process industries. Methodologies vary, but evaluation involves the identification and assessment of all conceivable risks and interactions, and the prediction of their individual or collective operational consequences. Application of these essential principles to reservoir safety, using an appropriate methodology, has much to commend it.

7. It is considered that a quantitive hazard evaluation would have an important contribution to make to effective dam surveillance in relation to:

1. determination of priorities for resource allocation in surveillance and also for remedial works

2. 'worst-scenario' contingency planning for selected high-hazard and vulnerable reservoirs

3. introducing a limited element of indirect discretion to operation of the Reservoirs Act

and 4. logical assessment of insurance requirements.

This paper reviews approaches to the assessment of reservoir hazard level and introduces the concept of a Reservoir Hazard Rating (RHR) as an operating parameter in embankment dam surveillance. The RHR is developed with particular relevance to conditions pertaining in Britain.

HAZARD ANALYSIS - HISTORICAL REVIEW

8. The heightened awareness of safety issues so evident at the present time evolved as the natural response to a succession of particularly serious disasters. The catastrophic failure at Malpasset (France) in 1959, and later disasters at Vajont (Italy (1963)), Teton (USA (1976)) and Macchu II (India (1979)) each had a seminal influence on dam engineering. Collectively, these and other lesser disasters served to focus attention on safety. The effects were most marked at national level, e.g. with the 1975 Reservoirs Act in Britain and comparable developments overseas, but was also significant at international level through the work of ICOLD.

9. In the United States a Federal programme to inventory and review the national stock of dams was commenced in 1972. By 1975 some 49000 dams had been identified, 91% of which are embankments. (It may be noted that extension of the scope of the exercise later revised the total identified to 68000). Some 40% of dams were considered to present a significant potential hazard. A disconcerting 33% of 8800 dams subjected to screening and inspection for the first time were considered technically inadequate,

2% being immediately classified as 'emergency unsafe'. The foregoing statistics demonstrate the enormity of the dam safety problem in the United States at that time, and provide a backdrop to consideration of different approaches to evaluation of reservoir hazard level. As an illustrative comparison, the total British stock of dams is believed to approach 5500, of which 44% fall within the ambit of the 1975 legislation.

10. Initial interest in the structured analysis of reservoir hazard level can be traced to 1973 and the Dam Hazard Potential Index (DHPI) proposed in the United States by Sarkaria.(2) The semi-empirical DHPI concept and its limitations are reviewed later. Recent and mathematically more elegant alternatives in hazard evaluation are exemplified by the probabilistic risk analysis work of Gruetter and Schnitter (3) in Europe, and Bury and Kreuzer (4) in Canada.

11. In Britain, interest in reservoir hazard analysis and indexing was first expressed in 1976 by the Author (5) who advocated a refined variant of the DHPI approach. In 1980 Law defined a Dam Risk Index (DRI), conceived as a simplistic parameter for use with draft spillway standards for Korean dams.(6) More recent British work has included a comprehensive and most interesting exploratory investigation of the feasibility of probabilistic risk assessment for reservoirs (Clifton et al.(7)). This research has centred upon the deployment of techniques first established for nuclear safety studies, and may be considered a much more rigorous approach in terms of complexity, depth and sophistication.

OPTIONS IN HAZARD ANALYSIS

Selected options

12. The degree of sophistication associated with the several alternative approaches to hazard evaluation, only a few of which have been referred to, is almost infinitely variable. Three selected approaches illustrative of thiss are below subjected to a brief critical review. They are presented in ascending order of complexity.

13. Law: Dam Risk Index (DRI)

$$\text{DRI} = S.(H)^{2.5}$$

with S = storage volume ($\times 10^6 m^3$)
and H = max. dam height (river level to crest (m))
(exponent 2.5 is drawn from the observed relationship between dam-break flood flow and H.)

14. The limitations of the DRI as a simplistic two-parameter function are self-evident and extreme. Tested against any representative selection of British reservoirs its inadequacies are readily demonstrated; no logical relationship is apparent between DRI and subjective, but nevertheless valid, perceptions of 'real' hazard levels for widely differing situations.

15. Sarkaria: Dam Hazard Potential Index (DHPI)

DHPI was proposed as a function of several primary variables:

$$DHPI = fn. (V, H, {}^{1}/_{L}, D^2, P, T_d)$$

with: V = volume of reservoir
H = maximum height of dam
L = plan distance to centres of downstream development
D = elevation difference, reservoir surface to centres of development
P = 'value index' of life and property at risk
and T_d = 'type index' to represent relative security of different dam types.
(suggested relative values: arch = 1.0; gravity/buttress = 1.5; rolled fill = 2.0; hydraulic fill = 3.0)

Sarkaria suggested that additional factors could be incorporated as desired.

16. On first inspection DHPI is a valid but relatively straightforward and attractive concept. The expression is superficially complete in its recognition of primary inputs. It will be appreciated, however, that each input is in itself a much more complex and imprecisely determinate function of many sub-variables than is first apparent, e.g. L in relation to flood plain profile and section etc. It will also be noted that the functional relationship of DHPI to the inputs is not fully specified in terms of the weighting to be attached to individual 'fixed' physical parameters, e.g. L,H etc., in comparison with those attaching to the time-variable and essentially less determinate and all-embracing dam 'type index', i.e. parameter T_d.

17. Probabilistic Risk Assessment (PRA)

The methodology of PRA has developed in response to the safety needs of high-risk and high profile industries. In general terms it draws upon established procedures for Event-tree Analysis, Fault-tree Analysis, and System Reliability Analysis. It makes use also of the associated techniques for Probability Assessment and Damage Assessment.

18. Event-tree Analysis details possible outcomes from a specific initiating event, e.g. PMF flood in the case of a dam. A simple 'yes-no' logic is employed, moderated by a weighting in terms of known probabilities. Events are therefore traced in a 'forward' direction. Fault-tree Analysis, on the other hand, operates a similar logic and probabilistic weighting to detail the combination of states which could lead to a specified event, e.g. overtopping. Events are therefore now traced in a 'backward' sequence. Probabilistic weightings in both analyses are derived from statistical evidence, i.e. historical records of failures and their cause. The actual weightings assigned in Event-tree or Fault-tree analysis are time and decision dependent.

19. System Reliability Analysis examines individual failure modes and their interactions. The principal elements involved are mechanism, location in dam, and time (load condition).

20. Reporting upon the detailed study of PRA undertaken by the Safety and Reliability Directorate of the UKAEA, Clifton et al [7] concluded that,

despite having identified many difficulties, PRA could usefully be applied to reservoirs. They recommended the trial application of a rigorous PRA analysis to a selected and supposedly 'average' Pennines-type embankment. As summarised by Parr and Cullen [8], this exercise quantified property damage from catastrophic failure at £170 million, with loss of life ranging from 4 deaths up to a projected maximum of 2300 in the very unlikely event of sudden and total collapse of the embankment.

21. PRA is a flexible and powerful tool in the right hands, but its limitations must be appreciated. Effort required per dam is at present prohibitive, at least as regards general application, in terms of specialist expertise required and cost. Some probabilistic inputs used are also of questionable validity, employing statistical evidence derived from historical records which are incomplete, in many instances inaccurate, may be inappropriate, and are heavily biased towards large, i.e. World Register listed, dams. The latter group accounts only for an unrepresentative 530 British dams, i.e. 21% of those subject to statutory legislation on safety. Further, that group of dams is largely publicly owned, and therefore arguably better maintained than the 1900 lesser, and commonly private-ownership, dams. Many of the latter stand in particular need of hazard evaluation analysis by virtue of their condition or situation.

22. Notwithstanding the limitations outlined above, PRA has a useful contribution to make to reservoir surveillance in Britain given that it is applied selectively and with due regard to its strengths and weaknesses. Its real value may lie in its selective application to abnormally high-risk dams previously identified by some simpler and more rapid screening technique. Suggestions for the latter have been made by Parr and Cullen, and a suitable starting point is Sarkaria's DHPI or a derivative thereof such as the Reservoir Hazard Rating proposed iun this paper.

Conclusions on selected options

23. It is considered that none of the approaches to hazard evaluation described is of itself adequately matched to the operational requirement. A satisfactory approach must be realistic in its appreciation of the relative importance of different parameters. It must also be sufficiently sensitive in its application to properly illuminate, and not to mask, significant differences in hazard level. It must not, however, be over sensitive to inevitable variations in input quality. Most importantly, it must be readily and rapidly applicable by dam engineers without great need for specialist assistance.

24. The crudity of Law's DRI renders it sensibly valueless in hazard assessment. Sarkaria's DHPI, on the other hand, is insensitive to important factors such as age, history etc. Standing at the other extreme, PRA is too complex and too demanding in specialist expertise. It is therefore too slow and costly for application to primary-level hazard analysis.

25. For simple initial screening and primary-level analysis it is suggested that a solution lies in application of an enhanced and refined variant of Sarkaria's DHPI, i.e. the Reservoir Hazard Rating, or RHR, referred to earlier. The remainder of this paper presents an outline perspective of

embankment dams in Britain and introduces the concept of the RHR in that context.

EMBANKMENT DAMS IN BRITAIN - A PERSPECTIVE

26. The large-scale construction of dams in Britain was initiated by the canal-building activity of the late 18th century, and subsequently enhanced by industrial demand for water power. Many smaller and some large dams date back to this period. In the 19th century, growing industrial demand was compounded by the effects of urbanisation, which continued to 1900. Almost all dams to that point were earthfill embankments.

27. Population densities and reservoir numbers in Britain are extremely high in relation to land area. This is particularly so in the traditional industrial areas, e.g. in the North West and Central Scotland. The potential hazard represented by the considerable concentrations of smaller but elderly dams in such regions is accentuated by their situation in confined valleys in close proximity to quite extensive urban areas.

28. The great majority of British dams, in excess of 80%, are embankment structures. Most older and larger embankments were constructed to the traditional 'Pennines' type profile dating in concept from the 1840's, i.e. steep slopes with a slender and vulnerable puddle core and cutoff. Many lesser dams have no detectable core or cutoff. 'Design' was on the basis of successful precedent, and safety often marginal. Construction standards fell well below those to be expected today. Spillways and outlet works are commonly inadequate in capacity, badly deteriorated or vulnerable, and maintenance effort may have been minimal.

29. The perspective sketched above highlights four cardinal points of particular significance to embankment surveillance and safety in Britain:

1. median age is high, (c. 90 - 95 years)

2. many dams are located in high-hazard situations

3. design and construction standards vary greatly

and 4. 'structural' integrity may well be marginal.

These points are also most relevant to the development of a satisfactory RHR.

THE RESERVOIR HAZARD RATING

Introductory observations

30. The essential requirements for a system of hazard evaluation can be summarised as realism, relative simplicity, flexibility, adaptability and economy. These requirements can only be fulfilled if due regard is paid to a number of additional considerations which must influence the structure of any Index or Rating for hazard level. Considerations of particular concern to the RHR are outlined in the paragraphs below.

31. RHR is not an end in itself; it is merely a tool within the overall surveillance and inspection function. It is inappropriate for the RHR to be rendered too complex in misguided attempts to produce 'absolute' rather than indicative 'relative' values for hazard level.

32. There must be a mechanism or factor to properly reflect inspection findings and any follow-up actions required or taken. The RHR should also be sufficiently flexible in its structure to be influenced by events in the known technical history of a dam. RHR is therefore conceptually dynamic rather than static.

33. In principle RHR must be applicable to the entire dam population. In practice, it is necessary to effect differences in emphasis to accord with the main types of dam and their relative vulnerability to specific destructive mechanisms. In the context of embankment dams it is reasonable to differentiate between rockfill and earthfill and, at second level, perhaps also to take account of core and cutoff type and numerous other particularly significant design features.

34. The RHR must be structured to reflect any unusual features of the national dam scenario which may have important implications with respect to safety. Obvious examples are, as in Britain, high median age or, as a further illustrative dimension, ownership (i.e. public, private etc.).

35. All probabilistic weightings should be based upon the historical frequency of serious incidents as well as of total failures. This would considerably extend the data base available, and hence the reliability of probabilistic predictions. It is important to appreciate the limitations of available sources of data. Most published analyses refer to World Register dams only, i.e. larger structures, and they frequently use international rather than national sets of statistics. This can result in serious bias.

<u>Form and structure of the RHR</u>

36. The RHR is a logically constructed and semi-rigorous expression for the assessment of reservoir hazard level, primarily for purposes of comparison. It is sufficiently sensitive and responsive to permit essential flexibility in operation. The expression is also receptive to additional and semi-subjective input based on direct knowledge of a dam.

37. RHR is intended for first-level and selective second-level application as characterised below:

<u>First-level</u> - general screening

1. identification of high-hazard situations

and 2. resource allocation, non-critical purposes

<u>Second-level</u> - selective assessment

1. priority allocation, non-emergency safety works

and 2. dam-break contingency planning, nominated dams

Second level application is more rigorous, employing a definition of RHR which is more demanding in terms of input quality.

38. The expression for RHR is constructed from functions representing the dimensions of reservoir hazard defined in 3., ie.:

RHR = fn. [Risk [R], Consequence [C]]

In this context Risk and Consequence are defined as:

Risk [R] : predicted probability of breaching, or of a major incident requiring immediate action to obviate probable breaching.

Consequence [C] : predicted loss of life and estimated material damage which would result from catastrophic breaching.

39. Risk and consequence are complex, functions of numerous inter-related physical and statistical parameters. Many do not readily lend themselves to accurate mathematical modelling other than in simplified form. This restriction is acceptable, given that the constraints it imposes on 'accuracy' of the RHR are fully appreciated.

Probability of breaching and Risk [R]

40. The probability of an embankment breaching is influenced by many factors. At the simplest level, it will be related to embankment type, construction era, history, ownership etc. Narrower controls operate through details of design and/or construction, e.g. spillway inadequacy, lack of internal drainage, and similar factors. Few influences of consequence are determinate; their significance may be inferred, given reliable statistical data.

41. Published data on failures and incidents for British dams is incomplete or inadequate. Data analysed for application within the RHR must be compiled to a common standard. Information for over 740 British dams is held by the Author, and incident data is being processed. An early partial analysis of a sample 200 dams has been published in Reference (9).

42. For purposes of first-hand analysis it is considered sufficient to express [R] on a simple statistical basis. Appropriate factoring, derived from statistical evidence, can be applied to account for less tangible factors such as past ownership, previous history, age, etc.

43. For second-level analysis the RHR must be determined in more rigorous fashion. It is nevertheless important to avoid unjustifiable complexity. Statistical data relating to occurrence of the common mechanisms associated with incidents, eg. seepage erosion, overtopping etc. can be used in conjunction with corresponding statistical data relating to primary causes, eg. geotechnical, hydraulic, etc.: Interactions will exist between Causes and Mechanisms in most instances.

Consequence [C]

44. The immediate determinant of Consequnece is the dam-break flood hydrograph. A first approximation of dam-break peak outflow can be made using empirical relationships such as those published by MacDonald and Langridge-Monopolis [10]. These relations also permit limited dam-breach sensitivity analysis in terms of breach volume (and hence size and shape) versus time.

45. Maximum flood levels at successive stations on the floodplain are determined by the locally prevailing slope and degree of containment. Flood rise and time to peak in a confined valley can be estimated by routing the dam-break flood. An approximation is sufficient in the first instance.

46. For first-level analysis, Consequence can be estimated in terms of the predicted floodplain, employing data on population and land-use to quantify loss value. Sufficiently representative areal data for this purpose can be established through study of local maps.

47. Second-level analysis is more demanding on input quality and quantity. The use of established commercial software to establish the floodplain limits is appropriate, eg. DAMBRK. A valuable study of dam failure modes and consequential flooding has been prepared by Binnie and Partners under contract to Department of the Environment [11].

Marriage of Risk with Consequence

48. Obvious difficulties emerge in reconciling Risk and Consequence within the RHR. Philosophical problems emerge in terms of equating loss of life and material damage. Should life be converted to a national costing as is done in highway engineering? How are situations with low Risk but high Consequence to be compared with the reverse situation when considering priorities? The only practicable solution is considered to lie in a final value judgement being made at an appropriate level, eg. the Inspecting Engineer.

49. Considered in relation to either level of analysis, further difficulties are apparent. First-level analysis requires the marriage of overall or global Risk with Consequence. Second-level analysis requires detailed study of several individual values of Risk relating to mechanisms and/or primary causes. In both instances it is suggested that RHR is best expressed in a format such as:

First-level : RHR = [overall Risk; lives; material cost]

Second-level : RHR = [individual Risks; lives; material cost]

CONCLUDING OBSERVATIONS

50. It is impossible to do more than outline the RHR concept in this paper; much development work remains to be done. RHR is applicable also to non-embankment dams, and can be extended to special situations, eg. reservoirs in cascade.

51. The RHR is not suggested as a partial substitute for proper surveillance and maintenance. It is proposed as a relatively

unsophisticated and inexpensive aid to correct decision making, and represents an 'engineering' approach to the immediate problems of priority and resource allocation in surveillance programmes. As such it merits further study.

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DISCUSSION: TECHNICAL SESSION 6

EMBANKMENT DETERIORATION

Session Chairman: Dr A K Hughes (Group Manager, North West Water)

In this session we have Douglas Gallagher talking on deterioration of cores within dams; Dr Engineer Martin Schmidt from West Germany, we are pleased to welcome him, talking about some fascinating old dams in Germany; Bill Carlyle on the subject of waves and wave damage and Ian Moffat on Reservoir Hazard Index.

S K SHAVE (Southern Water)

With reference to Mr Carlyle's paper I wish to refer to the warnings of 16th October last year, after the hurricane, when there were about half a dozen of us in the office out of about 100, and I got a third hand message from our emergency control room that the slabbing of Bewl Bridge Reservoir had been broken up, that there was a hole in the dam and that they thought someone, i.e. me, as supervising engineer, should take a look. I think what they really meant was that the supervising engineer was in, thank God; it was a very hot potato to be got off their hands as quickly as possible.

As I explained yesterday, nothing happens to new reservoirs and when I put my hand up to become a supervising engineer, I didn't agree to get involved in the supervision of reservoirs that were going to start falling down. I agreed to inspect them and make reports, which would at worse say something might go wrong if....

Conveniently for me on this occasion, neither the inspecting engineer nor the construction engineer could be contacted, and I was left to my own devices to start with.

When I got to the dam and took my first look from the end I was obviously relieved to see that the damage was of no real consequence and that I could relax. The closer I got, the more concerned I became. At least two rows of the 125 mm nominal thickness slabs, which are 4 metres square, had been broken up over a length of almost 80 metres.

The second row from the top of the revetment had slipped down and most of the slabs that had been displaced at this level were split in half. There was significant erosion to the drainage layers under the slab to the point where the face of the dam had begun to erode and I would estimate that, at the worst, almost a metre thickness of the dam construction had been damaged.

Following a meeting on site with Ron Cole and Michael Kennard it was agreed that temporary repairs only would be carried out to see us through the winter, and that the reservoir would be filled to take any further storm waves away from this area of worst damage, and that permanent repairs would made this summer. A survey was undertaken by divers in the reservoir, which indicated that the damage extended at least one and possibly two further rows of slabs below the top water line. The temporary repairs which we undertook comprised some very basic filling of sandbags and placing them in all the large gaps: these were then spiked to the embankment with 2 metre long steel rods and, similarly, along the bottom of row 2 to prevent any under-cutting before the reservoir could be brought up to its normal level. Areas of slabbing, which had been broken into smaller pieces, were covered with a weak 10-Newton concrete, which was merely to provide as smooth a profile as possible for wave action over this last winter.

We undertook no temporary works at all under the water, since we considered that it would not have been practical, even though a complete slab some 4 metres square was clearly visible on the top of the beach of drainage material which had been washed out which had become established under the water. With the reservoir drawn down for the permanent repair we saw that it had withstood the winter fairly well with no damage to the mass concrete at all and little damage, somewhat surprisingly, to the bagwork.

I am sorry that the contractor has only just started work and we haven't got any photographs to show you what he is doing, other than to give this very preliminary idea of how he is tackling it. We have removed all the concrete, the top row of slabs is supported against further movement using intermittent sheet piles, and the concrete was broken up and used quite conveniently to form riprap in an area towards the east abutment, which was beginning to erode.

If I can give you some basic information on our approach to the permanent repairs, we believe the storm to have a return period of perhaps 1 in 200 years and we are advised by the inspecting engineer after discussions with Hydraulics Research that we take account of the current state of research and model testing. We took a wave height of 2 metres and we took a formula of H/D equalling 3 to be used for the derivation of our slab thickness, which has given a replacement thickness of 500 mm. The original design allowed a granular-filled gap at the top of the revetment between the slabbing and the wave wall base, and in order to minimise entry for wave runup into the drainage layer via this slot in the future, this is to be sealed along the length of the dam.

We are undertaking this part of the works on the basis that flotation of the slabbing was a factor in its failure.

As a footnote, Hydraulics Research were looking for a reservoir in the South of England to carry out their measurements but rejected the idea of using Bewl Bridge, which was actively considered, on the basis that they would be unlikely to get severe enough conditions.

D GALLACHER (R H Cuthbertson & Partners)

Bill Carlyle's paper refers to Megget Reservoir but actually doesn't give much information on it. So I just thought I would fill in a few facts and general comment.

Megget was in fact damaged at the same time as Kielder. It was the storm of 1983/4 and I think the storm actually started before the New Year break; it continued through Janaury and it just so happened, I think on the 13th of February, that someone 'phoned us and said that there appeared to be damage on the dam.

Megget is at an elevation about 337 metres; its maximum fetch is four kilometres and the valley itself runs in the prevailing wind direction, the bearing is about 245. The dam is about 600 metres long, so your maximum fetch is 4 kilometres; the effective fetch adopting the Saville technique reduces down to about 1.2 kilometres. The upstream face to the dam is 1½ to 1 and the protection on the upstream face has a total protection thickness of 1.8 metres made up of three layers of 300 mm, 300 mm, and the heavy riprap layer 1.2 metres thick that contains rock up to 900. In fact rock in excess of that was placed in the layer.

S There is a tower in the reservoir, the embankment is curved and the prevailing wind direction is down the very straight reservoir, is in this direction, so that you've always got an angle of obliquity for the main section of the dam. What happens in fact is that the tower influences the wave attack on the embankment and there is a sheltered area behind it. Towards the north end of the embankment the angle of obliquity is large and in this area here also is next to no damage at all. There may be some displacement of stones but nothing that would warrant any remedial work. The worst area of damage is over a length of about 200 metres in the middle and at the far end there is little damage.

So there is obviously a displacement of the energy from the tower into this area here. On investigation immediate repairs were carried out on this section and we recommended to our client that an investigation be carried out on wind and wave.

Very briefly, a wave riding buoy was established in the reservoir upstream of the tower and over the period of 12 months readings were taken of wind and wave to establish the relationships for the reservoir.

This has shown that (taking 15-minute means) the maximum significant wave height is about 1.3 metres with a maximum wave of 2.13 metres. I was actually there when we had a wave of 2.13 metres, standing at the top of this tower and was able to watch the wave come in and smack the tower, and I could run inside and take a reading. It was all very impressive and I can assure you that waves of that magnitude on a reservoir are really quite frightening. It was more like a raging sea. The 60-minute average, hourly, Hs was 1.33 at that time.

What happens of course is that these figures are inflated by the reflection from the dam, which HRL estimates to be about 25%, which really brings the true wave height down from 2.13 to about 1.6 metres and Hs down from 1.15 to 1, in that sort of range.

The maximum wind speed we recorded at that particular time was 30 metres a second, which was well below the design wind speed for which the upstream face had been designed. For data for designing for wind for the reservoir we had in fact consulted the Met Office, who didn't have data locally in the area. So they referred for similar conditions to a very exposed site in Edinburgh, and they also had the rider that the narrow valley would possibly offer a braking force on the wind speed. Their hourly mean velocity with a hundred year return period at that time was 29.95 metres per second. So we were experiencing wind speeds of 30 metres per second and these are hourly averages.

Further repairs have been done since that day and obviously we've increased the stone size. We've made the stone as big as possible; we've also introduced areas where we have additional reinforcement, areas where we have put in concrete between the stones with fibrous reinforcement. All this is performing very well at the moment, but it's obviously going to be taken into account in a final report.

G A MILNE (Crouch & Hogg)

My remarks relate to Paper 6.3, page 9, Table 3. Mr Carlyle indicates that of the seven dams listed only Keilder has suffered damage.

The first dam on the list, Loch Turret has also suffered damage. The first statutory inspection by Mr Arthur Allen in 1974 established the need for blockwork repairs in areas extending to 7.0 m below overflow sill level i.e. about 19 m down the slope. In 1978 about 2100 sq m of blockwork was lifted and relaid, with some further small areas relaid in 1980.

When I inspected the dam in 1984 with water levels 6.4 m below overflow level it was apparent that while no blocks had been dislodged in the areas visible, movement of blocks was continuing and was quite widespread. The typical movement was rotational with the upper edge being raised by up to 50 mm relative to the adjacent block, this being similar to the movements in pitching illustrated in a slide of Lower Carno Dam shown by Mr D E Evans during the Session No 4 discussion.

D E EVANS (Binnie & Partners)

I thought it would be of interest to outline a comparatively cheap but effective method recently used for the repair of a leaking puddle clay core.

Boddington Reservoir (British Waterways) north embankment built nearly 200 years ago had been leaking at several places in recent years whenever the reservoir rose to within 0.5 m of top water level. In 1987 Binnie & Partners recommended reworking and compacting the top 1.5 m of the core to restore the watertightness of the 200 m long embankment using the following method.

1 The work was carried out as a continuous operation working from one end of the embankment to the other.

2 The work was led by Machine 1, a hydraulic backhoe which commenced by stripping away the crest road to expose the top of the core over a length of a few metres.

3 Machine 1 then excavated a 1 m wide trench approximately 1.3 m deep in the puddle clay core and fed the clay to Machine 2 at the trailing end of the open trench, about 6 m long typically.

4 Machine 2 with a hydraulically powered vibrating plate fitted to the boom first compacted the bottom of the open trench over a length of 3 m. It then spread, broke down, kneaded and compacted th clay from Machine 1 in thin layers until the 3 m length of trench was full. It then moved forward to repeat the cycle taking care to bond each 3 m length with the preceding one.

5 The crest road was then rebuilt with crushed stone over the core using a layer of woven filter cloth to prevent stone punching into the clay.

The use of tracked machines minimised damage to the embankment crest. There was no need to support the sides of the open trench as men did not have to work inside it. Only a short length of trench was open while work was in progress and this was filled in at the close of each day's work. A typical day's progress was 30 m and the overall cost of the operation about £25 000.

Locally the trench had been deepened to 2 m over a short distance and with care working down to 3 m depth appeared possible in the given circumstances (clay core supported by clay shoulders).

The results have been satisfactory, no leakage having re-appeared in the 12 months since the work was completed and the reservoir refilled.

N W H ALLSOP (Manager, Coastal Structures Section, Hydraulics Research Wallingford)

On Mr Carlyle's Paper 6.3:-

1 The author is to be congratulated on identifying some of the more important examples of protection failure on UK dams. The paper has highlighted the main apparent difficulties in two areas:

(a) predicting the severity of wave action on an inland reservoir;

(b) estimating the size of protection unit needed for stability.

In neither instance is the problem simple, and engineers and researchers need to consider the main processes carefully to give their clients the most cost effective solutions. HR noted the request made at the last BNCOLD conference for further research and advice on good practice in this field. The DoE have been aware of these problems and research work to address two areas has been commissioned at HR:

We have not yet been commissioned to conduct further work on the stability of rip-rap, but are presently working with CIRIA who are producing a revised guide on this subject. We are hopeful of supplementing the original work of Thompson & Shuttler, CIRIA 61, and further work in Holland and at Wallingford, by tests on the effect on stability of rock armour or rip-rap grading.

2. In recent research work Michael Owen has addressed wave prediction methods in reservoirs, and some of the early findings are discussed in the paper. A literature review has been compiled and copies can be sent to anyone who contacts myself or Michael Owen at Wallingford.

A report on the wave measurements referred to in the paper is presently in preparation, and Michael Owen will be happy to send copies out as soon as the report is approved by the DoE Reservoir Safety Research Committee.

3. Turning to concrete blockwork, it may be useful to note that the influence of joint spacing, under-layer permeability, and of wave period or steepness has been studied by research in the UK, USA and Holland. A summary of published work has previously been presented by Powell, Allsop & Owen. (HR Report SR 54).

In this discussion I would like to provide more recent research advice to designers, based on the model tests at HR supported by DoE, and reported by Herbert & Allsop (HR Report EX 1725).

The two most obvious parameters of interest are the wave height, H, and wave period, T, sometimes used to define a deep water wave length, $L_o = g T^2/2\pi$ These parameters may be used to define the Iribarren number:

$$Ir = \frac{\tan\alpha}{(H/L_o)^{1/2}}$$

This dimensionless parameter is widely used to describe the effect of structure slope angle, α , and wave steepness, H/L_o, on armour stability.

4. In phase one the tests at HR were conducted on fixed blocks under regular waves to determine the net uplift forces. Blocks equivalent to those at Kielder were modelled at a scale of 1:8. Examples of test results are shown, and it may be seen that uplift pressures greater than could be resisted by block weight alone were generated even for $H/ D = 2.5$, the equivalent of the old 1/6 rule. It may be noted that this was with regular waves only, and implies that the threshold level for H_s would be commensurately lower!

These tests clearly showed that the survival of many concrete block revetments depends on more aspects than had been considered in the very simplified case tested.

5 In phase 2, un-restrained blocks were laid on carefully prepared underlayers, with specified joint widths, and were subjected to random waves.

For wide joints $\sim$ 0.08D, and a permeable underlayer, the block stability was dependant on the Iribarren number, and hence on wave period or steepness. A minimum stability, given by the onset of block movement, was reached at $H_s/\Delta D = 2.5$, where the block depth =D, and the relative density $\Delta = \rho c/\rho - 1$. This corresponds well with the 1/6 rule for $H = H_s$, not Hmax.

The effect of a reduced joint gap can be seen in the test results for joint width = 0.03D, where no block movement occurred even for values of $H_s/\Delta D = 3.4$. This change alone therefore reduces the block thickness, and hence concrete volume, by about 35%.

6 Relatively thinner blocks cannot mobilise quite the same restraint forces and therefore shows a slight reduction in the stability limit to $H_s/\Delta D = 3.0$ In this instance the joint gap has also increased to 0.05 D.

7 A dramatic improvement is offered by reducing the permeability of the underlayer. This had previously been shown by Whillock (HR Report IT 195), and others, to increase block stability. It may be seen here that, for a slightly permeable underlayer, the limit of stability was not approached at $H_s/\Delta D = 5$ in the range of conditions tested. An impermeable underlayer further improves the stability. These general conclusions are well supported by full-scale test results from the large flumes in Holland and Germany that suggest values of $H_s/\Delta D$ in the range 3.5-6 may be used where blocks are laid directly on a clay foundation layer (PIANC Supp. Bull 57, 1987 available through ICE).

8 It may be seen therefore that a significant increase in safety, and/or reduction in cost, may be produced by careful attention to blockwork placing, the specification of appropriate underlayer permeability, and interblock restraint such as partial grouting. Where such savings are of significance, further careful study of the complex phenomena may be needed to reduce design uncertainty, and hence construction cost and/or failure risk.

9 It may also be seen that the available data is not always complete, and may be contradictory. It is essential to study the performance of wave protection in service. HR and CIRIA intend to extend work covered here, and will be seeking support from owners and engineers shortly.

P HORSWILL (Watson Hawkesley)

I wanted to draw your attention to a relative minor, but I think, rather significant, variation on the observations that were made at Bewl Bridge. This was also a consequence of the October 16th storm last year. The violent storm crossed the Thames Estuary, you may be aware, and went into Essex and Hanningfield Reservoir suffered some damage, although the significance of my contribution was that the damage was delayed.

Hanningfield Reservoir was completed in about 1955. It has a puddle clay core and gravel shoulders; the upstream slope is 1 on 3 and is protected by 150 mm thick concrete slabs and these are 1.8 metres square. The supervising engineer observed damage immediately after the storm. The pre-cast concrete coping blocks had been moved, but there was no obvious damage to the slabbing at the time. Later, during March of this year the water level was lower and in the area where the coping stones had been moved most there had been damage akin to that at Bewl Bridge, but on a much smaller scale. There had been tilting and movement of the blocks and removal of the permeable gravel underlayer over a significant area. We believe that the storm of October last year caused a loss of fines through the open joints between the slabs, causing voids to be produced underneath. There was a small amount of inter-slab movement, nothing that appeared significant at the time, but the lesser storms of March this year caused the slabs to move in a fashion which we believe is very similar in essentials to the Bewl Bridge failure.

The inspecting engineer has asked that remedial works be carried out by replacing the 150 mm slabs with slabs 400 mm thick, so there is definitely a "beefing-up" in Mr Carlyle's words. I would also like to add the point made elsewhere by other contributors, that offshore technology/coastal protection technology would suggest not only a "beefing-up" but perhaps a much greater emphasis on the sealing of the joints between the slabs, so as to perhaps affect economies in the thickening up of the slabs.

W J CARLYLE (Binnie & Partners)

Thank you Mr Horswill. Could I just understand: are you intending to replace the whole of the slabbing on Hanningfield by 400 mm slabs?

P HORSWILL (Watson Hawkesley)

No, only the repair.

N CULLEN (Water Research Centre)

I wish to thank Mr Moffat for airing this subject - perhaps in a future conference Hazard and Risk Assessment should have a session of its own.

I wish to make 4 points:

1 The most complete recent summary of the state of the art is given by N B Wellington in a paper to ANCOLD, presented in ANCOLD bulletin December 1987.

2 The recent Binnies/UKAEA work has highlighted the fact that dams - a number of them in the Yorkshire-Lancashire area - present the largest man-made hazard to life and property in the UK. Chemical and industrial plants presenting lesser hazards now fall within the CIMAH regulations which require the preparation and publication of emergency plans. Although responsibility for emergency planning beyond the site of a dam falls to the local authority, it is civil engineers who have the skills to forecast the nature and extent of flood damage, and I believe that civil engineers have a professional if not a moral responsibility to promote awareness with dam owners and the emergency services of the likely consequences of a failure.

3 Detailed examination of the Binnies/UKAEA working documents indicates that a simple approach to preliminary risk analysis is possible. This amounts to writing down all the possible causes of failure, estimating a probability for each, then adding up the result. This will be a means of ranking reservoirs and of highlighting potential areas which give the greatest concern; it will not produce absolute risk values. In a survey of the subject, it appears no-one else had been able to take the subject significantly further than this for risks associated with the geotechnical aspects of embankment dams.

4 The attempt at risk analysis has highlighted the lack of good, consistent data on British failures and I support Ian Moffatt wholeheartedly in a plea for a "near-miss" database so that design, inspecting and supervising engineers could have access to a more comprehensive list of "what can go wrong".

A I B MOFFAT (University of Newcastle-upon-Tyne)

Thank you for those points, Mr Cullen. I could go further on one thing, however. I think the question of responsibilities for contingency planning is a subject we don't talk about; well, fair enough, it is the responsibility of the local authorities or the enforcement authorities to actually put these plans into effect, but that will not save you gentlemen employed by the regional water authorities, who own the dam. You are the ones, I would suggest, who have to prepare the contingency plans, to detail to local authorities what the consequence would be in the unfortunate event of breaching.

A D H CAMPBELL (Fairhurst & Partners)

There are two points in Mr Carlyle's paper. His Silent Valley slide showed heavy stone beaching placed on weak concrete. Nevertheless that forms a fairly rigid mass, and you can get swallow-holes on the upstream face of a dam, similar to those shown by Mr Gallagher on one of his slides. If you have a rigid structure, this can bridge the gap. A case in point is in the North of Scotland where one pitching stone was missed and a hole seen underneath it. When probed, the hole was 22 feet deep.

The above point is about the Megget dam. The protection on the slope, which was no doubt designed to appropriate design methods at the time, is beaching on a 1½ to 1 slope, and one of the troubles there is that, when stones are dislodged they don't readjust themselves, but they go right down to the bottom of the slope. Perhaps Mr Gallagher would comment on that?

D GALLACHER (R H Cuthbertson & Sons)

The 1½ to 1 slope: when the reservoir has been drawn down, the bulk of the displaced stones are still sitting on the slope. They do tend to form a ledge, but there is no doubt that odd stones do disappear to the bottom of the reservoir. When you do get an attack, you get a notch along the line and stones above may tend to slip out, but gradually an arching effect forms over that particular area; then if you get a collapse of that arch, you have the stones above falling in to where that gap is, so it does tend to be self-healing. When you get that type of movement then there is the possibility that stones will move further and slip to the bottom of the reservoir.

WRITTEN CONTRIBUTIONS

B KEMPES. (Hesselberg Hydro Ltd)

I would like to comment on the paper by W J Carlyle; namely 6.3.

I would like to describe the basic rules for the design of erosion protection constructions and relate specifically to the development of open stone asphalt revetments.

The development of open stone asphalt revetments began in Holland and for more than 20 years such revetments have been used in coastal defence works and inland waterways.

Over those 20 years they have also been successfully applied in many other countries such as, Belgium, Germany, Denmark, France and also in the United Kingdom.

In addition I would like to draw attention to one of the most sensitive areas nowadays: the environment open stone asphalt revetments blend and adapt well with the environment.

In my opinion wave heights resulting from severe storms are not directly the cause of damage but only bring to light already existing damage underneath the surface, caused by a failure in design or execution.

As long as permeable revetments are considered for erosion protection then the design must include three basic rules:

1 Permeability should increase from the inner to the outer side and every layer (sub-base, filter & revetment) should have an optimal filter stability to the adjoining layers.

2 The extent of permeable characteristics should be as similar as Possible for each layer - sub-base, filter and revetment. Water-tight areas with small open joints (e.g. concrete blocks) on top of a mineral or geotextile filter should therefore be avoided.

3 The revetment should be sufficiently flexible to be able to follow, besides the normal expected settlements, other deformations caused by failure in design or choice of products. Thus every deformation of the dam will be made visible enabling measures to be taken before a possible sudden and dangerous collapse occurs. A rigid construction with open joints (e.g. concrete slabs) and a lack of filter stability with the required permeability should be avoided.

Certainly, taking into account 1 and 2 above, we should be able to eliminate:-

(a) the uplift pressures.
(b) the downward ground-waterflow parallel to the slope.

Additional Notes:

(a) Uplift pressures.

For instance in concrete block revetments - continuous movement, causing in its turn the erosion of the subbase. An 'S' shaped slope would then result with negative consequences. Such deformations easily resulting in leakage and washing-out of the subbase material. Concrete blocks/armourstone being forced out of the revetment, as happened on the Kielder Dam!

(b) A parallel ground-waterflow could cause progressive erosion of the sub-base and so accelerate the process as mentioned above in note 'a'.

Using the three basic rules above we believe construction using Open Stone Asphalt Revetment to be the optimal solution. Additional details regarding design, tests and practical experience are described in text supplied to BNCOLD.

PROCEEDINGS: TECHNICAL SESSION 7

GRAVITY DAM DETERIORATION

PAPERS:

DISCUSSION:

Index 7 (cont)

REPLACEMENT OF EXPANSION JOINT SEALS AT CLYWEDOG DAM

P G Mackey DLC BSc MSc MICE MIWES
Principal Engineer, Severn Trent Water Authority

A C Morison BSc MICE
Senior Engineer, Sir William Halcrow & Partners Ltd

SYNOPSIS

The paper describes the investigation, design, implementation and performance of replacement expansion joint seals on the downstream face of Clywedog Dam.

INTRODUCTION

Clywedog Dam

1. Clywedog Dam is located in Mid Wales near Llanidloes, Powys on a tributary of the upper Severn. It has a catchment of some 50 km^2. The reservoir retains 50,000 Ml and is used to augment dry weather flow in the River Severn to support water supply and other abstractions.

2. The dam is a mass concrete, round-head gravity buttress dam 72m high and 230m long. It consists of eleven buttresses, basically equilateral triangles in elevation. These are flanked by two gravity blocks on the west abutment and three on the east. It is the highest concrete dam in the UK.

3. The spillway is capable of passing the PMF and comprises the crest of all the buttresses. Water discharges over the sloping downstream face of the dam on in-situ slabs and precast beams spanning between the buttresses, then down side spillway channels to a stilling basin at the toe. The surface of the spillway beams is stepped to dissipate energy. The arrangement of the dam is shown on Figure 1.

The Expansion Joints

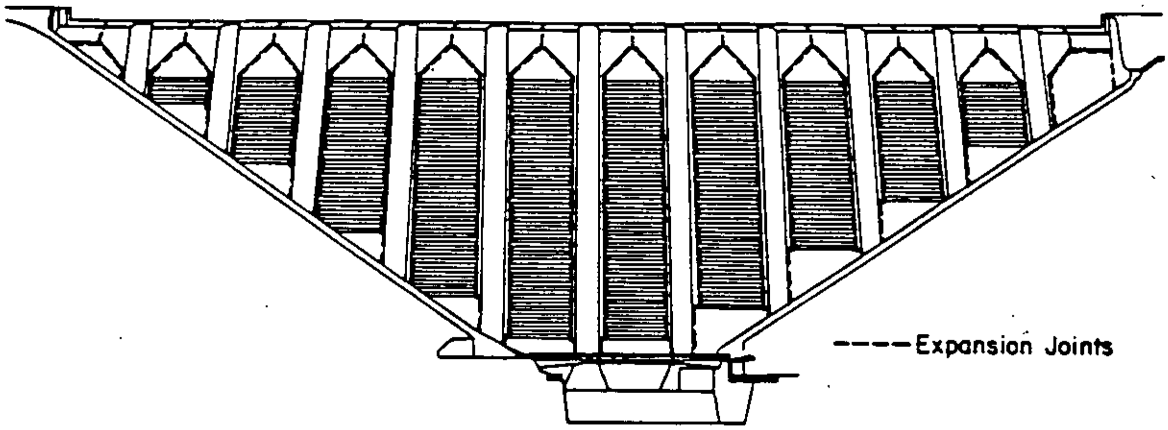
4. The upstream seal consists of two water bars, a rubber-bitumen plug and a drain. No problems have been experienced with this.

5. The crest roadway joint seal consists of a rubber water bar and a surface sealant. The upper gallery runs below the crest roadway.

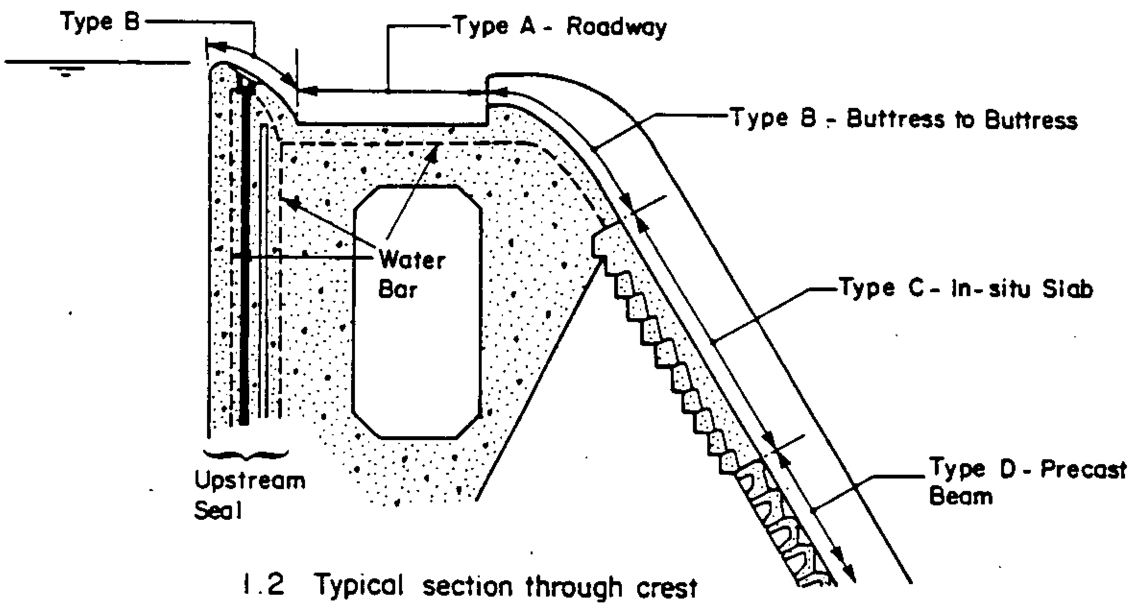
6. The expansion joints between the buttresses and the in-situ slabs and at the ends of the precast beams are sealed only with a surface sealant.

7. In the original construction all expansion joints were formed with 12mm bitumen impregnated cork filler and the joints sealed with polysulphide sealant.

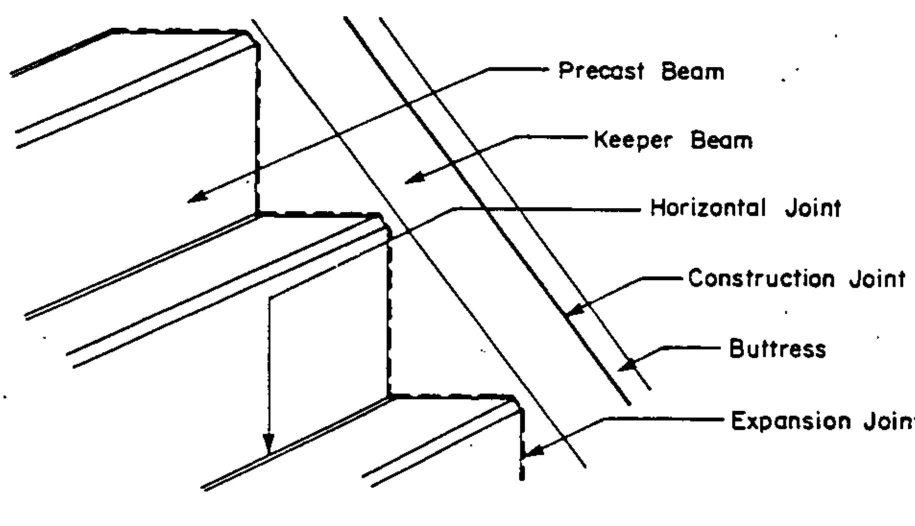
8. The horizontal joints between precast beams were sealed with a neoprene section in preformed channels in the beams.



1.1 Downstream elevation



1.2 Typical section through crest



1.3 View of joints at precast beam ends

Figure 1: Arrangement of Expansion Joints at Clywedog Dam

The Problem

9. There has always been some leakage through the joints on the downstream face during rainfall or spilling. Following the cold winter of 1981/82 leakage was seen to have significantly increased, with water running down the buttress faces and dripping extensively from the precast beams during prolonged rainfall. A piece of spalled concrete was also found, apparently from one of the precast beam supports and it was thought that frost action and the leakage at the joint could have caused this.

10. In 1982 Severn-Trent Water appointed Sir William Halcrow and Partners to investigate and subsequently to design and implement replacement of the sealant in the expansion joints on the crest and downstream face.

INVESTIGATION

Inspection

11. Inspection of accessible areas revealed that the original polysulphide sealant installed in 1967 had formed a hard crust or become completely brittle and had failed over almost all its length by parting from the concrete or by splitting. On the crest roadway repairs had been carried out in 1973 using polysulphide sealant and these too had failed, exacerbated by a variable sealant cross section due to a circular foam backing strip. Concrete nosings had also failed at a number of the roadway joints.

12. At the time of the investigation there was no access to the downstream face of the dam and so no inspection was carried out. From the evidence of accessible areas and the known age of the sealant, it was concluded that the sealant had reached the end of its useful life and should be replaced. This conclusion was amply justified by the condition of the joints and sealant found during the replacement contract.

Construction Records

13. Examination of the original contract drawings showed the 12mm expansion joints rebated for a 25mm square section rubber bitumen sealant. During construction a 12mm polysulphide sealant was substituted.

14. The original design records contained no information on expansion joint widths, sealant section or assumed sealant properties.

15. Construction records revealed a dispute with the sealing sub-contractor over responsiblity for preparing and cleaning the joints.

DESIGN

Objectives

16. Severn-Trent Water required that the sealant system be designed for maximum durability and so a study of alternative systems involving enquiries of previous users was carried out.

Joint Types

17. Four types of joint were considered:

- A the crest roadway where new nosings were required (about 75m)
- B the spillway crest and upper part of the downstream face where the joint was between the two adjacent buttresses (about 75m)
- C the upper and lower in-situ slabs where the movement was split between two joints on either side of the slabs (about 500m)
- D the ends of the precast beams, where the buttress and longitudinal beam expansion was taken by the two joints (about 880m)

Joint Movement

18. The temperature of the buttress concrete is monitored. From records the thermal range of the buttress sections was taken as 15°C. This produced movement of 2.6mm at joint types A and B and a contribution to the movement at joint type D of 0.67mm.

19. The spillway beams and in-situ slab are lighter than the buttresses and are inclined to the south and south-west. From BS 5400, for bridge deck design, a temperature range of 35°C was adopted for the beams, producing a movement of 4.45mm. Subsequent observations of surface temperatures suggested a range of 38°C.

20. Dam instruments suggested some possible seasonal movement of the valley sides. An allowance for this of 0.4mm per buttress was included in the joint design.

21. The total calculated joint movement at type A and B joints was 3.0mm. This compared with a maximum recorded movement of 2.8mm at any of the inter-buttress joint movement stations over a 10 year period.

22. At type D joints the total calculated movement was 5.5mm, divided between two joints. If evenly divided this would be some 2.8mm at each joint but even movement was thought unlikely in practice. For design a 2:1 movement distribution was assumed, producing 3.7mm movement at one joint. If one joint end were fixed, the total 5.5mm movement would occur at one joint. Movement at type C joints was considered similar to but less than at type D and the same criteria were adopted for design.

Movement Accommodation Factor

23. The movement accommodation factor (MAF) is defined in BS 6213(1), as "the maximum movement that a sealant is capable of tolerating throughout its working life, expressed as a percentage of joint width".

24. For type A and B joints ie joints directly between buttresses, the MAF required of the sealant was 25% for a 12mm joint and 15% for a 20mm wide joint.

25. For type C and D joints ie at the in-situ slab edges and precast beam ends, the required MAF was 31% for the 12mm joint at 2:1 movement distribution. This increased to 46% for a beam fixed at one end. Increasing the joint width to 20mm reduced the required MAF to 18% and 28%.

Materials

26. The following materials were considered for resealing the joints. Because of their shorter life, bitumen products were not considered.

- Compression seals in joint
- Surface sealing strips
- One-part and two-part polysulphides
- One-part and two-part polyurethanes
- Silicones

27. Compression seals were considered both on their own and as a backing to a conventional gunned sealant. In both cases the shape of the spillway steps would have made installing the material difficult. Control of joint width is also of major importance with this type of seal and the variation in width expected in the existing joints would reduce the effectiveness of the seal. Following discussions with manufacturers and contractors, the use of compression seals was considered impracticable.

28. Surface sealing strips were rejected because of the shape of the joint on the spillway steps. They would also have been extremely conspicuous on joints B and C to which they were technically best suited.

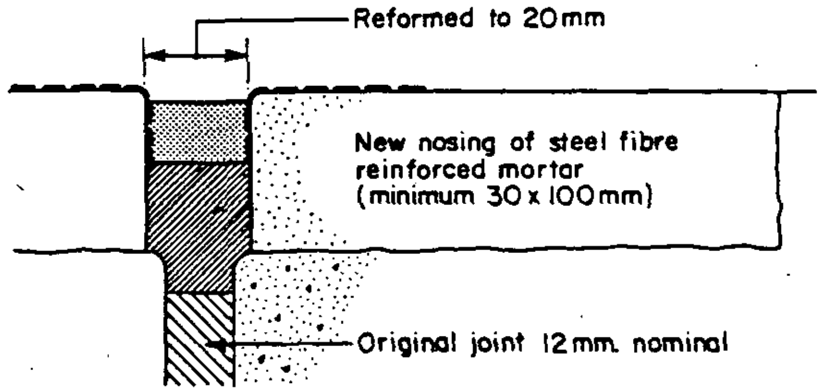
29. Two-part polysulphides and two-part polyurethanes were found to be the most widely used sealants and the two materials suggested by BS 6213(1), which considers them equally suitable. Manufacturers of the materials made conflicting claims about their suitability and companies who manufactured both gave no consistant opinion as to which would be most suitable for Clywedog. MAF quoted by manufacturers varied in the range 20-33%. Adequate site mixing was reported as a significant factor in subsequent problems with both materials.

30. One-part polysulphides and polyurethanes seemed to be similar to the two-part materials, except that they cure more slowly and most have lower MAF. They are not recommended by BS 6213 for this type of work. A one-part polyurethane with MAF of 50% (Sikaflex 15LM) was considered in some detail but, as a new material with little field record, it was considered imprudent to use it in a large-scale application.

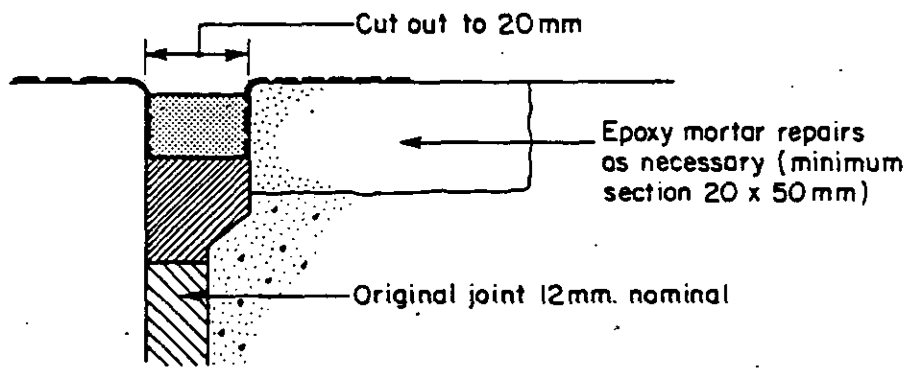
31. Low modulus silicones with MAF of up to 50% were suggested by some manufacturers. However, few have a history of use in civil engineering structures, although some have been used in buildings.

32. Only polysulphides have 20 years of use but BS 6213 considers the expected service life of all the above sealants to be 20 years. Polysulphides are known to age harden. Polyurethanes and silicones are expected to remain more elastic once successfully installed.

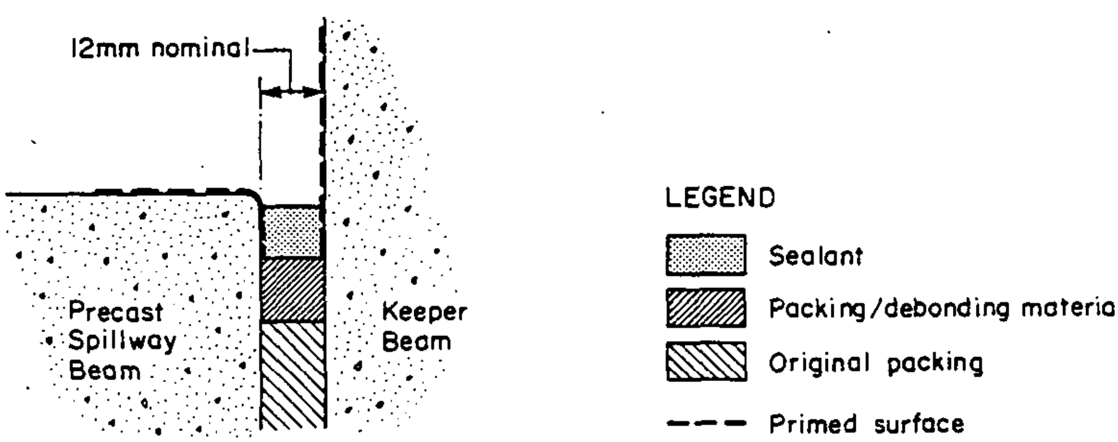
33. Having reduced the choice to two-part polyurethanes and two-part polysulphides, details and case histories of both were obtained and discussions held with manufacturers. Advisory bodies and a number of previous users in similar circumstances were also consulted. From these enquiries it was apparent that a stong, but not universal, body of opinion considered the performance of polyurethanes superior to polysulphides if carefully installed. Several past users commented that they had used



2.1 Roadway Joint (Type A)



2.2 Buttress to Buttress and In-situ Slab Joints (Types B and C)



2.3 Precast Beam Joint (Type D)

Figure 2: Joint Details

polyurethanes successfully in circumstances where they had experienced difficulties with polysulphides and other materials.

34. It because clear that one particular polyurethane sealant, Flextron 2 manufactured by Berger Elastomers, was most frequently recommended and moreover had an MAF of 33%. This material was specified for the work.

Joint Width

35. The 33% MAF of Flextron 2 was the minimum acceptable for the nominally 12mm spillway beam joint which in the design condition required a MAF greater than 31%. (Para 25). Some design factor of safety would have been preferred, especially to allow for possible fixing at a beam end. Consideration was given to increasing the joint width by concrete cutting or grinding, but this was rejected because the arrangement of the spillway beams made access to the joint difficult. In fact most of the joint was found to be more than 12mm wide due to concrete shrinkage. Where practicable the joint was formed or cut to a nominal 20mm width. Figure 2 shows the adopted arrangement of the joints.

Trial Materials

36. Had a suitable proven sealant with a movement accommodation factor of 50% been found this would have been preferable. Two unproven materials with MAF of 50% had been recommended by manufacturers and trials of these were proposed. The materials were Sikaflex 15LM, a one-part polyurethane and Silpruf, a low modulus one-part silicone.

Specification

37. Failure of expansion joint seals relates as much to preparation of the joint as the sealant used. A particularly high standard of preparation was required and the following were covered in the tender specification:

- attendance by the sealant manufacturer during installation of initial sample sections and at intervals during the works
- cleaning of the joint by grit blasting
- use of rectangular rather than circular foam backing strips
- use of debonding tape where appropriate
- use of primers for sealant
- concrete repairs to joint sides as required
- arrangements for suspending work in poor weather conditions, including payment for abortive work and standing time
- a 3 year maintenance period

Timing of the Work

38. Flextron 2 is an elastic sealant, although it does have some initial plastic properties. It was, however, recommended for installation under average rather than extreme joint opening conditions. Annual movement peaks occur in February and August. April-June and October-December were therefore the preferred times for installation. The April-June period offered better weather conditions, however this period coincided with

maximum retention level in the reservoir. Retention level was limited to lm below crest level during the work, giving reasonable notice of spilling.

IMPLEMENTATION

Tenders

39. Tenders were issued to 11 contractors in February 1983. Nine of these submitted tenders which ranged between £22,900 and £89,500. The contract was awarded to Kottler and Heron Construction in March 1983.

Access

40. Access to the downstream face of the dam was always recognised as a major constraint on the works, and a significant factor in the cost. Access equipment was also required for follow-up inspections and any other work on the downstream face. Severn-Trent Water therefore awarded a separate contract for £9000 to Stevens and Carter to design and supply a purpose-built permanent platform. This was designed to carry either two men and 200lb of equipment or three men. The platform was based on a triangular truss some 10m long running on wheels on the keeper beams at the ends of the precast beam and supported on wire cables through two electrically operated "Power climber" units, one at each end of the platform. The platform was handed over to the Contractor for the contract, although Stevens and Carter moved the platform between bays on the dam.

Site Establishment

41. Work from the cradle remained the critical activity throughout. This limited the contractor's site team to three men, two of whom worked on the cradle. The third provided support to them as required and carried out work on the crest and elsewhere. One of the three was foreman/ganger and the contractor's agent visited as required.

42. Supervision was provided by a full time inspector of works and a visiting engineer's representative.

Progress

43. The work was originally programmed between 1 April and 30 June 1983. Work actually commenced on 11 April and proceeded slowly through April and into May in very poor weather conditions. The weather improved during May and the work, including additional work, was completed by 29th July, within an agreed extension of time. Delays were due to problems with the cradle and exceptional weather conditions in the early part of the contract.

44. Some time could have been saved, particularly in checking, if the cradle had been capable of supporting three men and their equipment.

Workmanship

45. The Contractor's workforce were all full-time employees experienced in sealing work who knew the techniques to achieve a good quality result. On completion all agreed that the quality of the joints was high.

Contract Value

46. The contract was finally valued at some £19,500 compared with the tender total of £24,274. The difference was largely due to inclusion in the tender of provisional items for concrete repairs subsequently not required. The contractor also carried out work valued at some £6000 covered by separate work orders, mostly concrete repairs and crack sealing.

SUBSEQUENT PERFORMANCE

Leakage

47. Within a few months of the completion of the contract it became clear that, although the amount of water entering the downstream face was less than before the joints were sealed, significant leakage still took place. Inspection of the replaced expansion joints showed them apparently intact.

48. During the contract an open construction joint had been noted beside the keeper beam retaining the precast beams. This and the joints between the precast beams were tested using hoses on the downstream face and both were shown to conduct leakage to the interior of the dam.

49. It was not possible to confirm that the expansion joint was completely sealed but, except for a few minor defects, it appeared so. Movement monitoring of the expansion joint and construction joint were carried out through the winter of 1983/84. This showed almost as much movement at the construction joint as at the expansion joint. Following deliberation on whether this was a sealing or structural failure, it was decided that the keeper beam should be restrained.

Further Works

50. A second contract was undertaken in 1984 to fix the keeper beam using resin bonded stainless steel anchors. The construction joint and some joints between precast beams were also sealed. Design and installation of the anchors is not covered here, however it is possible that tensioning the anchors may have opened the sealed expansion joint.

51. This second contract produced a further reduction in leakage to the interior of the dam, but not its total elimination. This was confirmed by inspection during overtopping between 11 and 13 June 1985.

Inspection, August 1985

52. An inspection of the whole downstream face was carried out in August 1985. This confirmed that most of the 1550m of joint installed under the original contract was in good condition. Particular leakage had been noted between Buttresses 9 and 10 and some 1.5m of joint was found to have failed by loss of adhesion. The sealant here was lighter in colour, much softer than the remainder and still tacky. The manufacturer ascribed this to incomplete mixing of the sealant. This and a similar length found elsewhere were replaced by the contractor during the maintenance period.

53. A further 50 or so minor defects were also found. These were isolated areas which could allow seepage, but not significant flow to pass the

joint. The most common of these were areas, typically about 50mm long, where adhesion had been lost at one side of the joint. Some of these seemed due to loose sand or dust on the joint surface. No other problems were found relating to the joint sealant. Other defects included open concrete cracks, spalling of concrete or local honey-combing and vegetation growing through the sealant. The combined length of these was less than 5m.

54. The lengths of each trial material on the downstream face and the crest were particularly inspected. The Sikaflex 15LM showed a number of areas of adhesion failure. The Silpruf appeared generally satisfactory.

Inspection, March 1987

55. This confirmed the general findings of the previous inspection. The worst areas had been repaired and were satisfactory. The minor defects had not extended or increased significantly. The Silpruf trial showed some adhesion failure.

CONCLUSIONS

56. The need for reasonably waterproof seals on the downstream face is peculiar to buttress dams with a spillway on the downstream face. However, this has provided a suitable test bed for sealants, with the unusual *characteristic of their performance being visible. It is hoped that the* experience gained will be useful elsewhere.

57. With care taken in the design, specification and installation a joint has been produced which, after repair of one significant problem, contained some 0.3% of defects. A significant proportion of these relate to the structure rather than the seal. Visible signs of leakage are still apparent within the dam during overtopping and after persistent rainfall, although it is suspected that much of this orginates from sources other than the resealed joints. Futher efforts to achieve a total seal could continue, but would provide diminishing returns.

58. The original designers considered a complete seal of the downstream face unnecessary, or they would have included water bar in the joints.

59. The use of surface joint sealants with an expected service life of 20 years or less on a dam with a virtually unlimited service life requires particular consideration at the design stage.

60. This project preceeded the publication in 1987 of CIRIA T N 128(2). Had this been available, it would have greatly simplified the design task. It is commended to anyone undertaking a similar project.

ACKNOWLEDGEMENT

The authors thank Severn-Trent Water for permission to publish this paper.

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RENOVATION OF TRES AND COREDO RESERVOIRS AND DAMS IN PROVINCE OF TRENTO

Associated Professor P Mazzalai
Associated Professor S Odorizzi

University of Padua

SYNOPSIS

1. Some of the old dams (masonry and earth dams), that were built in Italy at the beginning of this century, are now under reevaluation of safety, according to the latest guidelines and regulations. Their hydraulic and structural characteristics are to be upgraded in order to combat long term deterioration.
This paper describes two well documented case-histories.

2. The reservoir of Tres is enclosed between two gravity masonry dams. The reservoir and the dams, since construction in 1930, have suffered from bottom seepage problems.
Waterproofing work in 1963-64, proved to be sufficiently effective for a few years. Successively, investigations on the masonry dams were carried out with the aim of stating the soundness of the structural work.
Based on these investigations, a plan of intervention was designed, mainly devoted to the consolidation of the dams by a grout injection, a drainage and waterproofing of the up-stream surface with an auxiliary concrete wall. All these operations are works presently under construction, as well as an adequate spillway and a bottom outlet
The reservoir of Coredo consists of two earth dams constructed in the 1930's.

3. The two structures are lacking in any waterproofing. Moreover, the greater of the two dams is made of poor and heterogenous material. Again a plan of intervention was designed, and this is presently starting. Waterproofing of the smaller dam, whose body is just reached by the reservoir water, was restored by a concrete diaphragm excavated and jetted in place in the centre of the section. The greater structure, even if it were equipped with suitable waterproofing, would not present sufficient safety guaranties. Therefore, the demolition of the existing structure and its replacement with a new dam constructed with classified materials, equipped with drainage systems, and with an impermeable mantle on the upstream face and with a foundation waterproofing consisting of a concrete diaphragm were decided. In this case too, the construction of an adequate spillway and bottom outlet, absent until now, is foreseen.

COREDO RESERVOIR AND DAM

Features of the Reservoir and Dams

4. The Coredo reservoir is located in the Province of Trento in North-Eastern Italy. The reservoir, which has the purpose of irrigation,

is placed in a North - South orientated depression (figure 1).

5. In order to form the storage, two earth dams were built in the 1950's (figure 2). The obtained water volume is about 350 000 m3 and maximum water level is 875.36 m a.s.l.

6. Downstream the North placed dam, called main, in contrast with the other one called auxiliary lies a natural lake, with irrigation regulation, licking the toe of the main dam.

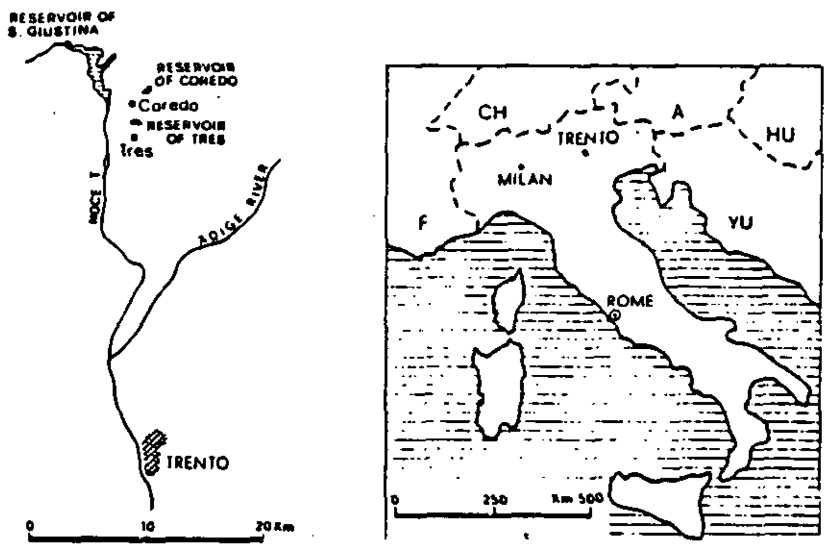


Figure 1: Location map of the reservoirs

7. The body of the main dam is 7.7 m high and its length is about 250 m. The crown, with a width from 11 m to 16 m, has an altitude of 877.5 m a.s.l. The face slope is 0.5. The dam doesn't have waterproofing or drainage system and is formed of materials of poor mechanical characteristics and high permeability analogous to those of the foundation.

8. The auxiliary dam consists of a massive structure with a length of 180 m and a crown elevation of 879.5 m a.s.l. The embankment material and the foundation soil have slightly better characteristics in comparison with those described for the main dam.

9. In the last operation years, softening in the East abutment of the main dam and considerable seepage in the dam body occurred. With respect to the auxiliary dam, an occasional softening of the soil downstream the embankment has been noted. These observations, added to the lack of efficient spillways and bottom outlets, suggested that the reservoir should be put out of service.

Geological Features

10. The depression where the reservoir lies is of glacial origin. The emerging soils are made up of microcrystalline and intraclastic dolomites, crystalline limestones and, at the bottom of the valley, a bank of moraines and alluvial soil belonging to the Quaternary. Tectonic faults have not been found.

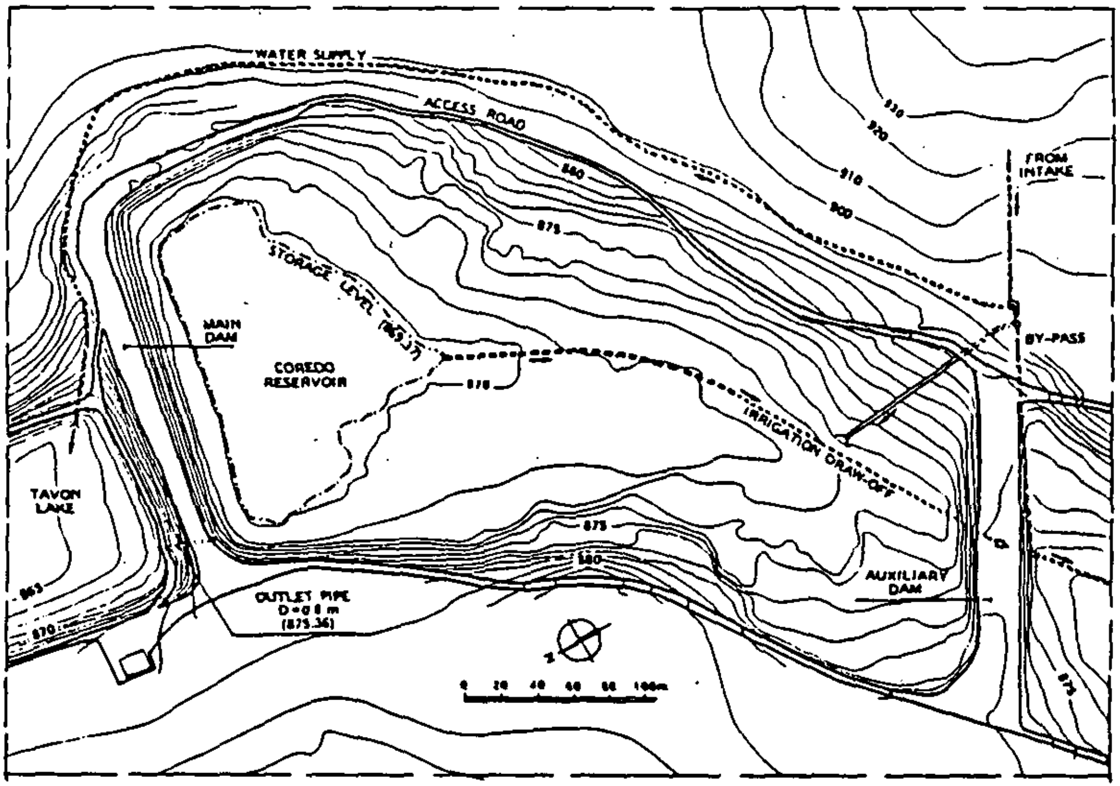


Figure 2: Lay-out of Coredo reservoir: actual situation

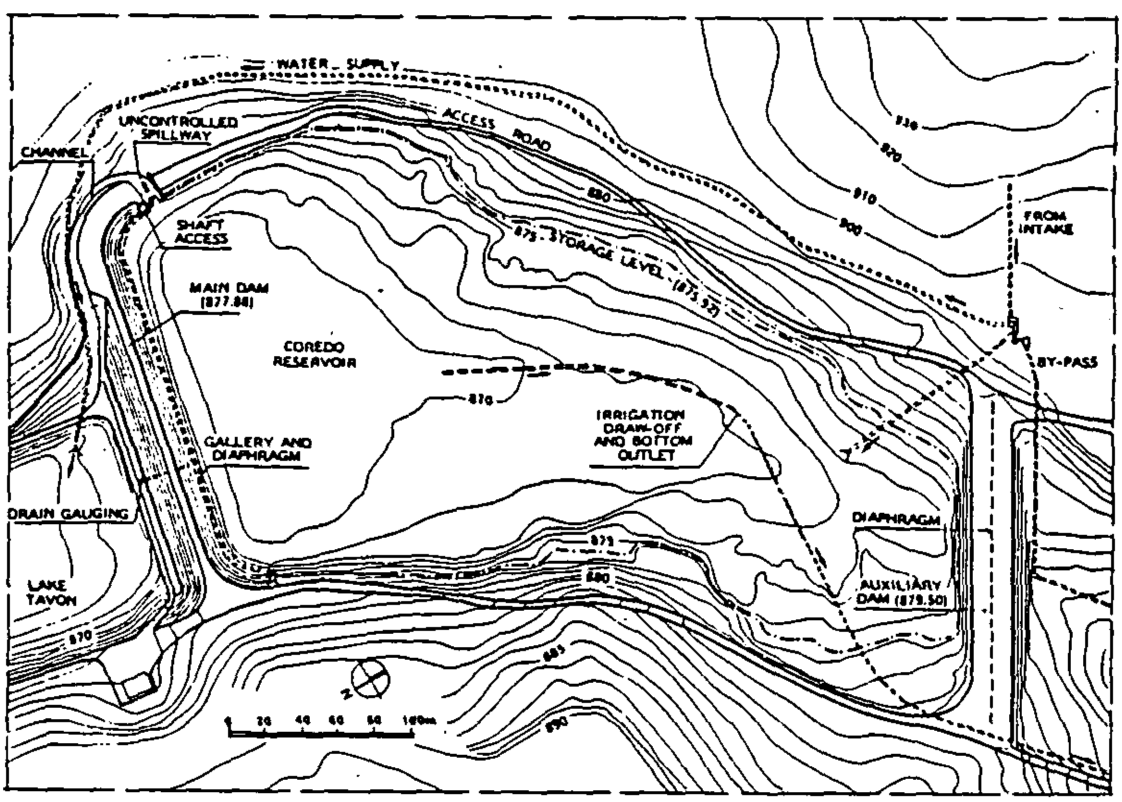


Figure 3: Lay-out of Coredo reservoir: designed situation

Geognostic Investigations

11. In the embankments and in the relative foundations borings and S.P.T. were carried out; permeability tests and undisturbed samples were taken. Dynamic penetrometric tests, in situ density tests and seismic explorations were also carried out.

12. The boring logs obtained have indicated the existence of morainal bank in a whitish slimy ground in the foundation soils and in the embankments the presence of sand, gravel and brown clay in a clayey ground. A relevant soil dishomogeneity, both vertical and horizontal was noticed

13. The granulometric composition, in accordance with ASTM D-653 is: gravel 40 %, sand 25 %, clay or silt 35 %.

The S.P.T. tests indicate not much dense soil in the dams and middle dense to dense soil in the foundations. Permeability, moderately changing along with depth, shows values from 1*10E-6 cm/s to 1*10E-7 cm/s, that are high values for the structure in consideration. Water table measurements have indicated spatial variableness of the levels, correlated to the different soil thicknesses. The penetrometric investigations reveal a low number of blows, varying highly from test to test. In situ density tests, performed on the embankments, indicate unit weight from 1.91 g/cm3 to 2,25 g/cm3 and natural water content from 4 % to 37 %. Over the refracting bottom stratum (2300 m/s - 3100 m/s), the seismic explorations showed a middle dense and saturated stratum (dam foundations 1200 m/s - 1700 m/s) and a low dense stratum (dams 400 m/s).

14. The laboratory tests have furnished the following results:

Table 1 - Laboratory test results

test	main dam		auxiliary dam	
	embankment	foundation	embankment	foundation
Pocket penetrometer [kg/cm2]	0.63-0.87	1.26-1.56	0.50-0.64	2.70-3.17
Torvane [kg/cm2]	0.20	0.76	0.15	> 1
Natural water content [%]	17	17	20	20
Liquidity limit [%]	31	28	33	38
Plasticity limit [%]	18	18	20	20
Plasticity index [%]	13	10	13	18
Bulk density [kg/cm3]	2.01	2.01	2.05	2.05
Specific gravity mass [km/cm3]	2.71	2.71	2.73	2.73
Friction angle [degrees] ϕ	24	24	24	24
Cohesion c [kg/cm2]	0.4	0.4	0.4	0.4
Grain size distribution [%]				
gravel	33	26	28	22
sand	19	20	21	16
silt	40	46	46	49
clay	8	8	5	13

Criteria of The Redesign

15. The aim of the renovation design is to waterproof the body of the dams and the foundations soils, to drain under the waterproofing system and to build adequate spillways in order to discharge maximum likely flood, and to build bottom outlets in order to empty the reservoir in a short time.

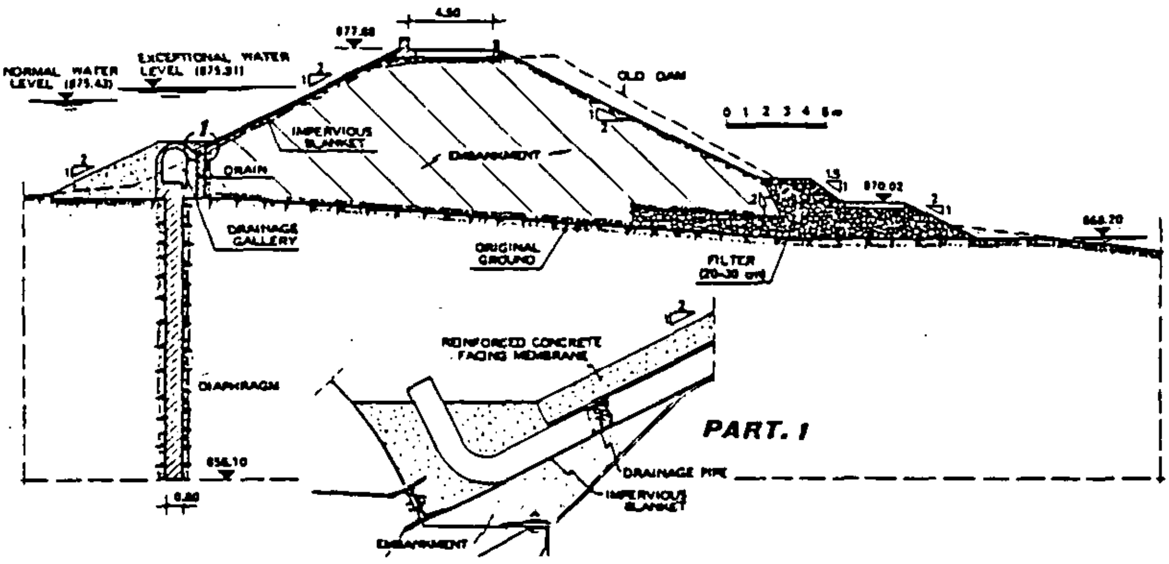


Figure 4: Cross section and particulars of the Coredo main dam

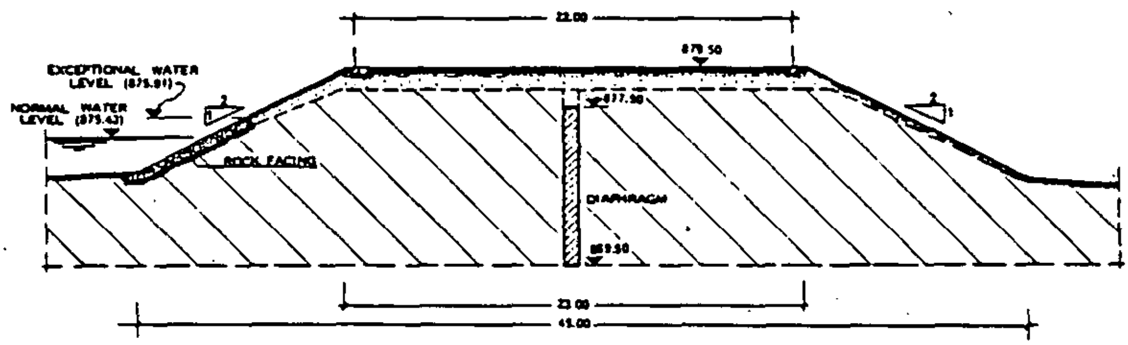


Figure 5: Cross section of the Coredo auxiliary dam

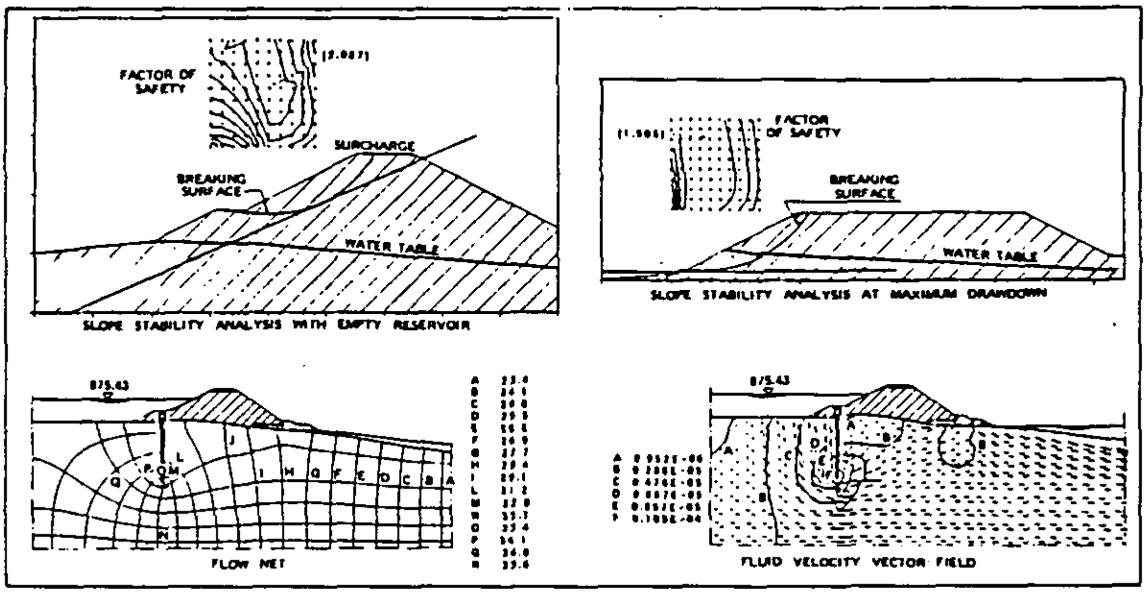


Figure 6: Stability and seepage analysis of the Coredo dams

16. Developing the re-design of the dams led to doubts as to whether the whole work should be rebuilt or reinforced. This was decided mainly because of the fact that the actual physical continuity of the dam was not depicted by the tests performed and that the static and hydraulic checks were consequently insufficiently reliable. Above all, even though additional tests were carried out, quality of the materials and their properties seemed not to be assessed as having the necessary safety. Hence, this being an indispensable condition for the renovation of the work, it seemed more convenient and safe to decide on complete re-building.

17. The auxiliary dam is different. Here the materials quality is better and, moreover, the dam is only licked by the reservoir water level itself. Consequently, there is the assurance that a convenient waterproofing of the dam and of its foundations can remedy the deficiencies noted.

18. For the main dam rebuilding the use of quarry material is foreseen The quarry has been already chosen and laboratory tests consisting of granulometric analysis, compaction tests, permeability tests and consolidated, drained triaxial tests have been carried out. The particle size characteristics indicate the practical absence of silts and clays (5% passing the n.200 sieve), 25% sand, and the remainder is gravel and cobbles. Standard compaction tests indicate the possibility of reaching unit weight from 2.31 kg/cm3 to 2.35 kg/cm3 with moisture content from 6.7% to 5.5% . Permeability, on compacted samples, varies from 1*10E-8 cm/s to 1*10E-9 cm/s . Triaxial tests have furnished an average friction angle of 38°, with 95% of compaction.

19. The design section of the main dam (figures 3 e 4) foresees a homogeneous embankment with a 0.5 face slope. Waterproofing is performed by a concrete revetment, which is spread on the upstream face combined with polythene sheets. This revetment is fixed at the base to an inspection and drainage gallery that is connected to a concrete diaphragm that reaches a depth of 15 m The contact between the dam and its foundation is adequately drained by a porous layer which is connected to a filter placed at the base of the downstream face.

20. Auxiliary dam waterproofing (figure 5) was obtained by a watertight diaphragm, built at the middle longitudinal section, reaching a depth of 10 m. The upstream face toe has been protected by a rock facing rip rap of adequate sizing.

21. The modalities and the functional characteristics of the works have been dimensioned by static and hydraulic verification. Both embankments were tested by the Bishop modified method on circular straight section cylindrical surfaces, also affecting the bottom alluvial formation. Filtration calculations have been made by a finite element model, considering the continuity at the nodes and in the permanent flow condition (figure 6).

22. The minimum values of safety factor obtained were:

for the main dam with empty reservoir equal to 2.03;
for the main dam with reservoir at maximum water level equal to 2.28;
for the auxiliary dam, at maximum drawdown water level equal to 1.51.

23. Waterproofing checks indicated that, in the main dam, there is no hydraulic rise in the dam body. The global seepage discharge is less than 0.01 m3/s and the flow velocity is about 2*10E-5 m/s . In the case of the secondary dam it is verified that there are not, in the maximum water level condition, downstream water bearing stratum levels that could give soil softening. The calculated global seepage discharge is 0.0085 m3/s and the flow velocity is 5*10E-5 m/s.

New outlets

24. Maximum flood discharges have been calculated starting from the hourly precipitations, extrapolated in accordance with Gumbel for a 5000 years return period, applying a kinematic approach. The unit contribution resulted 17 m3/s/km2 that furnishes, considering the catchment basin (1.04 km2) and the reservoir lamination storage, a discharging flood of 11.3 m3/s . Maximum flood is discharged by a spillway placed in the East side near the main dam in a channel entering the Verdes stream.

25. The bottom outlet has been obtained rebuilding and increasing the actual primary irrigous draw-off. The lay-out has been improved, and has been removed from the auxiliary body dam. At the end of works the maximum discharging flow will be 0.7 m3/s. The bottom outlet will allow the complete emptying of the reservoir in 5.5 days.

TRES RESERVOIR AND DAMS

Abstracts

26. Tres reservoir was built for irrigation purposes. It is placed in a depression that crosses the crest at the North side of Tres in the Province of Trento (figure 1). The morphology of the site required (figure 7) the construction of two dams to create the necessary reservoir. The two dams are placed in the North and in the South of the depression. The reservoir storage is 180 000 m3 with a maximum water level of 839.43 m a.s.l. The dams, built in 1929, are masonry gravity dams with blocks inlayed with mortar on the upstream and downstream surfaces. The foundation is a concrete non-reinforced slab with quite coarse granulometry. The maximum height is 12 m and the lengths are 275 m and 153 m respectively. The crown elevations are both 840.54 m a.s.l.

27. The reservoirs have not been used to their full capacity because, since the beginning of their use, there have been water losses, both through the bottom of the reservoir and under foundations. Once the construction work was completed, a waterproofing was effected but this didn't proved to be well done. Consequently storage was limited to 30 000 m3. In the 1960's the bottom of the reservoir was waterproofed with the addition of polythene sheets and clay. This work limited water losses, without solving the problem completely.

28. The dam and the reservoir was fully redesigned in order to achieve total reservoir storage. The new project also included the adjustment of the bottom outlets, in accordance with required safety regulations.

Geological Features

29. The reservoir area and the site of the dam are mainly composed of Jurassic red ammonitic limestone. The stratification layer is variable, so that the strata tend to dip toward outside the examined zone.

Geognostic Investigation

30. Borings and permeability tests were carried out in the body of the dams and the relative foundations. Specimens of concrete and bed rock were sampled.

31. The dam face blocks are made of rose coloured limestone with no deterioration resulting from freeze-thaw cycles and cleavage. The inner masonry is made of mortar limestone formed of scarce hydraulic lime and coarse sand. Mortar is porous, often cohesionless and difficult to be sampled, also showing many discontinuities. The concrete of the foundation shows better homogeneity and quality. The foundation rock is calcareous, and quite compact with veins and breaks closed by calcite re-crystallization. However, there are zones presenting Karst phenomena.

32. Lugeon tests generally showed high permeability values. In the body of the dams values are in the range of 1*10E-1 m/s to 1*10-2 m/s, in foundation rock in the range of 1*10E-4 m/s in the cracked zones to 1*10E-1 cm/s in the Karst zone.

33. Mechanical laboratory tests performed on a sufficiently large range of samples have indicated compression crack loads from 81 kg/cm2 to 117 kg/cm2 for dams and from 1350 kg/cm2 to 2010 kg/cm2 for foundation rocks. The coefficients of elasticity have been drawn in the range of 200 000 kg/cm2 to 160 000 kg/cm2 , for the concrete, while the foundation showed higher values. The foundation rock has a coefficient of elasticity of 60 000 kg/cm2 .

Criteria of The Redesign

34. The materials of the dams proved to be inadequate both from the mechanical and homogeneity point of view. On the other hand, the tests showed good concrete injectability, including the layer in contact with the foundation. But of course the injections reduce permeability drastically while affecting the mechanical characteristics only marginally.

35. The foundation rocks have fairly good mechanical quality, and high average values of permeability. In this case too the execution of grout curtain and foundation soil injection will raise the foundation impermeability to acceptable values. In the case that the screen encounters Karst holes, suitable pre-obstructions by course dry materials have to be done.

36. Therefore, from the permeability point of view, the works are shown to be perfectly recoverable, by means of usual measures in hydraulic work and restorations. From the mechanical point of view, the stress state is too high and needs an auxiliary reinforced concrete structure connected to the upstream face of the old dams.

37. Other aspects to consider are those relative to the verification of the

existing watertight facing on the reservoir bottom and those relating to the outlets renovation.

Dams Renovation

38. The present step of the renovation work is the consolidation of the masonry structures by perforations and injections of binary mixtures containing additives (figures 8 and 9). Successively, when the bottom level of the foundations is reached by excavations, a screen of injections will be completed to consolidate the upstream face of the foundation itself. After that, the auxiliary reinforced concrete structure will be shaped. This will be connected to the existing structures with steel bars in order to achieve monolithicity. Between the old and the new structures a drainage system will be inserted and extended into the foundation where a collecting tunnel will be built. Finally, sewing and injections in the foundation will be executed. All the injections will be performed on the base of progressive thickening to keep their effectiveness under control.

39. To determine the best composition of the concrete, investigations and tests have been done, taking into account the mineralogic characteristic, sizing and distribution of the aggregate, choice of the cement type and additives. The chosen aggregates are made of acid intrusive rocks and of metamorphic rocks. The size of aggregates are stated in four classes, according to a parabolic curve with a power of 0.33 including cement, and to a maximum size of 76 mm. The cement is an A.S.T.M. class of type V. The dosage is 210 kg/m3 with a 0.50 water/cement ratio, and a proportion of different additives. The resulting concrete is homogeneous and not segregable, with a 10 cm slump, 2474 kg/m3 specific weight, 194 kg/cm2 7 days compressive stress, 336 kg/cm2 28 days compressive stress, 250 000 kg/cm2 coefficient of elasticity and 1*10E-9 cm/s permeability.

40. The static and dynamic answer to the behaviour of the old structure and the auxiliary one have been investigated using finite element models. The results obtained have been compared with those of the original structure both of which have been modelled by finite elements. The discretizations adopted are shown in figure 10 . The constitutive laws of the materials have been drawn from the outcome of the above tests. The loads take into account various combinations: the self weight, the hydrostatic load and the pressure of ice. The main results are given in table 2.

Table 2 - Principal stresses [kg/cm2]

principal stress	old configuration	new configuration
Maximum compression stress	-3.12	-1.86
Maximum tensile stress	+6.14	+1.01
Tangential stress at the interface between old dam and auxiliary structure	- -	average value +0.15 max value +1.43

The first nine eigen modes of the structure were investigated giving periods in the range of 0.065 s to 0.007 s for the original structure and in the range of 0.072 s to 0.008 s for the new structure. The safety factor on the equilibrium of the horizontal forces in the worst condition

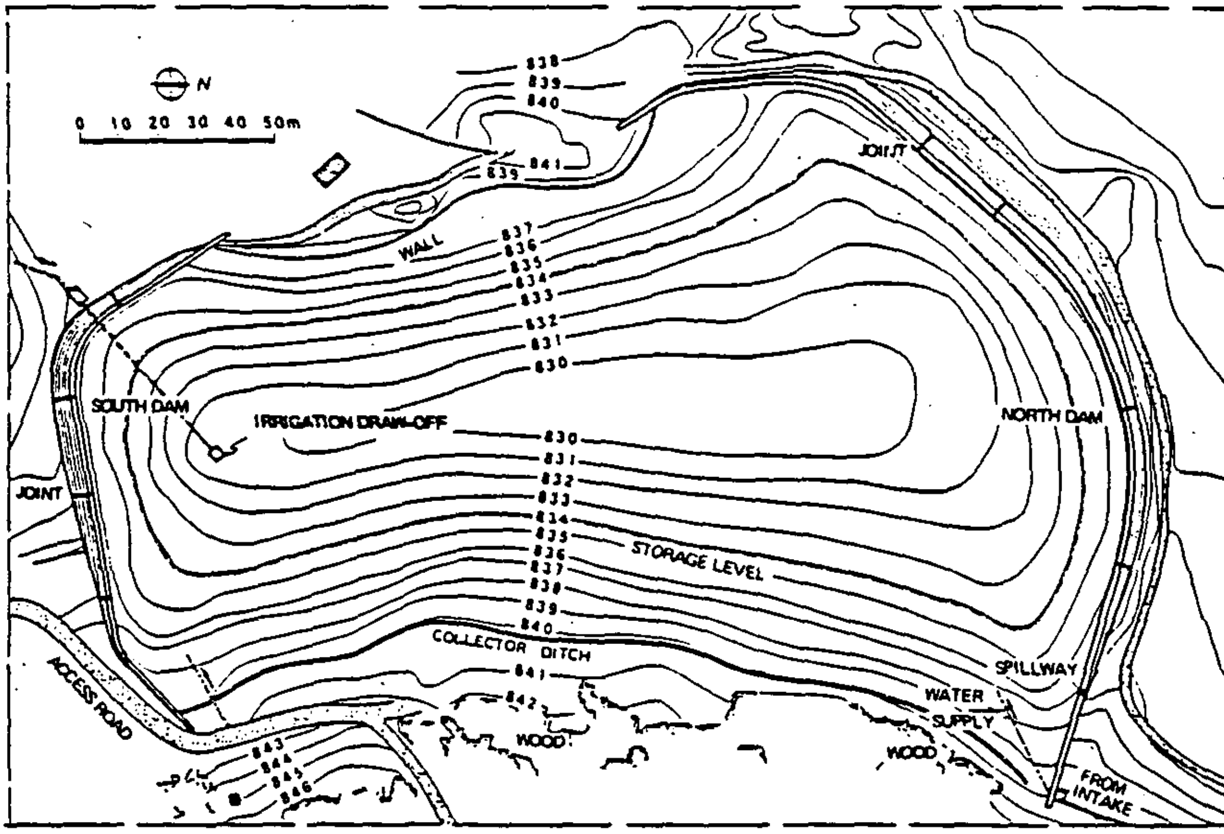


Figure 7: Lay-out of the Tres reservoir: actual situation

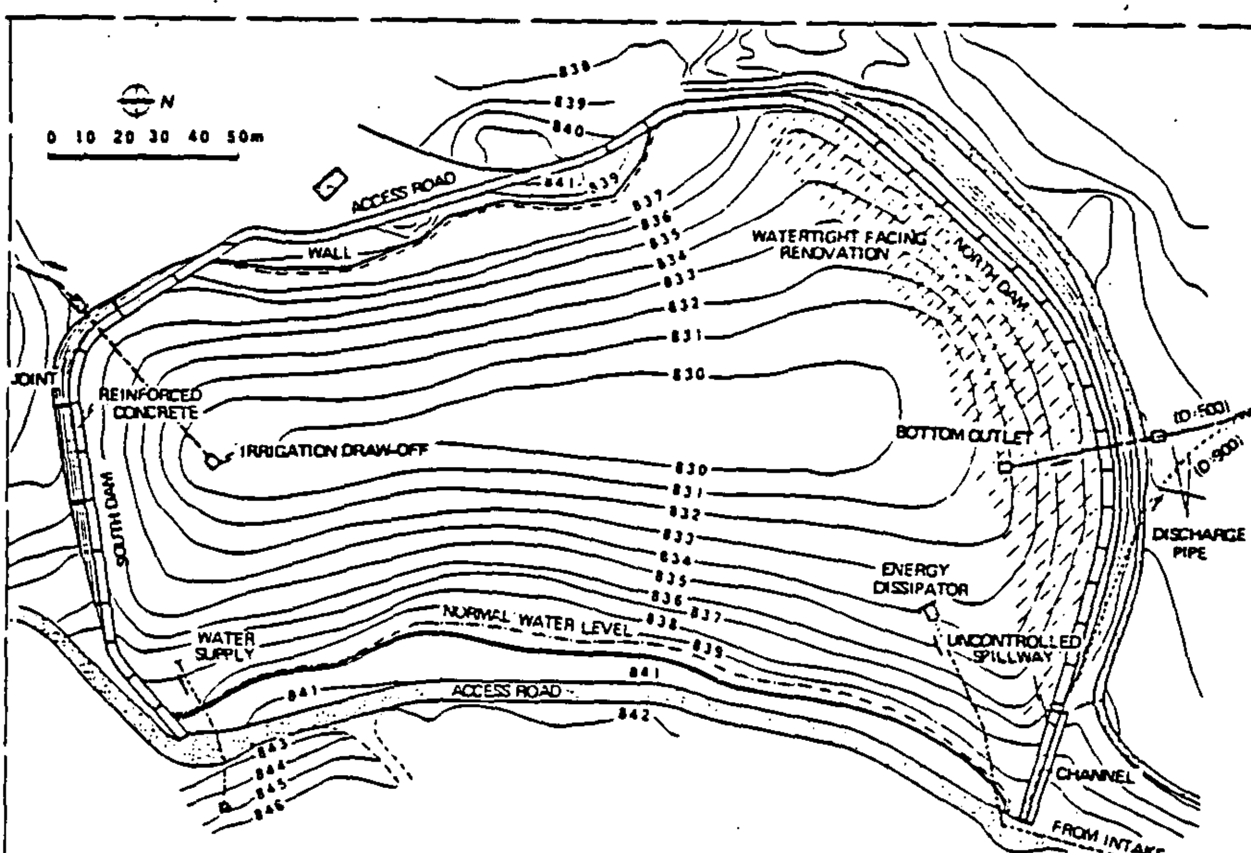


Figure 8: Lay-out of the Tres reservoir: designed situation

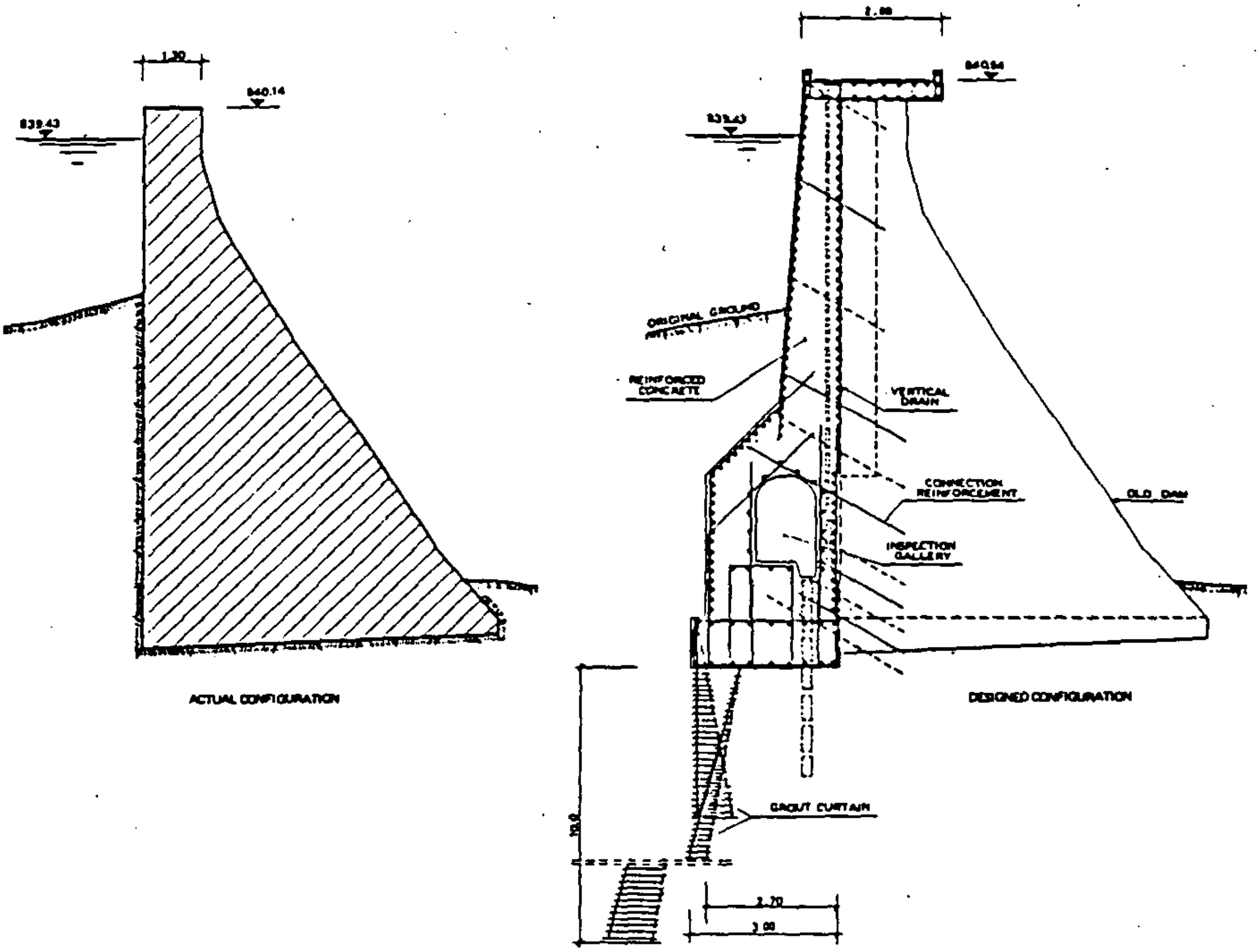


Figure 9: Typical sections of the Tres dams: actual and designed configurations

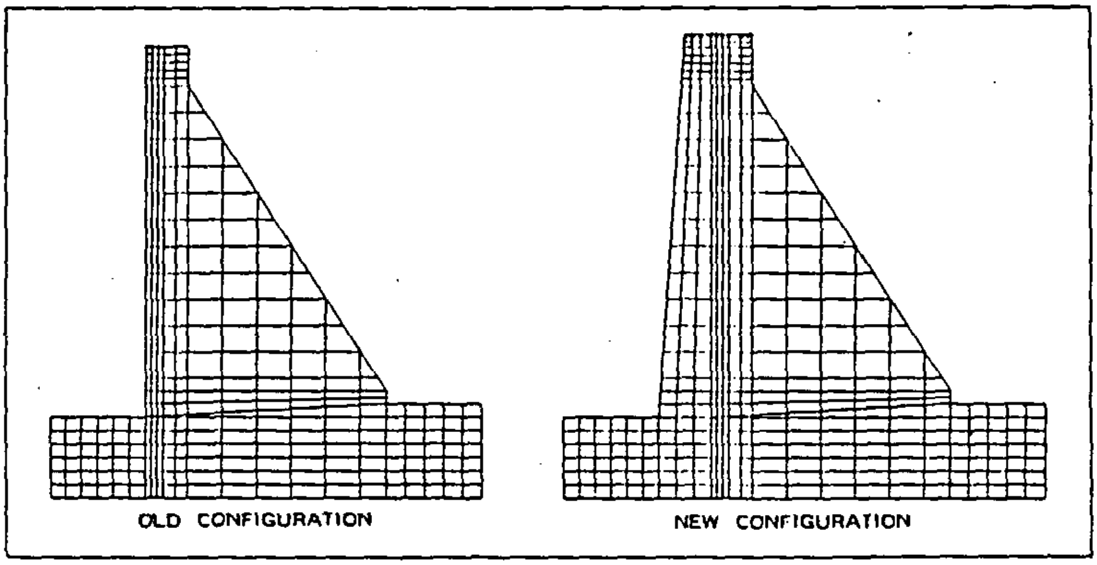


Figure 10: Tres dams: Mesh of the actual and designed solutions

was only 1.19 in the old structure while it is up to 2.56 in the new one, also because of the change in the soil saturation. It should also be stressed that the intervention doesn't substantially change the structural behaviour. This is proved both by the stress distribution and by the dynamic answer. The monolithicity of the complete structure is also well assured by the connection reinforcing bars, which are slightly overdimensioned to take the shear actions.

New Outlets

41. Maximum flood has been calculated using the unit peak discharge of 36.6 $m^3/s/km^2$. The catchment area is of 0.145 km^2. The flood is discharged by a new spillway placed on the north-dam, as indicated in the lay-out (figure 8).

42. The bottom outlet has been designed for a maximum discharge of 1.5 m^3/s. The same outlet will allow the complete emptying of the reservoir in 17 hours.

43. The discharges coming from the spillways and from the bottom outlets are taken downstream by pressure pipe-line, supplied with impact-type energy dissipators until they delivered into the Sette Fontane stream.

Conclusions

44. The approach, which was followed to state the principles of the renovation of the works here described, highlighted the importance of a thorough and complete investigation programme to be carried out on the site and in the laboratory. Only in this way can the necessary elements be collected to comply with the necessary safety while designing the intervention. In particular it must be decided whether the dam should be rebuilt as a whole, which would be technically and economically preferable, or whether it should be restored by renovation.

REMEDIAL WORKS TO BRAYTON BARFF SERVICE RESERVOIR COMPARTMENTS 1 & 2

C J A Binnie MA DIC CEng FICE FIWEM (Director)
T E A Askew BSc CEng MICE MIStructE (Senior Group Engineer)

W S Atkins & Partners

SYNOPSIS

Brayton Barff reservoir had been constructed in the early 1960's and was known to require attention to joints internally. During an inspection under the 1975 Reservoirs Act it was concluded that the leakage from the reservoir had formed voids beneath the reservoir floor. Ground probing radar was used to determine the extent of the voiding and visual confirmation was made using a boroscope through holes in the slabs. The voids were subsequently filled with grout and the reservoir joints reconstructed and resealed. This paper reviews some of the difficulties and successes of the remedial works undertaken.

BRAYTON BARFF RESERVOIR

1. The service reservoir at Brayton Barff is the main buffer storage facility on the Yorkshire Derwent aqueduct between the abstraction and treatment plant at Elvington and the main storage point at Hoober prior to delivery to Sheffield. It was constructed in two phases and now consists of a divided tank of 32 million litres capacity and a single tank of 32 million litres capacity. This paper relates to remedial works undertaken on the divided tank which is the older of the two reservoirs and was constructed in 1962. It was inspected in November 1986 under the Reservoirs Act by C J A Binnie and at this time various problems became apparent.

Problems at Brayton Barff

2. The reservoir stands on the top of a small hill formed of glacial till overlain by fluvio-glacial sands. Since construction severe leakage had been recorded and repairs to the joints inside the reservoir were undertaken within 3-5 years of the original commissioning. These repairs appear to have been inefficient in recent years and a drop test had revealed that substantial leakage was occurring.

3. A visual inspection of the interior of the reservoir indicated that the most likely cause of the greater part of the leakage was the failure of the floor expansion joints. Other factors such as the ageing and debonding of the rubber bitumen sealant in the contraction joints were possibly contributing to the leakage to a lesser extent.

Design of the Original Reservoir Joints

4. The reservoir is a traditional reinforced concrete design in most aspects, consisting of cantilever retaining walls and division wall, with a flat

roof slab supported on columns each with its own foundation independent of the reservoir floor. The reservoir floor itself is a 100mm thick reinforced slab on a 100mm thick concrete sub floor. Expansion joints are positioned between the floor slab and the cantilever wall bases and also through the floor slab in a north-south and east-west direction dividing the floor into 4 unequally sized portions in each of the two compartments. The two compartments are linked permanently by an opening in the division wall constructed subsequent to the main structure. Figure 1 shows the layout of the reservoir complex and indicates the position of the expansion joints in tanks 1 and 2.

5. The expansion joints at the perimeter of the floor and in the wall cantilever bases include water bars, but where the expansion joint transverses the floor slab no water bar was shown on the original construction detail and was not then built into the floor. This is not the case in the newer single compartment tank where all expansion joints have water bars. Where no water bar is present the water tightness of the floor becomes wholly dependent on the effectiveness of the sealant in the joint. Figure 2 shows the expansion joint details.

6. In the originally constructed detail expansion joints were built into both the sub floor and floor contrary to the construction drawing which shows expansion joints in only the floor. The joints were filled with fibrous joint filler of the flexcel type. The old remedial works referred to above, had involved removing the flexcel and filling the expansion joints to full depth with bitumen putty similar to plasti joint. A semicircular strip of plasti joint was formed proud of the joint and the whole joint capped with a thin glass fibre strip some 200mm wide bonded to the floor. The semicircular plasti joint formed a corrugation in the fibre glass thereby allowing it to flex without rupture. This joint was damaged during recent remedial works inside the tank and appeared to have been leaking copiously in recent years. Further examination of this joint revealed that the filler had debonded from one or other side of the joint over much of its length and simple leakage paths were evident.

7. Discussion with the operations staff at the reservoir revealed that sand had been collecting in the underdrain collection manholes periodically and that in recent years a significant volume of sand had been removed from the underdrain collectors and manholes. With the reservoir full there was flow detectable in the underdrain collectors and this flow varied in intensity according to the time of year. Larger flows were evident during the coldest periods.

8. It was therefore a reasonable assumption that the sand in the drains had been washed from beneath the reservoir and that there would be corresponding voids beneath the floor which needed to be located and filled before the reservoir could be returned to service. It was therefore agreed with Yorkshire Water that investigation work should take place to determine whether voids were existing beneath the floor and further what remedial measures were required to fill them.

9. The expansion joints would need reconstructing and resealing and since this operation was to be a significant exercise it was decided to reseal all of the bitumen joints rather than attempt to identify existing sealant which could be retained.

Investigation of Sub Floor Voids

10. It was agreed that a nondestructive survey technique would be most suitable if an effective one could be found. Both ground probing radar (GPR) and dynamic response techniques were considered but the former was thought to be more likely to provide good results if the correct conditions were found in the reservoir. Heavy clay and reinforcement are two notorious obstacles to GPR. Clay was not present in the subgrade and it was felt that the light mesh present in the reservoir slab would not present a problem. It was thought prudent though to appoint a survey contractor who could provide both techniques under review so that should GPR prove ineffective, the dynamic response technique could be used. The appointed contractor, Structural Testing Services of Southampton, also had some experience and success with GPR on reinforced concrete slabs and this encouraged its use in a trial. We were able therefore to expect that whilst the radar print-outs would indicate the position of reinforcement, sufficient information should be obtainable from the radar penetration between the bars providing the spacing of the reinforcement was reasonable.

11. An initial radar survey was undertaken to establish the effectiveness of the GPR technique and the most suitable wavelength of aerial required. Patterns on the radar print-out were observed which were thought to be voids and this was then confirmed visually by a boroscope placed directly into the void through a hole drilled in the floor slab. A typical radar print is shown on Figure 3. In the event the light reinforcement in the upper floor slab did not seriously reduce the effectiveness of the radar survey.

12. A second more rigorous survey was then conducted of the whole floor slab. Various locations where voids or loose formation were suspected were recorded on a master plan. A visual inspection, using the boroscope, of groups of these areas showing similar disturbance patterns, was undertaken to establish the print-out patterns representing voids and those representing loose sand. Detailed analysis of the radar print-outs was then possible and the contractor prepared a floor plan showing suspect areas of voids and loose formation. Figure 4 shows a typical floor plan with voided areas marked on and Figure 5 shows a more detailed plan of the voided area.

13. Further exploratory confirmation of the contractor's interpretation was then carried out using the boroscope confirming that his conclusions were correct. The nature of the voids was variable between 'warren like' tunnels to an extensive area of approximately $1m^2$ in one place.

The Nature of the Sub Floor Voids

14. The survey results indicated that some 95% of the voids coincided with the line of the expansion joints. A small percentage were found to lie under contraction joints.

15. There was also substantial correlation between the location of voids and the route of the reservoir floor underdrainage as shown on the original scheme plans.

16. The underdrainage system is separate for each compartment and in each case comprises three herringbone shaped collector drains leading to a perimeter drain running around the sides of the reservoir. When this herringbone pattern of underdrainage was superimposed over the pattern of the voids predicted by the survey virtually all of the voids not along the line of the expansion and contraction joints were found to coincide with the position of underdrainage collectors.

17. Initially it was considered that a fracture of the lower unreinforced slab, due to hydraulic pressure, must have occurred since the record plans available indicated that the expansion joint was in the top slab only. During the remedial work it was revealed however that this expansion joint was in fact the full depth of the two slabs and the leakage path was thus much more straightforward.

18. The leakage path was thus concluded to have passed through failures of sealant in the expansion joints and then taken the shortest route to the underdrainage collector. Migration of eroded sand from under the joints was possible since only a coarse gravel had been used for the underdrainage bedding.

19. Calculations were carried out to check the adequacy of the reinforced concrete floor slab. In the area of the voids these calculations revealed that the structural integrity of the reservoir floor would be at risk unless the voids were filled to give support to the slab particularly where the void was at the slab edge.

20. It was concluded that grouting of all voids should be undertaken and that pressure grouting was necessary along expansion joints to ensure that adequate support would be provided to the edge of the slab.

21. Whilst it was thought that grouting of the voids would inevitably fill the underdrains it was thought that the relatively permeable subgrade would allow adequate drainage and hydrostatic pressure distribution beneath the reservoir.

Remedial Works to Under Floor Voids

22. Grout injection holes were drilled on each side along the line of the expansion joint at regular intervals and also in a grid in other locations where voids had been indicated on the survey. At each drill hole an inspection was carried out and the depth and apparent size of the void present was recorded.

23. To fill the voids a two stage grouting process was specified. The first stage used a sand/cement grout with plasticiser for good flow characteristics. This was injected under a controlled head by way of fixed pipes projecting 500mm above the floor of the reservoir. It was hoped that this grout would fill voids but be less likely to flow into the underdrains.

24. Initially trials were conducted to monitor the suitability of the grout mix and the passage of grout was observed at pilot holes adjacent to the grout injection holes.

25. Following completion of the first stage grouting and after allowing sufficient time for shrinkage of the sand/cement grout to occur a second stage grouting was carried out. A cement/bentonite grout was injected under pressure and the floor slab was monitored for any sign of significant movement using dial gauges mounted on the column bases. Pressure grouting was stopped when the upward deflections recorded approached 0.5mm and this proved an effective way of monitoring the performance of the second stage grouting process.

26. After grouting was complete the total grout take at each injection hole was reconciled with that expected from the GPR survey and from the inspection of the voids. A number of further holes were drilled to examine apparent inconsistencies.

27. It was only necessary to carry out additional grouting at one of the further drill hole locations.

Remedial Works to the Joints

28. Rubber bitumen sealant had been used in the original construction and it was thought most prudent to repeat this practice because of the difficulty of cleaning joints completely so that an alternative could be used.

29. For the replacement of the joint filler it was considered necessary to use a fillerboard readily compressible laterally but capable of supporting the sealant in the joint without rupture when subjected to high water pressure. It was recognised that whatever filler was used it should completely fill the joint. Rigid filler board was therefore considered unsuitable since it would be difficult to place in a compact manner or grout in without bridging the joint. A closed cell polyethylene filler board recently developed by a leading sealant manufacturer was therefore selected for this purpose. It was agreed with the manufacturer that this compressible board would be suitable for the remedial works.

30. The joints were duly cleaned out and it was revealed that the expansion joints were in fact full depth joints as mentioned earlier in the text and not as shown on the construction drawings. It was concluded that this was an attempt to ensure the full movement capability of the joint. The expansion joints in the floor were found to be not properly rectangular in section or completely straight and it was very difficult therefore to fully remove all traces of previously placed plasti joint and indeed traces of the original flexcel filler. However ultimately great care was taken to remove the original filler and sealant material using specially forged tools. This proved far more costly than had been originally envisaged but was thought to be necessary to ensure a firm base for the joint. The filler board was placed in the joints and packed into place. The floor joints were then sealed with hot poured bitumen sealant and the wall joints and wall base joints sealed with plasti joint putty. The joints were cleaned and dried and primed before sealing.

31. The reservoir was tested and it became apparent very soon that some failures of the resealed joints had occured both because of the excessive water level drop during testing and there was also evidence of water flow in the underdrain collector manholes.

32. The reservoir was drained and inspection revealed that for the most part the remedial works had been successful but that the deep expansion joint in the body of the floor slab had failed in a number of places.

33. Close investigation of the nature of these failures revealed that the filler board had been unable to provide adequate support for the sealant when subjected to hydrostatic pressure over about 3.5m. This depth was measured at the time of the test when flow appeared in the underdrains.

34. These failures were examined in more detail and it was clear that the closed cell filler board required extremely firm lateral support in order to resist yielding in the vertical direction under hydrostatic load.

35. Whilst it was a possibility to go back over the reservoir floor joint and with great care repack the hydrocell filler there was a question mark over its inherent compressibility and suitability for use on these remedial works because of the distorted nature of the joint. It was therefore necessary to review other possible means of sealing the floor expansion joints. The new proposal was required not only to take into account the support of the sealant but also the ease of installation into the deep and variable section joint.

36. Whilst the deep joint in the body of the floor, where there was no water bar, had proved extremely vulnerable during the test the expansion joint on the perimeter of the floor between the floor and the cantilever wall bases had proven to be satisfactory. A review of the temperature ranges to which the reservoir floor would be subjected showed that under operational conditions, rather than construction conditions, the floor would be required to expand and contract very little under normal circumstances and only a few millimetres in the extreme. It was concluded that this expansion and contraction could be taken up in the perimeter joint and that the expansion joint in the body of the floor could be effectively fixed. It was therefore proposed to fill the floor expansion joint with suitable grout and seal as if it were a contraction joint. This proposal would also easily accommodate the joint irregularities which had proven difficult with the other filler.

37. The closed cell filler and bitumen sealant was completely removed from the floor expansion joint and the joint sealed with a proprietary grout leaving a 25 millimetre deep rebate for the hot poured sealant. A bond breaking tape was used on top of the cured grout and the joint was duly sealed. This proved satisfactory and the reservoir was returned to service.

Conclusions

38. Ground probing radar proved to be an effective technique for surveying reservoir floors which were lightly reinforced. The technique proved successful in identifying subfloor voids.

39. Reservoirs on sandy subgrades require good underdrainage. The drainage system should be wrapped with a geomembrane to prevent migration and removal of fines when flow from the reservoir to the underdrains occurs.

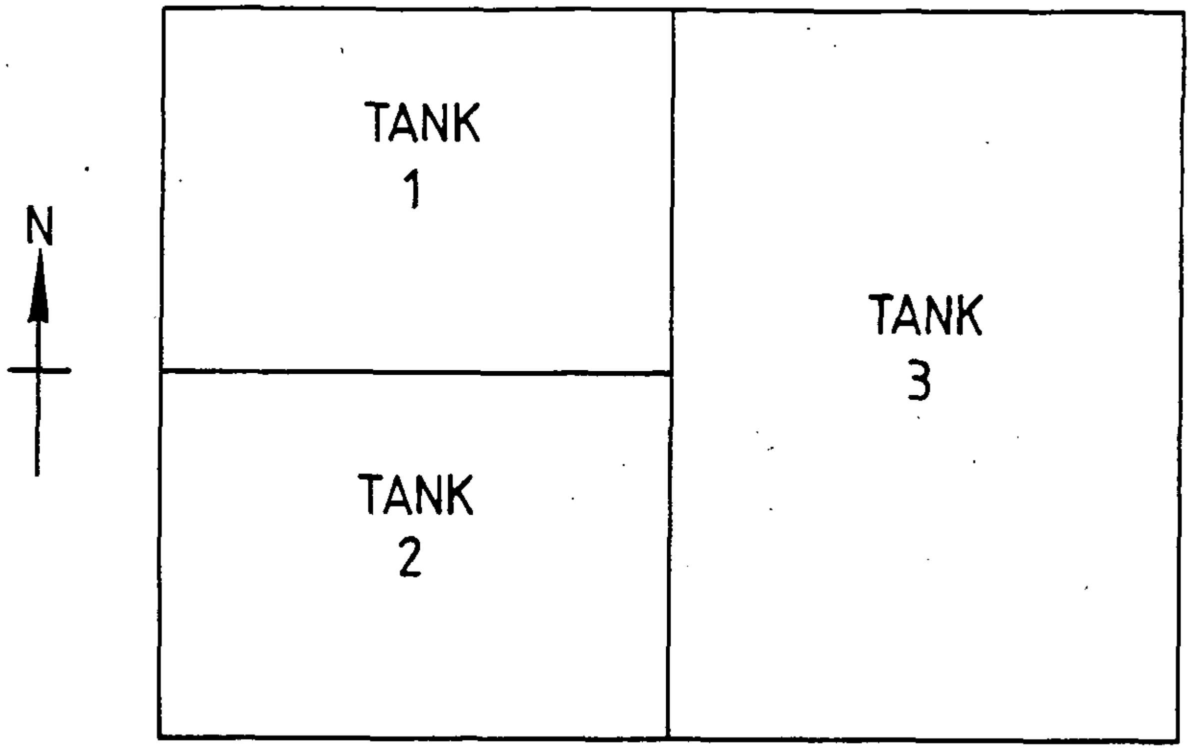
40. Movement joints in water retaining structures should have water bars or secondary means of leakage prevention.

41. Remedial works require a flexible approach and discrepancies between construction and as built conditions should be expected and contingency allowances included.

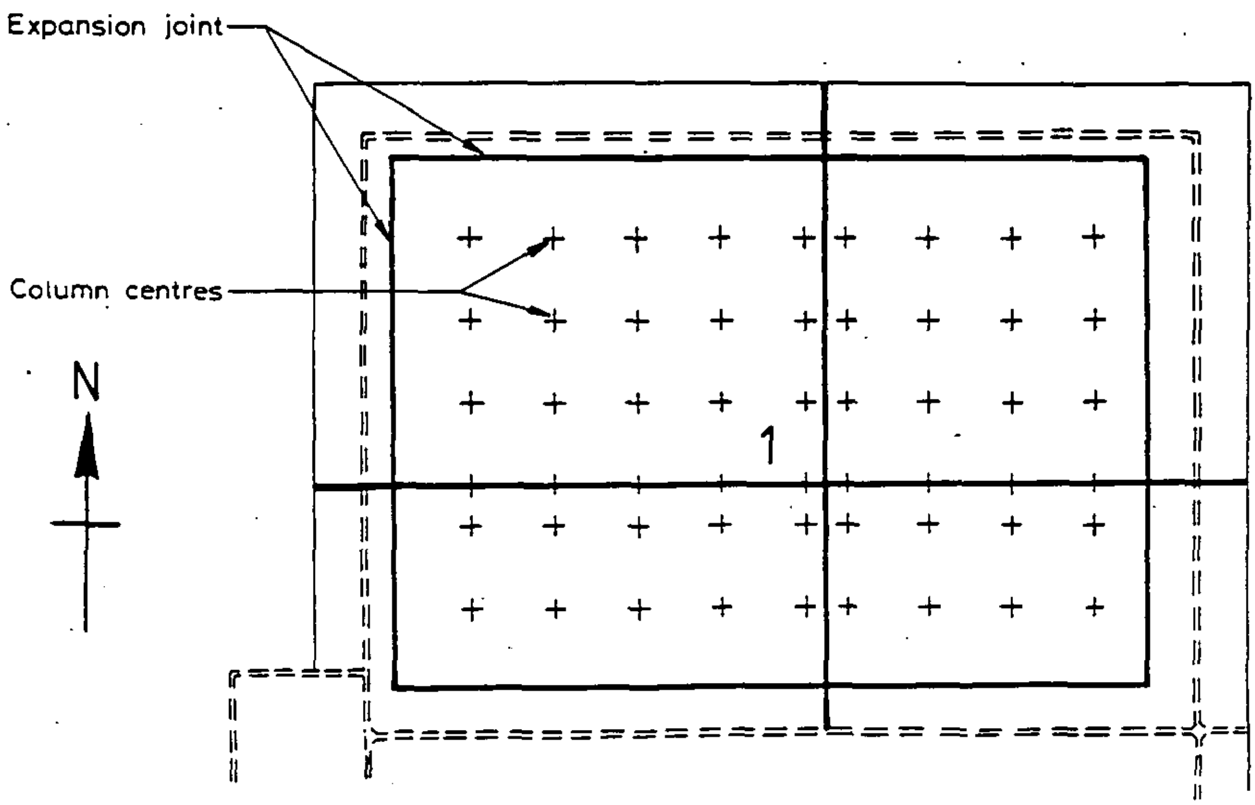
42. Very flexible expansion joint fillers should be used with caution where substantial hydrostatic pressures against expansion joints are expected.

Acknowledgements

The Authors wish to thank Yorkshire Water Authority for their co-operation and permission to publish this paper.

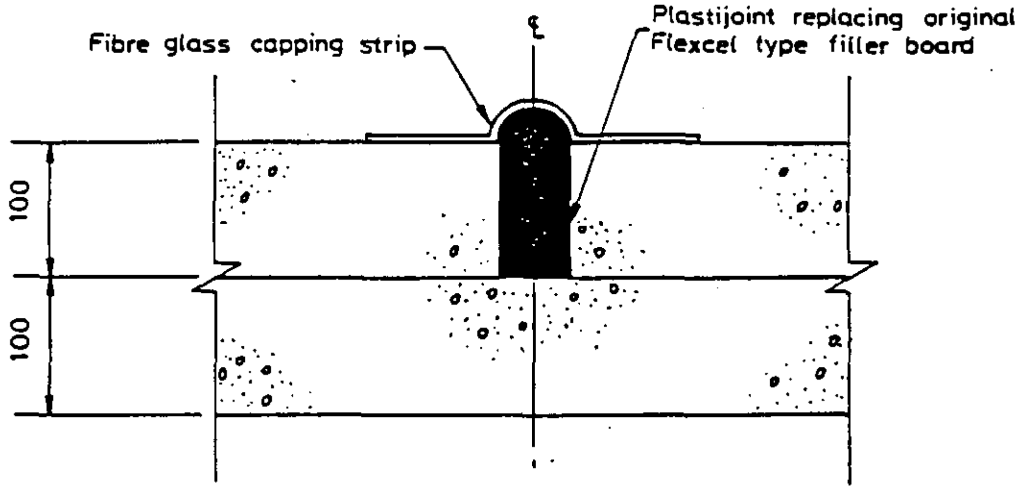


KEY PLAN OF RESERVOIR

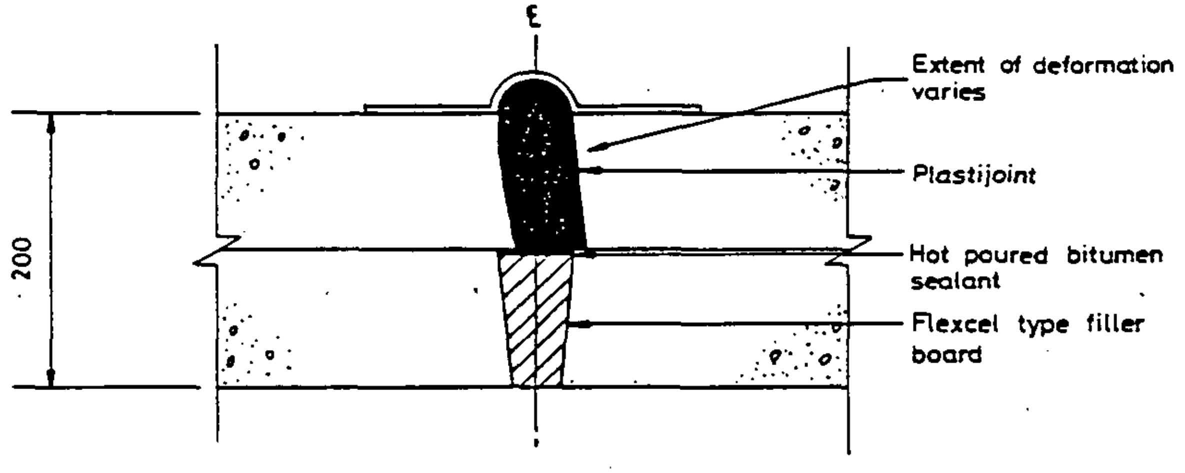


TYPICAL POSITION OF EXPANSION JOINTS IN TANKS 1 & 2

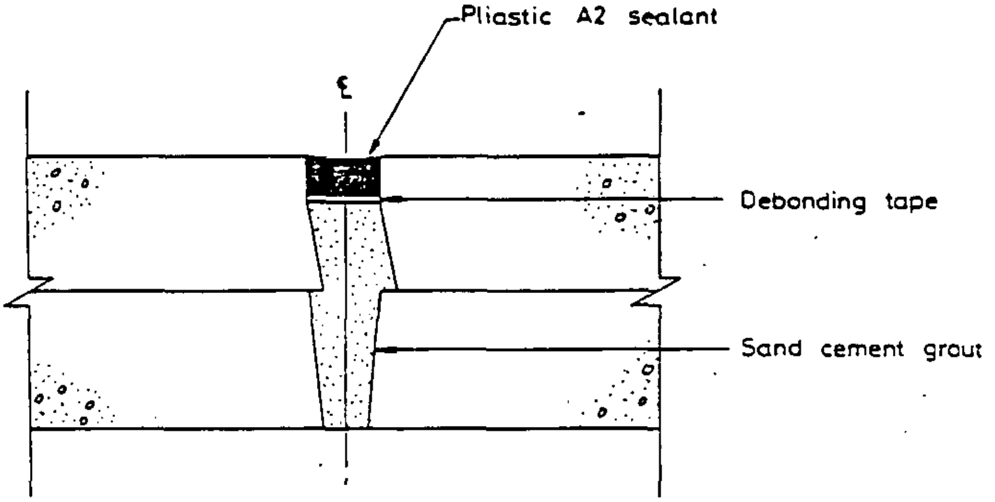
Figure 1 : Reservoir layout



FLOOR EXPANSION JOINT AS DETAILED



FLOOR EXPANSION JOINT AS BUILT



FLOOR JOINT REMEDIAL WORKS

Figure 2 : Joint Details

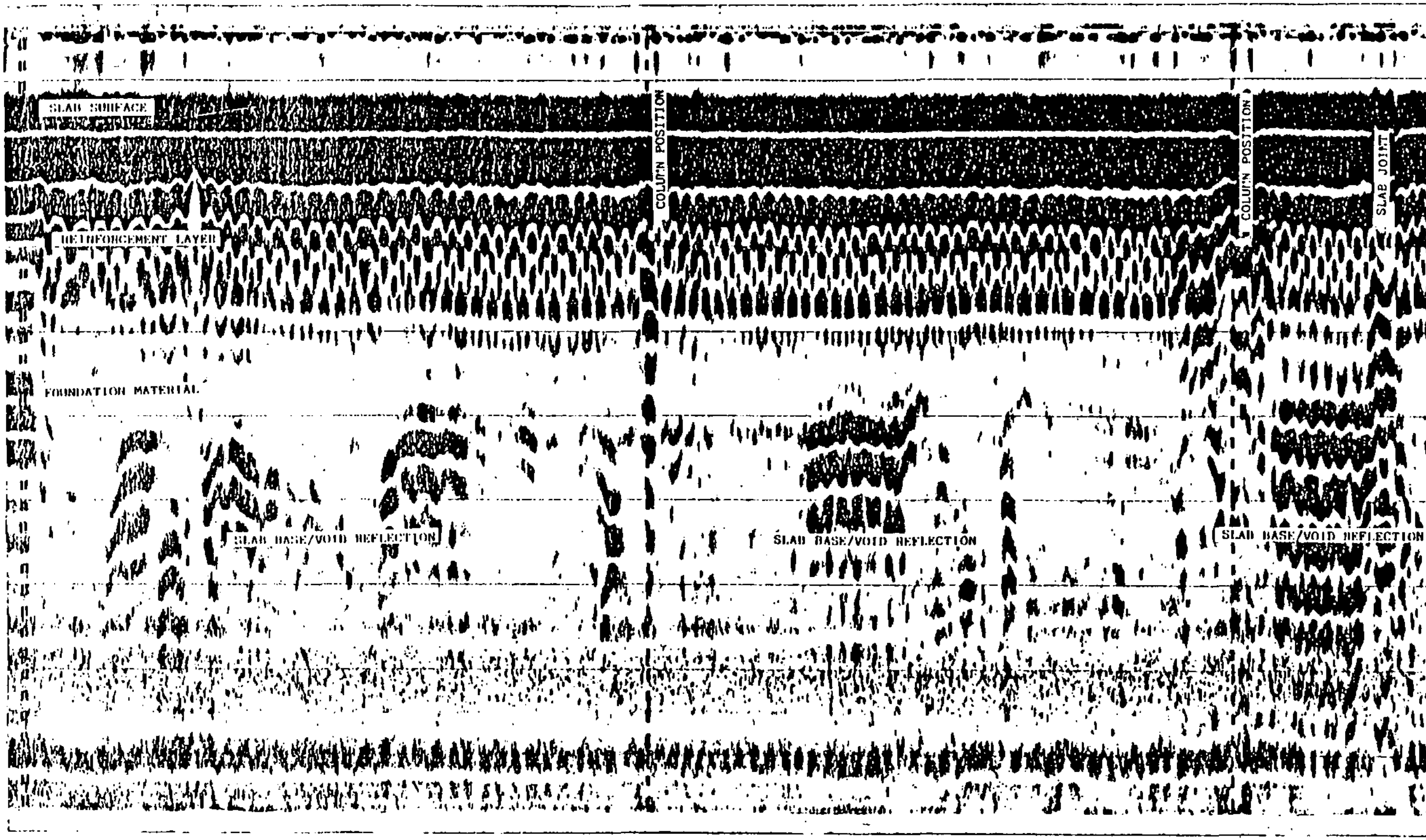


Figure 3 : Typical radar printout

Supplied courtesy of
STRUCTURE TESTING SERVICES (UK) LTD

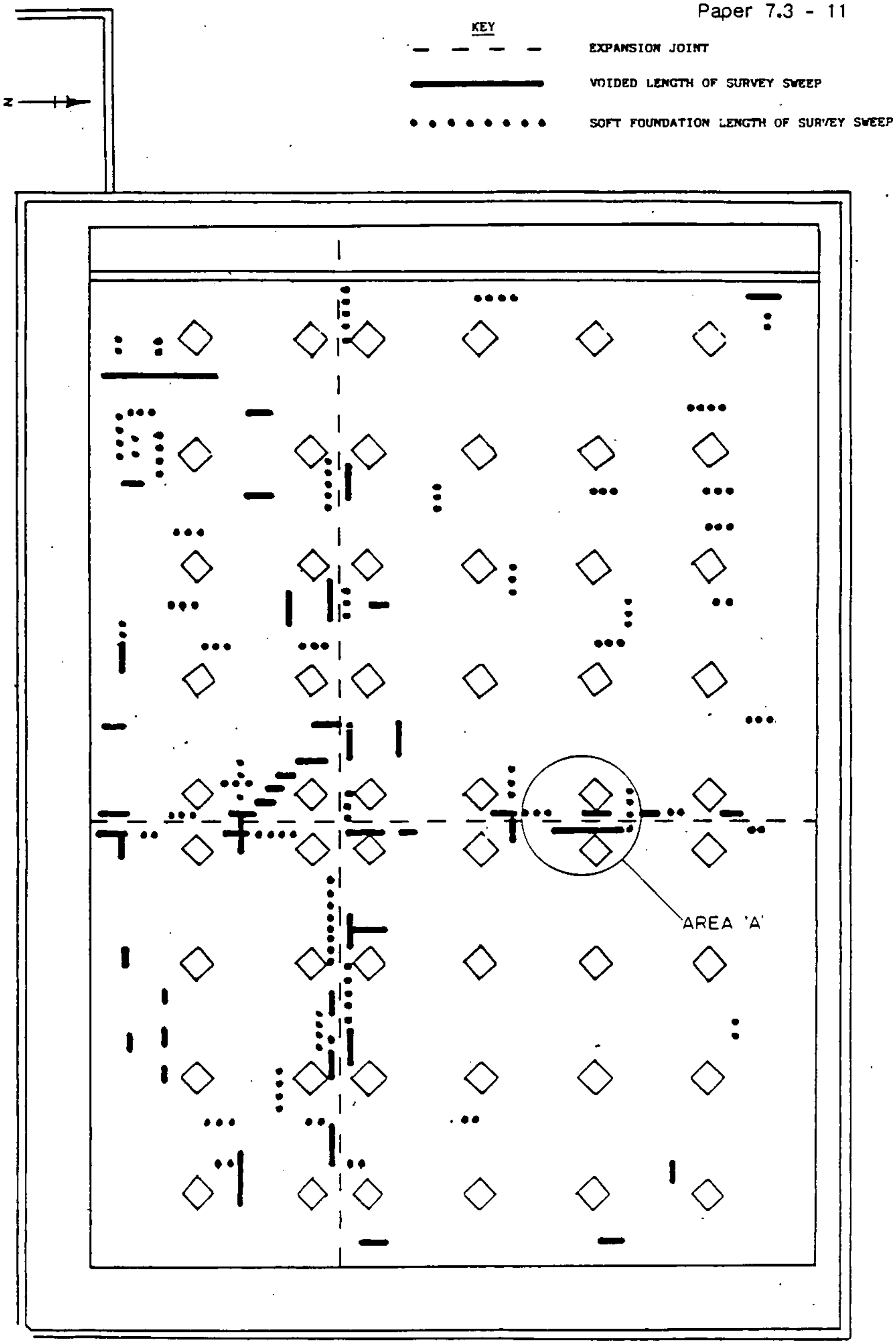


Figure 4 : Plot of voids and soft areas tank 1

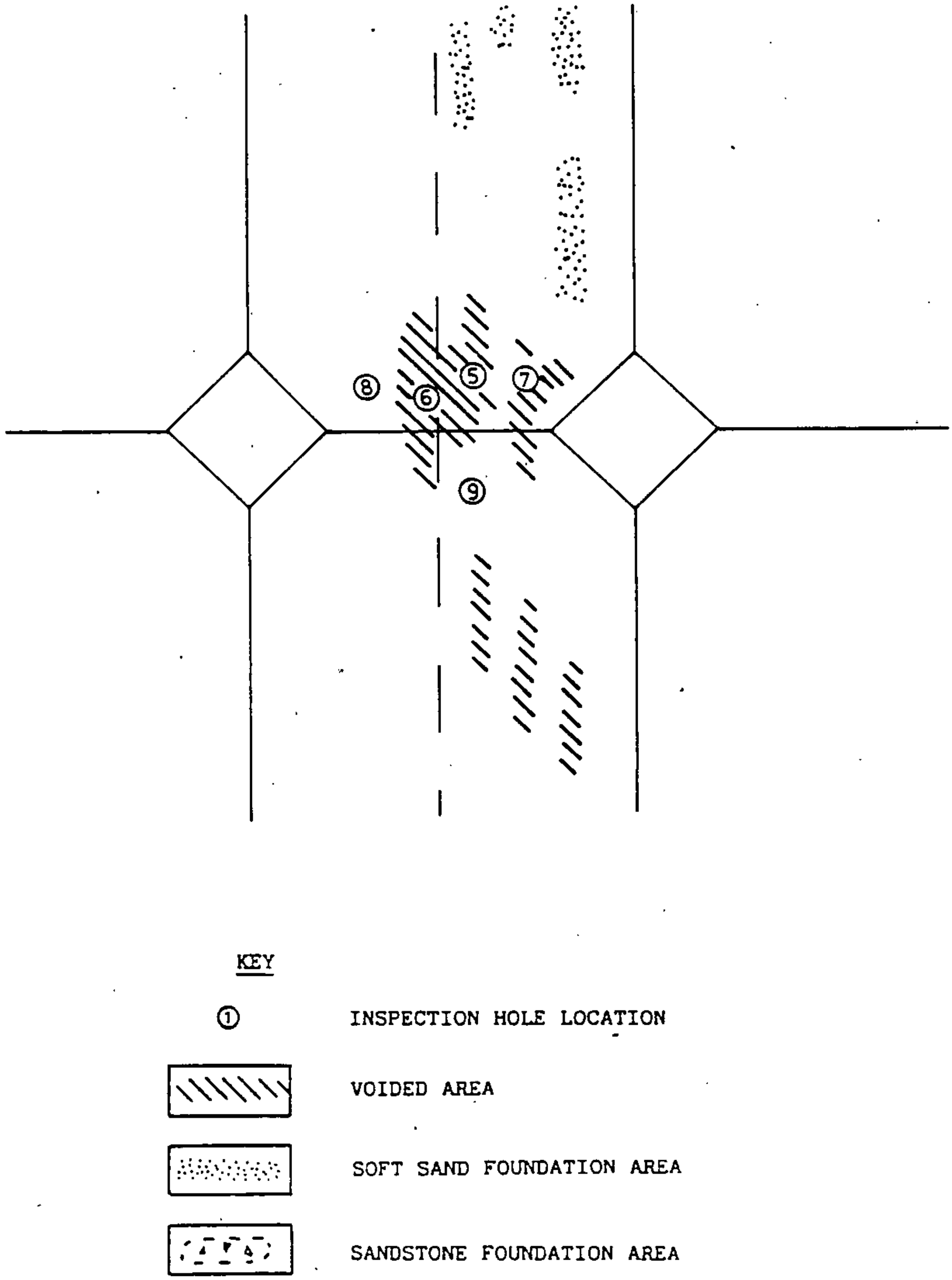


Figure 5 : Detailed plot of voided area 'A'

MILL HILL RESERVOIRS
THE LOSS AND PARTIAL RECOVERY OF STORAGE CAPACITY

J P Millmore (Babtie Shaw & Morton)
R T Heslop (Sunderland & South Shields Water Company)

SYNOPSIS

The service reservoirs at Mill Hill have a history of cracking and leakage associated with coal mining subsidence. The reservoirs were structually repaired and made watertight with a rubber lining. After 25 years the rubber lining had deteriorated. While preparing for the replacement of the lining part of the reservoir collapsed. An investigation was undertaken to establish the cause of the collapse and identify means of recovering safe storage of the water. The investigation lead to the construction of new service reservoirs at an adjacent site. Provisions were made to accommodate severe ground movement and instruments were installed to monitor the behaviour of the foundations. In addition part of the damaged structures were repaired.

INTRODUCTION

1. The service reservoirs at Mill Hill in the County of Durham were constructed in two stages by Sunderland and South Shields Water Company. Reservoir No. 1 was built in 1926 with mass concrete walls and reinforced concrete floor, roof and columns. In 1939 the adjoining Reservoir No. 2 was constructed to double the capacity to store 24 million gallons of potable water. Each reservoir was divided internally by a half height wall to provide northern and southern compartments. A mixing chamber controlled the inlet to both reservoirs.

2. Mining of coal seams adjacent to and beneath Mill Hill Reservoirs was carried out between 1929 and 1961. There is no record of any subsidence affecting the reservoirs prior to 1938. However, the mining in 1938 had a substantial effect on the reservoirs.

3. Reservoir No. 1 developed leaks in 1938 and one month after its commissioning, Reservoir No. 2 also developed problems. From that time large cracks developed in the floor and large cavities formed under Reservoir No. 1. These were grouted solid as they occurred and as described by McLellan (1) the structure was made watertight by lining the compartments with a rubber membrane.

4. Reservoir No. 2 was lined with rubber in 1953 while Reservoir No. 1 was not lined until 1964. Maintenance repairs to the rubber lining were carried out at regular intervals to reduce the leakage from the reservoirs. No structural repairs were required during this period.

5. In 1978 following an inspection under the Reservoirs (Safety Provisions) Act 1930 it was recommended that the rubber lining in Reservoir No. 2 and the mixing chamber be replaced due to excessive deterioration. The Inspecting Engineer also noted that - 'There was no apparent recent settlement or movement of the reservoir or surrounding land which might affect the stability of the structure'.

6. In view of the recommendations the Water Company reviewed their service reservoir requirements and concluded that storage at Mill Hill was necessary. It was therefore decided to proceed with the relining contract.

COLLAPSE OF RESERVOIRS

7. On Monday 22nd October 1979 within one week of the commencement of the repairs a sudden subsidence occurred in the south west corner of Reservoir No. 1 and part of the structure collapsed. Within 2 hours Reservoir No. 2 and the common division wall were also affected. The two northern compartments appeared to be unaffected and still remain in use.

8. The volume of water stored in the two damaged south compartments drained completely away together with the storage above the level of the half height division wall in the north compartments. Approximately 15 million gallons was lost in a period of about six hours. The water drained away into the subsided area and then into the underground strata since there was no trace of water at ground level around the reservoirs. There was no damage to property other than the reservoir and no life was in danger. The water level in a borehole at the site rose shortly after the collapse of the floor. The loss of the water from the reservoir was monitored by the Water Company's telemetry monitoring system.

9. It was apparent from the subsequent inspection that a void caused the floor to collapse and the large flow of water enlarged this void. The adjacent mass concrete division wall also collapsed as the void continued to enlarge under the floor of reservoir No. 2 south compartment.

10. The floor slabs and column foundations subsided over an area of up to 1,000 sq.m and caused differential settlement in the floor slab of up to 1,200mm. The roof also settled and two large depressions formed on either side of the division wall. The roof settled by up to 1000mm over an area of 600 sq.m. The columns in the affected areas showed signs of substantial distress, many having both tension and compression failure due to their extension and rotation of the column head and base. It was estimated that more than $500m^3$ of glacial till was eroded from the reservoir foundations. Further small movements of the structure continued for several months probably due to stress transfer within the structure. A record of the damage to the structure associated with the October, 1979 collapse is summarised in Figure 1.

GEOLOGY OF SITE

11. The general sequence of the geology of the site shown in Figure 2 was obtained from the water supply borehole adjacent to the reservoir.

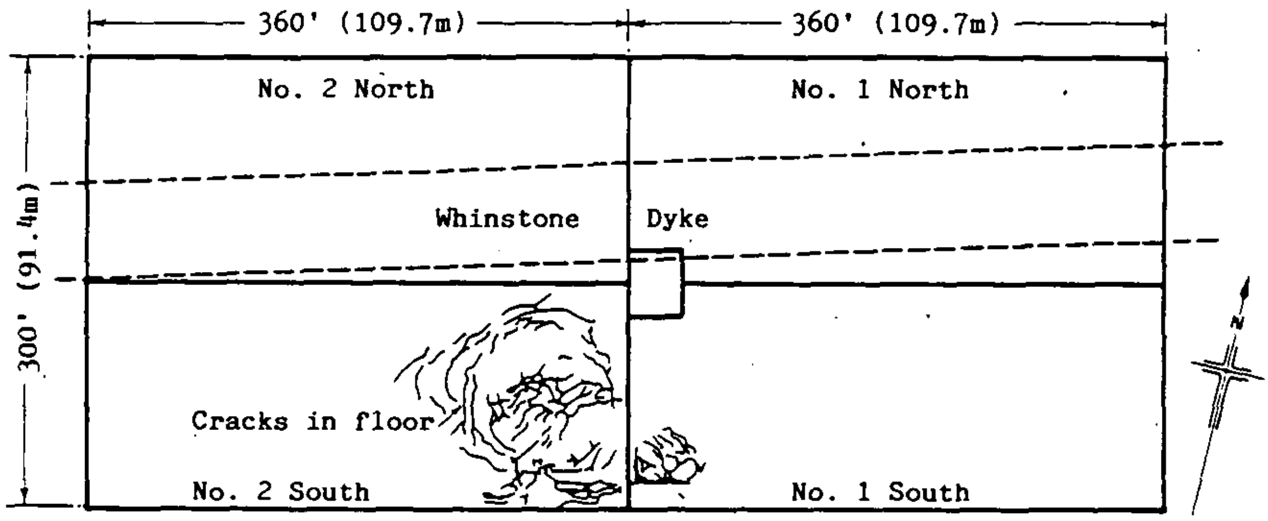


Figure 1 : Plan showing cracks in floor

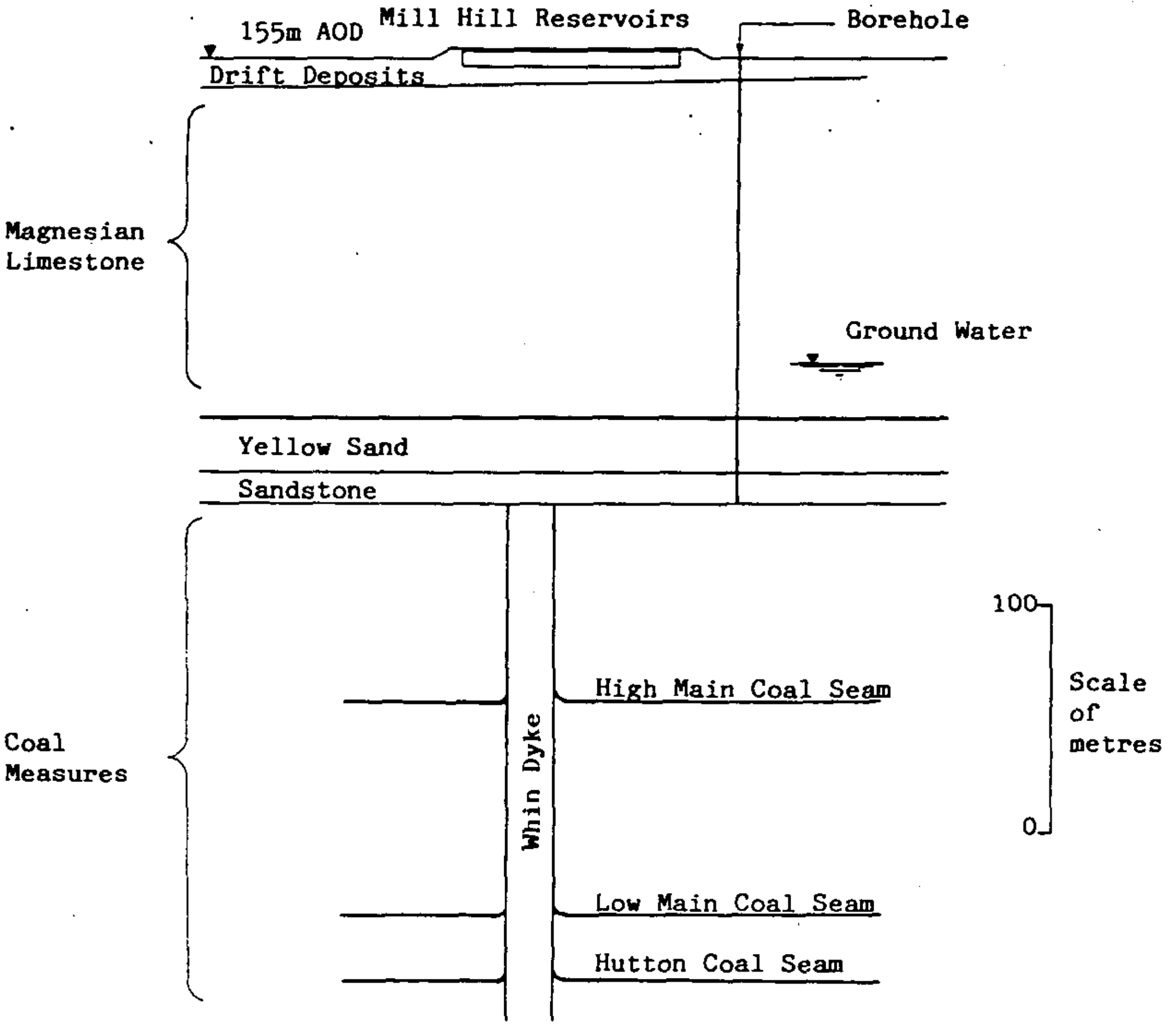


Figure 2 : Geological succession

12. The drift material overlying the rock is described by the Institute of Geological Sciences in their published map of the area as 'thin clay over gravel and gravely clay' except at the eastern part of the site which is described as a 'brown stony clay'.

13. A vertical whinstone dyke some 20 metres in width known as the Ludworth Dyke has been recorded as running beneath the site in the Coal Measures. (Fig. 1 and 2) It predates the Permian Sands and Magnesian Limestone. A major fault in the Coal Measures runs parallel and close to the dyke, but there is no evidence of it affecting the overlying Magnesian Limestone. The Magnesian Limestone was known to be fissured.

14. The reservoirs were situated over the Ludworth Dyke. It was thought prior to the construction of Reservoir No. 1 that the foundation would not be affected by coal mining subsidence since the coal seams adjacent to the dyke were expected to be 'cindered' and not commercially workable.(1)

15. Extraction of the coal in the Coal Measures occurred as follows:-

Seam	Depth (m)	Thickness (mm)	Extraction Date
High Main	283.5	1330 to 1370	1961
Lower Main	378.0	940 to 1195	1939 to 1943
Hutton	408.5	1370 to 1550	1929 to 1930

16. Examination of the mining records showed that the Lower Main coal seam was worked to a limited extent within the zone of influence below the southern side of the reservoirs, with almost total extraction further south. It was considered that these workings caused the initial damage to the structure. There is no evidence of further movement at the time of the 1961 extraction north of the reservoirs.

17. Sink holes were observed west-south-west of the reservoirs aligned to the Ludworth Dyke. One sink hole, approximately 1100 m from the reservoirs was noted by Northern Gas in 1978 to have exposed several metres of their trunk gas main. Other sink holes are noted by the Institute of Geological Science in their published map of the area dated 1963.

18. In view of the size of the pillars left in the coal workings and the long period that had elapsed between the mining and the subsidence it was unlikely that the collapse could be directly attributed to mining. It was therefore decided to undertake a site investigation.

SITE INVESTIGATION

19. The purpose of the investigation was to assess:-
(a) the mechanism of the failure of the foundation of the collapsed southern compartments of the reservoirs,
(b) whether the two northern compartments could continue in use and if so whether their capacity could be increased.

20. The investigation of the overburden deposits was carried out to determine the depth of the material and its susceptibility to the formation of sink holes. The investigation of the rock strata was carried out to determine the quality of the rock and in particular the extent and nature of the fissuring.

21. The site investigation consisted of trial pit excavations, cable tool (shell and auger) boreholes, and inclined rotary boreholes. The inclined rotary boreholes were drilled beneath the collapsed reservoirs and around the intact northern compartments to investigate the material below the collapsed area and to intersect vertical fissures which were unlikely to be revealed with vertical boreholes. In addition holes were drilled through the concrete floors of the collapsed reservoirs.

22. The rotary drilling rig was instrumented using equipment developed and owned by CIRIA (2) who loaned it to the contractor Geotechnical Engineering Ltd., for the period of this contract. The equipment allowed the monitoring of the drilling operations by: bit penetration rate; hydraulic thrust; torque; r.p.m.; flow rate and pressure of flush water. A chart recorder produced a record of all functions for correlation with other site investigation information.

23. A CCTV survey of each borehole with forward and lateral-viewing camera heads enabled the length and depth of fissures and voids to be assessed. Air pressure measurements were taken of air which flowed into and out of the cracks in the floor of the reservoir to monitor its behaviour.

24. The overburden was seen in the trial pits to be loose and there was evidence of a void in one trial pit. The material was silty slightly sandy clay with sandy pockets which is typical of this type of glacial till. Permeability testing in boreholes suggested that there was no interconnection between the sandy pockets. The soil characteristics indicated that the majority of the overburden could collapse into wide voids and fissures. Local pockets of sandy material however could collapse into narrower fissures.

25. Open fissures through the overburden material were observed in the CCTV survey and were also demonstrated by the free flow of air into and out of the rock through cracks in the reservoir floor. Monitoring of the pressures associated with this air movement confirmed that it reflected variations in atmospheric pressure.

26. The holes drilled through the floor of Reservoir No. 2 south, at the area of the subsidence, indicated that the floor of the reservoir was not supported on the soil and was spanning across a very large void.

27. The Magnesian Limestone was found at a depth below the reservoir varying from 3.5 to 9 metres across the site. It appeared to comprise three broad layers referred to as Upper, Middle and Lower layers.

28. The Upper layer extended from rockhead to about 20 m below the reservoir floor and consisted of a moderately strong dolomitic limestone with irregular pockets and bands of weak material. Many cavities and fissures were observed by the CCTV survey and instrumentation records in this layer. The individual fissures were estimated to be up to 200 mm wide, and where several fissures interconnected cavities up to 1000 mm were observed. There was evidence of overburden material in several of these fissures particularly near rockhead, and many were still open. Fissures were encountered over the entire site but were more predominant in the boreholes beneath the collapsed southern compartments of the reservoirs.

29. The Middle layer, which extended from 20 m to 30 m approximately, consisted of moderately weak dolomitic limestone and calcitic dolomite. Seams or partings of soft silty clay were common throughout this layer. Only small voids were observed during the CCTV survey in the Middle layer. However the instrumentation records indicated that rock existed at these locations and it therefore appears that the drilling operations washed the material out.

30. The Lower layer extended below 30 m depth and consisted generally of a weak friable calcitic dolomite. The fissures in this layer were generally less frequent than in the top layer of the rock and tended to be fairly small, in the region of 50 mm wide.

31. Only the voids and fissures found in the Upper Magnesian Limestone layer were large enough to permit the loss of the overburden which led to the collapse of the reservoirs.

POSSIBLE CAUSES OF SUBSIDENCE

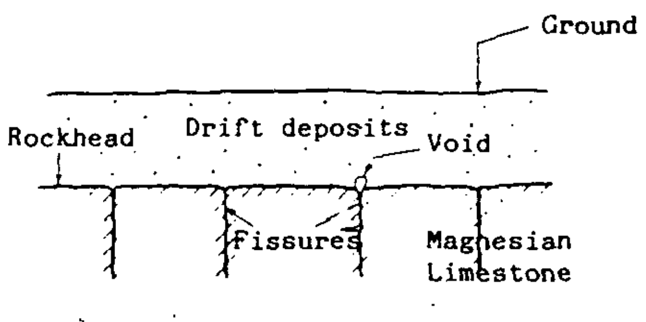
32. It is unlikely that mining subsidence as such was the direct cause of the loss of ground which brought about the collapse of the reservoirs.

33. The loss of the foundation material and subsequent collapse of the structure appears to have been caused by the migration of the drift deposits into fissures in the rock. These fissures are natural geological features but they apparently widened when coal mining disturbed the rock and soil strata. This mining also caused substantial cracking of the reservoir which resulted in leakage from the structure. The widened fissures in the rock together with the leakage from the damaged reservoir accelerated the migration of the drift materials into the rock. In this way the mining operations could have precipitated the collapse of the reservoirs.

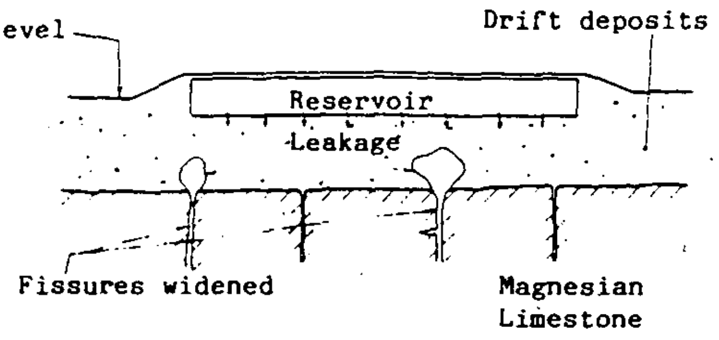
34. The probable sequence of events leading up to the collapse is as indicated below and as shown in Figure 3.

(i) Over the past millennia, fissures in the Magnesian Limestone have been solution widened by natural ground water flows, mainly rainfall. The more gravely and sandy drift deposits gradually collapsed into the widened fissures forming voids near rockhead.

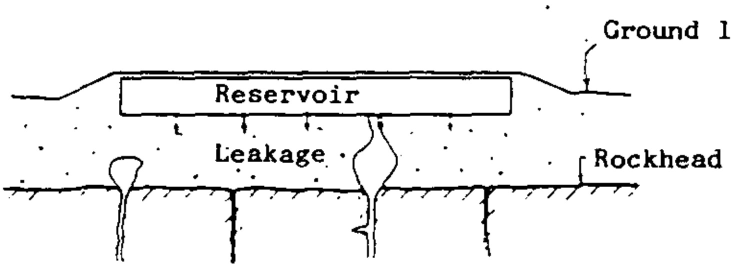
(ii) Following construction of the reservoirs, mining took place which disturbed the ground and opened the fissures in the limestone even more. The movement of the limestone appears to have been in discrete blocks causing abnormally large opening of some of the fissures. The mining also formed tension gashes in the drift deposits which enabled water to flow readily from the ground surface to the rock. This disturbance of the ground caused movement joints in the reservoir to open and the walls, floor and roof of the structure to crack. This damage resulted in a substantial leakage from the reservoir which in turn washed away the drift deposits forming cavities beneath the floor. In the period 1940 to 1950 the cavities were grouted up, the structure was repaired and later lined with rubber to make it watertight. With time the rubber deteriorated and water again seeped into the ground. The mining in 1961 may have disturbed the ground even further.



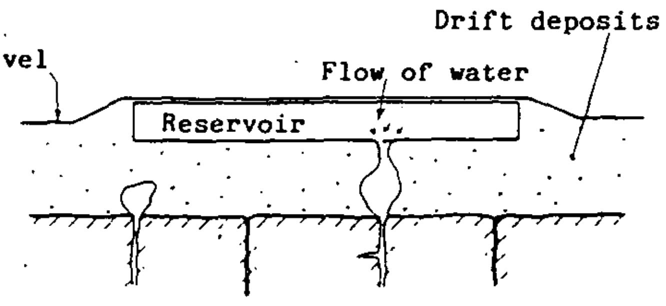
Stage I.
Gradual collapse of drift into fissures



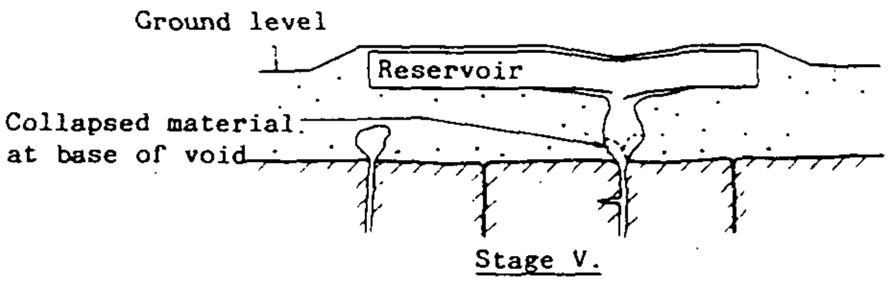
Stage II.
Some fissures widened due
to mining activities



Stage III.
Void or fissure penetrates reservoir



Stage IV.
Collapse of reservoir floor



Stage V.
Erosion of large volume of soil

Figure 3 : Probable Cause of Subsidence of Reservoirs

(iii) The widened fissures in the limestone together with the increased leakage from the damaged reservoirs rapidly accelerated the collapse of the drift deposits beneath the reservoirs. A void in the drift penetrated to the reservoir foundations forming a sink hole beneath the floor.

(iv) The loss of ground support caused the floor slab and column footings to collapse into the void rupturing the rubber lining and thus allowing the water to escape from the reservoirs.

(v) The substantial flow of water from the reservoirs during the next few hours carried a much larger volume of the drift deposits into the voids and fissures in the rock thus causing a major subsidence of the ground immediately under the reservoir floor. Later collapse of the drift deposits by gravity extended even further the area of subsidence.

35. The possibility of a collapse of the remaining northern compartments of the reservoir similar to the recent collapse of the southern compartments could not be ruled out therefore measures were considered to prevent this occurrence.

RECOVERY OF WATER STORAGE

36. Various options were identified for the required storage of water at Mill Hill. One option entailed the abandonment of the site and the construction of adequate storage elsewhere. A second option was the demolition of the existing structure and the complete reconstruction of a reservoir on the same site. The remainder of the options included various degrees of partial recovery of storage on the site and construction of new reservoirs elsewhere.

37. To solve the problem of the overburden and rock at the distressed site, it was proposed that the voids and fissures be grouted to form a raft below rockhead. The grouting would not entirely prevent the formation of new voids and fissures in the Magnesian Limestone. However, the development of partings by solution widening to a width comparable with the existing fissures would take many thousands of years.

38. Even with this grouting there would always be uncertainty about further movement at the site. It was therefore decided that new reservoirs would be constructed at alternative sites and those parts of the existing reservoirs which had not suffered in the collapse would be made safe and used until any further movement at the site rendered them unusable.

39. Two separate reservoirs, each of 11.4 megalitres (2.5 mg) capacity, were constructed at Mill Hill on a site one kilometre north west of the damaged reservoirs to meet the immediate water requirements. This site was selected since it was at the required elevation and it provided a sound foundation. It was beyond the influence of the Ludworth Dyke, the faults in the rock and the potential coal-mining subsidence.

40. Work was then undertaken to reinstate the northern compartments of Nos. 1 and 2 Reservoirs where there was little fissuring and cavitation. Cracks to the concrete in the northern compartments of No. 2 Reservoir were repaired and the rubber lining was replaced up to the level of the half height division wall. In the northern compartment to No. 1 Reservoir the rubber lining was repaired. The damaged parts of the structures were severed by an inspection trench cut through the south compartments adjacent to the division wall. The trench was afterwards filled with concrete. Along the top of the half-height division wall a concrete blockwork screen wall was constructed to enclose the northern compartments.

41. Extensive grouting below the floor slabs and at rockhead was carried out by vertical and inclined holes under the division walls. High grout takes were recorded in the vicinity of the mixing chamber.

42. Rock bolt anchors were applied to two diagonal cracks in the half height division wall using 20 mm Macalloy bars set in epoxy resin and stressed to 50 tonnes.

43. The damaged southern compartments remain in place with access restricted. Safety work was undertaken to prevent any sudden catastrophic collapse. A large void under the column in the south west corner of No. 1 Reservoir was filled using some 150 tonnes of grout. Blockwork props were also constructed between the delaminated layers of the partially collapsed division wall between Nos. 1 and 2 Reservoirs.

INSTRUMENTATION

44. Prior to the construction of the new service reservoirs at Mill Hill, a site investigation was undertaken involving a resistivity survey, exploratory boreholes and trial pits. A CCTV survey was undertaken in the inclined boreholes. The results of this work indicated that the site was suitable for the construction of the new reservoirs and there was no evidence of cavitation in the overburden or underlying rock.

45. Provision was made for continued monitoring of the foundations for the formation of cavities. A grid of stainless steel probes was installed below formation level and connections were taken to the probes to permit resistivity measurements in various probe configurations.

46. The underdrainage system was also arranged so that a CCTV camera could be introduced into the pipework from a junction manhole to permit inspection of each drain run of the grid.

47. The structures were extensively articulated with movement joints. The floor of the reservoir was designed to span a void of 2 metres diameter since this was the discriminatory limit of the resistivity monitoring system.

LONG TERM MONITORING OF RESERVOIRS

48. Periodic inspections of the old reservoirs are carried out including annual site levelling and horizontal movement monitoring. A twice yearly leakage testing routine has been established. Since renovation the reservoirs have remained stable and watertight.

49. Quarterly resistivity measurements at the new reservoirs have been recorded since 1984 with no indications of potential cavitation. The monitoring cycle has recently been relaxed to annual measurements.

50. The new reservoirs were also included in a programme of dynamic testing of service reservoir foundations adopted by the Company. The method involves monitoring the dynamic response of the structure and foundations to vibrations applied at the column heads. (3) Changes in the stiffness of the foundations can be identified and potential cavitation identified.

CONCLUSION

51. The collapsed reservoirs had been sited in an area thought to be safe from mining subsidence. The ground beneath the new replacement structures and the original structure still in use have been instrumented to warn of possible future cavities and subsidence.

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DISCUSSION: TECHNICAL SESSION 7

GRAVITY DAM DETERIORATION

Session Chairman: Mr F G Johnson

CHAIRMAN: F G JOHNSON (North of Scotland Hydro Electric Board)

We've got four papers this afternoon. I am going to call first of all on Mr Morison, Senior Engineer of Sir William Halcrow & Partners, to present the joint paper by Mr Mackay and himself on the replacement of expansion joint seals at Clywedog dam.

We are delighted to welcome Professor Mazzalai all the way from Padua, with a very interesting paper on the re-building of a masonry dam and the waterproofing of an auxiliary dam.

Our third paper is remedial works to Brayton Barft Service Reservoir and this is from W S Atkins & Partners and Mr Askew who is a senior group engineer is going to present it on behalf of himself and Chris Binnie.

Our last presentation is the horror presentation judging from reading the paper. We have two presenters, Mr Milmore, who is an associate of Babtie, Shaw & Morton and Mr Heslop who is the Technical Services Manager of Sunderland and South Shields Water Department.

E M GOSSCHALK (Sir William Halcrow & Partners)

The comments that I want to make arise from work carried out for Sir William Halcrow & Partners on behalf of the North of Scotland Hydroelectric Board and I really want to say some cautionary words about problems arising from and around inspection galleries in concrete gravity dams.

Galleries are installed in concrete gravity dams for three main reasons. One is to facilitate inspection of the interior of the dam and the second is to facilitate the use of instrumentation in the dam and the third is to facilitate drainage and relief of hydrostatic pressure within the concrete. My remarks really lead to the suggestion that there will be fewer problems that needed inspection if you didn't have an inspection gallery, so in other words, if the only need were to inspect the interior of the dam, it might be better not to have a gallery to do it.

The problems I am referring to really arise from thermal stresses, which are generally considered secondary or indirect stresses, but when there are cold external temperatures and warmer core of the dam, the cold shell of the dam tends to contract and crack because it is restrained by the warm interior, and conversely the core tends to crack.

I have recently directed some linear and non-linear finite element modelling of stresses round a gallery in a concrete gravity dam.

The results of non-linear 2-dimensional finite element modelling with a 5½ degree temperature differential between the upstream face of the dam and the core, the upstream face being colder. The modelling was actually carried out by Bristol University, and the method was to apply the load in 5 small increments, the first increment involving application of the hydrostatic loading from the reservoir and the gravity loading of the dam and also one-third of the thermal loading. In the next two steps, the remainder of the thermal loading was applied in equal increments, and the final two steps involved applying hydrostatic pressure from the reservoir into the cracks that propagated through the water face.

After each increment of loading, where tensile stresses exceeded 0.375 megapascals (375 kilonewtons per square metre or roughly 50 lbs per inch) the stresses were redistributed because it was assumed that that threshold of stress would cause cracking to occur and therefore that particular location would no longer be able to take tension.

After all the loads were applied, and tensile stresses nowhere exceeded the values of 0.375 megapascals, there tended to be a concentration of cracking between the reservoir and the gallery near the upper levels and also at low level, and this cracking tended to propagate downstream of the gallery in towards the core. Cracking elsewhere, at least in this vicinity, was estimated to be fairly insignificant. There were some cracks on the upstream face above the gallery and isolated cracks on the downstream face.

Of course the reversal of temperature, warm outside and cold inside, tends to reverse the situation and the cracking develops within the core. I would go on to say if longitudinal expansion of the dam is restrained, through lack of transverse expansion joints for example, then thrusts develop longitudinally in the dam, which also lead to more or less horizontal cracking.

So the conclusion of this really is that, if you must introduce galleries into concrete gravity dams, you should consider carefully the need for reinforcement around the galleries to resist the tensile stresses that can be estimated, or you should consider carefully the shape of the gallery, perhaps making it round or egg-shaped or oval, so that is a non-stress raising shape, but I have no particular evidence to confirm that that would solve the problem.

J D HUMPHREYS (MRM Partnership)

I just wanted to say that I was delighted having heard the account of Messrs Milmore & Heslop to discover that, after all, we didn't really have any problems at Winscar at all.

F G JOHNSON (Session Chairman)

I am going to start by addressing a question to Professor Mazzalai. Do you think that it is necessary to provide a gallery for a 12-metre high dam?

PROFESSOR P MAZZALAI (University of Padua)

This is an interesting question. The first answer is connected with the particular dam illustrated in the paper. In that case, there had been a lot of losses of water, not only in the body of the dam, but in the bottom of the reservoir. It is really necessary to have more time to explain all the restoration works that have been done in the past. Before this renovation work one refurbishment consisted of the placement of one thick layer of impermeable sheet on the bottom of the reservoir. This did not resolve all the problem of losses of water and we decided to measure the losses with the gallery and to increase the static characteristics of the dam.

I listened with interest to the previous contributor who illustrated the tensile stresses in the concrete around the tunnels. We did reinforce with steel bars all around the tunnel.

Another answer to your question is connected with the particular regulations in Italy. The water authority required the provision of a tunnel for measuring the losses of water and the pressure between the old and the new structure with some new instruments.

W J CARLYLE (Binnie & Partners)

I have a question for Mr Mackay and Mr Morison about the seals in Clywedog dam. I see no reference in their paper to the performance of the sealing material under spillway flows and I wonder if that was a contributory factor to the behaviour or the performance of the seals themselves. It is not often that a designer gets the chance to put his nose so close to a dam in such detail as this and get at every aspect with a fine toothbrush. I wonder if the Authors have anything to say about the condition of the concrete in such an exposed site.

A C MORISON (Sir William Halcrow & Partners)

There seems to be no clear indication on the old sealan that there had been problems of plucking large areas off it by spillway flows. It was something we looked for during the contract, when we were taking out the old sealan, but although we were restricted in our access while the contractor was working, we did ask for any large areas missing to be reported to us and none were.

At the 3-year inspection, after the sealan material had been put in, there had been, I think, 2 spills, but neither of them particularly large, and again there was no sign of damage to the sealant material by plucking.

On the surface concrete of the spillway and on the pre-cast beams, which were of higher grade concrete than the main concrete of the buttresses, there is some slight indication of exposed aggregate appearance on the tops of the beams in particular, but not to any great extent, and there is very little sign of frost damage or spalling on the downstream face.

H PERFECT (Babtie, Shaw & Morton)

I refer to Messrs Milmore & Heslop to enquire whether any attempt was made to measure methane from the underlying Coal Measures and if such measurements were taken whether monitoring continues?

R T HESLOP (Sunderland and South Shields Water Company)

We did apply methanometers to the air issuing from the reservoir foundations. It was clean, and indeed we do take periodic measurements because we do have about 10 underground resources which are in the Permian limestone and we do a routine monitoring of all these sources and also our voided reservoirs along with them.

C D ROUTH (MRM Partnership)

A question for Mr Binnie and Mr Askew. I was interested in the ground-probing radar technique. Did you have to calibrate the system in any way so that when you got what you thought were voids on the print-out you were reasonably confident they were actually voids?

Secondly, is there a depth limitation to this system? Does it, for example, mean that there could be further voids below those that we saw or could it be applied to other techniques, for maybe small swallow holes in natural ground, for example?

T E ASKEW (W S Atkins & Partners)

When we decided that there were probably voids underneath the floor, we did not really know what to use and ground-probing radar was the technique that was available, and fortunately it did the job. It is used for all sorts of things and it measures a change in consistency in anything and so it will detect an interface between concrete and air. As long as the dielectric constant changes it will get a reflection, so when we went to Brayton Barft we had to experiment with different wavelength aerials, and when we had the radar printout it was quite a while before it was possible to determine what may be voids and what may not be. Of course, once we found what we suspected were voids, because of the location and because of the consistency in certain areas, then we drove a hole through the floor and established the positions.

We were able then to go back through all the readings we had taken and view areas that may be voids, but the only safe way to establish that this was the case was, in fact, to drill a hole through the slab at the point where you thought the void existed and look for the void. So that is what we did and we actually did two radar surveys, the first one to give us some results to conclude what may be voids and then a further more detailed radar survey.

C D ROUTH (MRM Partnership)

You didn't get any indication of the depth of the voids from your radar surveys?

T E ASKEW (W S Atkins & Partners)

No, it won't do that, all it will determine is a change in the dielectric constant.

The depth of penetration of the radar depends on the frequency of the aerial and also the consistency of the material. If you have got very dense materials and very dense sub-grades, those are notoriously bad for penetration, you can't detect very deeply. We were quite fortunate because the reservoir slab was fairly thin and we were able to use a high frequency aerial which hasn't got great penetrative capability, but gives you better definition. I don't know what the range is, possibly up to 5 metres, but by the time you get into that sort of depth you would be losing a lot of definition.

We found the voids were by and large warren-like, running along the expansion joints over the top of the under-drainage and about 2-300 mm in diameter, with some areas where these warrens had expanded to small caverns about a metre wide and 3/400 mm deep.

A I B MOFFAT (University of Newcastle-upon-Tyne)

There are two different themes I would like to pick up if I may. Firstly, appreciating that Mr Gosschalk had his tongue slightly in cheek when he referred to the non-desirability of inspection galleries, I would merely mention the cautionary tale of Upper Glendevon dam, well-covered in various papers presented to BNCOLD meetings and, indeed, to ICOLD meetings, where many very serious problems are attributable at root to the failure to provide any form of gallery for any form of inspection or any type hydrostatic pressure relief.

The second point is something I would like to address to Mr Milmore with respect to the Mill Hill site. I am slightly surprised that you are a little bit dismissive, if I may use that expression, of mining activity as having lain at the root of the later troubles. Did you in fact make any attempt to correlate the records of the advance of the coal face underneath the site with known changes in the appreciation of what damage had occurred?

J P MILLMORE (Babtie, Shaw & Morton)

At the time of the 1939 coalmining there was quite a bit of work done by the Water Company and it was at that time seen that the recent coalmining was a major contributor to the events at that time, and there really were some enormous voids. It was then thought that the coalmining effects had ceased and it was therefore permissible in about 1955 to return the structures to service by relining with rubber.

There was some later coalmining again in the 1960's but there was no evidence of any movement whatsoever at that time. When the collapse occurred in 1979, we engaged Professor Potts of Newcastle University to advise on whether coalmining had been the likely cause of the latest events and whether further subsidence could take place.

Indeed, Professor Potts had done a tremendous survey in that area on the effect of coalmining in Peterlee, so he was very helpful; but he wasn't as conclusive as we would like him to have been, on the effect of the coalmining in each individual seam that had been mined, and whether coalmining was going to continue to play a part in collapses in the area. I think it is fair to say that the work at Mill Hill with the rubber lining is seen as a temporary situation, which is buying time for Sunderland and South Shields Water Company while they are constructing new reservoirs elsewhere.

C DE SOUZA (Anglian Water)

My question is addressed to Binnie and Askew.

You had herringbone under-drains and then you grouted the floor. Wouldn't you have blocked the drains, or was there no advantage in having under-drains as opposed to peripheral drains? Did they work?

T E ASKEW (W S Atkins & Partners)

We were concerned that we were going to block the underdrains and so we chose to grout in two stages. I think that we had to accept that the drains may be fully blocked up so we considered the permeability of the sub-grade to the reservoir, to make sure that there was some kind of drainage there anyway, even if the under-drains were blocked. In practice, I suspect that they were actually full of sand anyway. We found from the volumes of the stage 1 grout, which was coarse sand and cement which we hoped would be less likely to penetrate all of the drains under the reservoir, that the grout take was more or less as we would have imagined to fill the voids only, so I think there was very little penetration beyond the void beneath the floor. We chose to do it in two stages. Does that answer your question?

C DE SOUZA (Anglian Water)

What was the advantage of under-drains when they tend to block up naturally?

T E ASKEW (W S Atkins & Partners)

They do register flows because when the reservoir is leaking the under-drains do take water away, but I would say that a lot of the under-drains are blocked with sand that has migrated from beneath the floor. In addition, you don't want trapped hydraulic pressures beneath the floor because it is so thin, and if you drain down the reservoir very quickly, you could get uplift. Also you have to consider release of hydraulic pressures underneath the retaining walls; if you are draining down a reservoir with two compartments and you've got a division wall that depends on the stability of the sub-grade for overturning, you don't want any trapped pressures underneath the toe of a cantilever wall for instance.

T A JOHNSTON (Babtie, Shaw & Morton)

Referring to Tres dam, Professor Mazzalai has described the design tenson stresses in Table 2 of his paper. What is regarded as the maximum acceptable tenson stress in masonry and concrete in Italian practice?

Could Professor Mazzalai please explain the procedure for installing pressure relief drainage at Tres, and what assumption has been made regarding the distribution of hydrostatic uplift pressures on the base of the dam.

What was the timing of the installation of the grout curtain?

PROFESSOR MAZZALAI (University of Padua)

I'll try to answer. For the first question about the tensile stress in Italy along the masonry dams it is very very low. If you see on table 2, I reported that the maximum tensile stress in the new configuration is 1 kilo per square cm and the maximum value is about 2 kilos per square cm maximum tensile stress, but the tensile stress in this case is supported by the reinforced bars that are put in the new structure.

The second question is how is it possible to construct the grout curtain from the gallery. It was impossible to construct from the gallery because the gallery is too small to allow this operation. The grout curtain is constructed before the construction of the reinforced concrete structure upstream of the old dam. The last operation was the connection between the new structure and the rock foundation as shown in the figure 9, on the left hand side of the new structure. The second curtain will be formed at the end of the construction of the new structure to permit the complete connection between the new structure and the rock foundation. This is formed at the end of the construction because you need some weight over the new structure to balance to the pressure of the injection.

The third question is why we constructed one drain from the gallery. This drain is not constructed from the gallery because it is not possible. It is constructed from the top of the dam at the end of the construction, draining all the new structure in which it is inserted. This permits the reduction of the uplift pressure of the water to a minimum value in correspondence with the drainage holes, hence you can reduce the uplift to one-tenth of the maximum value of the maximum upstream point.

J C BROWN (British Waterways Board)

I was interested to see the results of the ground-probing radar survey to detect voids under the Brayton Barft reservoir. British Waterways Board have used this system to locate filled construction Shafts on canal tunnels with considerable success over the last few years. Are you aware of any other use of the system, particularly with regard to the detection of voids in earth dam embankments?

T E ASKEW (W S Atkins & Partners)

No simple answer to that. I can't quote examples, but it has been used before, but because it's not a technique specifically to determine voids it will detect any change in consistency and any change in dielectric constant. Anywhere you can get a change you can get a radar reflection and I suppose the limitation then is on what's screening the radar from what you are trying to look at.

C J A BINNIE (W S Atkins & Partners)

I do know that it has been used for looking for sewer pipes in roads.

PROFESSOR MAZZALAI (University of Padua)

I want to add another reason for the installation of an inspection gallery in such a small dam. Downstream of the dam there is one village very near the dam, with more than 1000 people and after the disaster of Seva in Italy, the sensitivity to dam problems is very very high, and so this is the third reason for installing a gallery to improve monitoring.

R T HESLOP (Sunderland and South Shields Water Company)

Could we just add one word of advice. If you are dealing with a concrete tank filled with water, insure it for fire.

F J JOHNSON (Session Chairman)

In concluding this session, I would just like to say that the first reservoir I ever designed, and was supervising construction, used Flexcel in the joints and had a bitumen sealer. We filled it and the water went. We had to replace the joints just as you have done and we did exactly the same, we filled the lowest slab joint with cement mortar and put a bitumen joint above, so there is nothing new under the sun.

PROCEEDINGS: TECHNICAL SESSION 8

NEW MATERIALS FOR THE RENOVATION OF DAMS AND RESERVOIRS

PAPERS:

DISCUSSION:

Index 8 (cont)

REHABILITATION OF UPSTREAM FACINGS ON MASONRY AND CONCRETE DAMS

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SYNOPSIS

The factors which cause deterioration of the various types of upstream face of concrete and masonry dams built in Italy are examined; the solutions adopted in those cases where it was necessary to rehabilitate the face entirely are illustrated. Recent trends in construction are discussed.

INTRODUCTION

The expansion of dam-building in Italy, both in masonry and in concrete, started at the beginning of the century with the increasing interest in hydro-electric power development; the most intense periods of construction were 1920-1930 and 1945-1965; after that most dams were built for the purposes of irrigation or drinking water.
The material used for the body of dams was initially stone and mortar; with these materials 73 dams were constructed, the last one being built in 1956. Concrete was first used in 1920, but only after 1945 was this material fully exploited with the development of the necessary technology, as the material came into general use. Up to today, 273 dams have been built in concrete, many of them of great height and volume.
The very variable climatic conditions in Italy influence the behaviour of the dams, which can be divided according to their natural geographic regions as follows:
- dams in the Alpine region, in the North of the country, characterized by low temperatures, frost and formation of ice fields during the winter, wide seasonal temperature range (-20° +30°) and approximately 100 freeze/thaw passages per year; dams are situated up to 2.700m elevation a.s.l.;
- dams in the Appenine region, which covers the entire peninsula; the climatic conditions are normally mild; dams are situated up to 1.400m;
- dams in the islands (Sicily, Sardinia), characterized by a wide daily temperature range.

In some parts of the country, areas with intense seismic activity are present. Of a total of 326 dams, 173 are situated in the Alpine region, 91 being at elevations between 1500 and 2700 m.; 99 are in the Appenine region, 10 are in Sicily and 44 in Sardinia (see Table I).
During construction, in order to obtain the necessary qualities of compactness and watertightness of the upstream face of the dam, various types of facings were used, in keeping with the

construction technology available at the time. Facings to be found are: (see Figure 1)

- hand-finished cement rendering, or gunite, often reinforced with metal mesh; these were widely used until 1950;
- squared stone pitching with mortar joints; this was widespread before 1959, especially at high elevations;
- bitumen facing, later abandoned;
- compact watertight concrete face, in general use after 1945 on concrete dams with the perfecting of the necessary technology;
- two cases of sheet metal facing, the first applied experimentally in 1931, the second in 1964 in relation to the particular construction technology of the dam.

It should be remembered that before 1925, the statics of dam-building was still in its formative period; the first Italian Code for Design, Construction and Operation of Dams was published in 1925 and 1931, and there was a new edition in 1959, recently revised in 1982.

This report does not mention the numerous partial interventions; these have been necessary over the years on the upstream faces of dams (for example grouting of the face, rendering with cement and renewal of gunite for the portion above low water level); we deal here only with cases of rehabilitation of the complete face.

DETERIORATION OF UPSTREAM FACE AND NEED FOR COMPLETE RENEWAL.

The deterioration of the upstream face, to be found in the older dams in Italy, is essentially determined by the following factors:

- the characteristics of the construction materials: permeability; reaction between concrete and environment, in particular the carbonation process and washing carried out by the soft alpine water; low resistance to freezing/thawing; cases of weakening of the siliceous aggregate of the concrete also exist.
- external actions; high variations in external temperature, with long periods well below 0°C in mountain areas; ice erosion; uplift; the large-scale earthquakes which took place in 1976 in Friuli and in 1981 in Campania did not damage the dams in those areas.
- structural behaviour: types of original facings used were various, but not conclusive; the lack of joints in dams built before 1925 has caused some cracking, which was satisfactorily treated with local waterproofing;

No deterioration due to stresses at the heel (tensile stress, concentration of stress) has been found. There are no cases of deterioration of the upstream face due to the settling of the bedrock: the condition of the foundations is, generally speaking, satisfactory, due also to consolidation grouting, technique which has meanwhile been developed.

The first instances of the rehabilitation of the complete face were the applications of sheet metal in 1935 and 1948 to the Gabiet and Vacca dams; during 1954-62 concrete facings were built on five dams; after that date, rehabilitation with meshed gunite began; after an experiment in 1971, the first refacing with a geomembrane took place in 1982 (Lago Nero dam). The bulk of rehabilitation activity starts from 1970; Table II gives a list of the interventions carried out to date.

Rehabilitation of the face is particularly necessary for gravity dams in masonry situated at high elevations; concrete facings were applied to those dams not in line with the current regulations; sheet metal and more recently geomembrane has been applied to those dams where particular problems of availability of aggregate on site or of transport of materials exist; meshed gunite has also been applied to high altitude dams, depending, amongst other factors, on the possibility of preparatory high-pressure sand blasting (200-400 atm.) of the surface to be treated.

On dams situated in areas of temperate climate, and on multiple arch dams, meshed gunite is often used; five thin-arch dams were refaced with epoxy resins, but this operation does not produce lasting results; the possibility of using geomembrane to reface a multiple-arch dam would be interesting.

Lastly, three small dams at low altitudes, whose upstream faces were successfully rehabilitated by grouting the body of the dam, must be mentioned.

From this picture, the essential factors which determine the choice of type of facing emerge: checking of the static condition of the dam; difficulty of supply of materials (in some cases, materials can only be brought up to the dam by helicopter or by cableway); another important factor is of course the need to limit as far as possible the time the reservoir is out of service.

CONCRETE FACINGS ANCHORED TO DAM BODY

This rehabilitation technique has been used on 13 dams in Italy, most of them situated in the Central or Western Alps at elevations between 1500 and 2600 m.a.s.l.. All are slightly curved massive gravity dams, in concrete or in masonry, built between 1923 and 1953.

Two distinct approaches were used, depending on the existing situation. One of these is specific to masonry dams, in which leakage by seepage through the body of the dam, due to deterioration of the seal of the original facing, can reach very high levels over time; the steady seepage through the structure produces a progressive impoverishment of the mortar between the stones, with a consequent reduction in weight of the entire body of the dam; in other cases the deterioration of the facing of the Levy-type in some old dams causes a notable increase in uplift, again with consequent influence on the static condition of the structure. In all these cases the new concrete covering is an effective protection against seepage through the dam, as well as increasing the overall weight of the structure, bringing the safety factor against sliding into the limits contained in the Code. The second approach is specific to concrete dams, where refacing is necessary because

of serious deterioration of the original face; its function is therefore exclusively protective.
The first type of rehabilitation of an upstream face , which we can define as static-protective, has evolved along the following lines: in at least four of the first dams to be rehabilitated, between 1954 and 1964, the new face was conceived as a massive structure built up against the existing one and, in practice, part of a new construction. The minimum thickness at the top is of 1.50 m.; maximum thickness at the foot of the dam is of 4.60 m.; the ratio between mean thickness of new face and height of dam is in the order of 7×10^{-2}; the joints, all vertical, are predisposed every 12-15 m. and drainage is by means of large diameter vertical drainage wells, which collect leakage in inspection galleries, within the facing itself.
Subsequently, planning criteria were refined; in three dams requiring a purely static integration, the maximum thickness at the foot was reduced to 2.50 m. and 0.80 m. at the top, with the corresponding ratio mean thickness:height being 3.6×10^{-2}; with the development of F.E.M. analysis the concept of solidarity between the two structures was enhanced; and the spacing of anchorages was reduced, and horizontal joints were introduced; semicircular vertical drainage wells were still connected to inspection galleries within the new facing or in specially created seats at the foot of the dam.
The second type of rehabilitation of an upstream facing, with purely protective function, (five dams in number) follows a construction pattern which started in 1966 and which has changed little since then. The concrete facing, well reinforced with double mesh and with constant transverse section, does not normally exceed 90 cm., and the ratio mean thickness : height is about 1×10^{-2}; anchorage bars were very closely spaced and horizontal and vertical expansion joints divide the face into regular rectangles of side 8 - 12 m.. Drainage of the facing is again resolved with sub-horizontal holes running from the galleries or from the existing drainage wells, to the area of contact between old and new facings.
Very satisfactory results have been achieved, in terms of both reduction of leakage and rehabilitation of deteriorated concrete. The types of concrete used, frost-resistant and carefully checked both for grading and quality of aggregates, using portland pozzolana cement or blast furnace cement according to the aggressiveness of the waters in the reservoir, showed a good degree of conservation. The operation is very expensive and complex and often requires closure of the reservoir for several years, but it does appear to give results which are reliable and long-lasting.

COATING WITH SYNTHETIC RESINS

The first experiments with resin coating of the upstream face of dams in Italy were in 1968. The evolution of waterproofing products was so rapid as to prevent the development of a definitive technological solution. The types of covering used

are in fact very diverse, indicating the still-experimental nature of the technique. Eight dams have so far been treated with synthetic resins in Italy; mainly arch dams, multiple arch dams or Ambursen dams. In some cases the upstream faces have been entirely covered with resin; in others, the application has only been partial, either because the deteriorated area was of limited size, or because of testing in order to compare the results with other protective techniques (gunite or waterproof cloths), used on other parts of the same dam. The first known application is to two small cement-rendered masonry dams, at a very high elevation (2850 m.a.s.l.); application of the resin coating permitted an evaluation of the resistance of this material to an extremely severe environment, where the continual alternation between action of ice and exposure to ultraviolet rays could provide information not easily deducible in the laboratory.
Essentially, the resin is applied as follows: a primer is applied for anchorage, then two or more coats of epoxy resin are spread (these have coal tar pitch or bitumen additives). In more recent cases the resin will polymerize even in the presence of water; a layer of glass fibre mesh is interposed between the two coats and a layer of finishing pigment to protect the resin from ultraviolet rays is superimposed. Overall thickness is normally about 1-1.5 mm. (including the mesh).
As far as has been established, the mean life of the coating can be said to be satisfactory (10 or more years) so long as the conditions suitable for application are met: condition of face, method of application, temperature, protection from ice through a suitable compressed air de-icing bubble plant.

COVERING WITH REINFORCED GUNITE

This protective technique, in use since the era of the construction of the first concrete dams, has also been used for extensive rehabilitation of the original facing; since 1954 it has been used on the upstream face of 13 dams.
Facings in reinforced gunite have been adopted both for gravity dams and other types of construction (Ambursen, multiple arch dams); for the former the reasons for the application are to be found in the high seepage gradient, for the latter it is mainly the need to protect the upstream concrete face. In the most up-to-date application techniques, several factors are extremely important: preparation of the face; anchorage to the body of the dam, ; the execution of a perimetral cut-off trench for anchorage at the heel of the dam. The overall thickness does not usually exceed 10 cm.. An autonomous drainage system is not usually envisaged, since it is held that for masonry dams, drainage is ensured by the thin cracking or permeability of the mortar; for concrete dams, the continuity between gunite and structure is so close (often enhanced by glueing with epoxy resin) that the drainage system of the dam may be considered sufficient also for the new coating.
The covering is without joints for low elevation dams without

own joints; in the case of dams in the high mountains, a number of joints are incorporated into the facing, more in some cases that those of the dam itself; in one case, horizontal joints were also incorporated. Particular care is taken over the joints and the perimeter anchorage in areas with a harsh climate, after the numerous negative experiences with this type of facing due to the weakness of this construction detail. When certain rules are followed, i.e. preparation of the face and careful study of the grading and of the joint systems, this technique can be reliable; in severe climates, however, the life is rarely above twenty years .

COVERING WITH STEEL PLATE

In Italy, the waterproofing of deteriorated faces with steel plate has been successful in some cases; six dams built between 1922 and 1931 (raising the dam in one instance: in 1942 for the Venina dam) in an Alpine environment at a rather high altitude (1824 - 2360 m.) were so rehabilitated. The technique of protecting the face with sheet metal was developed from 1931, mounting the sheets by bolting them between anchorage plates and supporting frame sealed with hemp and lead.
Following this, (Vacca dam) the technology was simplified, by welding small dimension metal sheets (1.8x7 m^2) and inserting these into steel ribs predisposed vertically on the face, again with hemp and lead seal.
The latest developments in steel technology (steel plate with low C content, weldable, corrosion-resistant) have made it possible to mount the fully welded steel sheets, welded both to each other and to the vertical anchorage struts (Diavolo and Truzzo dams), thus avoiding joints and the relative seals. The interspace (0.20 m. thick approx.) between steel sheet and face of the dam was then filled with porous concrete provided with vertical drainage wells connected at the bottom by an inspection gallery already existent in the body of the dam. In the cases described, to improve protection against corrosion anti rust products bonded with epoxy resins were applied, and protected by paints (chlorinated rubber or chrome-vinyl with light-coloured barium pigments), and a compressed air de-icing bubble plant was installed; in the case of Venina and Cingino dams, cathodic protection was also instituted.
The results obtained with steel facing are good, as long as the metal is well-sealed to the bedrock and quality steel is used.

COVERING WITH WATERPROOF SYNTHETIC GEOMEMBRANE

This protection technique was first adopted in 1970 and has now been applied to seven dams in all. Facings with synthetic geomembrane have been used on both gravity dams (in masonry or in concrete) and multiple arch dams, but it would also be suitable for application on double curvature arch dams or Ambursen dams.
The first attempt consisted of glueing a polyisobutylene membrane directly onto the face; since this method has proved

unreliable, in subsequent cases a waterproof synthetic PVC-based geomembrane thermocoupled to a geotextile was applied without glueing directly onto the concrete face.
The geomembrane (the only type used so far) is produced with a special formulation to resist UV rays and low temperatures, with good performance regarding breaking load, springback, resistance to strain and abrasion.
The geomembrane is anchored to the face of the dam by using a patented system consisting of two sections of stainless steel rib which fit together to form a continuous connection. An adjustable anchorage unit, in three pieces, connects the two rib components, holds the geomembrane in position and permits its pre-tensioning and positioning, and also fixes the structure so formed solidly to the face behind.
With this system, uniform and continuous anchorage of the waterproof membrane is achieved, while the same is made to adhere to the face beneath, and pre-tensioned to avoid creases or bulges. The vertical ribs, which are installed at intervals of about 185 cm., act also as drainage wells for the general drainage system formed by the geotextile itself. The ribs take the water flow to the foot of the dam; from here, through holes in the dam body, it is taken to the internal galleries, or to the downstream face, where it can be measured and eliminated.
Before laying the geomembrane it is necessary to demolish the most deteriorated parts of the face and replace them with new concrete; it is advisable in some cases to build a concrete perimetral cut-off trench at the heel of the dam to fix the geomembrane; for dams situated at a very high altitude it is also best to provide for a de-icing bubble plant.
In conclusion, this type of protective membrane presents the advantages of protecting the face of the dam including the joints, as well as creating a new drainage system by means of the vertical fixing ribs; other advantages are reduced installation time and ease of maintenance. Experience to date indicates that it will probably have a life of some decades.

CONCLUSIONS

As can be seen from the above outline, the present-day tendencies in rehabilitation are as follows:
- concrete facing for gravity dams (which are the oldest), in particular for masonry ones and for those with static weakness with respect to the current Code; the advantage is of the long life, but it is expensive, also in terms of out-of-service period of the reservoir;
- reinforced gunite for any type of dam in a temperate climate, although it can also be used at high altitudes, where particular complements are necessary: joints, anchorage trench at the heel of the dam, de-icing plant;
- geomembrane for gravity and cylindrical arch dams; the advantage is of rapid application and limited preparation of the underlying face, but it requires a perimetral anchorage trench or the equivalent; good results have been obtained on dams at high elevations, where a de-icing plant is advisable.

TABLE I - Italian masonry and concrete dams subdivided by type of original upstream facing.

TYPE	MASONRY DAMS			CONCRETE DAMS		
	n°	Y	R	n°	Y	R
Rough stones	8	1879-1925	AL-SI	-	-	-
Concrete poured	-	-	-	18	1920-1942	AL-AP-IS
Concrete vibrated	-	-	-	157	1938-1987	AL-AP-IS
Cement coating	20	1901-1940	AL-AP	34	1916-1954	AL-AP-IS
Gunite reinf.	11	1883-1944	AL-AP-IS	23	1914-1956	AL-AP
Squared stone pitching	28	1905-1956	AL-AP-IS	14	1925-1959	AL-AP
Concrete elements	-	-	-	2	1924-1950	AL-AP
Reinf. concrete slab	1	1926	AP	1	1937	AP
Lévy facing	4	1931	AL	2	1923-1930	AL
Metallic facing	1	1931	AL	1	1964	AL
Bitumen coating				2	1924-1928	AL-IS
	73	1879-1956		254	1920-1987	

Y = year of completion of dams; R = geographical region; AL = region of Alps; AP = region of Appennines; IS = islands (Sicily Sardinia).

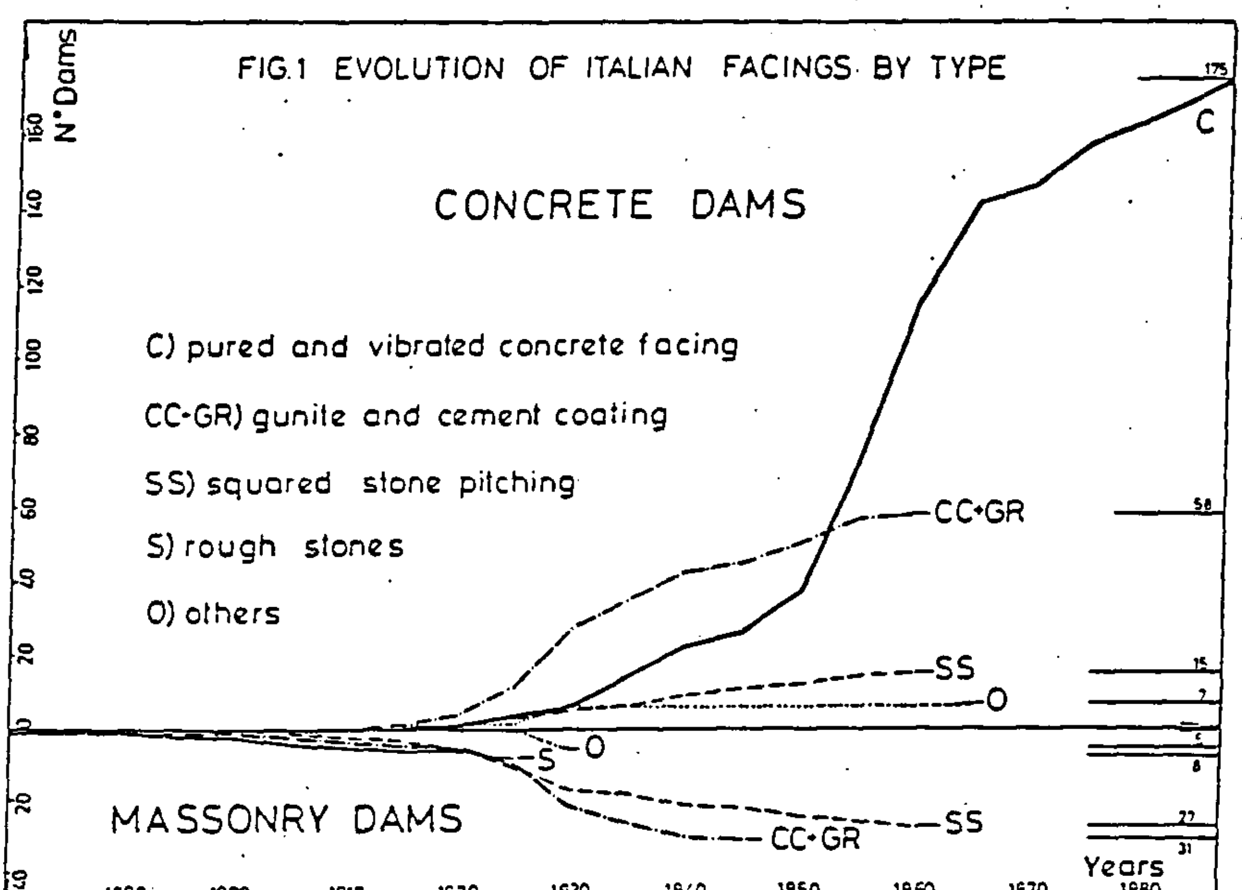


FIG.1 EVOLUTION OF ITALIAN FACINGS BY TYPE

TABLE II - Facings entirely retiabilitated ou dams in masonry and concrete.

NAME	T	H	EL	Y1	TF	A	Y2	TH
A) Reinforced concrete								
Lago d'Avio Grande	PGM	39	1910	1929	LF	4600	1947	150-400
Lago Salarno	PGM	41	2071	1928	LF	8900	1960	150-360
Toggia	PGM	44	2193	1932	SS	6250	1961	150-385
Morasco	PG	57	1818	1940	C	16900	1962	60
Lago d'Arno	PGM	36	1821	1927	LF	2900	1966	50-400
Careser	PG	62	2601	1934	C	12600	1966	40
Cavia	PG	24	2101	1949	C	3500	1970	40
Alto Mora	PG	40	1538	1953	C	3950	1970	90
Melezet	PG	21	1493	1921	C	600	1971	40-90
Rochemolles	PG	63	1975	1930	CC	7800	1976	60
Campliccioli	PGM	70	1364	1928	SS	11800	1979	60
Lago Busin infer.	PGM	21	2390	1923	CC	2540	1986	80
Giacopiane	PGM	40	1014	1926	CC	4500	C	60
B) Reinforced gunite								
Lago Lavezze	PGM	40	648	1883	GR	2400	1954	5
Lago Lungo	PGM	21	685	1901	CC	4900	1966	6
Agnel (*)	PGM	20	2297	1938	CC	600	1972	5
Riolunato	MY	27	690	1920	CC	1650	1972	13
Fregabolgia	PGM	65	1960	1952	SS	5000	1979	10
Ozola	MV	25	1229	1929	GR	1500	1969	5
Lago Colombo	PG	33	2059	1929	SS	750	1982	9
Pian Casere	PGM	47	1819	1946	SS	3150	1981	9
Lago Benedetto	PGM	32	1931	1940	SS	6700	1984	9
Lago della Rossa	PGM	26	2720	1932	GR	4200	1986	9,5
Campo Tartano	PGM	58	958	1929	GR	1500	1987	9
Trepidò	PGM	37	1274	1928	CC	3600	1987	8
C) Metallic facing								
Gabiet	PGM	48	2378	1922	SS	4800	1935	0.50
Lago della Vacca	PGM	21	2561	1927	CC	750	1949	0,27
Lago Venina	MV	50	1844	1926	GR	7200	1965	0,35
Lago del Diavolo	PGM	26	2150	1931	MF	900	1970	0,20
Cingino	PGM	46	2264	1930	SS	5700	1973	0,30
Lago Truzzo	PGM	30	2080	1927	SS	5100	1971	0,50
D) Geomembrane (* partial)								
Lago Baitone	PGM	37	2283	1930	LF	3500	1971	0,20
Lago Nero	PG	43	2027	1928	GR	4000	1982	0,20
Molato (*)	MV	55	362	1928	CC	1000	1986	0,20
Pian del Barbellino	PG	66	1872	1931	GR	4000	1987	0,25

Cignana I	PG	58	2173	1928	C	9000	1988	0,25
Publino	VA	40	2135	1951	CC	4200	C	0,25
E) Epoxy resins								
Corfino	VA	38	515	1914	GR	1200	1970	-
Lago Grande	PGM	9	2847	1980	CC	400	1970	-
Lago Balanselmo	PGM	9	2734	1931	CC	150	1970	-
Gurzia	VA	49	432	1925	GR	1670	1974	-
Pian Sapeio	MV	16	965	1926	CC	2000	1974	-
Combamala	MV*	33	915	1916	CC	2000	1977	-
Rimasco	VA	34	887	1925	CC	1050	1978	-
Riofreddo	VA	41	1206	1956	GR	4165	1972	-

T = Type of dam; **PG** = gravity concrete; **PGM** = gravity masonry; **MV** = multiple arch dam; **VA** = arch dam

H = height (m.)

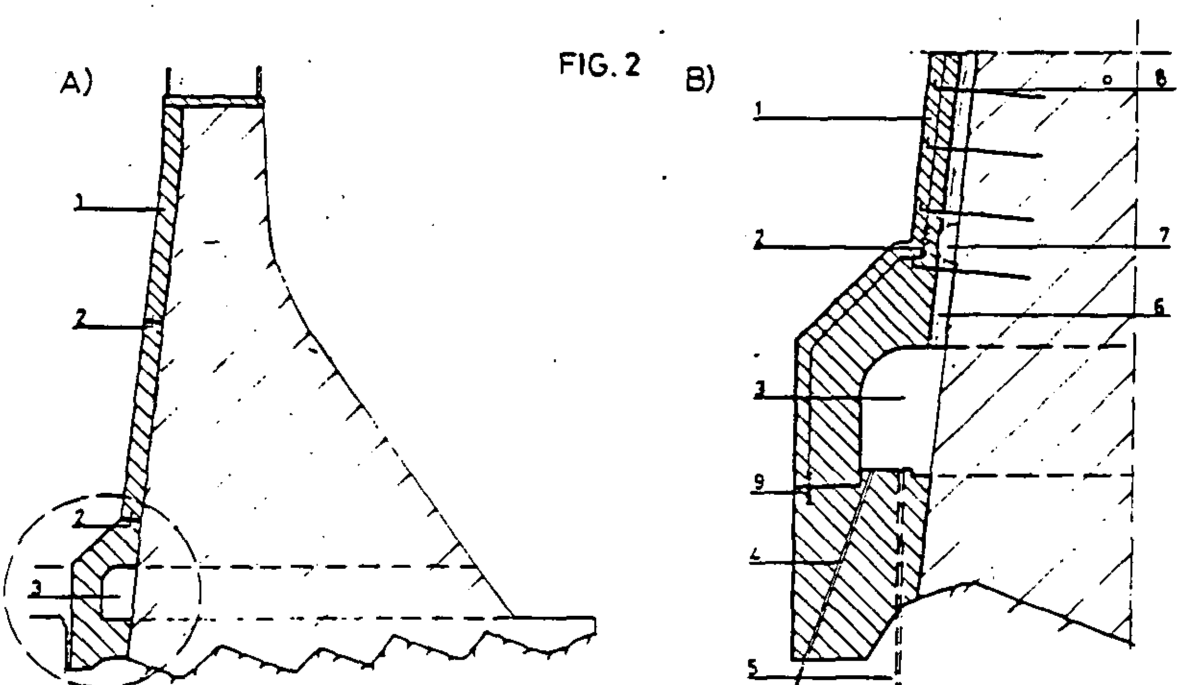
EL = elevation above s.l. (m.)

Y1 = year of completion

Y2 = year of completed remabilitation

TF = type of original facing; **C** = concrete; **CC** = cement coating; **GR** = reinforced gunite; **LF** = levy facing; **MF** = metallic facing; **SS** = squared stone pitching with filled joints; **A** = facing surface (m^2); **TH** = thickness of new facing (cm)

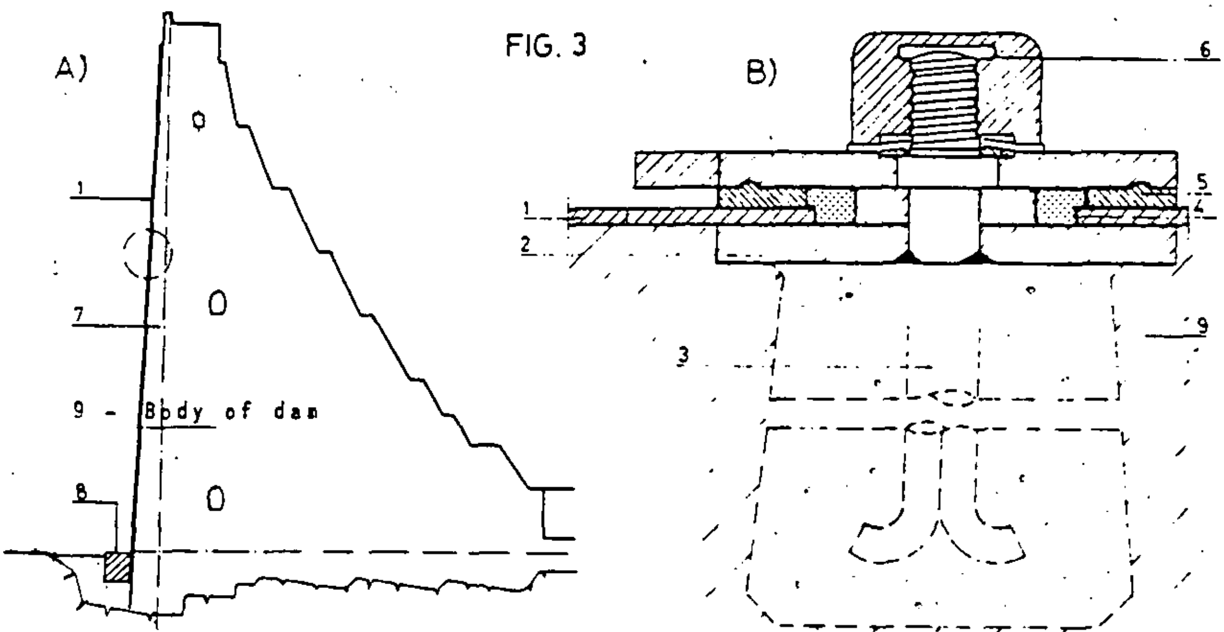
FIG. 2



A - Typical section through a masonry dam with new facing in concrete slabs
B - Section through the concrete slab facing at vertical joint

1 - Reinforced concrete slabs
2 - Horizontal joints
3 - Gallery to collect seepage
4 - Grout cut-off
5 - Pressure relief pipes
6 - Vertical drainage wells
7 - Horizontal drainage galleries
8 - Anchorage bars, D = 24 mm.
9 - Water-stop sealing strip

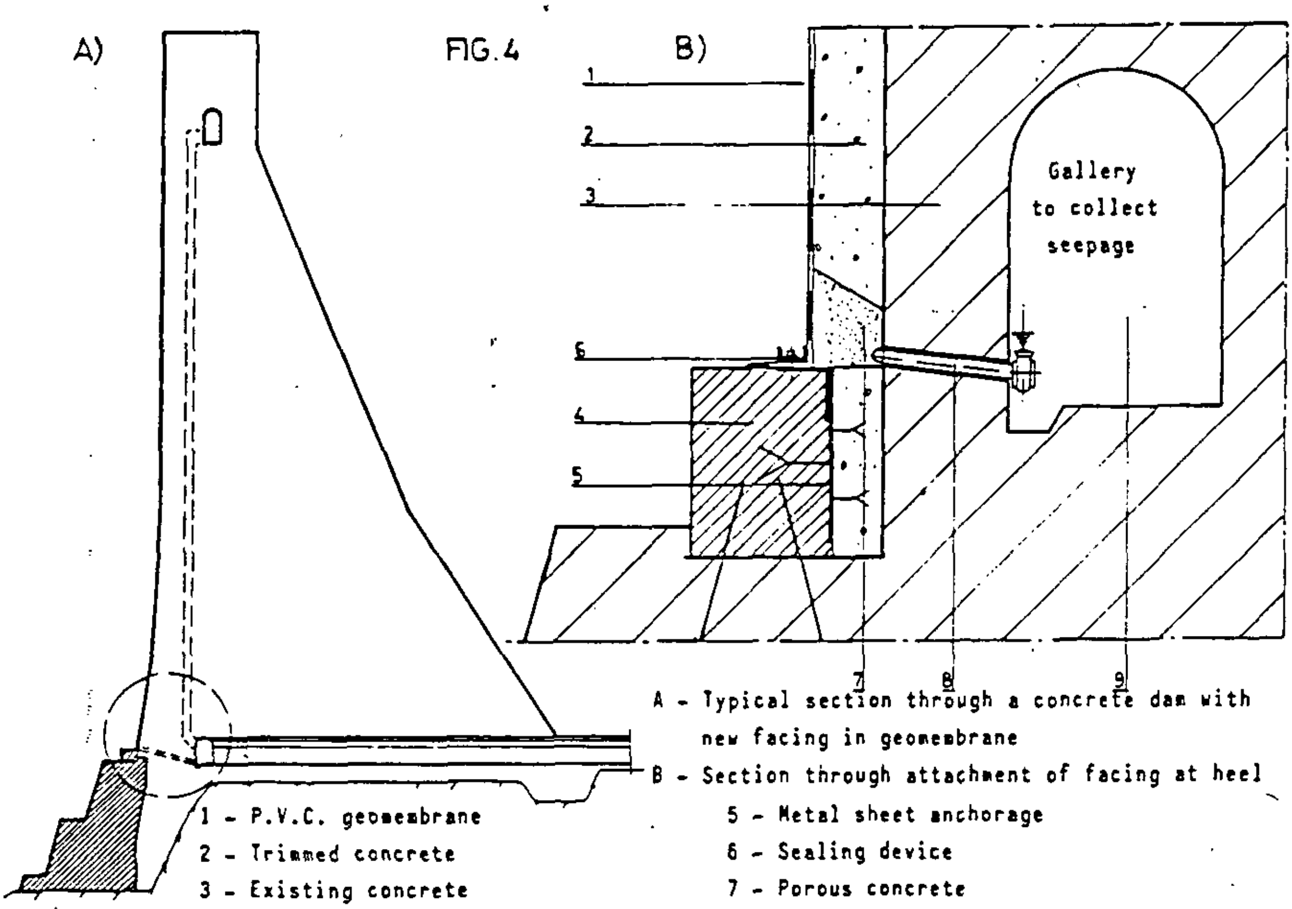
FIG. 3



A - Typical section through a masonry dam with new facing in sheet metal
B - Detail of sheet metal anchorage joint

1 - Steel facing sheet
2 - Metal plate fixed against masonry
3 - Anchor bars
4 - Greased hemp
5 - Lead gasket
6 - Closing nut
7 - Pressure relief pipes
8 - Concrete encasing at heel

FIG. 4



A - Typical section through a concrete dam with new facing in geomembrane
B - Section through attachment of facing at heel

1 - P.V.C. geomembrane
2 - Trimmed concrete
3 - Existing concrete
4 - Concrete perimetral cut-off trench
5 - Metal sheet anchorage
6 - Sealing device
7 - Porous concrete
8 - Pressure relief pipe

REHABILITATION OF DAMS WITH ENGINEERED SURFACE MULTILAYER PVC MEMBRANES

P Sembenelli (Consultant)
J Cuniberti (Consultant)

SYNOPSIS

The use of synthetic membranes produced specifically for the rehabilitation of dams is now possible. Examples of repair of two reservoirs using a reinforced PVC membrane with polyester geotextile backing, installed with special adhesives, both designed and tested for predetermined requirements is given.

INTRODUCTION

1. The importance of this conference is emphasized not only by the ever increasing number of "older" dams but above all by their deterioration. Deterioration means only two things: loss of water or loss of strength. The loss of water, besides being the antithesis of a dam's primary function, normaly entails and promotes the loss of structural strength through erosion and leaching or a decrease in overall stability due to hydraulic uplift forces. Therefore if we can eliminate, diminish or postpone leakage we can prolong the useful life of the reservoir.

NEW PRODUCTS

2. The logical place to inhibit water loss is at the point of contact between water and the dam. This has led to the development of surface membranes which have undergone rapid growth in materials and types, and improvements in structural details and application techniques. Improved chemical technology, manufacturing methods and experience have all contributed to the success of polymer surface, or near-surface, membranes offering, today, substantial advantages in ease of application and price.

REQUIREMENTS

3. An effective surface membrane must have a very low permeability, at least 1000 times less than the underlying structure. It must have sufficient tensile strength to resist any deformation of the dam and even more, to bridge across large cracks or a sharp step-shaped deformation of the old dam surface. It must be tough enough to remain unharmed by minor accidents during construction or later. It must be rigid enough to be placed smoothly, without wrinkles. It must be resistant

to aging due to environmental agents such as the ultra-violet rays of sunlight, high temperatures, freezing, hydrolysis by the impounded water, ice action and damage from floating debris.It must be chemically stable and have a reduced rate of internal breakdown or loss of its components. Last, but not least, its application must be easy, safe and rapid, since the upstream surface can be exposed for only a short period of time.

4. Taking advantage of advanced polymer membrane technology a recently developed concept makes it possible to design a membrane for each renovation project to suit the structure's characteristics and the local conditions on a case by case basis. GEODAM (R) is the commercial name of a waterproofing system consisting of three basic parts: GEODAM GEOCOMPOSITE (membrane plus backing), GEODAM ADHESIVE (different for concrete or bituminous surfaces) and GEODAM PRIMER for consolidation of the structural surface. The GEODAM geocomposite consists of a multilayer polyvinylchloride Geomembrane adhered to a geotextile support. The thickness of the PVC can be varied from 1.5 to 4.0 mm and the weight of the geotextile can range from 120 to 500 g/m2.

5. The multilayer concept for the geomembrane permits a combination of an ultra-violet light resistant outer layer, an inner sandwich of stronger layers enclosing a glass fiber reinforcement, an underlayer of PVC specially formulated to resist the attack of noxious chemicals from the support such as alkalies from cement or aromatic compounds from a bituminous surface. The geocomposite is produced wholly or in part by the spray-on process based on the application of liquid PVC over the previously hardened layers. Any possible tiny defect in one layer will thus be blocked by the other layers. The geotextile layer functions as a protective cushion, as a good adsorbant base for the adhesive and also as a drainage layer in certain cases. To optimise all these functions the thickness of the geotextile may also be adapted to the particular surface to be repaired. The GEODAM Adhesive must also be formulated as an integral part of the multilayer system since it has to be adapted to the reservoir surface, the drainage conditions and the mode of application.

MECHANICAL CHARACTERISTICS

6. A second fundamental concept introduced in this geocomposite is the bi-modal stress-strain relationship. At low stresses the layer of stiff glass fibers insures a dimensional stability which is very desirable to reduce aging from thermal cyclical changes and, during placing, permits rapid, wrinkle-free application. This characteristic is evident in the marked bend in the stress strain curve at about 10% elongation shown in Fig. 1. At higher stresses the embedded glass fibers yield and the tensile stress is shared entirely by the PVC geomembrane and the geotextile backing over a large deformation. The very large deformation potential of the PVC membrane is a very

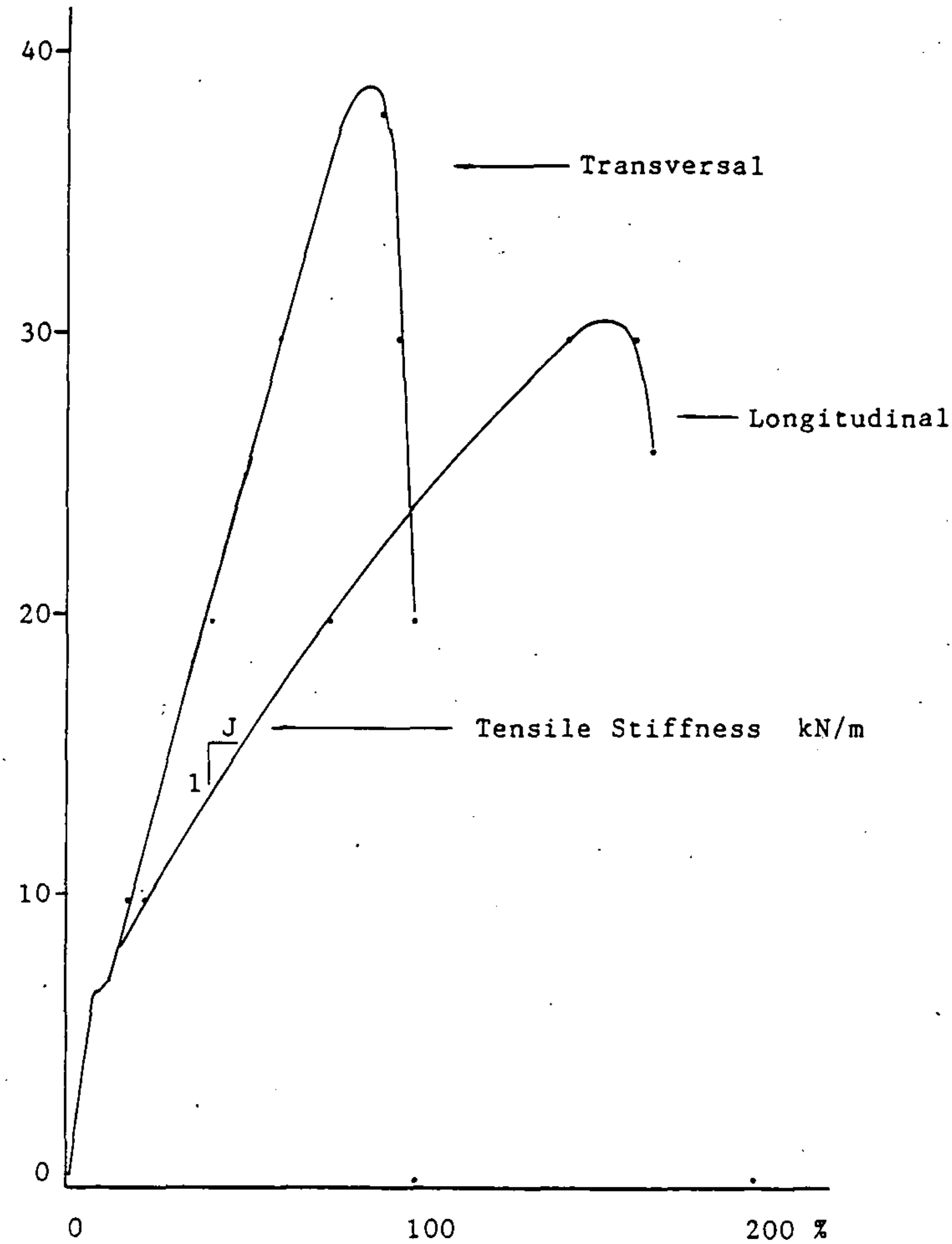


Fig. 1. Tensile Force (kN/m) - Elongation (%) test results of GEODAM (R) Geocomposite (geomembrane+geotextile)

favorable quality as far as bridging cracks or accomodating distorsions of the reservoir surface. The characteristics of the geomembrane and the geotextile are matched so that the stress created in the membrane by sharp local deformation is redistributed over a a larger area by the backing. Thus, although the work density (stress x strain) remains constant, the stress is reduced and the strain spread over a larger portion of the membrane.

7. In addition to the tensile strength tests a series of standard tests, including dynamic punching, tearing strength, static punching and cold bend were performed as a measure of general resistance to accidental damage.

8. In order to assure the resistance of the membrane, even in the case of a large crack in the supporting dam, special tests were performed under water pressure. The pressure chamber shown in Fig.2 was modified internally to provide a deformable supporting surface which could be spread apart to simulate a crack or displaced vertically to form a step. The local elongation was observed for each type of deformation and for various water pressures. The membrane did not fail or leak at water pressure of 90 m when deformed across either an opening or a step of 40 mm.

9. As an example of the pressure chamber test results the Fig. 3 indicates the elongation measured at three different positions at pressures up to 900 kN/m2 for a step deformation of 35 mm. Although the strain at the sharp angle (o) is high, the strain concentration remains relatively uniform at the three measured points due to the concomittant action of the glass fiber and of the geotextile despite the large increase in pressure.

10. It has been found ,through test like the one above, that a useful dimensionless parameter called the deformation potential, A, can be defined as :

$$A = Sh/J$$

where S = the magnitude of the deformation m.
h = the normal water pressure kN/m^2
J = the tensile stiffness kN/m
(force per unit width per unit strain)

Using this parameter the safe deformation level acceptable for the geocomposite may be compared with the probabilty of the occurrence of a deformation of magnitude S.

HYDRAULIC CHARACTERISTICS

11. The permeability of the PVC membrane is about 10^{-14}cm/s. That is about ten thousand times less than concrete. Any real leakage must come from microscopic defects in the PVC film. Since this is built up from three separate sheets this type of

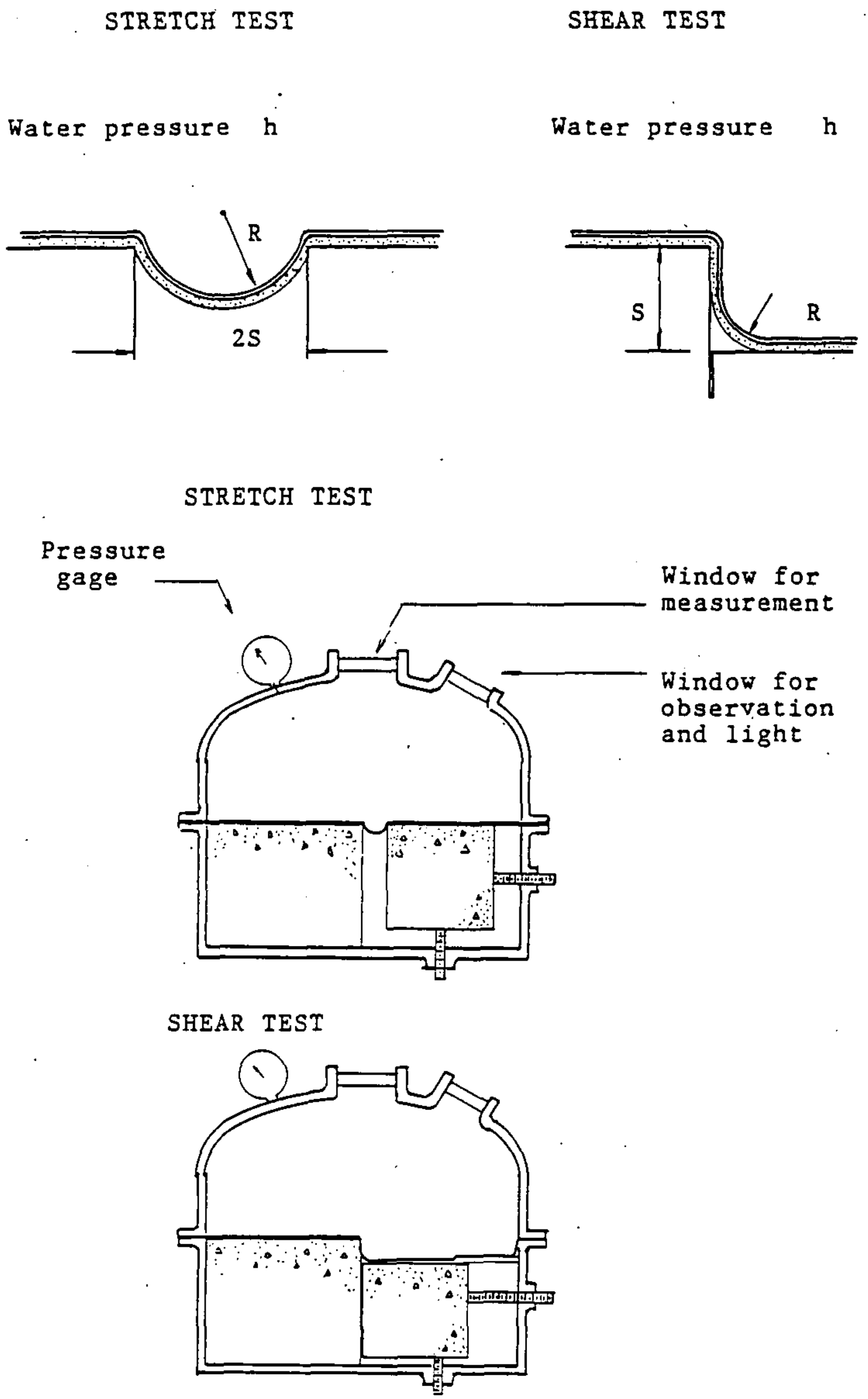


Fig.2 Pressure chamber modified for stretch and shear tests and definition of parameters measured..

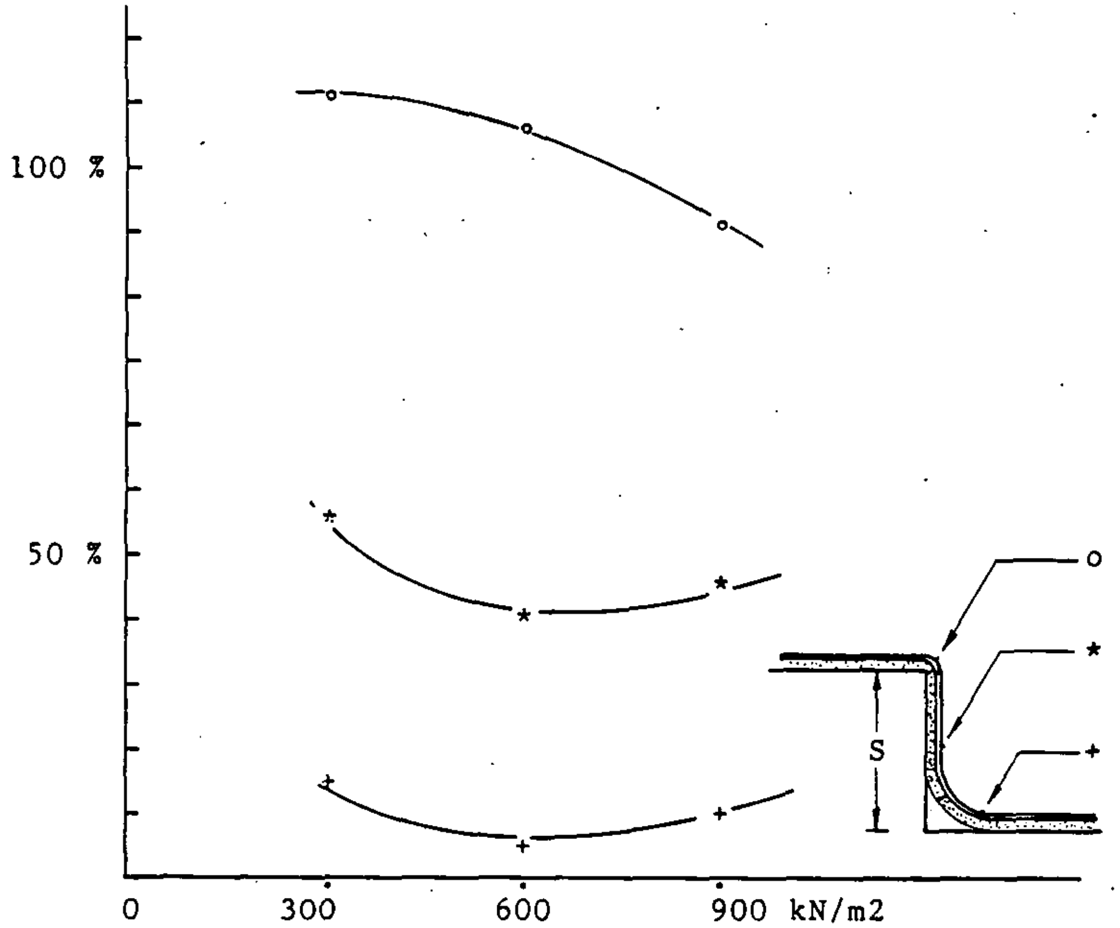


Fig. 3 Water Pressure (kN/m2) - Elongation (%) results of Shear test across a step of S = 35 mm

leak is practically eliminated. More important is the permeability of the geotextile backing which may function to provide lateral drainage when placed over a relatively impermeable concrete surface. The geotextile is compressed under the effect of the water pressure but radial permeability tests have been performed to prove that an adequate permeabilty of $3x10^{-5}$cm/s remains even under 150 meters of normal water pressure.

DURABILITY

12. Durability should include not only passive resistance to deterioration with time due to loss of plasticizer, break down of polymer chains and leaching of chemicals, but also resistance to variable forces which in time lead to fatigue. In practice the membranes are placed with very low stresses and should a catastrophic defomation occure the membrane can be expected to bridge the crack temporarily, until the damage can be repaired or the structure reinforced after draw-down. The deformation potential should nevertheless be sufficiently large so that normal seasonal changes do not induce fatigue problems. Of course the membrane has passed abrasion tests and simulated

hail tests. An added adavantage of the PVC membrane is that ice has no measurable adhesion on it.

13. As to the passive durability, extensive tests were performed to measure the loss of tensile strength after exposure to ultraviolet light, hot air and hot water, and various organic solvents. Although it is not possible to correlate these results exactly with actual field conditions the results indicate a useful life expectancy of many decades. Data available at present indicate that a 20 year life should produce not more than 6% loss of tensile strength and not more than 20 % loss of plasticizer initially present in the membrane materials.

ADHESIVES

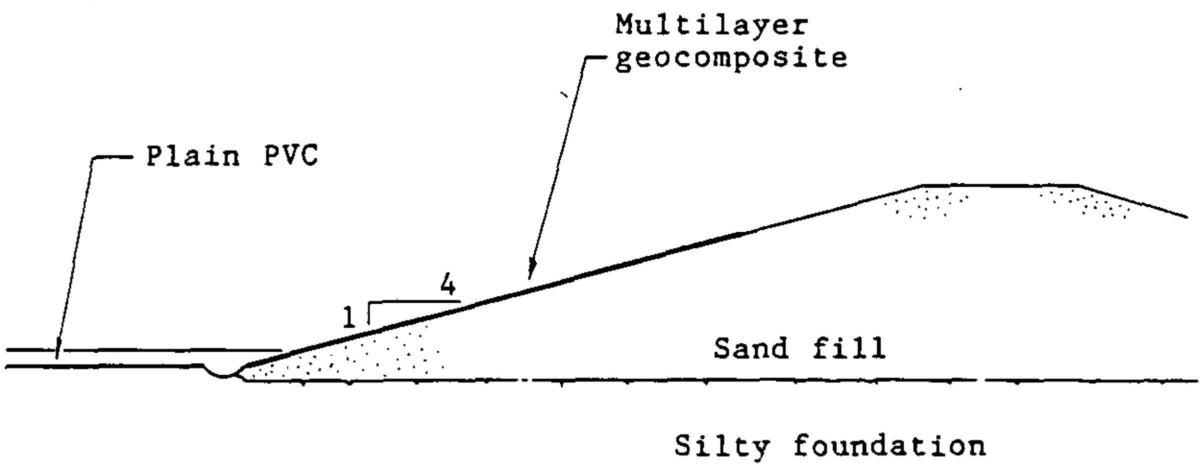
14. A third important feature of GEODAM is the application of the geocomposite using adhesives. This permits an even support with a minimum of stress concentrations in the membrane. The adhesive may be formulated to penetrate only the surface of the geotextile for use on an impermeable concrete surface, thus allowing any seepage to flow within the thickness of the geotextile toward a conveniently placed drain. In the case of a dam with a bituminous concrete surface a different formulation of the adhesive, than that used for cement, is used so that although the geotextile is saturated by the adhesive it remains slightly permeable allowing any seepage to penetrate the relatively porous underlying asphaltic surface without excess pressure increase.

APPLICATIONS

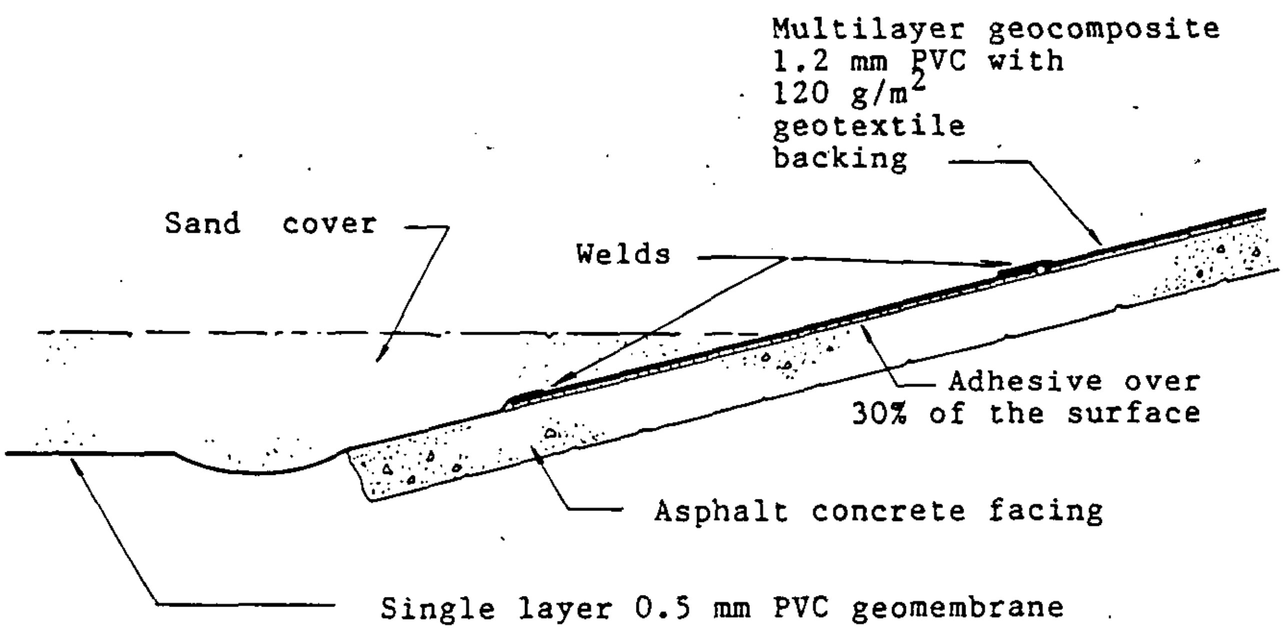
15. The GEODAM system has now had several successful applications. The first was on a dam in northwest Nigeria. The dam is part of an irrigation project which stores the only rain at the end of the summer. The dam is made of compacted silty sand with an asphaltic concrete facing.

16. Piping occured during the first filling through a weak spot in the foundation of the dam causing a sinkhole before the reservoir was full. Investigation revealed that the seepage path across the relatively porous bituminous surface and into the foundation was too short for the sandy soil of the compacted fill to act as a barrier.

17. The breech was repaired and the bituminous surface replaced, but for increased security , since the dam was made of silty-sand , it was decided to place a multilayer surface membrane on the aphaltic concrete face. The 13'000 m^2 of GEODAM geocomposite made of 1.2 mm PVC and 120 gm/m^2 geotextile backing was applied to the 1 on 4 sloping bituminous face at a rate of about 100 m^2 an hour including welding of the geocomposite rolls and application of the adhesive. The climatic conditions were favorable in the sense of being dry and not over 30° C but the dusty wind from the Sahara to the



a) Cross section



b) Detail of joint at toe of dam.

Fig. 4 Bituminous concrete faced sand fill dam repaired with a surface multilayer PVC geocomposite fixed with adhesive-GEODAM (R) SYSTEM

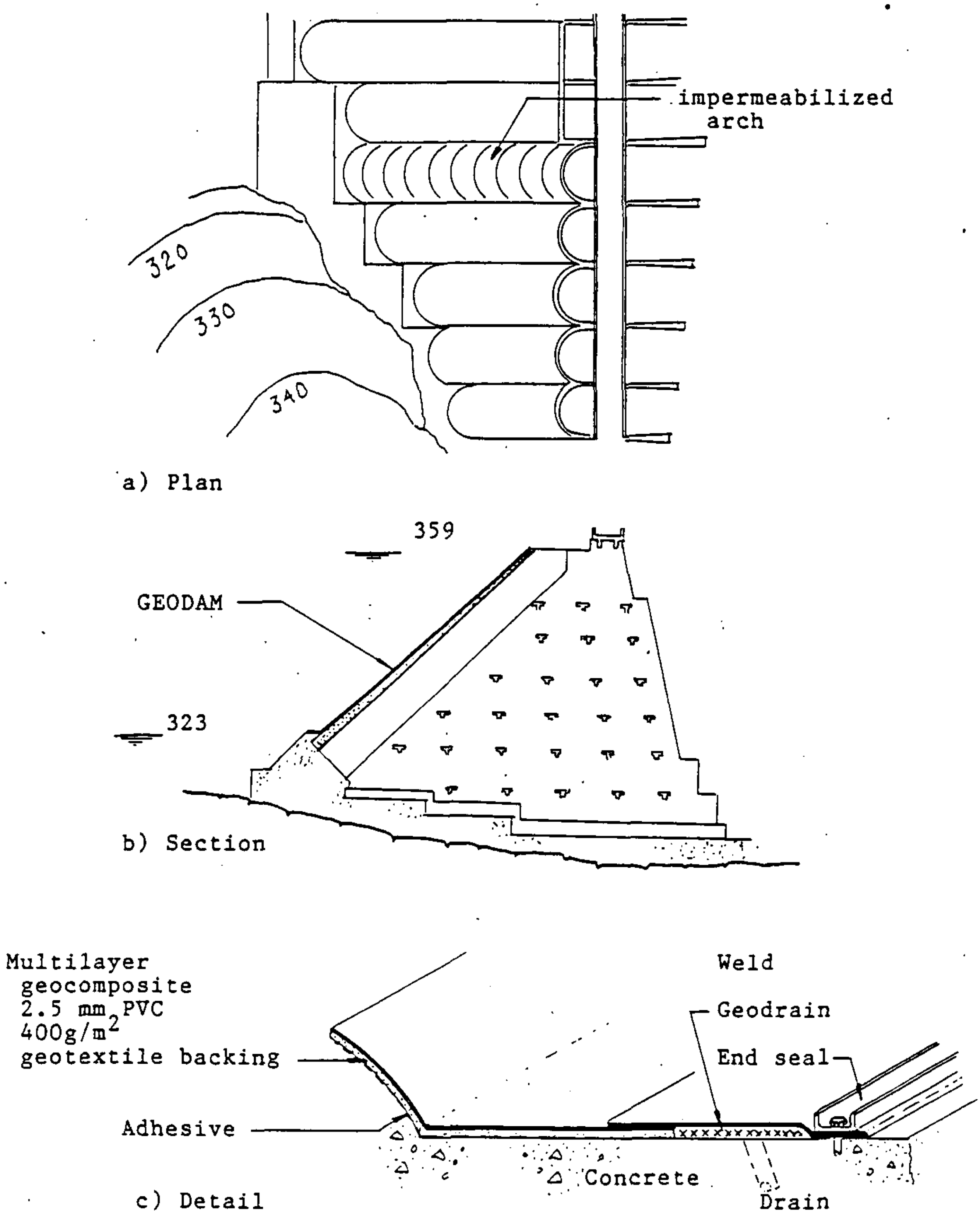


Fig. 5 Concrete multiple thin shell dam repaired with a surface multilayer PVC geocomposite fixed with adhesives. GEODAM (R) SYSTEM.

north required meticulous cleaning of all edges before welding. It should be added that a horizontal 0.5 mm thick plain extruded PVC blanket was extended upstream from the geocomposite and covered with a 50 cm sand protection. This plain PVC geomembrane was put down at a rate of 3'500 m^2 a day. Fig. 4 gives a typical cross section and a detail of this particular repair job.

18. Another example of dam rehabilitation is illustrated by the application of Fig. 5. This inclined multiple thin-arch dam was constructed to store water for irrigation purposes in Italy during the 1930's. The arches are supported by a series of thin concrete frames.

19. This dam has had leakage problems from the beginning, both diffused and at the construction joints. Over 50 years of operation and the average quality of the original concrete have resulted in a condition of substantial weakness. Both the quality of the concrete and the static conception of the dam make it difficult to find a solution. Attempts to cover the arches with another layer of reinforced concrete were abandoned. The success of mastics or brushed on preparations was limited by the adherence to the low quality concrete.

20. In a cautious search for adequate solutions based on full-scale tests it was decided to apply a multilayer PVC surface membrane to one arch. The GEODAM geocomposite was applied in horizontal strips, field-welded and glued to the thoroughly cleaned but rough concrete arch. The fact that the inclined membrane is an integral element which is held in place by the water pressure makes its adherence to the concrete of lesser importance. The adhesive will hold the membrane in place at low water stages while the geotextile will drain any underpressure. This repair was done at the annual emptying of the reservoir in late December. Despite the difficult weather conditions the 700 m^2 surface was applied in 4 working days. Naturally the rate would be faster for a larger area where repetition would improve the productivity of the application crew and the use of an appropriate lift platform could be justified.

SUMMARY

21. We have presented an application of high quality,factory produced engineered polymer geocomposites to improve the overall performance of dams. The renovation of dams and reservoirs using appropriately designed geocomposite membranes applied with adhesives presents a proven technique which has many potential applications.

THE USE OF POLYURETHANE GROUTS AND GROUT TUBES

Bert Kriekemans (De Neef America Inc)

SYNOPSIS

Most polyurethane grouts used for seepage control are "one-component" and water activated. They are prepolymers of a polyisocyanate with a polyol. Some of the polyurethane grouts use an accelerator to regulate their geltime. When they come in contact with water, they react, foam and solidify. There are hydorphobic and hydrophilic water activated polyurethane grouts.

HISTORY

The use of polyurethane as grouts is young compared to grouting with silicates (since 1886) and acylamides (since 1950). The first use of polyurethane grouts used for water control in civil construction was 21 years ago (1967) in Japan by Takenaka Komuten Ltd. (product name TACSS). TACSS has been used worldwide in the field of water control (for approximately 15 years in Europe and 7 years in the United States). In the 1970's, the 3M Company brought polyurethane grouts on the market, but they were used exclusively for sewer rehabilitation. TACSS polyurethane grouts have been used for the last 20 years for watercontrol in civil works, whereas the other polyurethane grouts have only been used for the last 5 years for this purpose.

ADVANTAGES

Advantages for using polyurethane grouts for seepage control, compared to other chemical grouts or epoxy grouts:

1. Being a one shot injection method, they are one-component grouts (Accelerators are mixed with the grout prior to injection, no pot life)

2. Adjustable gel times, from seconds to hours

3. Due to the CO_2 generation there is an expansion or volume increase of the injected grout (sometimes up to 15 times)

4. Most water activated polyurethane grouts are EPA approved to be used in contact with drinking water

5. Polyurethane grouts generally give higher strengths than other chemical grouts. But most times they are not intended to be used for structural repair (unlike the epoxy grouts)

6. Choice between rigid or flexible polyurethane grouts depending on the application

PROCEDURES

*The one-component polyurethane grouts are pumped into cracks, joints, fissures, or soil by hand-operated piston pumps, airless spray pumps, air or electric operated piston pumps (example: Graco), or pressure pots.

Pump pressures vary from slightly higher than the water pressure present, up to several thousand psi, volumes from ounces to gallons. When uplift pressures can cause structural problems, it is the responsibility of the owner, or the owners engineer, to establish allowable pumping pressure.

Being expanding grouts, it is sometimes necessary to drill extra relief holes with packers and pressure gauges. Also the injection pressure shall be measured at the point of injection and not at the grout pump.

*Most polyurethane grouts use an accelerator to regulate the induction and geltime, which is considered an advantage. The geltime also depends on the temperature.

*Injection into the strata can be done through mechanical packers or plastic injection ports for concrete, brick or rock. Manchette tubes or strainer pipes for injection into soil.

APPLICATION

1. Sewer Grouting:

   *Sewer lines are grouted following the double packer system

   *The problem of infiltration of ground water into manholes can be solved by injection of polyurethane grouts

2. Crack injection of polyurethane grouts in concrete or masonry structures:

   a. <u>Hairline Cracks</u>

      There are two (2) ways of sealing water bearing hairline cracks:

      1. Injection of the polyurethane grout behind the concrete or masonry wall in the substrate (soil) by drilling through the walls or driving pipes (strainer - manchette) from the surface.

      2. Injection in the crack or joint by drilling straight into it or under a 45° angle, installing packers or nipples and injecting under pressure.

   b. <u>Expansion Joints</u>

      *Route out old joint material, install an INJECTO tube, cover INJECTO tube with hydraulic cement, or epoxy, and inject the tube with flexible polyurethane grout.

      *Drill into expansion joint and inject a flexible polyurethane grout.

c. High Flowing Leaks

Canalize or control leakages through pipes (plastic, copper tubing, etc...) installed into the leaking area, or by drilling holes and intercepting the leakage deeper in the structure. Cover the remaining leakage area with a fast-setting cement. If necessary, cover this plug with an epoxy gel.

After all water is canalized through the pipes or drill holes, start injecting the polyurethane grout (fully catalyzed) in the lowest hole or pipe. If it appears in the next hole or pipe, close it off (by means of a valve) and keep pumping until the leakage stops.

One of the advantages of polyurethane grouts, is that you will have an immediate result.

3. Soil Stabilization

Polyurethane grouts can be injected into the soil through open-end, strainer or manchette pipes to strengthen the soil or to form an impervious curtain.

Advantages of using the polyurethane grouts are:

a. Higher strength than with other chemical grouts

b. Due to expansion of the polyurethane grouts during reaction, less grout will be needed

c. Adjustable gel times allow for further penetration into the soil.

d. Hydrophobic polyurethane grouts will not dilute or be washed away in underground water flows.

What are the criteria's for a polyurethane grout:

*First you need to determine if a flexible or rigid polyurethane grout is to be used (moving or non-moving cracks or joints).

*Polyurethane grout needs to be 100% solids (or contain no solvents). Solvents are flammable and cause shrinkage of reacted material when evaporating.

*Can not be water soluble, as they could then be washed out by waterflow.

*Have a good adherence to concrete.

*Will not shrink when no longer in contact with water (due to drying out (CFR hydrophilic)).

*Have a high flash point, expecially when working in enclosed areas (tunnels, basements, manholes, etc...) with the grout.

*Be EPA Approved for use on structures in contact with potable water.

*Have a controllable gel time.

SPECIFIC APPLICATIONS FOR DAMS

Sealing leaks in vertical joints. There are two(2) methods for sealing vertical joints in concrete dam structures depending on the accessibility of the joints.

1. If the face of the dam is readily accessible, the joints can be cut out in a V-shape. An INJECTO tube is installed in the V for the total length of the joint. A sealant is put at the outside of the joint over the tube (See Figure 1).

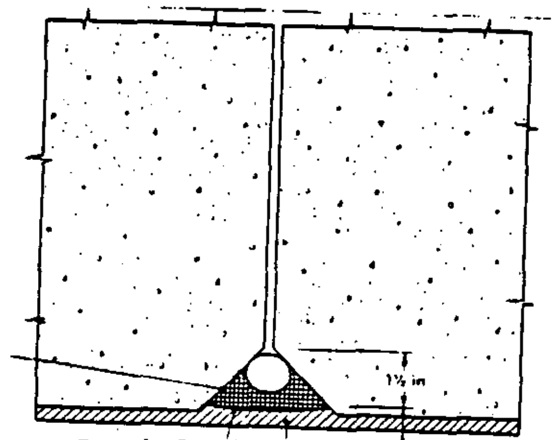


Figure 1. Design for repair of vertical joints in Richard B. Russell dam - Savannah, Georgia

After hardening of the sealant, the FLEX or TACSS 020 NF grout is injected from the bottom of the joint until it protrudes out of the top. The total length of the joint is injected with grout. By increasing the injection pressure, the grout will protrude out of the grout tube into the joint and seal it.

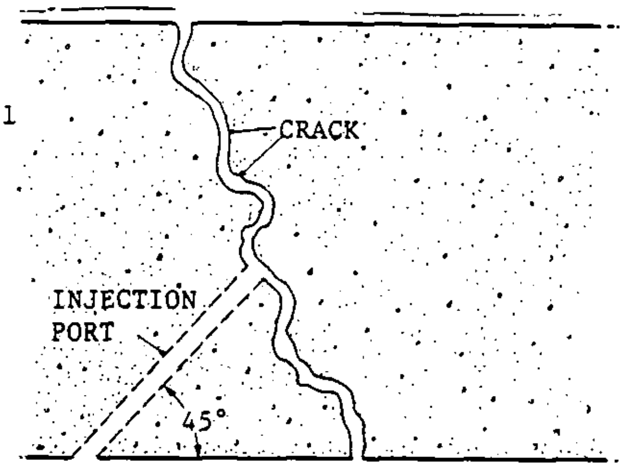
2. If the face is not accessible, 3 or 6 inch (7.5 or 15 cm) holes are drilled vertically over the total length of the joint. The hole is filled with FLEX 44 flexible grout to form a new flexible waterstop. Because of the expansion of the FLEX grout, the total joint and fissures will be filled with the grout.

Sealing leaks in horizontal joints. For horizontal joints or cracks, it will be necessary to drill 1/2 inch holes under a 45° angle from the sides of the joint intercepted by the drill hole deeper in the structure. An injection port is installed in the 1/2 inch hole (See Figure 2). The outside of the crack or joint is covered with hydraulic cement. TACSS 020 NF or FLEX 44 grout is injected into the cracks through the drill hole. The polyurethane grouts will react, expand and seal the crack.

Curtain Grouting. TACSS 025 NF polyurethane grout is mainly used for curtain grouting, because of its low viscosity and high strength after solidification.

The polyurethane grouts can be injected through strainer pipes or through Tube-A-Manchettes. Injection pipes are generally spaced between 1.5 to 3 feet apart.

Figure 2. Technique for installing sealant in horizontal joints or cracks.



Preventing seepage in new construction. INJECTO tubes are installed between the different lifts of concrete (instead of a waterstop). After the concrete cures and a construction joint occurs, the tubes are injected with the polyurethane grouts. The grouts will penetrate into the construction joint and form a permanent waterproofing of the joint.

This system can be applied for vertical, horizontal or circular construction joints in concrete or RCC dams.

BNCOLD CONFERENCE 1988

THE GROWING ACCEPTANCE OF RCC FOR DAM CONSTRUCTION

by

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THE GROWING ACCEPTANCE OF RCC FOR DAM CONSTRUCTION

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SYNOPSIS

Roller-compacted concrete (RCC) has very rapidly become an accepted material for use in dam construction. It has been proven that *in-situ* properties equivalent to those found in conventional concrete dams can be achieved. A number of high dams (up to 100 m high) have already been satisfactorily completed and very high dams (over 150 m high) are now being designed containing RCC. A further area of development for RCC is the rehabilitation of existing structures. It may be that this will prove to be a growth area, equivalent if not greater, than the use of RCC in dam construction itself.

INTRODUCTION

1. Since the construction of the first RCC dams in the early 1980s there has been a very rapid growth in the use of this method of construction and it is growing in acceptance throughout the world. However concern has been expressed in certain quarters regarding the performance of some of the early RCC dams(1). RCC dams have been likened to anything from a concrete-faced cement-stabilized rockfill dam(2) to a true concrete dam. A consensus is now appearing and RCC is becoming to be considered a true replacement of the conventional immersion-vibrated concrete dam.

2. Conventional concrete dams have an enviable reputation for safety. Dams of this type have resisted overtopping well beyond their original design criterion, they do not suffer from internal erosion and have been subjected to significant seismic activity without deterioration. It is suggested that unless RCC dams can be built with a performance as least as good as that of a conventional concrete gravity dam, the full potential of the method of construction may not be realized. The *in-situ* testing of recent RCC dams has shown that such a performance can be achieved with a very significant cost saving compared with conventional concrete dams and, given a suitable foundation, also at a lower cost than an equivalent fill structure .

METHODS OF CONSTRUCTION

3. During the early development of RCC dams, three different design philosophies emerged. These have been classified[3] as follows:-

a. The lean RCC dam (sometimes known as the DLC (dry lean concrete) dam in the UK[4]); the interior concrete used in this form of dam has a cementitious (i.e. Portland cement and pozzolan) content of less than 100 kg/m^3, of which up to 40% can be pozzolan (usually low-lime flyash). It is placed in layers with a thickness of approximately 300 mm. Some of these dams have contained aggregates with a high proportion of fines. Examples are Willow Creek[5], Grindstone Canyon and Monksville[6] dams.

b. The RCD (rolled-concrete dam) method; this method has been developed in Japan and is rather different from that used elsewhere in the world. The concrete is placed in thick lifts (700 to 1000 mm) and joints are cut from the upstream to the downstream side of the dam. The interior roller-compacted concrete is protected by a skin (2 to 3-m wide) of conventional immersion-vibrated concrete and a comprehensive system of waterstops and drainage is used at each joint. The cementitious content of the concrete used in this form of dam is between 120 and 130 kg/m^3 of which 20 to 35% is flyash. Typical examples are Shimajigawa and Tamagawa dams[7].

c. The high-paste content RCC dam; the concrete in this form of dam is designed to have a very low air voids and good bond between layers without treatment of the surface of each lift. The concrete is placed in thin layers (usually approximately 300 mm) and the cementitious content is usually in excess of 150 kg/m^3, but a high proportion of this, generally 70 to 80%, is pozzolan. The Portland cement content is thus low. An example of a high paste-content RCC dam is Upper Stillwater[8].

4. There is some overlapping of the various approaches but each has a slightly different philosophy towards the design of the dam. The lean RCC dam uses an upstream watertight membrane to protect the lean roller-compacted interior concrete. This membrane can either be an immersion-vibrated concrete facing (up to 500 mm wide) placed at the same time as the interior concrete and cast against conventional formwork, or precast concrete panels with or without an attached PVC membrane. Bedding mixes are frequently placed between each lift near the upstream face in order to try to improve the bond and reduce seepage between the layers of RCC.

5. The RCD method is a modification of the Alpe Gera method of construction developed in Italy in the early 1960s[9]. The final structure is very similar to a conventional concrete gravity dam with separate monoliths although these are post-formed by cutting the joints as opposed to pre-formed with formwork. The method is somewhat faster than the conventional concrete dam, approximately 10 to 15%, but is rather labour intensive.

6. Behind the philosophy of the high-paste content RCC dam is the acceptance that an unreinforced facing concrete laid without joints will almost certainly crack due to changes in air temperature(10). Therefore the centre of the dam - that is the roller-compacted interior concrete - is considered to be the watertight barrier and the outside face is purely a durable skin protecting and insulating the interior concrete. Thus the RCC has to be designed to bond layer to layer and to have an *in-situ* permeability equivalent to that of a conventional concrete dam.

7. Despite the different concepts, in recent RCC dams there has been a move towards the middle ground with the cementitious contents rather higher than those of the early lean RCC dams and a further classification is now required - the medium-paste RCC dam. Such a dam has a cementitious content of between 100 and 150 kg/m^3.

8. A further trend has become noticeable; that is the increasing workability of the roller-compacted concrete. Whereas the early lean RCC had no measurable workability (both the concretes used in the RCD method and the high-paste content RCC had a workability equivalent to a loaded VeBe of some 25 to 30 sec), the majority of the latest RCCs now being used in dams (of all four classifications) have a workability which is easily measurable (equivalent to a loaded VeBe of 10 to 20 sec).

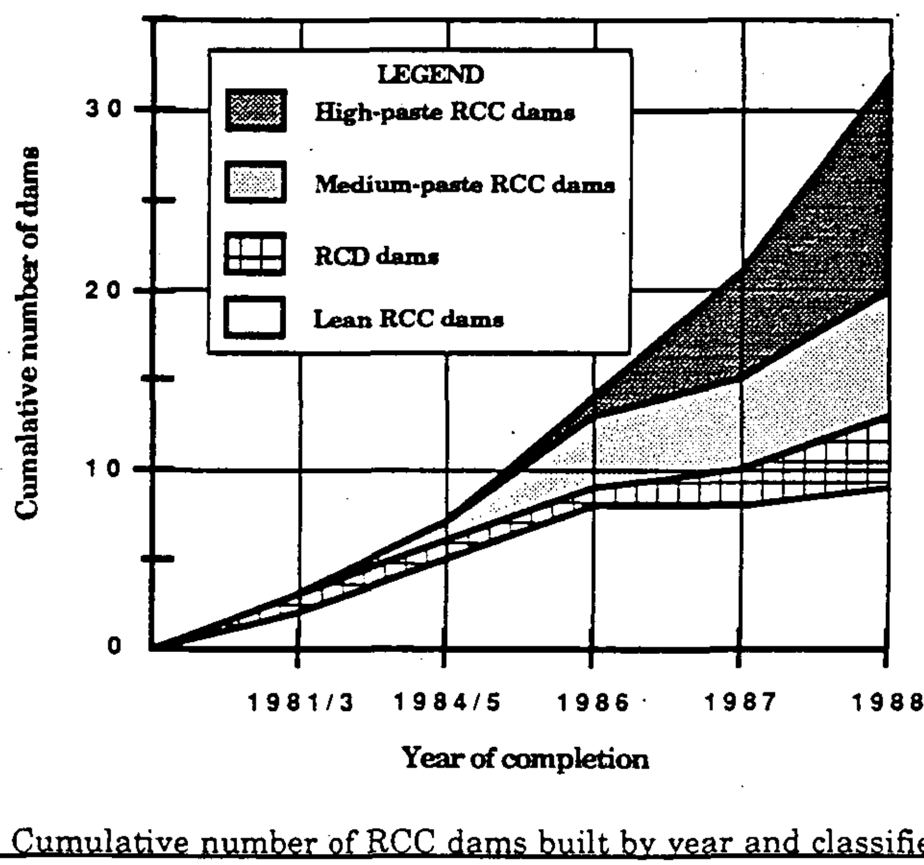

Figure 1: Cumulative number of RCC dams built by year and classified by type

9. The move away from the lean roller-compacted concrete used in the early RCC dams, which did not have an entirely satisfactory performance, is shown in Figure 1. This shows the cumulative number of RCC dams

constructed during the 1980s together with the classification of those dams. Whereas up to the end of 1986, eight out of the 14 dams completed were classified as lean RCC dams and only one a high-paste content RCC dam, in the past 20 months (up to August 1988), only one out of 18 completed dams was a lean RCC dam whereas 11 contained high-paste content RCC.

10. A contributory reason for the increasing paste of the RCC, and thus improved performance, has been the increasing size of the RCC dams being built. Up to the end of 1986 only Shimajigawa in Japan, using the RCD method, had a height greater than 60 m, whereas since that time nine (i.e. 50%) are in excess of this height and two are over 90 m in height. At the present time RCC is being considered for a number of dams in excess of 150 m in height, including for arch-gravity and arch dams.

IN-SITU PROPERTIES OF ROLLER-COMPACTED CONCRETE AS USED IN DAMS

11. With the wide range of approaches to the method of construction and the wide range of cementitious contents used in RCC dams, there has been an equally wide range of *in-situ* properties. It is the properties as found from tests on cores rather than the properties from tests on laboratory specimens that are significant. Probably the most important property for a concrete to be used in a dam is permeability.

Permeability

12. As well as being one of the most important properties, the *in-situ* permeability, is the property which has caused greatest concern to designers of RCC dams. Although the permeability of the parent (unjointed) material may be low, it is the joint between the layers that are the main cause of the difficulty. For example at Willow Creek, water appeared on the downstream face of the dam 12 hours after impounding(11). It is the permeability at the joints that has led to some commentators to suggest that it should be utilized in order to reduce the potential for uplift in a dam when combined with a conventional concrete impermeable upstream face(2). Nevertheless it has been shown that it is possible to obtain an effectively monolithic and impermeable structure when roller-compacted concrete is placed in layers. Figure 2 shows the results of *in-situ* permeability testing on 24 different water-retaining RCC structures from 11 different countries. This is a development of a relationship that was postulated during the expert summary to Q.57 of the Lausanne ICOLD Congress during which roller compaction of concrete for dams was discussed(12). Total *in-situ* permeabilities (including that at joints) ranging from 10^{-4} to 10^{-13} m/s have been found and there is a consistent relationship between the permeability and the cementitious content.

13. The structures, tests of which are shown in Figure 2, were constructed in a number of different ways and the permeability results were obtained by a wide range of methods; some tests were by measurement of the total flow of water through the dam, some by tests on core holes in the structure and others on the cores themselves. With some of the structures, full treatment of the surface of each layer was carried out and full bedding mixes used, with some

there was little treatment and a partial bedding mix while with some others there was no treatment and no bedding mix. It is difficult to differentiate between the various method of construction and the various methods of test from the data in Figure 2. It seems that the overriding consideration is the cementitious content of the roller-compacted concrete.

14. Although the use of bedding mixes may not have a significant effect upon the overall permeability, this does not mean that a bedding mix should never be used. There will be cases where a layer has been left for a long time (for example after a winter close-down) where it may be advantageous to fully treat the surface of the layer and use a bedding mix, or more likely to increase the cementitious content of the whole of the lift to be placed on the old layer.

15. It is of interest to compare the results in Figure 2 with those of conventional concrete dams. Figure 3 shows the same relationship for the latter and Figure 4 shows a comparison of the two relationships. It can be seen that below a cementitious content of approximately 150 kg/m^3 there is a significant difference between the permeability of the two forms of concrete. It seems likely that this difference is a result of the increased permeability of the joints. Nevertheless it is clear that RCC can be, and has been, designed and placed so that the overall *in-situ* permeability is at least as good as that of an immersion-vibrated concrete as used in conventional dams.

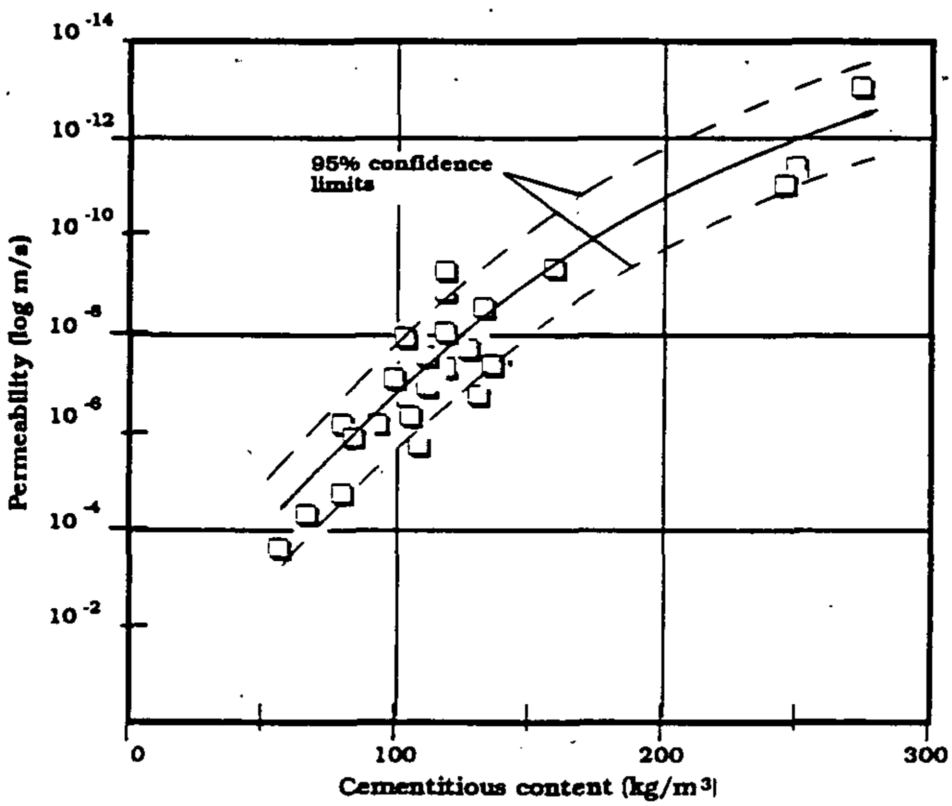

Figure 2: Relationship between *in-situ* permeability and cementitious content for concretes from RCC structures

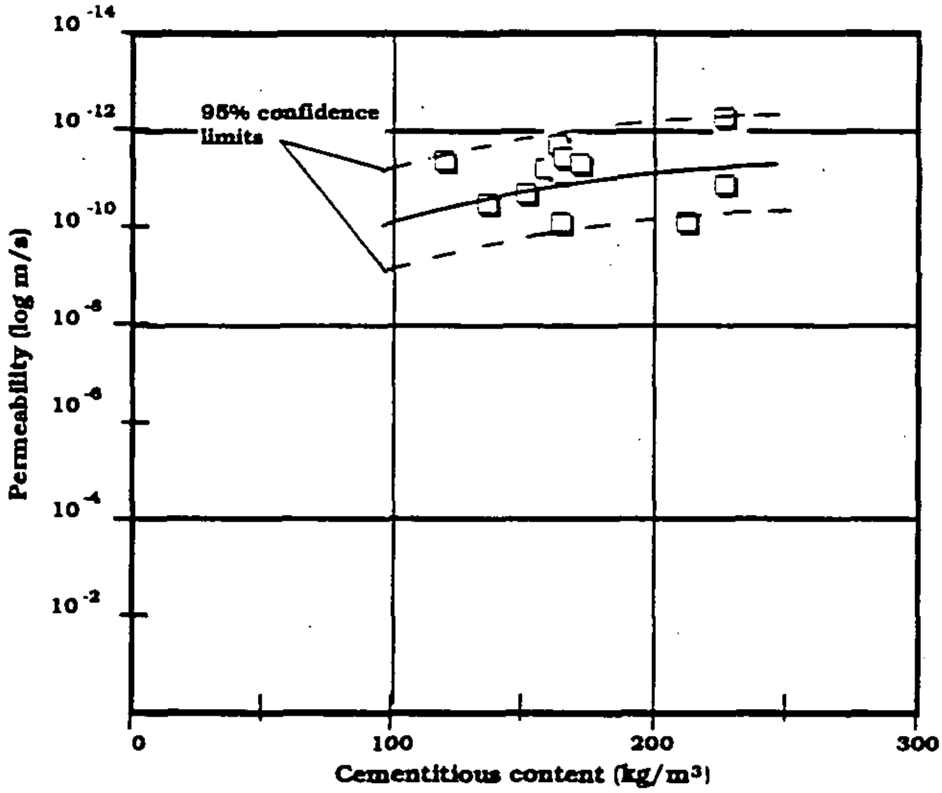

Figure 3: Relationship between *in-situ* permeability and cementitious content for concretes from conventional dams

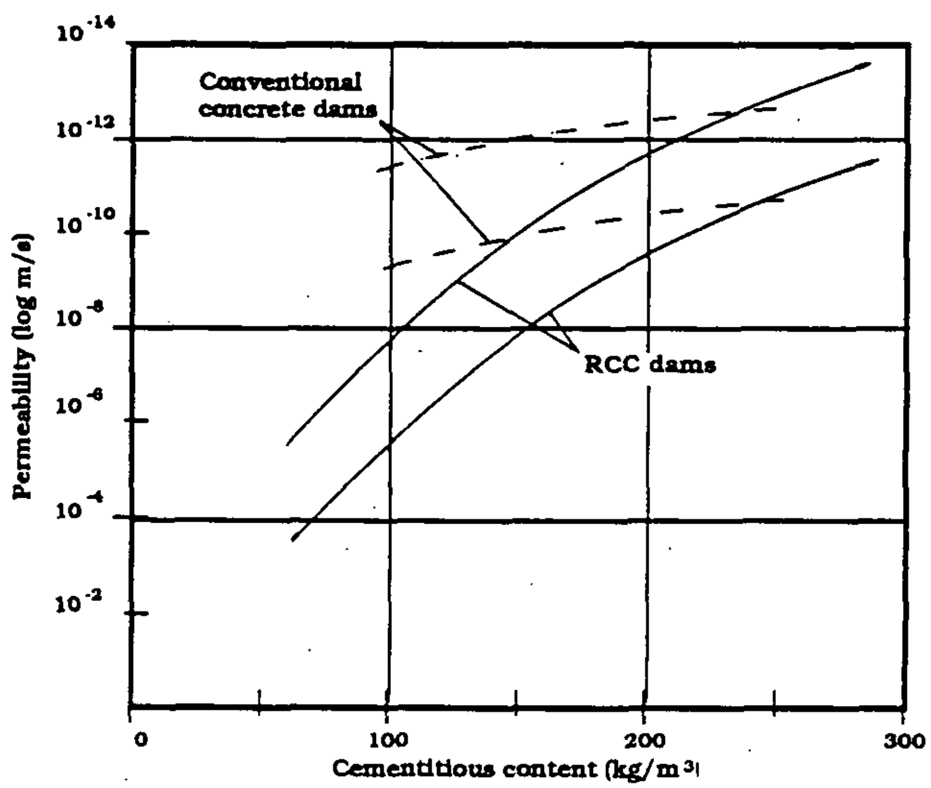

Figure 4: Comparison between the *in-situ* permeabilities of concretes from RCC structures and conventional concrete dams

Summary of properties

16. As roller-compacted concretes with a wide range of mixture proportions have been used in dams, a whole range of properties are available. It has been found that by whatever method the mixture proportions are designed, using concrete technology or soils technology, the concretes conform to the same set of relationships(12). Table 1 summarizes the properties achieved to date. It has been suggested that RCC dams over 200 m in height could be constructed using concretes that have properties that have already been tested and proven in the field(13). The Engineer thus has a whole range of properties from which to choose. He can make a judgment between concretes with different properties leading to different cross-sections of dam and thus optimise the design of the dam.

Table 1: Summary of *in-situ* properties (at an age of 91 days) found during testing of RCC structures

Property	Unit	Lean RCC	MPC RCC(1)	RCD	HPC RCC(2)
Density	%taf	95.0 to 98.0	96.0 to 98.5	97.0 to 98.5	98.5 to 99.5
Permeability	m/s	10^{-4} to 10^{-8}	10^{-6} to 10^{-9}	10^{-7} to 10^{-9}	10^{-9} to 10^{-13}
Direct tensile strength	MPa	0.5±	-	-	1.4 to 2.1
(at joints)	MPa	0.2±	-	-	1.4 to 1.7
Indirect tensile strength	MPa	1.0 to 1.3	1.3 to 1.8	1.4 to 1.7	2.0+
(at joints)	MPa	-	0.3 to 0.8	0.4 to 0.7	0.9+
Shear strength (cohesion at joints)	MPa	0.5 to 1.0	1.0 to 2.0	2.5 to 3.0	2.2+
Compressive strength	MPa	8 to 10	12 to 20	12 to 16	20 to 40

Note: 1. Medium-paste content RCC 2. High-paste content RCC

RCC AS A MATERIAL FOR RENOVATION OF DAMS

17. In spite of the rapid growth of RCC for dam construction itself, an even greater growth may already be occurring for renovation of dam structures. One of the first major uses of RCC, and still the largest volume placed on a dam site, was at Tarbella in Pakistan. Between 1975 and 1982 several million cubic metres of RCC were placed to rebuild an outlet tunnel and to protect the service and auxiliary spillways. This was a special case and renovation of dams with RCC can generally be classified under three headings: repair of damage to overflow structures, protection of embankment dams against overtopping and strengthening of existing structures which now have

unacceptably low factors of safety (either due to deterioration of the structure or due to reconsideration of the design criteria).

Repair of damaged overflow structures

18. An example of the use of RCC to repair a dam overflow structure was Kerrville Ponding Dam. This dam is 7 m high and 180 m long and was designed as a concrete-capped clay embankment which could be overtopped. A 90-m section was left low to act as a service spillway. In 1984 the dam was overtopped by some 3 m of water and the downstream spillway section of the dam was badly damaged.

19. To repair the structure an RCC section was built downstream of the dam using the old dam as an upstream form(14). A month after completion of the repairs, some 300 mm of rainfall fell in a short period and the dam was overtopped by 4.5-m depth of water for between four and five days. No damage occurred to the RCC.

Protection of overflow structures

20. One of the early examples of the use of RCC in dam construction was the rehabilitation of Ocoee No.2 Dam by the Tennessee Valley Authority in 1980(15). The initial rehabilitation design consisted of a riprap berm placed on the downstream side of the 10-m high rock-filled timber-crib dam. After four washouts of the riprap from flash floods during the reconstruction a change of design was considered to be necessary. RCC, which was mixed in a ready-mix plant some 28 miles from the site, was placed in stair steps over the riprap on the downstream side. Since completion of the renovation the dam has been periodically overtopped with no damage noticeable.

21. One of the most recent uses of RCC for this form of protection of an existing fill structure are the Addicks & Barker Dams near Houston. These are very long flood-prevention earthfill structures. RCC has been placed on top of each dam and down the downstream side. The concrete is placed by paver and then compacted by vibratory roller. Some 32 hectares of RCC is being placed.

22. A CIRIA report(16) has been prepared describing, amongst other things, the potential use of RCC to protect old embankment structures against potential overtopping.

Strengthening of existing dams

23. One of the more interesting uses of RCC will be the strengthening and raising of Gibralter arch dam with RCC. This dam was built in 1921 and has a maximum height of nearly 60 m. The width of the dam varies from 20 m at the base to just over 2 m at the crest. A finite-element analysis of the structure under probable maximum flood (PMF) and maximum credible earthquake (MCE) conditions indicated that the stability of the dam may be questionable. Three methods of rehabilitation of the dam were considered; placing a reinforced rockfill section on the downstream side against the dam, the

addition of a steel-fibre reinforced shotcrete 'blanket' on the upstream and downstream faces near the top of the dam and the addition of an RCC gravity section against the downstream face of the dam. Of these methods, the RCC repair was considered to be the best approach(17).

24. A further feasibility was carried out to assess the possibility of also raising the dam. After studying a number of alternatives it was decided to repair the dam as a first phase, the design of which would allow for raising at a later date.

CONCLUSIONS

25. Roller compaction of concrete is now an accepted method of construction for dams. The initial experimentation has come to an end and although the method of construction will continue to develop, there is now sufficient data available on which to base engineering decisions. Large dams are under consideration - several dams over 150 m in height are being investigated. With dams of this height, the properties will have to be at least as good as that of conventional concrete dams, the permeability will have to be 10^{-11} m/s or better and the shear strength will necessitate relatively high cementitous contents or full treatment of each lift surface and a bedding mix.

26. In order that RCC can be considered for dams of all reasonable heights, and become a true replacement for the conventional monolith method of construction (and for arch-gravity and even arch dams), the properties of the interior concrete will have to be improved above the level of some of the lean RCC dams constructed in the early 1980s. It has been shown that this is possible and the cost is no greater than that of a larger cross-section of cheaper material.

27. The RCD method is a potential solution but the advantage of the speed of construction possible with the roller compaction of concrete is partially lost and the simplicity of the method of construction is certainly lost. It is suggested that by simulating the conventional method of construction, the RCD method loses some of the advantages obtainable with other forms of RCC.

28. Roller compaction of concrete for dams is a new method of construction but it is not a solution for all sites. With a good foundation, an RCC dam will frequently be the most cost-effective solution, and the completed dam should have a quality at least as good as that of a conventional concrete dam.

29. A potentially greater use of RCC for dams, particularly in the UK, could be the use of the material to repair damaged structures, to protect embankment dams against overtopping and to strengthen structures which no longer conform to their original (or new) design criteria.

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BNCOLD CONFERENCE 1988

USE OF FLEXIBLE REVETMENTS IN DAMS AND RESERVOIRS

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USE OF FLEXIBLE REVETMENTS IN DAMS AND RESERVOIRS

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SYNOPSIS

This Paper reviews early applications of the use of a flexible revetment system on reservoir embankments in the light of the recently published CIRIA Report (1) on reinforced grassed waterways and the Author's own experience. Two embankment protection schemes are described and are shown to fall within the general recommendations made by CIRIA. In addition the underlayer design is also discussed.

INTRODUCTION

1. Flexible armoured revetment systems comprise those systems with discreet concrete armour units geometrically and physically connected together to form a continuous primary protection system. It is possible to catagorise these systems in the following way:
- dual cabled interlocking system (Petraflex)
- single cabled interlocking block system (Armorflex, Dycel, Terrafix)
- interlocking block systems (Tri-Lock, Armorlock)

All these systems are produced using modern manufacturing methods providing for economic unit components and a high degree of quality control. The use of the semi-dry mix for this type of product results in an accelerated attainment of the concrete design strength over the more traditional wet mix product methods enabling supply and installation to be carried out within a matter of days of production. In addition a wide range of concrete strengths can be catered for together with defined limits of material type and content, water absorption, product strength and freeze thaw.

2. It has to be stated that generally the Consulting Engineer or Design Engineer working on behalf of the client pays little attention to detail in regard to the definition of the specification for these types of systems, indeed quite often only a system weight is specified. This can, and does, lead to situations where products may be used on projects which have not been tried and tested in the manner described in the CIRIA Report (1). This, of course, has provided a framework of formulating a design incorporating flexible armoured revetment systems for use as a reinforcement for increasing the erosion resistance of a grass surface subjected to overtopping and fast flows. It contains many useful recommendations with regard to the design and practical experience with examples of many projects incorporating the above mentioned products. The next section describes two projects where flexible revetment systems have been used in reservoir engineering.

EARLY APPLICATIONS

Stanford Reservoir

3. Possibly the earliest application for the use of a flexible armoured revetment system used in a dam project is that at Stanford Reservoir described extensively by Mackey (2). This project was conceived prior to the reservoir programme culminating in the CIRIA Report (1) mentioned earlier.

4. Stanford Reservoir was constructed in 1928 and comprises a 1:3 pitched upstream face, a 5m wide crest and a 1:2½ grassed downstream slope. A full account of the engineering operations identified by Severn-Trent Water Authority as a result of an intensive review of the flood hydrology and dam hydraulics is contained in Mackey (2). Suffice it to say, the concept of allowing the dam to overtop under PMF evolved, and also methods of preserving the dam intact whilst at the same time accepting some damage to the structure could take place albeit insufficient to cause a breach. In addition, in the event PMF did take place such a flood would be of unprecendented magnitude, a factor perhaps in the final choice of the reinforcement system.

5. At that time the only conclusive document on grass reinforcement was the CIRIA publication "A Guide to the Use of Grass in Hydraulic Engineering Practice" (6) and reference to that provided the following engineering objectives:
- engineering specification and geotechnical criteria of the site
- design life of 50 years
- easily maintainable
- aesthetically and environmentally acceptable
- cost effective

6. A number of different types of systems were then considered, some of them very recent additions to the reinforced grass concept. These included:
- polypropylene nets
- flexible grid slabs
- insitu laid perforated concrete slabs
- interlocking concrete blocks
- mulched or uprated grass slopes

7. Of prime consideration was the effectiveness of these systems at all times. Stanford Reservoir had existed for 60 years and having an established vegetative cover was less susceptible immediately after provision of the reinforcement armour surface than, say, a new reservoir embankment freshly seeded. Consequently, because it was judged that the interlocking concrete block system was less susceptible to drought or other seasonal factors the system chosen in preference to others was the Petraflex revetment system.

8. The particular technical design requirements for the system were:
- inundation to a depth of 300mm at the crest
- a terminal velocity of 6m/sec 2.5m down from the crest
- capable of withstanding a 0.6m wave height on the upstream face
- flexibility to accommodate minor settlements
- high durability

Design for Fast Flows

9. Mackey (2) details the results of conventional boundary layer theory resulting in critical depths of 0.19m with flows of $0.25m^3$/sec/m, and velocities ranging from 2.6m/sec - 6.9m/sec depending upon Mannings 'n' for grass or concrete being 0.03 - 0.08. Clearly if the slope is never maintained a roughness of 0.08 could be possible, however the value of 0.03 was the more appropriate value to take. Ven te Chow (3) states that a roughness of value of 0.08 should be used for an unmaintained channel, reeds and brush uncut, with dense weeds as high as the flow. CIRIA (1) concluded that the roughness value for estimation of velocities should be 0.02 with established grass at less than 50mm in height.

10. Applying this value to the equations results in velocities in excess of 10.0m/sec. Indeed, CIRIA (1) advises being conservative on the choice of Mannings 'n'. If one assumes the manufacturer's Mannings roughness of 0.025 for Petraflex, arrived at purely on the basis of the roughness height being approximately 0.05m, (half the depth of the blocks) then a velocity of 8.2m/sec is calculated. Continuing the CIRIA design procedure then such a velocity would require the use of a concrete block system with good interblock restraint. Restraint from the chosen system is provided by:
- the individual blocks intermeshing
- positive interpanel connection between panels
- two-way cable system for anchoring

which are particular to the Petraflex system. The system chosen for this project was that which would be within the recommendations of CIRIA (1) that is, system weight $150Kg/m^2$, individual block weight $14Kg/m^2$ with depth of block 100mm.

Wave Attack Design

11. As far as wave design condition is concerned Petraflex is designed according to the following formula:

$$\frac{H}{Sd} = k$$

where H = design wave height
S = relative density
d = block depth
k = co-efficient

From full scale tests on the Petraflex system k is usually taken to be 6 for a slope of 1:3. For steeper slopes k is reduced normally to a minimum of k = 5. This means that for a design wave height of 0.6m the required minimum depth of block is 92mm. When designing for wave conditions it is adviseable to consult the specific full scale tests for cabled systems on which the design recommendations are based. It is not the case that all cabled systems perform to the same degree, indeed non-dimensionalising the test wave data results show the dual cable system out performing other systems. In addition recent American full scale tests (4) involving performance of flexible revetment systems subjected to fast flows have also provided a wide variation of performance with the dual cabled Petraflex system easily out performing other systems.

Underlayer Design

12. At Stanford it was decided not to install a membrane or filter layer beneath the protection system and installation of the system took place directly on the existing grassed embankment. The reason given for this was that a membrane could inhibit root growth resulting in reduced frictional resistance to sliding at the armour-membrane interface. Certainly from an aesthetic view point the grass at Stanford has hidden the blocks exceedingly well, but it is debateable that without the soil in the cells of the blocks the roots themselves would provide any such resistance. The CIRIA Report (1) includes insitu tests of root pull-out resistance and shear strength which are performed on blocks whose cells have been seeded and filled with top soil. Of course, it is a matter of months before the root structure can be relied upon to provide such additional stability, by which time the flood condition could have taken place. In addition the root structure would not be able to provide an adequate structural connection with the main embankment fill material if a drainage layer was incorporated in the design. Consequently, if the Design Engineer is relying on the root structure for a stable system then the use of a drainage layer is automatically discounted. The CIRIA Report (1) also refers to other methods of increasing the shear resistance at these interfaces including the installation of shear pins and the inclusion between the block sides of particles of grit. It is the Author's view that such methods, whilst well proven in scaled laboratory experiments and by nature simplistic, are, in reality almost impossible to predict whether such resistance will be within the system in the long term. The CIRIA Report (1) described the use of pins of 25mm diameter, 500mm long at 1m centres providing a shear resistance of 1.5KN/m^2. Given that an embankment could be in the region of 10,000m^2 this represents over 20,000 pins would be installed. It would be more practical to consider increasing the weights of the armour system rather than adopting the use of such pins.

13. At Stanford the use of anchors was adopted namely 1.4m long x 14mm square sections of galvanised mild steel. The reason was because it was felt the high strength polyester cables to be susceptible to creep. Such anchors were placed to provide an additional surcharge of over 4KN/m^2.

14. No consideration was given to uplift created on the system by the flow of water. Indeed the CIRIA Report (1) does not consider this either, as a basic assumption is made that fluctuations in pressure at the boundaries are neglected. Of course, in an ideal situation with a perfectly laid system this would probably be acceptable, but again this assumption is considered inadequate. Einstein and El-Sami (5) conducted flume experiments in which turbulent flow was established and concluded that the average lift force (P) is related to a reference velocity (U) at a specific distance (proportional to the roughness of the bed) above the bed. The Prantle-von-Karman velocity distribution is also assumed, the expression obtained is in the form:

$$P = C\left(\frac{U^2}{2}\right)p$$

where P = average uplift pressure
C = lift co-efficient
U = relative velocity
p = density of water

15. Using the same lift co-efficient (0.178) established by Einstein and El-Sami (5) and applying data obtained from Mackey (2), that is,

$d = 0.19m$
$v = 8.2m/sec$
$n = 0.025$

the average lift force is calculated to be $1416N/m^2$, which is considerably in excess of the submerged weight of the installed system at $858N/m^2$. However, given the fact that additional anchors were installed (for reasons associated with the alleged susceptibility of the polyester cables to creep) providing an estimated $4,000N/m^2$ additional surcharge, then the integrity of the design can be seen to be maintained.

16. At present there is no definitive publication on the relationship between drag and uplift forces due to fast flows, and the beneficial, or otherwise, effects on providing a drainage layer underneath the primary protection. Indeed recent American trials (4) on fast flows have not provided clarification on this as different primary armour systems with identical underlayers, that is, with and without a definitive drainage layer, have performed equally as well (or badly). This leads the Author to believe that performance of the total system is contained within the primary armour system itself. It will, therefore, come as no surprise that the dual cable system, stack bonded, sustained a 4ft head ($2.8m^3/sec$) of overtopping for a 30 hour period without failure, whilst a single cable system, stretcher bonded, could sustain only 1ft of overtopping ($0.7m^3/sec$) for a period of less than 1 hour. The estimated velocities of flow down the 1:2 slope were 2.1m/sec and 4.9m/sec for the condition of 1ft and 4ft overtopping depth respectively. These velocities were well within the limits imposed by CIRIA (1) for the use of a flexible concrete protection system, however, two of these systems failed at these values. The publication of this work is awaited with interest.

Furzton Reservoir

17. Following the general principle outlined above the recently installed flexible revetment system at Furzton Reservoir in Milton Keynes was designed to withstand the following:
- maximum velocity 7.5m/sec
- design wave height 0.48m

the flood discharge at ½PMF is estimated at $83m^3/sec$ over a 50.0m wide spillway. The system chosen was the Petraflex H41212i system providing a surcharge of $160Kg/m^2$ and was installed in panels of 5.3m length x 2.44m wide placed on top of a UV stablised woven geotextile with the following specification:

$O90 = 270$ microns
$k = 4 \times 10^{-4} m/sec$
Warp strength = 48KN/m
Weft strength = 25KN/m

Each panel was designed to be stable in its own right and 4 No. 40mm tie cabled anchors of length 600mm were used to anchor the main longitudinal cables of 1.6t breaking strain. It is interesting to note that both the primary armour system and the geotextile comply with those recommendations in the CIRIA Report (1). The specification of the geotextile is, of course, very important, permeability and sand tightness criteria being of a high priority, however, the thickness of the fabric is also being indentified as a priority parameter particularly where the so called "problem soil" is being filtered. Such a soil is identified by the proportion of the grading which is less than 0.06mm. It is argued that only a thick needle punched non-woven fabric can provide the necessary long term stability for filtration performance where a problem soil is encountered.

Acknowledgments

The permission of Anglian Water Authority for the use of information on Furzton Reservoir.

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DISCUSSION: TECHNICAL SESSION 8

NEW MATERIALS FOR THE RENOVATION OF DAMS AND RESERVOIRS

Session Chairman: Mr A I B MOFFAT (University of Newcastle-upon-Tyne)

CHAIRMAN A I B MOFFAT (Senior Lecturer, University of Newcastle-upon-Tyne)

This is I think, a particularly interesting session in the sense of being truly international. I am very pleased to welcome on your behalf the representatives from Italy, from Switzerland and from the United States. Dare I say it may be symptomatic of our approach to new materials in this country, that Malcolm Dunstan and Charles Tuxford are left to wave the flag for new materials in a UK context.

The first paper is being presented by Dr Paolina who led a working group of the Italian National Committee. The second paper is by Messrs Sembenelli and Cumbeti. Mr Kriekemans from the United States presents the third paper. The fourth paper will be presented by Dr Dunstan whilst Charles Tuxford will give a paper on the use of flexible revetments to finish the Session.

G A MILNE (Crouch & Hogg)

I have a comment in connection with Paper 8.1. I was very interested to note the Italian experience with synethec resins and wished to draw attention to the ICOLD Bulletin Number 43 on the use of synthetic resins for facings of dams which was prepared by the British Committee under the chairmanship of a member of the Italian Committee. A number of other Italian examples are included in that Bulletin together with many from other member Countries of ICOLD. That Bulletin may also have a bearing on the work at Haweswater discussed in Technical Note 3. American experience, which was reported, stressed that it was essential in freeze/thaw situations to identify the basic cause of dampness before applying a relatively thin surface repair coating. I think that caveat has a fairly general application: you treat the disease and not the symptom.

J HAY (Rofe Kennard & Lapworth)

Could I ask a question of Mr Tuxford? When we are thinking of flexible revetments in dams it seems to me we are thinking of fairly longterm provision. I would like to ask him what the significance of the cables are in the performance of these revetments, because I feel these galvanised steel or mild steel fixings are of limited life.

C TUXFORD (Ardon International)

Different systems provide for different types of cables within their system. Some have galvanised steel, others have polyester cables with a very tightly braided covering on the core. The longterm strength is important for the long term durability of the cable. It is impossible to precisely predict the strength of the cable after 50 years but given that the polyester cables have been used in marine environments for some considerable time now, the best that can be said is that after 50 years of life some 40% of strength of the cable has been lost.

H HEWLETT (Rofe Kennard & Lapworth)

Mr Tuxford suggests that no consideration was given to uplift created by the systems in the CIRIA report. Consideration was given to these forces and the recommendations gave velocities low enough and block weights high enough such that uplift was considered to be no problem, particularly when the anchorage given by the grass roots is considered.

C TUXFORD (Ardon International)

I am particularly interested in the system performing on day one. There is no grass root structure on day one and it is important that the system does perform at that time. I cannot find any details in the CIRIA report that relate to establishing the uplift generated on a rough system. If you make the assumption that you have a smooth primary system on your slope then you also discount the need to quantity what potential uplift there might be. If you make the assumption there is roughness then clearly there is a need to look at uplift.

A D H CAMPBELL (Fairhurst & Partners)

Would Mr Kriekemans please comment on the use of polyurethane grouts in soils with the object of decreasing permeability.

B KRIEKEMANS (De Neef Inc)

Polyurethanes have been used in soils primarily for strengthening but also for curtain grouting. Obviously the soil has to be permeable to the polyurethane: other chemical grouts tend to penetrate better than the polyurethanes and consequently they are mainly used for soil stabilisation.

A D H CAMPBELL (Fairhurst & Partners)

What sort of viscosity does the polyurethane have?

MR KRIEKEMANS (De Neef Inc)

There is one polyurethane on the market that has a viscosity of 25 poises at 70° F.

J B BOWCOCK (Sir Alexander Gibb & Partners)

I have a question for Dr Dunstan. I was very impressed by the photographs of Upper Stillwater and particularly by the quality of the rolcrete in the core. Could he say something about the extent to which the high paste content is now being picked up by other design organisations? In the paper it would appear that the tendency in the USA is towards the lean RCC dam but I would be very interested to know if the high paste method is being used by others.

DR M. DUNSTAN (M Dunstan Associates)

In the paper I give a graph which shows that up to the end of 1986 fourteen RCC dams had been completed. Of these only one was a high paste content dam in China, one was an RCD in Japan, and eleven were lean RCC. Since that time 18 RCC dams have been completed; of these only one has been a lean RCC and eleven have high paste content. Every country that has built more than one RCC dam have now switched to high past content and generally the cementitious content varies from 150 to 240 kg/m^3.

In cores at an age of about a year, the cohesion varied between 3 and 4 MPa, the direct tensile strength varied between 2 and 2½ MPa, and the compressive strengths were about 40 MPa from a cement content of 70kg/m^3 with a fly ash content of about 170 kg/m^3.

DR D J COATS (Babtie Shaw & Morton)

What use is made of the tensile strength of the concrete? If use is made of the tensile strength do you make any use of that tensile strength that you might think you get between the concrete and the base rock?

DR M DUNSTAN (M Dunstan Associates)

I believe that the tensile strength is a benefit. As you are aware early lean RCC dams did have a problem of permeability between joints. Efforts have been made to improve permeability. The only way I think you can do this is to have a high paste content which also gives benefits of cohesion and tensile strength. Upper Stillwater was in fact designed with some tension under maximum loading.

A I B MOFFAT (Session Chairman)

Of course one of the ways of overcoming the problem of permeability is to instal drainage.

On another issue we have had two papers from overseas dealing with a protective skin on the facing of dams. This has not been a popular solution in the UK and I would like to ask in particular if the gunite facings that have been used have been successful.

DR R PAOLINA (ENEL)

Our experience of gunite is that it is a useful, economic method at times for dams at high altitude. We have found the life of such reinforced gunite facings to be of the order of 20 years.

F G JOHNSON (North of Scotland Hydro Electric Board)

I listen with great interest about these new plastics etc but we are dealing with structures which are required to endure for many tens of years and desirably many hundreds of years. As dam engineers we are reluctant to put in new materials discovered within the last year or two which really have no longterm proof of their durability. I would like to ask the two speakers from America and Switzerland what durability tests have they had to do with the materials they are advocating and also how long they think the materials will last.

J S CUNIBERTI (Consultant, Switzerland)

With regard to longevity part is obtained by extrapolating laboratory tests which have certain limitations. However, compared with older formulations of PVC we have very low losses of plasticisers which is one of the limiting problems for PVC. The other is ultra violet deterioration: we subject samples to 5-6000 and some up to 10,000 hours of intense ultraviolet radiation, equivalent to 50 years application in temperate climates.

In addition all of this experience is based on many years of PVC application for roofing materials, and relatively new additives have been developed to prevent the polymers from breaking down under ultra violet life. The indications are that we can expect many decades of satisfactory performance. Dams in Italy in mountainous regions have performed well over 25 years with no ice adhesion.

B KRIEKEMANS (De Neef Inc)

The first use of water-activated polyurethane grouts was about 20 years ago, not for dams but in sealing flows in tunnels.

R M ARAH (Binnie & Partners)

About 25 years ago expanded polyurethane grouts were used to seal piezometers into embankments. I have not heard of any problems but by this time piezometers are often in trouble anyway. It would be useful to know if any users are aware of problems.

At that time it was extremely expensive: could Mr Kriekemans give an indication of present cost, perhaps in terms of cost per cubic metre treated?

B KRIEKEMANS (De Neef Inc)

It is quite an expensive material but we consider it is easy to apply and the results of its use are very good. It is used extensively in the USA and is cost effective is sealing leaks compared with chemical grouts and cementation materials.

R M ARAH (Binnie & Partners)

I would be interested in our Italian guests' reaction to the use of RCC. They have these extreme climatic conditions and I wonder if they envisage the use of RCC for their sort of dams on a large scale.

DR R PAOLINA (ENEL)

In Italy we say that the first RCC dam was Italian! At the moment no RCC dams are being designed or are under construction.

H JONES (M J Gleeson Ltd)

A question to Mr Kriekemans. Can your material be used in a seawater environment?

B KRIEKEMANS (De Neef Inc)

Yes - it has been used and will react with sea water. It has been used to seal sea walls where washout has occurred.

T A JOHNSTON (Babtie Shaw & Morton)

A question for Mr Paolina. As I read it the work in Italy has been done to comply with the Italian Code for the design, construction and operation of dams. Has the Code been produced by the Government and has it the force of law?

DR R PAOLINA (ENEL)

Yes. The Italian codes are renewed periodically.

The first edition was 1931; the second 1959, and the latest revision was in 1982/83.

1t should be emphasised that the CIRIA Report does not actually recommend the installation of shear pins or the provision of grit between blocks; these are only discussed as means by which shear resistance and root edging could be improved. The basic recommendations assume that sufficient shear resistance will be provided by grass roots and are based on the results of the field trials at Jackhouse Reservoir rather than laboratory experiments.

It is suggested in para 14 of the paper that the CIRIA Report did not consider the effect of hydrodynamic uplift on the system. The design criteria recommended in the CIRIA Report included sufficiently low velocities and high block weight for uplift not to be a problem. In addition, the presence of grass on the surface will mask the shape effects of the block causing turbulence and uplift. Finally, the grass roots create a composite system with the concrete blocks and the subsoil which reduces the response of the concrete blocks to any hydrodynamic forces.

Comparison is made in para 16 of the paper between some recent American trials and the CIRIA recommendations. There are not strictly comparable since the American tests did not contain any grass. It is not surprising that these failed at velocities less than those recommended by CIRIA for grass-reinforced systems since, as previously explained, without the effects of grass the response of the concrete block system would be different and probably much more dependant on (a) the characteristics of the different proprietary systems and (b) surface irregularities which could attract high localised drag and uplift.

PROCEEDINGS: FINAL SESSION

GENERAL TOPICS

DISCUSSION: FINAL SESSION

Session Chairman: Mr E T Haws (Chairman BNCOLD)

G R CURTIS (Consultant)

I find it particularly interesting that recent studies by the Hydraulics Research Laboratory at Meggat and Glascarnoch reservoirs now appear likely to confirm that wave heights may be underestimated by use of the Saville formula. It follows therefore that use of the recommendations in 'Floods and Reservoir Safety: An Engineering Guide' will also produce this result, at least under certain circumstances.

This underestimation was suspected while working with the North of Scotland Hydro-Electric Board when very comprehensive dam safety and reservoir flood management studies were being undertaken.

Glascarnoch Dam is partly earth embankment and partly concrete gravity structure. On a number of occasions considerable damage has been done to the facing of the embankment. Some of the sloping grouted pitching has turned into rough coarse rip-rap and has proved much more effective in destroying the wave energy.

A particular situation was observed at the vertical face of the concrete part of the dam, during somewhat short duration strong winds of distinctly less than gale or hurricane force. Several photographs were shown at a previous BNCOLD conference by Mr Frank Johnson. The water level was low at the time and yet the occasional wave shot up some 12m and was blown over the top of the dam. Regular waves on this face were about 2.5 to 3.0 metres high. Using Saville, the design wave height was just under 1 metre.

These unusual waves were recognised as the effect of clapotis (sic) or the superimposition of a reflected wave on an oncoming wave causing heights to be very much greater than at the adjacent embankment.

At that time we gave due consideration to adopting a factor of 2 for Hd/Hs whereas Table 2 of the Engineering Guide indicates a maximum of 1.3 and a reduction (not an increase) to 1.0 for vertical faces. I therefore look forward eagerly to the HRL report.

W J CARLYLE (Binnie & Partners)

Mr Curtis: thank you for your contribution and I am sure, like you, we are all waiting for these results to come out. I did try and bring out in my paper, that there is a considerable difference between the proposals in the guide, which are meant to help you to decide on wave run-up for freeboard purposes, and the necessity to arrive at a conservative approach towards the design of revetment or preventing wave damage. Certainly, in our studies, the derivation of a satisfactory wave height is less sensitive for the determination of run-up, because you have got to apply the run-up factor anyway and that isn't terribly well known, and in most cases the freeboard allowance is so great for the extreme floods, like PMFs, that a wave in the normal life of a dam is not likely to encroach on it. One must keep in perspective what the guide is trying to do, which is to lead you to wave run-up and freeboard, rather than slope protection.

MR G ROCKE (Babtie Shaw & Morton)

In his Paper Mr Carlyle reviewed the rather convoluted history of wave prediction methods and concluded that recent wave measurements on Megget and Glascarnoch reservoirs by HRL indicated that Saville's method underpredicts wave heights. This seems contrary to his introductory statement 'theoretical techniques exist to enable wave heights to be predicted with reasonable certainty'. However he is of the opinion that these two reservoirs are not typical and that any conclusions drawn from the HRL work would have only limited applicability to other reservoirs in the UK. The introductory statement itself requires further consideration in my view since I am not convinced that reasonable certainty does exist in the understanding of the interaction of wind and water when confined in valleys and projected onto dam slope protection systems. My reluctance to accept entirely present theories first arose while viewing the severe wind and wave action at Kielder Water in 1983. A desk study was subsequently made in 1984 of characteristics of 14 well known major reservoirs in the northern half of the UK. Some of the related embankment dams were concrete-faced and some were faced with rip rap. The results of the desk study of the dams selected are appended. Elevation, maximum fetch, wind lane width and each reservoirs' coincidence with the prevailing wind were listed.

The reservoirs were then examined under each heading to highlight those most likely to experience severe wind velocity conditions. The listings ranked Glascarnoch, Megget, Orrin, Daer, Turret, Carron in Scotland, Kielder in England and Llyn Celyn in Wales as being candidates for severe wind action. Many other reservoirs could have been so studied. Each of the above reservoirs except Carron are known to have experienced disturbance to their face protection systems, Turret being of a comparatively minor nature due to non-coincidence with prevailing winds.

Such simple desk studies highlight some factors not accounted for in present design methods. Convergence of the margins of a reservoir towards the dam structure may also tend to increase wave heights and this is a fairly common feature in glaciated valleys in Scotland. In addition to the above, at high velocities the wave forms near the dam face are very confused and local surging and plunging are likely to generate higher forces on the slope protection than from conventional wave forms. Further field studies seem essential to isolate such factors.

I would briefly comment on details given in the above Paper in relation to Kielder in paras 23 to 25. The blockwork revetment at Kielder was sized to be heavy flexible and to meet planning conditions.

WIND RELATED FEATURES AT VARIOUS RESERVOIRS

DF/3

DAM		ELEVATION ft	m	MAXIMUM FETCH ft	m	WIND LANE WIDTH ft	m	DIRECTION OF MAXᵐ FETCH	FACE PROTECTION AND THICKNESS (mm)
1. Carron	1935	740	225	10500	3200	2000	610	W	Pitching (Grouted)
2. Daer	1954	1122	342	10600	3220	2100	640	SW	• Concrete Blocks (150)
3. Orrin	1959	840	256	19000	5800	2000	610	W	• Concrete Blocks (300)
4. Selset	1960	1040	317	5600	1700	1640	500	W	• Concrete Slabs (200)
5. Turret	1964	1178	359	10000	3200	1200	370	NW	• Concrete Blocks (375)
6. Derwent	1966	725	221	10500	2900	2950	900	NW	• Concrete Blocks (375)
7. Backwater	1969	970	296	9000	2740	1500	460	N	Concrete Blocks (300)
8. Kielder	1982	607	185	15100	4600	3300	1000	W	• Concrete Blocks (300)
9. Glascarnoch	1957	826	252	22900	6980	1600	500	NW	• Rip Rap
10. Llyn Celyn	1965	975	297	11500	3500	2530	770	W	• Rip Rap
11. Balderhead	1965	1089	332	5250	1600	1640	500	W	Rip Rap
12. Scammonden	1970	825	252	2800	850	1000	300	N	Rip Rap
13. Megget	1982	1095	339	12000	3660	1800	550	NE	• Rip Rap
14. Carsington	1984	654	199	6200	1900	1300	500	N	Rip Rap

• Dams known to have suffered damage to facework

1122 Over 1000 ft altitude   700 500/1,000 ft. altitude   200 0/500 ft. altitude

FETCH RATING

	m	
Glascarnoch	6980	Long
Orrin	5800	Long
Kielder	4600	Long
Megget	3660	Long
Llyn Celyn	3500	Long
Daer	3220	Long
Carron : Turret	3200	Long
Derwent	2900	
Backwater	2740	
Carsington	1900	
Selset	1700	
Balderhead	1600	
Scammonden	850	

FUNNEL RATING = FETCH / WIND LANE

Glascarnoch	14·0	High
Orrin	9·5	High
Turret	8·7	High
Megget	6·7	High
Backwater	6·0	High
Carron	5·3	High
Daer	5·0	High
Kielder : Celyn	4·6	High
Carsington	3·8	Low
Selset	3·4	Low
Balderhead : Derwent	3·2	Low
Scammonden	2·8	Low

COINCIDENCE WITH PREVAILING WINDS

Glascarnoch	NW
Orrin	W
Kielder	W
Celyn	W
Daer	SW
Carron	W
Selset	W
Balderhead	W

The remaining 6 No. reservoirs do not have close alignment with prevailing winds.

NOTE:-

1. Kielder is at elevation of 185m, has third longest fetch, high funnel rating and is coincident with prevailing maximum winds.
2. A fetch (Maxᵐ) of the respective lane width given, is considered "long" if greater than 3,000 m.
3. The width of relatively unobstructed wind lane along the maximum fetch to the dam has been measured.
4. A funnel rating of 4 or above has been taken as "high".

BABTIE SHAW AND MORTON
GLASGOW

Movements prior to 1983 were noticed and closely monitored. They resulted in no net downslope movement whatsoever. Vertical jointing in the affected zone was specified to be 12 mm to 15 mm and was set to allow clamps to grip each block when it required resetting. The underlying filters were of crushed igneous stone (dolerite). The flakey nature of the local stone (i.e. minimum dimension 0.3 times the sieve size as set in the specification) enabled some of the filter stone to be removed under wave downrush conditions and blocks subsequently to rotate in position during severe storms. The pea gravel in the joints was sacrificial and only served to maintain joint widths during construction. In earlier dams, roof slates have been used (e.g. Backwater) and wooden wedges.

I trust the above comments will contribute to the general discussion of this useful paper.

D GALLACHER (R H Cuthbertson & Partners)

With regard to the point Mr Curtis was making, I can give some indication the use of the Saville method at Megget. Taking the actual measured wind speed of 30 metres per second and the deduced fetch of 1.2 km, that is by adopting the full Saville method, the significant wave height would be about 0.8. The actual measured significant wave height was 1 metre. This means that you need an effective fetch of about 1.94 kilometres and, if you work then back the angle to use is about 42 degrees. So there is a considerable reduction. Of course, this is measured in a long narrow valley and obviously that wouldn't apply to all other reservoirs.

R MELBINGER (Fed Min Agric Forestry & Water Management, Austria)

I should like to put a question to Dr Schmidt. In his paper he mentioned the inspection of the wooden pipes by a TV camera. It's a very small TV camera as far as I see. I think it's a very useful apparatus for inspection of outlet pipes. It is, in my opinion, of great importance, especially when these pipes run across an embankment dam and have the valves on the downstream side.

We did some inspections in Austria by a kind of submarine TV camera, which even was able to walk down an inlet tower and then proceed into the pipe, but these cameras only work at comparatively large diameters of outlets. Therefore my question to Dr Schmidt is what is the diameter of the pipes and what is the size of the camera?

DR SCHMIDT DR-ING M (Harzwass Erwerke Des Landes Niedersachsen)

Well, the size of the wooden pipe is about 20 to 24 cm square. The problem is that this wooden pipe consists of 2 or 3 pieces, so there is sediment between the tips of the steps. The size of the camera is about 5 cm by 5 cm. It is put in from downstream; the length we have to cope with is about 50 metres: 80 metres maximum.

J SAMMONS (Consultant)

I was very interested by Mr Alsop's presentation, particularly by the dramatic difference there appeared to be between protections that were on permeable layers and those on impermeable layers.

I was recently working on a job where a geomembrane is being used as the impermeable membrane of the dam on the upstream surface. It appears that there have been several dams built in France where these have been covered by relatively thin slabs of about 80 mm, probably 3 metres square or something of that size. This seems exceedingly light to me. It is also one of those examples quoted in the ICOLD bulletin on the use of geomembranes and geotextiles, so it seems to be an acceptable method of protection. I'd be very interested if anybody has any comments or experience.

Could I ask for the example you quote, whether they are blocks or slabs, which are placed in situ? There are other examples where blocks have been used, but they interlock on probably all 4 faces, which is rather a different case than the sort of blocks that have been used at Kielder.

W FLEMME (Strabag Tiefbau GMBH)

We are actually working on a job in Spain where a geomembrane was used for protection of a dam. This was the Gonzalez dam. It was originally planned to do it in concrete, later it was redesigned with a conventional asphalt lining and, finally, it was constructed with a geomembrane.

Last year, shortly after the first impounding, the membrane failed with 20 metres of water in the basin. The total height of the dam is approximately 50 metres. I have seen several dams which have very badly damaged geomembranes. These membranes were mainly damaged by wind action, by ice action and, in the last case, by probably small leakages and erosion underneath, which led to the collapse of the geomembrane. I would like to draw attention to these facts to those using membranes.

J S CUNIBERTI (Consultant, Switzerland)

We are aware of the failure of the Gonozalez dam. It depends a great deal on how the geomembrane is placed and, obviously. as in any asphalt surface or concrete surface, the dam has to be basically stable, even in the case of leakage. Use depends partially on whether you can use the advantages of the geomembrane. With regard to leaks in the membrane, the membrane can be tested. We can check every metre of it, each seam and also check for a minimum thickness of about 2 mm.

I am not familiar with the mechanism of the failure in those cases cited but we have examples which have withstood every possible attack other than actual vandalism. Obviously the entire structure including the geomembrane must be made in such a way that it's stable.

The problem of ice seems very improbable to me. Tests that we have tried showed that ice has absolutely no adhesion to the geomembranes.

W FLEMME (Strabag Tiefbau GMBH)

The damage was caused by pack ice in the reservoir.

J S CUNIBERTI (Consultant, Switzerland)

The coefficient of friction of ice is practically zero on the surface of these membranes. The membrane system that we have studied has to be glued on to at least 30% of the surface, so that there is no chance of creating a wrinkle. It is conceivable that you could create a wrinkle. However, we've tested the membrane under pressures of 120 metres of water over openings of 4 cm, and it resists without breaking and opening. It is a very durable material.

F G JOHNSON (North of Scotland Hydro Board)

I'd like to fully endorse Ron Curtis's views on wave heights at Glascarnock. We are very doubtful about Saville: certainly our experience does not endorse Saville's formula at all. I would like to broaden this topic to armouring of faces: We have dams with smooth faces, with stone pitching, with concrete blocks. To me the best form of armouring, the cheapest form of armouring and the easiest to place is good rock pitching. We have the minimum trouble with that, it's very easy to repair if you get any damage, and we have had very little damage.

We get the most damage with smooth concrete faces with smooth blocks. We've got one dam where the blocks are about 4 feet square and about 18 inches thick and they pop out like peas out of a pod every year, and every year we've got to go back and put them back.

Glascarnock is a very bad reservoir because we get the winds from the Minch coming right up the reservoir. We have got stone-faced pitching grouted together, which is the last thing you want. You should pin it and wedge it in position. We've got fed up of going every year and having to re-maintain that. We've tipped in heavy riprap at very little cost; it's very easy, it's most effective.

I believe you've got to destroy the energy as it rolls up the upstream face. If you want to use concrete blocks, use the design which I saw in Spain, where there were varying heights of blocks. The surface wasn't flat and you make it as steep as you can.

J L BEAVER (Sir William Halcrow & Partners)

I'd just like to add a comment or two to Chris Binnie's paper No 7.3, where he describes the use of ground radar to detect voids.

I've used this myself on Frankley reservoir, which is the terminal reservoir to the Elan aquaduct, Birmingham, with great success. Severn Trent had underfloor leakage of about 40 litres a second and, after the use of ground radar and remedial works, we found some very large voids under the floor slab, which we successfully filled and reduced the leakage to less than 10 litres a second.

We've also used it to determine the voids behind tunnel linings and I suspect that is perhaps its most well-known use.

CLOSURE

E T HAWS (Chairman BNCOLD)

There are 3 apologies; firstly Roy Coxon, for being unable to be here. He sends his greeting to conference and, as past Chairman of BNCOLD, he's very sorry he couldn't be here this time. Secondly, I said in my opening remarks, 'we have 4 dams under construction, aren't we lucky?'; we are even luckier, we've got 5. I didn't mention Maentwrog: my apologies for those involved with that project. I also failed to list Ireland as a visiting country.

The high point of the Conference was David Coats's lecture. It would be presumptuous of me to say any more about that, except that it was very thought-provoking, inspirational, and I think we will carry those messages away with us. We look forward to David's support in higher places, where we shall be exerting our influence.

Starting with our early sessions, how do you keep a dam discontinued and can an enforcement authority discontinue it? There's food for thought there, no resolution on that. Money for 10% of the reservoirs that give 90% of the trouble, where do we find that? What happens to the small dams? It's a question I raised in my opening remarks and we haven't really come near to a solution, but it's certainly been pointed out very adequately.

Does acid come under the Act? There's a situation I am aware of with a very large industrial organisation, that has had some enormous storage tanks with petroleum products in them. They're discontinuing the petroleum storage therein and moving over to store water. They believe they will come under the Act and, in terms of volume they certainly will. They are having to carry out very substantial refurbishment of the tanks to bring them under the Act, because they are now going to store water where previously they stored petroleum, so I think that shows a little bit of the force of our Act.

Flood relief dams. Clearly, we are going to have more of those. It was interesting to see those shown for the first time. They're a little problematical under the Act, they're a bit like ash dams and barrages. I've certified one, obviously others are certifying them, it does seem the right and safe thing to do. I was very glad to see the very thorough Hong Kong records on their dams and computerisation thereof; some of our enforcement authorities do this and it's obviously the way that big systems and big collections of dams will be looked after in the future.

Interesting to see some 100-year old valves, and even 130-year old gates.

Steel tubes for draw-off towers; these obviously have been very efficient means of getting rapid construction carried out and the 3-day possessions at Foel they were particularly impressive.

I liked the witticisms of the gate operators who had all the excuses for not turning the valves, and the Strathclyde vandals are rather impressive.

There was the resolution I have made not to drive to the Dudley Arms along the A449 if it's raining rather hard.

Jim Claydon's description of settlement and leaks and use of infrared, that was very interesting, a little bit of high-tech coming here. Arthur Penman let it drop that he's been supervising a dam 12,000 miles away by video film and fax, so we are moving into the 21st century.

Douglas Gallacher; tube a manchettes grouting, I'd like to hear more about that some time.

Dr Schmidt; very interesting, beautiful dams, lovely scenery; gas injection looking for leaks in clay cores, never heard of that before. We're very interested in all these new techniques. I was very surprised, Dr Schmidt, though, to hear you say that you couldn't get people to do grass sod sealing these days, that it's a lost art. As far as I am concerned, it's a third generation activity in my family in the Highlands.

Mr Moffat's hazard analysis: we've heard of this from time to time; it did seem a little more practical in this presentation, and we look forward to a workable one.

Professor Mazzalai showed a very interesting amalgam of masonry and concrete with ties, drains, diaphragm walling, finite element analysis - a very interesting collection of technical applications.

I've never inspected a reservoir like Mill Hill that's subsided, burned and exploded!

Dr Paolina and Dr Cuniberti, your facings are very new to us, clearly the Italians are doing things that we in the UK haven't yet moved to. We do look forward to more information as the life and experience with these things grows. Your little exchange with the gentleman from Strabag, we'd certainly like to hear the outcome of that later on.

Mr Kriekemans' polyurethanes: he wouldn't own up to what they cost, but I'm sure they do a good job.

Malcolm Dunstan and his RCC: we are impressed, we'd love to do one.

Those are passing thoughts on some items that I found of great interest and I said I would end up with mentioning some subjects that I think are clearly going to be on the BNCOLD programme. Bottom outlets, the desirability thereof and the capacity thereof - clearly there's variety of thoughts about it and, related to that, the rate of draw-down; Bob Arah suggested a metre a day but any rules of thumb are bad, codes of practice are very bad, David Coats told us last night. Clearly, the matter should be given quite a lot of thought and it does depend very much on circumstances.

Over-topping here and there in maximum design flood circumstances - clearly this is something that we, as responsible dam engineers, panel engineers, must expect. The PMF circumstance is one of expected damage; it is a catastrophical situation in the area and I am glad to see that sensible applications are being made to these enormous flows, for which some of the structures are now having to be designed.

Design curve limits for reinforced grass spillways, wedge block development - obviously interesting work continuing here. Messrs Bramley, Hewlett, Alsop and Tuxford have all had interesting things to say; I am sure this is a subject for BNCOLD. We have had a meeting on reinforced grass spillways, but it is a subject that is reaching more sophisticated design areas and I am sure we should go at it again.

Tipping gates. Again in my introductory remarks, I said we must go for cheap means of getting some of these old dams into action and making them survive, even with the high criteria we now set ourselves. Tipping gates, and I mention flashboards as an even cheaper equivalent, for medium return period flood: this is an area I am very glad we are moving into, I am sure it is the right thing to help some of these old dams survive.

Trees on crests, pollarding thereof: difference of opinions, but most of us I think are glad to see them so long as they are reasonable and are watched in terms of disease.

Then getting our 2 data bases consolidated. It does seem to me that data bases for UK dams should be one and the same thing. I mean obviously the gentlemen who have been putting so much time and effort into these should produce one definitive version.

Critical and non-critical filters: Michael Kennard introduced this subject. This is very important to us and obviously something that's still advancing and been advancing for many years.

Sealing and drainage of concrete dams' downstream faces against internal humidity: the Haweswater case, Clywedog, Ted Goscheik's talk. He came up with a very nice analysis and said shouldn't we reinforce around the galleries: I thought we always did, but maybe there are some galleries without reinforcement.

Then we have the very large subject to which many of you reverted: slope protection: slabs versus riprap. Frank Johnson is very definite about it and I'm pretty definite about it. I like rip-rap, but if you want slabs, what should the underlayer be?

As far as I can tell, the Act is going reasonably well.

We've got some progress on cheap spillways, with a lot more to do I believe.

I heard nothing at all on privatisation. I've learned a great deal, I hope you've all learned a bit at least. Thanks very much indeed to all the authors, the contributors, the chairmen, the projectionist. I think you've contributed greatly to a successful two days: thank you very much indeed.

TECHNICAL NOTES:

A SURVEY OF UK EMBANKMENT DAMS

J P Millmore (Babtie Shaw & Morton, Exeter)
J A Charles (Building Research Establishment, Garston)

SYNOPSIS

Information has been gathered from Inspecting Engineers and dam owners concerning the condition and problems of embankment dams in the United Kingdom. The information from the owners portrays the situation for dams owned principally by large water authorities while the information from Inspecting Engineers reflects a more general cross-section of British dams. Information is summarised about age, height, core, cut off, outlet pipes and function of dams and the occurrence of different types of problems, investigations and emergency measures.

INTRODUCTION

1. As part of a programme of research into reservoir safety in the United Kingdom (UK), the Department of the Environment has commissioned the production of an Engineering Guide to the Safety of Embankment Dams. The first stage of work on the guide was to collect information on the condition and problems of UK embankment dams. In 1987 questionnaires were distributed to engineers who had been responsible for dam inspections under British reservoir safety legislation[1][2] and to major dam owning authorities.

2. The reservoir safety legislation[2] applies to reservoirs capable of holding 25000 cubic metres of water above the natural level of any part of the adjoining land. The questionnaire was completed by 22 Inspecting Engineers and 21 dam owners who are all listed in the acknowledgements. The returns from the Inspecting Engineers covered 757 dams, the returns from dam owners covered 651 dams. The information from owners portrays the situation principally in large dam owning authorities, while the information from Inspecting Engineers also includes many small privately owned dams and is thus considered to be more representative of British embankment dams generally. As the two sets of data must overlap, they have been analysed separately.

3. It is believed that there are some 2000 embankment dams that come within the scope of British reservoir safety legislation. It is clear therefore that the dams reported on in the questionnaire were probably little more than a half of the total. It is reasonable however to assume that in general terms the information is representative of British embankment dams.

ANALYSIS OF RESULTS

4. The results of the questionnaire are presented in two tables. Basic

information such as age, height and function is given in table 1. Information about investigations, monitoring and serious incidents is presented in table 2.

Table 1: Basic information for UK embankment dams

		Inspecting Engineers		Owners	
		No	%	No	%
Number of dams reported on		757	100	651	100
Date of completion	Before 1900	533	70	431	66
	1900-1940	139	18	147	23
	1940-1960	36	5	29	4
	After 1960	49	7	44	7
Height	Under 15 m	594	78	397	61
	Over 15 m	163	22	254	39
Core/membrane	Puddle clay	273	36 (65)	447	69 (77)
	Rolled clay	38	5 (9)	25	4 (4)
	Upstream membrane	12	2 (3)	20	3 (4)
	Other	23	3 (5)	37	6 (6)
	Homogeneous	75	10 (18)	53	8 (9)
	Unknown	337	44 -	70	10 -
Cut-off	Puddle clay	159	21 (59)	331	51 (65)
	Concrete	40	5 (15)	94	14 (18)
	Rolled clay	30	4 (11)	6	1 (1)
	Grout curtain	26	3 (9)	25	4 (5)
	Other	3	0 (1)	3	0 (1)
	None	13	2 (5)	53	8 (10)
	Unknown	501	65 -	144	22 -
Unprotected outlet pipes in embankment	Upst control valve	118	16	173	27
	Control within dam	53	7	65	10
	Dst control valve	225	30	56	9
Purpose	Water supply	348	46	-	-
	Ornamental lake	228	30	-	-
	Industrial process	59	8	-	-
	Flood control	20	2	-	-
	Agriculture	13	2	-	-
	Power generation	17	2	-	-
	Other	73	10	-	-
	Impounding	687	91	516	79
	Non-impounding	70	9	135	21

Notes; (1) Figures in brackets for core and cut-off refer to percentages expressed as a proportion of only those cases where the situation is known.

(2) Where a dam includes more than one type of core or cut-off each type has been included in the table.

5. Information on embankment dams in Northern Ireland has been included in table 1, although the Reservoirs Act 1975 [2] does not apply to the province. Information on dams in Northern Ireland has not been included in table 2 since the comprehensive programme of investigation of dams carried out there[3] would disproportionately increase the figures in this table.

6. There are a large number of old dams in the UK. Inspecting Engineers reported that 70% were built before this century began. Less than 10% were built during the period when the theories of modern soil mechanics would have influenced design and construction. The dams owned by large authorites have a similar age distribution.

7. The majority of dams are small. Inspecting Engineers have recorded only 22% being greater than 15 m high. The information from dam owners would suggest that most of these high dams are owned by the large authorities.

8. The existence and nature of any core and cut-off is unknown in about half the dams reported on by Inspecting Engineers. Where information is available, it is found that a large majority have puddle clay cores. Most of these in turn have puddle clay filled cut-off trenches, with only a small minority having concrete filled trenches. As would be expected, large authorities know more about the composition of their embankments than the private owner. The arrangement of the core and cut-off at these authority dams is similar to the remainder of UK dams. It is worth noting that about 10 have peat or peat and clay cores.

9. Outlet pipes not protected in a culvert or tunnel are very common with approximately half the dams being in this situation. In these cases, the majority of the dams in private ownership rely on a downstream control valve, while the large authorities have mainly upstream control valves.

10. The returns from the Inspecting Engineers indicated two major functions for UK embankment dams; 46% were built for water supply, 30% were built to form ornamental lakes. A large majority are impounding reservoirs.

11. In table 2 the occasions on which evidence of deterioration or serious incidents have been observed have been listed. The Inspecting Engineers reported that most problems were identified by Inspecting Engineers; the owners reported that they usually identified the problems. This probably reflects the difference between the situation of the small private owner relying on the Inspecting Engineer to identify problems and the large dam owning authority with its own engineering staff and in-house expertise. A significant number of problems have been identified by Supervising Engineers although this role was only created very recently with the implementation in 1985 of the Reservoirs Act 1975[2].

12. The most common forms of monitoring at embankment dams are measurement of surface settlement and flow measurements. Measurement of crest settlement is common in the dams owned by large authorities but comparatively rare in dams owned privately. A similar picture is found with measurements of seepage and leakage. Instrumentation such as piezometers have been installed in about 10% of dams. The greater proportion of these are owned by large authorities. There is little difference in the two sets of data with regard to both emergency and precautionary measures.

Table 2 : Investigations, monitoring and incidents at British embankment dams

		Inspecting Engineers		Owners	
		No	%	No	%
Number of dams reported on		717	100	651	100
Identification of problem	Inspecting Engineer	62	-	37	-
	Supervising Engineer	10	-	16	-
	Owner	24	-	61	-
	Other	8	-	10	-
Investigation and monitoring after completion of impounding	Surface settlement	138	19	370	57
	Other surface movemt	19	3	58	9
	Flow measurement	113	16	211	32
	Tracers	4	1	21	3
	Diving survey	13	2	58	9
	Geophysical survey	2	0	14	2
	Boreholes	65	9	124	19
	Trial pits	30	4	65	10
	Instrumentation (eg piezometers)	66	9	110	17
Safety measures	Emergency (eg drawdown)	24	3	25	4
	Precautionary (eg restrict TWL)	51	7	69	11

Note; dams in Northern Ireland have not been included in this table.

CONCLUSIONS

13. The majority of British embankment dams were built before 1900. Most of them are under 15 m in height. The nature of the core and cut-off is not known at the majority of British embankment dams. Where the internal composition is known the typical embankment dam has a puddle clay core and often also a puddle clay filled cut-off trench. Unprotected pipes laid through the embankment are common and in many cases there is only downstream valve control. The principal uses of these embankment dams are for water supply and to create ornamental lakes. Surface settlement is now monitored at many of the larger embankments. Seepage or leakage flows are also commonly monitored. Not infrequently piezometers have been installed.

ACKNOWLEDGEMENTS

14. The Department of the Environment commissioned Babtie Shaw and Morton to work with the Building Research Establishment to produce an Engineering Guide to the Safety of Embankment Dams. The collection of information described in this Technical Note was the first stage of this work. The note is published by permission of the Director of the Building Research Establishment.

15. The data that has been analysed was provided by the following Inspecting Engineers; D W Berry, N Buchanan, N J Cochrane, J G Cowie, D C Crosthwaite, E S Dawson, D N W Earp, S G Elliott, D D Fraser, W G N Geddes, H Grace, T A Johnston, D J Knight, J E Massey, D Ormerod, D A Piesold, F F Poskitt, W J F Ray, A C Twort, P F Tye, J D Williams, A J H Winder: and the following dam owners; Borders Regional Council, British Alcan Highland Smelters, British Waterways Board, Central Regional Council, Central Scotland Water Development Board, Fife Regional Council, Grampian Regional Council, Hartlepools Water Company, Highland Regional Council, Lothian Regional Council, North of Scotland Hydro-Electric Board, North West Water, Northumbrian Water, South Staffordshire Water Company, Sunderland and South Shields Water Company, Tayside Regional Council, Tendring Hundred Water Company, Welsh Water, Wessex Water, Western Isles Island Council, Yorkshire Water. Their help is gratefully acknowledged.

16. Additional data not in statistical form has been obtained from other Inspecting Engineers and dam owners; Rofe, Kennard and Lapworth, Bristol Waterworks Company, Severn Trent Water.

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A NUMERICAL MODEL FOR CHUTE SPILLWAY FLOWS

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BACKGROUND

1. Over the past two years, a project has been carried out for CIRIA under the DoE Reservoir Safety research programme aimed at enhancing and making available to UK practitioners a numerical model for simulation of flow on chute spillways. DoE interests are primarily related to the assessment of spillway capacity on existing dams, but the model is also relevant to the design of new spillways. The project has built on earlier development of numerical techniques in spillway flow analysis by the author[1,2].

2. The first edition of the so-called "CHUTE" software will be demonstrated at the Conference. It is intended that by using the software the engineer will be able to extend the range of hydraulic calculations which can be carried out before having to revert to a hydraulic model.

3. The program has been developed in conjunction with a CIRIA steering group which has included practitioners in UK Reservoir Safety.

MODEL CAPABILITY

4. The model calculates values of velocity, both in magnitude and direction, and depth of flow at a network of grid points distributed both longitudinally and transversely within the spillway channel. Both supercritical and subcritical states of flow can be modelled, together with the hydraulic jump transition between zone of flow.

5. Spillway channel features which can be modelled using the "first-edition" of the model are:-

- o weir crests which are straight, curved or composed of straight-line segments in plan;
- o channels having both longitudinal gradient and cross-fall in the invert but with vertical walls to the channel;
- o bends, tapers and expansions of plan geometry;
- o chokes formed at bridges;
- o overtopping of the channel walls;
- o sets of steps of cascades in the channel invert.

6. Input to the model comprises geometrical data defining the channel and weir configuration and also flow conditions. Geometrical data defining the channel and weir form is in terms of x, y and z co-ordinates with references to a set of orthogonal axes. Resistance characteristics of the channel are defined by means of a roughness height which can be varied longitudinally and also transversely in the channel.

7. Flow conditions are in the form of either values of starting velocity components and depths at or below the spillway crest or in the case of simple forms of crest, the flowrate can be specified together with the head/discharge relationship for the overflow.

8. At the downstream limit of the model, the tailwater rating curve for the downstream river channel can be specified in order that the location of a hydraulic jump in the stilling basin may be determined.

AVAILABILITY

9. It is intended to release the first edition in disc form suitable to running on an IBM-compatible micro in late 1988 for one year user trials to UK organisations with interests in reservoir safety. Following this appraisal, a second edition will be developed for wider issues, and it is intended that future enhancements will be added to increase the ease of use and range application of the model.

10. The first edition model is accompanied by a related users' manual and a more general Guide to Hydraulics of Chute Spillway Flow which will be published as a CIRIA Technical Note. Information concerning the availability of the first edition and agreement covering the trials will be available at the Conference.

References

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REPAIRS TO DOWNSTREAM FACE OF HAWESWATER DAM

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SYNOPSIS

Haweswater Dam, of concrete buttress construction, was completed in 1942. It stands 27.5m high with a crest length of 470m.

Surface spalling of the downstream face, up to 100mm deep, has taken place since 1971. Continuing deterioration is thought to be due to freezing and thawing of saturated concrete.

Trials of different repair systems have been carried out since 1982. The combination of cementitious repair mortars coated with a silane and acrylic has proved to be most suitable.

INTRODUCTION

1. Haweswater Dam, situated 7kms west of Shap, Cumbria, was constructed between 1931 and 1942. (1) It is of concrete buttress construction with continuous upstream and downstream faces. The dam stands 27.5m high, has a crest length of 470m and provides a storage capacity of 84,550 Ml. The reservoir has a top water level of 270.79m AOD and the average annual rainfall at the dam is 1940mm.

DETERIORATION OF THE DOWNSTREAM FACE

2. Surface spalling of the downstream face of the dam was first observed in 1971. The problem accelerated from about 1975 for several winters and has continued to date albeit at a reduced rate. The spalling is generally restricted to the lower two-thirds of the dam with penetration up to a depth of 100mm. At present the spalling covers approximately 30% of the downstream face. Continuing deterioration is thought to be due to the effect of freezing and thawing on saturated concrete.

3. The commencement of serious spalling coincided with the installation of security doors within the dam structure. These doors restricted the free flow air through the internal chambers resulting in an atmosphere with a high humidity. The concrete surfaces within the dam remained constantly saturated. During 1985 louvres were installed within the doors. This has resulted in a reduction in the humidity within the dam, and a lower degree of saturation of the internal concrete surfaces. Rainwater passing through construction joints on the downstream face and crest provides much of the moisture found within the body of the dam. Other moisture comes from seepage through the upstream face and from uplift drains.

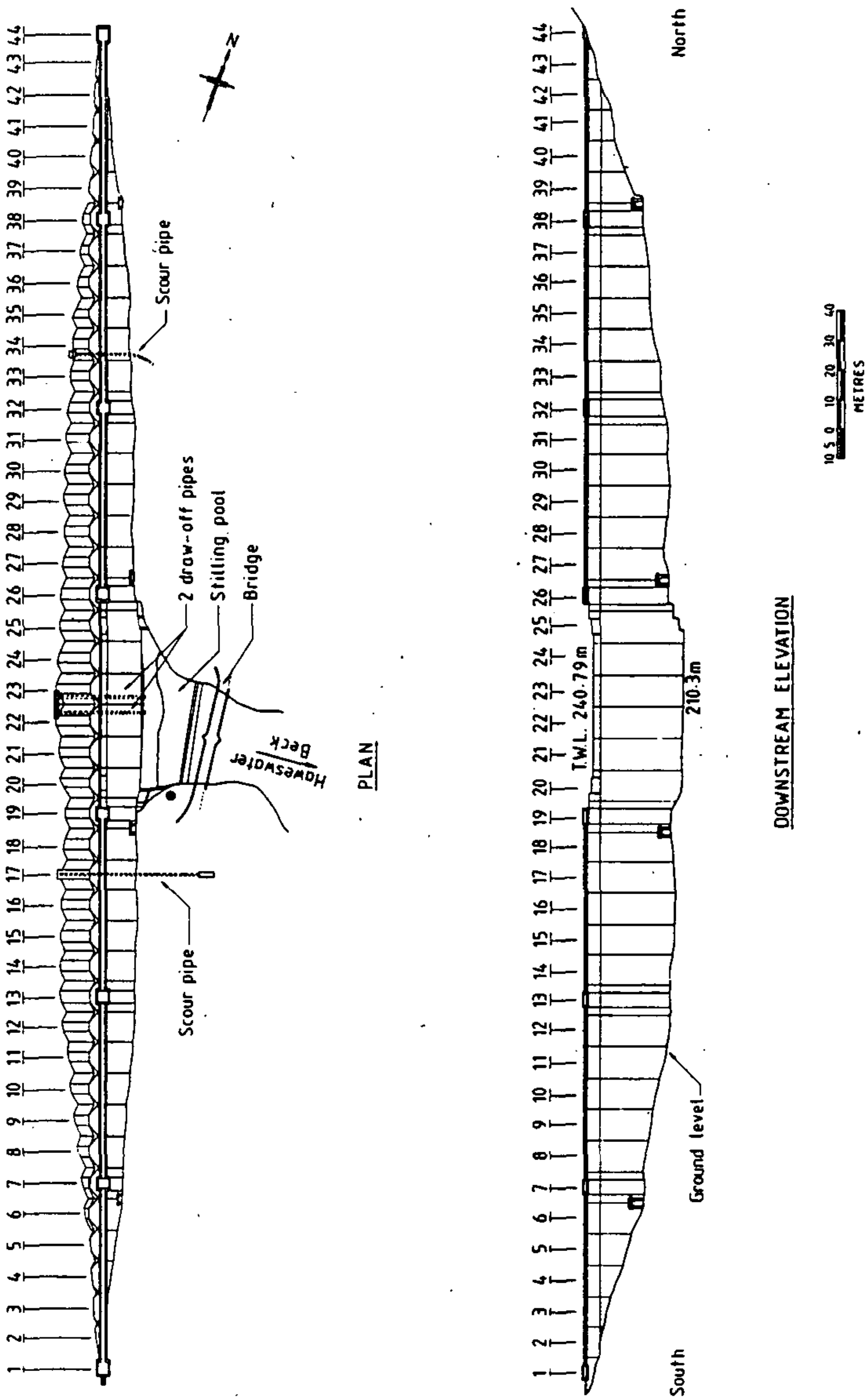

Fig. 1 - Haweswater Dam

TRIALS

4. Trial repairs were carried out in 1982 and 1985 and a preferred repair system established. A further trial was carried out in 1987 using a modification of the manufacturers materials.

1982 Trials

5. A number of material manufacturers were asked to recommend suitable repair systems. Three systems were selected for trials (see Table 1) and tender documents were issued to selected contractors on the suppliers approved list. The systems consisted of cementitious repair mortars with two acrylic and one epoxy surface sealer coatings.

6. On all panels the damaged concrete was mechanically cleaned to sound concrete, the surface was then cleaned with high pressure water jetting, the dam profile was restored with repair mortar, and finally a sealer coat or coats were applied.

Table 1

1982 trials

Panel No.	15	30	31
Supplier	CBP(Fosroc)Ltd	Sealocrete Co.Ltd	Inertol Co.Ltd
Applicator	TAS Surface Coatings Ltd	Whitley Moran & Co. Ltd	Whitley Moran & Co. Ltd
Specification	High pressure water cleaning at 6000 psi; Mechanical cleaning; High pressure water cleaning at 1500 psi.		
	Repair spalled areas with RENDEROC cementitious repair mortar.	Repair spalled areas with SEALOTAK Bonding Slurry and SEALOTAK Bonding Mortar. (SBR modified mortar).	Repair spalled areas with Bonding Bridge including ICOMENT additive, Repair Mortar including ICOMENT additive & thin layer PALESIT-MORTAR 520.
	Seal surface with WETEXI 623 acrylic sealer.	Seal with 3 coats epoxy WETCOTE.	Seal with 2 coats ICOSIT Concrete Cosmetic. (Metacrylate)
Comments	Cement coloured.	Uniform artificial colour. Much of repair failed in 1982.	Uniform artificial colour.

7. These trials were inconclusive. The epoxy sealed material failed shortly after application. However, both of the acrylic sealed materials have remained predominantly intact, but with minor areas of cracking, leaching and debonding. The areas of failure were considered to be partly due to the high fines content of the original concrete (2) and poor workmanship due to difficult working conditions, as well as any deficiencies in the repair materials and specification.

1985 Trials

8. As the initial trials had proved inconclusive, further trials were carried out in 1985, (see Table 2) using similar materials to the acrylic sealed systems used in the earlier trials, as well as low molecular sized chemicals which penetrate the surface of the concrete and form a hydrophobic layer.

Table 2

Panel No.	28	29	30
Supplier	CBP(Fosroc)Ltd	Celtite(Selfix)Ltd	Inertol Co. Ltd
Applicator	Advanced Concrete Protection Ltd		
Specification	Mechanical cleaning; Grit blasting.		
Area I	Repair & low build render with RENDEROC repair mortar. Coat with NITROCOTE DEKGUARD. (Silane & acrylic sealer)	Repair & low build render with RESIPATCH LIGHTWEIGHT mortar. Coat with RESICOTE AC2. (Acrylic sealer)	Repair with ICOMENT repair mortar 504 or 508. Low build render with ICOMENT DRY mortar. Coat with ICOSIT AQUASTOP (1 coat). (Siloxane)
Area II	Repair with RENDEROC mortar. Coat with NITROCOTE DEKGUARD.	Repair with RESIPATCH LIGHTWEIGHT mortar. Coat with RESICOTE AC2.	Repair with ICOMENT mortar 504 or 508. Coat with ICOSIT AQUASTOP (2 coats).
Area III	Repair with RENDEROC mortar. Coat with WETEXI 623 (Acrylic sealer).	Repair with RESIPATCH LIGHTWEIGHT mortar. Coat with RESICOAT AC3 (Pigmented acrylic sealer).	Repair with ICOMENT mortar 504 or 508. Coat with ICOSIT AQUASTOP (1 coat). Coat with ICOSIT CONCRETE COSMETIC (Metacrylate).
Comments	(I) - Cement grey colour. (II)&(III) - Patch repairs obvious.	(I) - Light grey colour. (II)- Patch repairs obvious. (III)- Uniform artificial colour. All areas show crazing.	(I) - Cement grey colour. (II)- Patch repairs obvious. (III)- Uniform artificial colour. Areas (I) & (II) show crazing.

9. Three panels were used. Combinations of materials were used to provide trials on nine sealing methods. Tender documents were issued to selected contractors, not necessarily on manufacturers' approved lists. A single contractor was selected to apply all of the materials.

10. On all systems the damaged concrete was mechanically cleaned, grit blasted and the dam profile was restored with repair mortar. Some panels were finished with a render layer of mortar. All panels were then sealed.

11. Minor cracking sometimes associated with leaching has been observed on all panels. This is thought to be due to fine cracks in the concrete of the original dam. It is considered that no repair system will totally prevent these from occurring.

12. In addition to this cracking most areas of panels 29 & 30 have shown crazing, which has allowed moisture to penetrate into the repair mortar. (It is more obvious after rain). This is thought to be due to minor shrinkage in the repair mortar.

13. All areas of panel 28 were considered to have performed satisfactorily. The recommended solution for permanent repairs to the whole of the downstream face was based on the materials used on this panel.

RECOMMENDED SOLUTION

14. The two systems considered in detail consisted of Renderoc repair mortar, in conjunction with a render layer of Renderoc plus sealing coats of Nitrocote Dekguard, or sealing coats of Nitrocote Dekguard without the render layer.

15. Renderoc is a cementitious repair mortar applied in conjunction with an acrylic bonding agent. Nitrocote Dekguard consists of a silane primer, which penetrates the concrete surface and reacts to form a hydrophobic block, and an acrylic topcoat.

16. Subsequent to the 1985 trials, Renderoc FC was formulated. This enables a very thin fairing coat of mortar to be applied instead of a render layer. A trial was carried out in 1987 utilising this material instead of Renderoc as a render.

17. The use of a render or fairing coat has the advantages of providing a smooth surface, which fills any small cracks and pockets within the unrepaired concrete, and provides a uniform coating, in this case natural cement grey in colour. If no render or fairing coat were to be used then on areas not repaired with mortar any small cracks and pockets would provide a weak spot in the surface sealer, and would provide a ledge for moss and debris. The face would be left with a patchwork appearance.

18. It is programmed to carry out the works during the summers of 1988 and 1989, using Renderoc repair mortar, Renderoc FC fairing coat and Nitrocoat Dekguard sealer.

JOINTS

19. Rainwater enters the centre of the dam via joints on the downstream face. These joints were not provided with any waterstop or sealant, except at the spillway.

20. A polyurethene sealant was used to seal the joints between the 1985 trial panels. This has been successful in reducing water entering the centre of the dam from the downstream face and has performed satisfactorily. It is now intended to seal all of the joints.

ESTIMATED COST

21. The estimated cost of the works is £450,000. The surface area of the downstream face is 10,300 square metres.

22. In comparison, to use the render layer instead of the fairing layer would cost £620,000 and to omit both render or fairing coat would cost £300,000.

CONCLUSIONS

23. Major concrete dams are often situated in areas of high rainfall and harsh weather conditions. Mechanisms of behaviour and deterioration are often complex.

24. It is important that any repair methods are carefully considered and where possible comparative trials carried out. Trials should be true comparisons of the materials as applied under normal contractural conditions. As variations in standard of workmanship, especially in sub-strate preparation, can distort the overall picture, it may be advisable to use a single contractor to apply all of the materials under test.

25. Close co-operation and assistance from the manufacturer(s) in drawing up the specification and during the application of materials is essential. Whilst manufacturers' approved contractors may be appropriate this does not guarantee that workmen skilled with particular materials will be used. A helpful attitude, a good general track record and a willingness to learn is seen as more important.

ACKNOWLEDGEMENTS

26. The Authors are indebted to the Authority for permission to publish this Technical Note and wish to thank their colleagues in Engineering (North) for their assistance in its preparation. The assistance of the Inspecting Engineer, Mr K T Bass, and the material manufacturers with the trials, is gratefully acknowledged.

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