UNIVERSITY OF NEWCASTLE UPON TYNE
DEPARTMENT OF CIVIL ENGINEERING

In conjunction with

BRITISH NATIONAL COMMITTEE
ON LARGE DAMS

INSPECTION OPERATION and IMPROVEMENT of EXISTING DAMS

PROCEEDINGS OF A SYMPOSIUM
HELD AT THE UNIVERSITY OF NEWCASTLE UPON TYNE

SEPTEMBER 24 - 26 1975

INSPECTION OPERATION and IMPROVEMENT of EXISTING DAMS

Proceedings of a Symposium sponsored jointly by the British National Committee on Large Dams and the Department of Civil Engineering, University of Newcastle upon Tyne.

Newcastle upon Tyne 24 - 26 September 1975

Edited by
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INSPECTION, OPERATION AND IMPROVEMENT OF EXISTING DAMS

THE THEME

'Inspection, Operation and Improvement of Existing Dams' is a broad theme, reflecting increased interest being shown in the complex and multifarious problems associated with safe operation of reservoirs.

The theme of the Symposium is particularly appropriate when set against the background of recent British developments relating to reservoir engineering. Revised reservoir legislation has been brought out, and an equally important development has been publication of the Flood Studies Report. A further relevant factor is the high proportion of older dams in the United Kingdom - some 60% of large dams in Britain are over 50 years old and are thus likely to demand increasing attention in the years ahead, and a similar picture may emerge in other countries.

THE EVENT

The Symposium was directed towards all engineers involved with the operation, maintenance and appraisal of existing dams. In recognition of the significance of the event the Symposium was marked by the presence of Mr C F Grøner of Norway, then President of the International Commission on Large Dams, who delivered an Introductory Address.

The Symposium was intensive, with five Technical Sessions concentrated into two full working days. Careful consideration was given to the balance of the Symposium, and the sub-themes corresponding to the Technical Sessions were selected accordingly:

Technical Session 1 :	'Reservoir Legislation and its Implementation'
Technical Session 2 :	'Operation of Reservoirs, including Maintenance and Instrumentation'
Technical Session 3 :	'Problems of Massive Dams and Remedial Measures'
Technical Session 4 :	'Flood Analysis, Prediction and Design in relation to Reservoirs, Dams and Spillways'
Technical Session 5 :	'Problems of Embankment Dams and Remedial Measures'

On the third and final day attention was focused on selected local dams, participants visiting Balderhead, Cow Green and Derwent.

SYMPOSIUM STEERING GROUP

The Symposium was organised by a Steering Group representative of the joint sponsors.

F G JOHNSON	MEng CEng FICE MIWES	(BNCOLD)
M F KENNARD	BSc CEng FICE FIWES	(BNCOLD)
W M KILKENNY	MSc CEng MICE FGS	(University : to 31 Dec 1974)
C B MARSH	BSc	(University : from 1 Jan 1975)
Prof P NOVAK	BSc (Eng) IngDr CSc DrSc CEng FICE MIWES	(University)
Organising Secretary:		
A I B MOFFAT	BSc CEng MICE MASCE FGS	(University and BNCOLD)

PREFACE AND EDITORIAL NOTES

The suggestion for a Symposium on 'Inspection, Operation and Improvement of Existing Dams' dates back to December 1973, the concept and the broad theme being endorsed by the British National Committee on Large Dams early in 1974.

As initially conceived the Symposium was to be a relatively small and purely British event, reflecting various developments then pending which were likely to influence activity in the field of dam engineering in the United Kingdom. The interest shown in the theme proved much greater than originally anticipated and led to 37 Papers being presented before 165 participants. It was particularly gratifying to note that some 20 overseas guests attended, representing a span of countries from Asia to Europe and 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, Session Chairmen and Reporters, who contributed so much to the success of the five Technical Sessions, and also with the participants who so actively involved themselves in discussion, formal and informal.

The Symposium Steering Group would like to record their appreciation of the help given by the Department of Civil Engineering, in particular by my secretary, Mrs G Faulkner, and by Mr J C Bennett, Mr A C Price and Mr A K Hughes. Our thanks are also due to Mrs M Moffat and Mrs V Saunders for their assistance with the Ladies Visits and for their patient help with so much of the preparation.

The General 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 the minimum, and every care has been taken to provide an accurate account of what was said. In those cases where participants later submitted an edited version of their contribution from the floor this has been incorporated in the Proceedings in preference to the tape record.

Many participants illustrated their contribution with the aid of slides, diagrams or other visual aids. Where this was the case the symbol S appears alongside a necessarily edited version of that contribution.

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.

A I B Moffat
June 1976.

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OPENING SESSION AND INTRODUCTORY ADDRESS

SYMPOSIUM SECRETARY : A I B MOFFAT (University of Newcastle upon Tyne)

Good morning, Ladies and Gentlemen. May I introduce the people sitting here on the platform today? We have Professor Elliott, Professor of Law and Pro-Vice-Chancellor of this University; we then have our distinguished guest the President of the International Commission on Large Dams, Mr Grøner from Norway; Mr Gerrard, who is Chairman of the British National Committee on Large Dams and Professor Isaac, who is Head of Department and Professor of Civil and Public Health Engineering in this University.

PROFESSOR D W ELLIOTT : (Pro-Vice-Chancellor, University of Newcastle upon Tyne)

Good morning, Ladies and Gentlemen. It is my pleasant duty to welcome you to this symposium, which your Steering Group has seen fit to hold in our University. We are gratified that they have done so, as indeed we always are when a learned body decides to hold their gathering here. As always, we are particularly delighted to welcome overseas visitors. On behalf of the Vice-Chancellor and the whole University I bid you welcome, and hope that your deliberations will be fruitful and pleasant.

I quite often get this task of welcoming learned bodies, and I am always very glad to do so, even though usually the subjects being discussed are of such an esoteric nature that I am quite unable to understand what is going on. It is usually the case that my ignorance is palpable, so I am all the more delighted that the subject being discussed this morning is not only comprehensible to a lawyer but one of interest to me personally.

The subject of dams and their failure has always had a very large amount of private law attached to it in this country; you may or may not know that the law about the liability of the operator of a dam for any damage done by failure of a dam in England developed out of the ancient law of cattle trespass. How it developed out of cattle trespass is, of course, an interesting story which, perhaps fortunately for you, I have not time to tell here!

It has always struck me that the public law on the subject has for long been remarkably lenient. It was not until 1930, I believe, that a reasonably comprehensive Act of Parliament first appeared applying to reservoirs in the United Kingdom. I gather that this statute is now being supplanted by a brand new Act of Parliament, and I also gather that the latter is regarded by some as in many respects disappointing. I look forward to learning more about this if you will allow me to eavesdrop on your Technical Session 1.

I usually implore delegates to Symposia to take time out to explore Northumbria, an area well worth exploring and which often surprises those who come here with misconceived preconceptions about what the region looks like. I do not have to do this with you, because I understand that your optional Field Trip will take you to some local dams. This will involve you inevitably in seeing the countryside unless, indeed, you are so immersed in technical conversation as not to look out of the coach windows, which I hope will not be the case. I note that your ladies are also being taken to see a large dam, one which failed several centuries ago. I refer to the Roman Wall, which for a long time held back a flood of barbarians from the Northern end of our island! However, its maintenance and instrumentation were evidently inadequate,. and long-continued seepage has eventually produced a situation in which names beginning with 'Mc' form the largest part of the London telephone directory! As a person of Scots ancestry myself I am persuaded that this is not a wholly bad thing for England, and that perhaps the Emperor Hadrian was misguided in what seemed a good idea at the time.

I hope you that your discussions will be meaningful and fruitful.

PROFESSOR P C G ISAAC : (University of Newcastle upon Tyne)

It is a pleasure to welcome you all not only to the University, which the Pro-Vice-Chancellor has done, but also to the Department of Civil Engineering here, which has long had an interest in all areas of water resources.

We are most flattered and happy that BNCOLD chose to hold the symposium here. It seems a very topical occasion as far as my Department is concerned, because you may be interested to know that the University has now approved an MSc postgraduate course in Reservoir Engineering with effect from 1976.

We are thus a Department actively concerned with water in the past and increasingly so in the future, still more with structures for retaining water. May I therefore, on behalf of my Department, welcome you here.

R T GERRARD : (Chairman, British National Committee on Large Dams)

Pro-Vice Chancellor, Ladies and Gentlemen, it is my pleasant task to tell you a little about our honoured guest.

Mr C F Grøner is Senior Director of a firm of Consulting Engineers in Oslo, a firm which specialises in hydro-electric and industrial developments. He was Chairman of the Norwegian Committee on Large Dams in 1960 and he is, or has been, a member of the Norwegian Engineering Society, Consulting Engineers Society, Geotechnical Society, Concrete Institute and Committee on Rock Blasting.

He was Visiting Professor of Dam Design and Construction at the Norwegian Technical University for three years and, of course, he has been active in ICOLD affairs, at least as far back as 1948.

The International Commission on Large Dams has a Congress every three years, and the Mexico Congress next year will be the ninth Mr Grøner has attended. He has been very active on ICOLD committees and has been author of numerous technical papers on dams and on dam deterioration, principally concrete dam technology. I have very much pleasure in inviting Mr Grøner to give his Introductory Address.

C F GRØNER : (President, International Commission on Large Dams)

Mr Chairman, Ladies and Gentlemen. I am sorry it is impossible for me to deliver this speech in Norwegian, and therefore you must excuse me if I have to read my Address!

It is an honour for me to have an opportunity to address you at this BNCOLD/University Symposium on dams. I shall speak on the topic of legislation for hydro-power in Norway and about inspection and observation of dams generally.

As many of you know, Norway's natural conditions are very favourable to water power development. We have heavy rainfall, especially on the West Coast and in the mountains in the West and Central parts of Southern Norway, with a rainfall of up to 2.5 m as against approximately 1.5 m for the country as a whole. We have a total lake area of approximately 13 000 km^2, some 4% of the total land area. These lakes are the basis for most of our reservoirs and dams. The total amount of water stored in our reservoirs amounts to about 37×10^9 m^3. Total electricity production for 1974 was 76 000 GWh, equivalent to about 17 000 kWh per inhabitant. Ninety-nine per cent of all our electricity in Norway is derived from water power.

From ancient times the farmers in Norway have owned the waterfalls in their own vicinity, and as a consequence of this the State, the counties or private industries intending to build a power plant and reservoirs have to buy the water rights in order to be permitted to commence development. Today the State owns 26%, the counties 51% and private industries 23% of the entire power production. In 1909 the Norwegian Water Resources and Electricity Board, for control of planned construction,was established; this was considered as a very important step. In 1917 we had two important legislative developments, the Act on Acquisition and the Act on Watercourse Regulation, and then we had legislation on reservoirs.

We in Norway require a concession to build a power plant and to regulate a reservoir. When a project is large enough it has to pass through Parliament. An application for a concession first has to pass the Electricity Board, that is the State, and if this Board found the plans acceptable they would be passed to the Parliament for a final decision. This procedure normally takes some two years, but in special cases it might be handled more rapidly. Norwegian law gives the State all rights to the power plants after 50 years of operation, at which time the plant must be delivered to the State in a normally well-maintained condition. This latter point can be a difficult question, but that is for the State to determine. According to our laws we also have regulations concerning the Water and Electricity Board.

Consortia are not allowed to design other than to Codes which the Electricity Board has worked out. The Electricity Board thus controls all calculations and all drawings. Normally this works well, but sometimes the Codes, in our opinion at least, are too rigid and cause considerable unnecessary expense to the owners.

Maximum flow must also be accepted by the Hydrological Department of the Electricity Board. There has been discussion in this field, concerning excess flow over the crest of concrete dams by comparison with flow over embankment dams. Flow over concrete dams outside the spillway section is normally of no danger to the valley downstream. In this respect an embankment dam is more critical. Rules for embankment dams are not yet specified in detail, but this will be done in the near future.

The Electricity Board requires all results from the control work on the dam site to be brought to their attention. All laboratory results, investigation and research into geology of the dam site, concrete and soils etc. have to be sent to the Board for acceptance. This may seem a little bureaucratic, but it does not cause as much trouble as one may imagine. Most of this paper control work for the Board is easily carried out during construction.

Dam sites are inspected by members of the Board on one or two occasions during construction. The rules require that the Consultant or the Owner shall have at least one site controller (or Resident Engineer) available on site at all times during construction.

During recent years we have had a special problem with the environmental question. Discussions were, for a period, mostly carried out between the involved groups of individuals, but often ended up shooting far beyond the goal. Procedure is now a little more relaxed and better balanced. The authorities have started to see some of the practical consequences, and thus that there is a limit somewhere - nobody can say exactly where, but it is there - to environmental consideration.

The results of this discussion are generally interpreted to mean that reservoirs must not be so large that it is not possible to fill them by the beginning of the month of June, and the level must be kept high until the end of August. (This is required to 'hide' the basin during the summer months.) Neither may the rivers be dry during the summer, and as a consequence of this we have built so - called 'ground' dams or weirs. Their purpose is to maintain a constant water level on parts of the river where people live. These dams are up to 4 m high and back up the stable water surface in certain areas.

We have also tried to avoid avalanches in the reservoirs when drawing down but that is not so easy. 'Draw-down' could cause damage to the valley which would tend to ruin the landscape from a tourist point of view. When drawing down during the winter season, which in the mountains is normally six to eight months, they are covered with snow and ice and no reasonable damage may be observed; such damage can only be seen in the summer time.

It seems to be a common belief that when a dam has been built it will last virtually forever without anybody looking after it. As everybody attending this Symposium knows, that is completely wrong. Any structure needs maintenance or it will gradually lose its ability to serve its intended purpose. A systematic inspection schedule each year or each half-year should exist for any dam, and whenever unusual conditions arise such as large flows, snow melts, storms, earthquakes etc. an inspection should be carried out immediately thereafter. It ought to be the same inspector each time for a specific dam. He must know what to look for; he must know the construction details of the dam and, if possible, the design parameters and stresses involved. The reports issued ought to be expressed in a standard format in order to make it easier to compare reports from different inspections from year to year. Photographs must be taken of irregularitics and filed with the records - this is very important for one cannot normally describe irregularities adequately without photographs. Any damage must be repaired by qualified personnel, and specialists will have to be called in if there is any doubt whatsoever.

Embankment dams are more difficult to handle than concrete dams. It may be far more complicated to find leakages, and the repair work is often complex and expensive.

It is often not appreciated that damage starts on a small scale and that this damage ought to be repaired as soon as possible. One should not wait until the damage has grown and more extensive work will have to be carried out at far higher cost. Whether or not there is damage, in addition to the regular dam inspection there should be a more extensive inspection carried out every second or third year by specialists to get an overall impression of the condition of the dam. Our firm has an agreement with some of the dam owners in Norway that an extensive inspection should be carried out by our company every year or every second year. On several embankment dams we monitor deformations each year, and we also monitor large dams for some years following their construction, in order to see if first filling of these reservoirs causes unanticipated deformations.

Operation of reservoirs is a very importantant point, especially when the reservoir stores only a small percentage of the annual run-off. The owners will normally try to hold the water level as high as possible especially during the autumn before the winter starts. When they hold the level high during the autumn, and heavy rain-fall occurs during this period on fresh snow this can be dangerous, and it occurs fairly frequently on the West coast. There can be 400 mm to 500 mm of snow, and when you get heavy rain on that difficulties will arise. The floods may often be larger than the maximum allowed under unregulated conditions, but if you narrow the overflow section to get a small initial flow and more self-regulation, for instance, you will require a higher dam to get the same degree of safety against overtopping. During Spring the flow may be easier to handle, and run-off can be predicted from Winter snow measurements. If one has a dry Spring, warm or not, melting of the snow will be slow and steady, but if at this time one gets rain in combination with warm weather one may face a severe problem. To cope with this situation reservoirs ought to be as empty as possible to be able to absorb the flow, and this is generally the case, but details can differ quite a lot.

There are some questions which, in conclusion, I would like to see this Symposium discuss:

1. How long do you generally think observations from embedded instruments in concrete and embankment dams will give you results you can rely on? I do not expect an exact answer!
2. How do you handle the large quantity of instrument data from larger embankment dams without it proving too expensive?
3. How do older dams, of 50 years to 80 years' age, behave? Do we really know, or do we not?

I thank you for listening to me. I believe symposia of this sort are of the utmost importance in connection with the safety of dams and for economic and environmental reasons. I congratulate the University of Newcastle upon Tyne and BNCOLD on conceiving this meeting, and I wish you all a successful symposium.

R T GERRARD :

Thank you very much for that Introductory Address. It is most interesting to hear from one who is pre-eminent in dam engineering how engineers in Norway view the themes which we are to discuss this week.

Now I think I have a small surprise for you, because among the visitors to this symposium we have Mr Willis from the United States Army Corps of Engineers, and he would like to address a few words to you.

H B WILLIS : (United States Army, Corps of Engineers)

Thank you Mr Gerrard. I am here as the representative of the Corps of Engineers, and perhaps I may commence by giving a little background information in explanation of their interest in this symposium.

Two hundred years ago, in June 1775, George Washington, who had been commissioned to raise a Continental Army, appointed a young Lieutenant to be Chief of Engineers, and from this initiative developed what is now the Corps of Engineers. In 1824 Congress formally authorised the first Corps of Engineers' participation in civil works, and involvement in surveys for transcontinental highways and canal construction commenced. The Corps' involvement in civil works has progressed to the point where, today, we have some 28 000 civilian employees and about 500 commissioned officers of the Corps active in such work. This group is now responsible for flood control and navigation activities at Federal level in the United States, design of dams, for example, being decentralised to some 40 district offices.

A few years ago the Chief of Engineers decided to establish a Design Award contest, to encourage excellence in project design. Entries are judged in the context of architectural design, landscape design and engineering design. Last year one of the Award winners was the Snettisham Power Project, built by the Corps in Alaska to provide power for the city of Juneau. In this project we tapped a lake in the mountains to bring water, via a tunnel, to the generating facility at sea level. Our Consultant, and the man largely responsible for much of the successful concept, was Mr Christian Grøner. It is customary for us to make a presentation to the Consultant for an Award-winning project. I have the plaque for Snettisham here, and it is my honour, Sir, to present it to Mr Grøner in the name of the Chief of Engineers, United States Army.

C F GRØNER :

Thank you very much. It has been a pleasure to work with the Corps since, I think, 1967. I hope Snettisham is a success, both technically and from the environmental viewpoint. Thank you.

PROCEEDINGS : TECHNICAL SESSION 1

RESERVOIR LEGISLATION AND ITS IMPLEMENTATION

RESERVOIRS ACT 1975

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SYNOPSIS

The Reservoirs Act 1975 repeals the Reservoirs (Safety Provisions) Act 1930 and re-enacts the legislation on reservoir safety in a strengthened and more effective form. The Act received Royal Assent on May 8 1975 and it is expected that most of its provisions will be brought into force some time before the end of 1975.

The Act distinguishes between the legal/administrative and the professional engineering aspects of reservoir safety. It provides for the first time: (i) a system of enforcement by local authorities and general powers of supervision by the Secretary of State; (ii) for large raised reservoirs to be kept under continual surveillance by a qualified civil engineer; (iii) the steps to be taken to remove a reservoir from the scope of the Act or to secure the safe abandonment of use of a reservoir. The paper describes the new powers and duties as they relate to qualified civil engineers, undertakers and enforcement authorities.

INTRODUCTION

The Reservoirs (Safety Provisions) Act 1930 was drafted following two reservoir failures in 1925, at Skelmorlie and Dolgarrog. It prohibited the design and supervision of construction of large reservoirs except under the supervision of a qualified civil engineer, that is to say, 'qualified' for the purposes of that Act by reason of his appointment by the Secretary of State to be a member of a Panel of engineers, or Panels, whose competence and experience qualifies them to undertake the different duties placed on them by the Act. In the forty four years of its operation there have been no major dam failures in Britain. Nevertheless, experiences in other countries have shown deficiencies in the law which the new Act seeks to remedy.

In 1966, the Institution of Civil Engineers' report on reservoir safety reviewed the operation of the 1930 Act and made detailed recommendations for amending the law. Some three years later the administration of the 1930 Act was transferred from the Home Secretary to the Minister of Housing and Local Government (now the Secretary of State for the Environment) and the Secretaries of State for Scotland and Wales. In 1970 the Government announced that they intended to introduce new legislation dealing with reservoir safety. Five years (and many bridges and hurdles later) the Reservoirs Act 1975 received Royal Assent.

Although the Reservoirs Act 1975 (in this paper called the Act) repeals the 1930 Act, it embodies and builds on what is best in the old and so contains much that is familiar. The Panel system of qualified civil engineers is still the linchpin of reservoir safety and the certification and periodical inspection procedures are still there, though modified and tightened up to close gaps.

The new provisions take account of the advice of the many interested bodies, especially that given by the Institution of Civil Engineers. The Act embodies, either in principle or in detail, most of the recommendation in the Institution's 1966 report. Only two main recommendations have not been met - one relating to third party insurance and the other for extending the ambit of the Act to structures storing liquids other than water.

The ambit of the Act is defined with more precision than previously. The 'large reservoir' of the 1930 Act is now a 'large raised reservoir' - the additional adjective serving as a constant reminder that capacity is not the sole criterion. The decisive volume of 25,000 cubic metres, a rounding up of the old 5 million gallons in metric terms, must also be above the level of the adjoining land. The Act specifically includes artificial lakes and lochs. The specific exclusion of canals and inland navigations unless they form part of a reservoir and of lagoons or tips for receiving refuse in solution or suspension, to which the Mines and Quarries (Tips) Act 1969 applies, may help undertakers to understand the nature of 'a reservoir for water as such'.

The 1930 definition of 'undertakers' as amended by the Water Resources Act 1963 is adapted to take account of the replacement of river authorities by water authorities under the recent reorganisation of the water services.

Those 'local authorities' who are required by the Act to maintain a register of large raised reservoirs are the Greater London Council and county councils in England and Wales and the regional and islands councils in Scotland. These same councils are also *enforcement authorities* except where they themselves are also 'undertakers', in which case, because a local authority who are also undertakers cannot enforce the law against their own reservoirs, the Act provides that:

a) where a local authority's reservoir extends into the area of other local authorities one of those local authorities shall be the enforcement authority;

and b) where a local authority's reservoir lies wholly within their area, there shall be no enforcement authority.

At first sight it would appear that, under b) above, a local authority is not subject to any oversight. Such is not the case. Under section 3, local authorities are required, both as enforcement authorities and as undertakers, to report to the Secretary of State at prescribed intervals on the steps they have taken to ensure that the Act is complied with, thus any failure by an authority as undertaker would be noticed. The Secretary of State has power, if he is satisfied a local authority is in default, to make an Order directing them to carry out their duties in accordance with the Act.

QUALIFIED CIVIL ENGINEER

Fundamental changes have been made in the constitution of and method of appointment to the panels of qualified civil engineers.

Under the 1930 Act, once a civil engineer was appointed to a Panel he was a member for life unless removed on the grounds that he was no longer fit. All appointments to existing Panels lapse five years from the date of commencement of the new Act or at such earlier date as the Secretary of State may decide after giving at least six months notice. In future, appointments will run for five years and members will have to apply and satisfy the qualification and fitness requirements before re-appointment for a further term. Panel appointments will automatically lapse when a Panel is abolished or altered. However, members affected will be allowed to continue to act as qualified civil engineers for up to four years in order to complete commissions undertaken before the abolition of the Panel, and they will be given time to apply for membership of any other Panel.

The Act provides the qualified civil engineer with a basic choice of four statutory hats to wear labelled *the construction engineer, the inspecting engineer, the supervising engineer*, and simply *the qualified civil engineer*. The Panel system ensures that the qualified civil engineer wears the appropriate hat of the right size.

THE CONSTRUCTION ENGINEER

In general terms, the 'construction engineer' is the qualified civil engineer employed by the undertakers under section 6 or as a result of enforcement action under section 8 :

(i) to design and supervise the construction of a large raised reservoir, whether it is new construction or alterations to increase capacity and (ii) to report on and supervise the re-use of an abandoned large raised reservoir under section 9.

The provisions dealing with the issue by the construction engineer of preliminary and final certificates governing the filling and operating conditions of a large raised reservoir are tightened. For new large raised reservoirs, the Act allows for successive preliminary certificates to be issued, but prohibits the issue of a final certificate earlier than three years from the issue of the first preliminary certificate. This time interval is intended to allow for a sufficient period of observation and testing during the crucial early years of a reservoir. Where under the 1930 Act a reservoir could be operated indefinitely on a preliminary certificate without any reason being given, the construction engineer now has a duty to explain to the undertakers why, if five years have elapsed since the issue of the first preliminary certificate, he has not issued a final certificate, and a copy of his explanation must go to the enforcement authority.

The Act introduces the 'interim certificate' for use by the construction engineer engaged on works to increase the capacity of an existing large raised reservoir so that pending the issue of a preliminary certificate he may revise the statutory filling level and operating conditions of the existing reservoir.

The construction engineer now has an additional duty when he issues his final certificate. He is required to annex to his certificate a note of any matters he considers should be watched by the supervising engineer until the first periodical inspection of the reservoir is made. This is an important new provision as it enables the construction engineer to draw the attention of the supervising engineer to any novel or critical features of his design or of ground conditions encountered during construction and so makes the day to day surveillance of the reservoir so much more worthwhile and valuable. It is perhaps relevant at this point to mention that the Act also empowers the construction engineer, as well as the inspecting engineer, to specify the frequency and manner in which the information for the prescribed record of a reservoir is to be given. Consequently, as the work proceeds the construction engineer will be able to arrange for appropriate prescribed information to be recorded at whatever stage in construction he considers it necessary.

There are special provisions relating to the issue of final certificates, where the construction engineer is acting under sections 8 and 9. Under section 8 he could be dealing with a reservoir that has been completed and in use for several years before non-compliance with the Act was discovered. In such a case if he is satisfied that the reservoir has been filled with water for at least three years and that it is sound and satisfactory and may be safely used for the storage of water, he may issue a final certificate forthwith. In other cases, because he has not been involved in the design of the reservoir and may not have supervised any part of its construction, he is not required to state on his final certificate that the reservoir is sound and satisfactory. If he does not do so however, and his report recommended safety measures, then he must instead state that those safety measures have been implemented.

THE INSPECTING ENGINEER

Once commissioning is completed and the construction engineer has issued his final certificate, the reservoir is subject to the periodical inspection procedure and the qualified civil engineer, in the role of 'inspecting engineer', assumes responsibility for recommending measures to ensure it is operated and maintained in a safe manner.

The new provisions governing the appointment of an inspecting engineer are more stringent than those in the 1930 Act and are in line with a recommendation in the Institution of Civil Engineers 1966 report. The inspecting engineer can no longer be an employee of the undertakers, he cannot be the engineer who acted as the construction engineer and he must not, when he acts as inspecting engineer, have any working or business connection with the construction engineer.

The act requires two certificates to be issued following a periodical inspection. Both are essential for the proper functioning of the enforcement system. In the first, the inspecting engineer certifies whether his report does or does not include recommendations as to measures to be taken in the interests of safety. In the second the qualified civil engineer, who must be appointed to supervise the implementation of the measures and who may be an employee of the undertakers, certifies that the measures have been carried into effect.

In his report the inspecting engineer would pick up those matters noted by the construction engineer or a previous inspecting engineer and amend, revise or add to them, thereby ensuring that instructions to the supervising engineer are continually updated to take account of those changes which are bound to occur in the reservoir structure, the catchment and the area downstream with the passage of time.

THE SUPERVISING ENGINEER

The only brand new statutory 'hat' introduced by the Act is that for the supervising engineer. A serious weakness of the 1930 Act was the absence of any requirement for the reservoir, once constructed, to be kept under continual surveillance by a civil engineer who is not only capable of interpreting operating data and records, but has the trained eye to notice and assess the effects of the unexpected event on reservoir safety. Between the 10-yearly periodical inspections required by the 1930 Act matters affecting reservoir safety were left to undertakers, most of whom have acted responsibly. Continual supervision is, however, too fundamental an issue to be left to the discretion of an undertaker. Perhaps the most important contribution to reservoir safety made by the new Act is the duty placed on undertakers to appoint a new class of qualified civil engineer to act as supervising engineer.

Once a construction engineer has issued his final certificate the supervising engineer is responsible for the day-to-day oversight of matters affecting the safe operation and behaviour of the reservoir.

In this he has a policing role and is guided initially by the matters drawn to his attention in the construction engineer's note and subsequently in the inspecting engineer's periodical reports.

Section 12 requires the supervising engineer to advise the undertakers of any aspect of the reservoir's behaviour affecting safety, to draw their attention to breaches of the Act (particularly as to compliance with requirements related to the storage of water), and to call for a periodical inspection at any time he thinks fit. At least once a year he must give the undertakers a written account of the action he has taken to implement the instructions of the construction and inspecting engineers.

A copy of the supervising engineer's account of his work is not required to be sent to the enforcement authority. Where, however, the supervising engineer advises the undertakers to arrange for a periodical inspection under section 10, or to take any action, say in connection with the implementation of recommended safety measures, then copies of that advice must be sent by the supervising engineer to the enforcement authority. Similar action is also required where the supervising engineer draws the attention of the undertakers to a breach of the mandatory conditions for the storage of water in the reservoir or the statutory requirement related to the keeping of records of prescribed information.

GENERALLY

Apart from the defined roles of construction, inspecting and supervising engineers, only a qualified civil engineer may:

a) be employed by the enforcement authority when they use reserve powers in default of an undertaker (section 15) or where the authority take emergency action (section 16);

b) approve and certify the satisfactory completion of alterations to remove a large raised reservoir from the ambit of the Act (section 13);

c) report on measures to be taken to ensure that a reservoir, when its use is abandoned, is incapable of filling accidentally or naturally so as to constitute a risk to public safety (section 14);

d) advise the enforcement authority on the time to be specified in a default notice served on the undertakers requiring them (i) to carry out works or measures before a reservoir is brought back into use (section 9 (8)), or (ii) to implement any measures recommended by an inspecting engineer (section 10 (6));

e) act as an independent referee on disputes (section 19).

Under section 20 it is the qualified civil engineer, whatever statutory hat he may be wearing at the time, and not the undertakers, who is required to send all certificates and annexes, as well as copies of reports (except a report of an inspecting engineer which makes no recommendation as to safety measures) to the enforcement authority.

An enforcement authority may consult any civil engineer as to the time in which measures , so long as they do not involve any alterations, shall be carried into effect to secure the safe abandonment of use of a reservoir. Similarly any civil engineer may design the alterations required to remove a reservoir from the ambit of the Act, but the alterations must be approved and supervised by a qualified civil engineer. Although there is no requirement for the undertakers to employ a qualified civil engineer to design and supervise alterations which do not increase the capacity of a large raised reservoir they are required, after the alterations have been done and if they have not been designed and supervised by a qualified civil engineer, to have the reservoir inspected under section 10 if such alterations might affect its safety.

UNDERTAKERS

Ultimate responsibility for the safety of a reservoir must rest with the undertaker. The new Act may at first sight appear more complex than the old, but in the long run the precision and underlying directness of its provisions should make life easier for the undertakers. The duty to employ qualified civil engineers for a wider range of specified functions and the improved certification procedures are typical examples. So, too, are the new provisions which make it easier and safer for the undertaker to carry out *discontinuance* works under section 13 to remove a reservoir from the ambit of the Act and those under section 14 which enable an undertaker safely to abandon the use of a reservoir without all the expense of complete demolition and removal. It should, however, be noted that where abandonment of use involves only measures and no alterations the reservoir remains on the register and is subject to the inspection and supervision requirements.

Undertakers with large raised reservoirs either under construction, or newly completed but still lacking a final certificate under the 1930 Act and excepting those completed before the 1930 Act or not within the ambit of that Act, are required by section 25 to comply with the Act as regards the provisions relating to construction or enlargement (section 6), construction engineers' certificates (section 7) and enforcement in event of non-compliance (section 8).

Section 26 requires undertakers of large raised reservoirs constructed before 1st January 1931, the commencement of the 1930 Act, which for any reason have not been inspected under that Act to arrange for a first periodical inspection as soon as practicable.

The Act tightens the arrangements for periodical inspection by requiring undertakers, under section 10, to arrange for a first periodical inspection of a new reservoir within two years of the issue of the final certificate, thereby ensuring that a reservoir is inspected by an independent qualified civil engineer in the critical early years of its life. The undertakers are placed under an obligation to implement, as soon as practicable, any measures recommended by an inspecting engineer. The enforcement authority is provided with a check on compliance by the requirement that the qualified civil engineer employed to supervise the implementation of the measures must issue and supply them with a copy of his certificate confirming that the measures have been implemented.

The right of the undertakers to refer complaints about recommendations made by an inspecting engineer to a referee is retained and extended to include recommendations made by a qualified civil engineer who has been appointed as a result of enforcement action or has reported on either the re-use of a reservoir or on the abandonment of use. Additionally, the referee is required to issue a certificate, with a copy to the enforcement authority, stating whether his decision does or does not amend an engineer's report.

The requirements of the 1930 Act related to the recording of essential information about the operation and behaviour of a reservoir are extended. A duty is placed on undertakers to provide any necessary instrumentation required for the proper recording of the information and to comply with any directions of the construction engineer or the inspecting engineer as to the frequency and the manner in which the information is given. The additional requirements related to instrumentation make mandatory what is already good practice. It is sensible that the construction engineer, who is responsible for certifying that a reservoir is sound and satisfactory, and the inspecting engineer, who recommends safety measures, should have discretion in deciding how often essential data should be recorded and how it should be presented.

Failure to comply with the Act now constitutes a criminal offence. The more serious failures are punishable on indictment by unlimited fines (section 22(1)). Fines on summary conviction range from £100 to £800. It is interesting to note that the fine of £800 relates to knowingly or recklessly giving, or making use of, false information to supervising engineers, inspecting engineers or any other qualified civil engineer employed for different purposes under the Act. The liability for an offence applies to officers and members of a corporate body as well as that body itself.

ENFORCEMENT AUTHORITIES

Just as technical matters are seen to be the preserve of the civil engineer, so the Act regards enforcement as a legal and administrative matter that should be dealt with primarily at local level. Matters related to public safety are appropriately a function of local authorities, who have among other things a duty under section 138 of the Local Government Act 1972 to act in emergencies. Enforcement of the Reservoirs Act 1975, which is after all an act solely concerned with public safety is seen as a rightful duty for local authorities to undertake.

Instead of a right, exercisable only through the Courts under the 1930 Act, local authorities (the Greater London Council and the county councils in England and Wales and the regional and islands councils in Scotland) now have a duty placed upon them under section 2 to see that undertakers observe and comply with the Act.

The nub of the enforcement procedure is the register of all large raised reservoirs which is to be kept by local authorities. To get the register started, undertakers are required within nine months from the date of commencement of the Act to provide the local authority with basic information about existing large raised reservoirs. Once on the register a reservoir can only be removed in accordance with the discontinuance provisions provided by section 13.

Information to be registered will be prescribed by regulation. The Act already requires a considerable amount of information to be provided to either local or enforcement authorities. This includes:

a) notices of intention to construct a large raised reservoir or to bring one back into use or of the fact of abandonment of use;

b) details of appointments of construction and inspecting engineers and appointments and cessation of supervising engineers;

c) copies of all certificates and reports used by qualified civil engineers;

d) copies of explanations why a final certificate has not been issued and copies of advice given by supervising engineer.

The registers will be available for inspection by any member of the public (not necessarily a person interested in or affected by a reservoir), who may, with the consent of the Director of Public Prosecutions, also institute a prosecution.

In the event of non-compliance enforcement authorities are empowered to require undertakers to appoint a qualified civil engineer, and in default to take action themselves:

a) to act as construction engineer for reservoirs in course of construction (section 8)

b) to inspect, report on and supervise an abandoned reservoir that has been brought back into use (section 9);

c) to carry out any periodical inspection as required by the Act or to implement measures recommended by an inspecting engineer (section 10);

d) to act as supervising engineer (section 12);

e) to report on the measures to be taken to ensure the safe abandonment of use of a large reservoir (section 14).

The enforcement authority may also take emergency action to protect life and property when a large raised reservoir appears to be dangerous, but only in accordance with the recommendations and under the supervision of a qualified civil engineer. Reasonable costs incurred by the enforcement authority in exercising their reserve powers in the event of default by an undertaker and in taking emergency action are charged on the undertakers

In order that enforcement authorities may carry out their functions under the Act enabling powers are provided to allow the authority's authorised agent entry on to land, subject to the giving of seven days notice, for the purpose of:

a) finding out whether a reservoir is a large raised reservoir or is one under construction or is in use as a reservoir;

b) taking enforcement action;

c) using their reserve powers under section 15;

d) taking emergency action, in which case there is no requirement to give seven days notice prior to entry and the power of entry also extends to neighbouring land.

Although enforcement authorities are given extensive powers, no enforcement provision breaches the principle that technical matters are the sole responsibility of the qualified civil engineer. It is therefore no impediment to effective enforcement that many local authorities, and their officers, will have had no previous experience of dealing with reservoirs.

CODA

The new system of administration for reservoir safety recognises that there are distinctive levels of activity and competence for each of the three principal participants (the qualified civil engineer, the undertaker and the enforcement authority). In their separate roles as either engineer, manager or administrator they have clearly defined powers and duties, the combination of which is designed to ensure the safe and efficient design, construction and use of reservoirs. The Act should, at least, allow everyone concerned with reservoir safety, and none more so than those members of the Institution of Civil Engineers who long ago recognised the deficiencies of the 1930 Act, to sleep more easily in their beds at night.

Any views expressed in this paper are those of the author and not necessarily those of the Department of the Environment.

POINTS OF LAW RELATING TO RESERVOIRS

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SYNOPSIS

The paper is a resumé of legal points in connection with reservoirs. The author is a Solicitor qualified to practise in Scotland and is not versed in English Law. The two legal systems are, however, similar in this respect and the main relative statute, the Reservoirs Act, 1975, applies throughout Great Britain.

Reference is made to the Common Law position with respect to land rights in relation to reservoirs, to the Reservoirs Act, and finally to Third Party Liability in connection with reservoirs.

THE COMMON LAW POSITION

Water being one of the four ancient elements of life it is to be expected that it occupy a large part of the legal systems of most countries. The title of this paper is intended to indicate that only one or two points of law are touched on, particularly in connection with reservoirs. To attempt otherwise would involve the risk of a flood of words. The author, having been associated with Engineers for over a quarter of a century, may be forgiven for the comment that if his experience is any guide the readers of this paper will have their own share of 'sea lawyers'.

The development of the law of water in different countries has naturally reflected the physical character of the country's topography. The obvious is recorded in that with Scotland being a land of liberal rainfall, it has fallen to the Courts in Scotland in early years to found principles which were subsequently worked out in later years by the more copious jurisprudence of England and America.

The basic rules of law for inland waters are fairly well ascertained, the principles having been founded in a series of Court cases at Common Law, that is in contrast to statutes, and stretching back over approximately the past 100 years. The basis is simply that an inland water is treated as a pertinent or part of the land.

If a competing claim of ownership relates to an inland water the title to the land itself is *prima facie* presumption of right to the ownership of the water, and this can only be overcome by proof of exclusive possession by another proprietor. In the absence of express title conditions the position at Common Law where the loch is surrounded by two or more estates is that the owners of the estates have the title to the loch and bed proportionally to their respective frontage up to the middle of the loch, but all owners have the common use of the water throughout its whole extent for the purposes of boating, fishing and fowling.

Thus at Common Law where the loch is surrounded by the land of one owner he is entitled to its full and exclusive use, and if he chooses he may drain and convert it to dry land provided, however, he does not destroy the source of any running stream which passes through the land of a lower landowner.

An early and authoritative case of competing interests in an inland loch was the Loch Rannoch case in 1798. Loch Rannoch, which is now one of the reservoirs of the North of Scotland Hydro-Electric Board, was in 1798 about seventeen km long and three km broad and, with the exception of a small area at one end, was contiguous with the lands of the Clan Chief Menzies of Rannoch, and of the lands of Robertson of Strowan. The Menzies title, dating from 1502, had a grant which specifically mentioned the Loch by name and also the islands in the Loch. The Strowan title dating from 1636 contained a general grant of lakes and fishings. It was held that Menzies had exclusive rights to the islands but that all the owners adjoining had a joint property in the Loch with common right of sailing, fishing, floating timber or drawing fishing nets on the shores adjoining their respective lands but not on the shores of the other lands.

The early cases also established the rules governing the boundaries of lochs, a leading one being that in 1769 relating to Duddingston Loch in Edinburgh. Here there were two adjoining owners, one with a title of a specific grant of the Loch and the other one excluding the Loch except for the right to water cattle.

It was held that the owner whose title specifically included the Loch had the sole and exclusive right of property in the Loch but the other owner had the right to water cattle and pasture down to the edge of the water in its natural state and upon any part of the *solum* which by natural drought was dry at any one time, these rights of servitude or easement being exercisable only adjoining the other owner's ground. The owner of the Loch had the right of boating and fishing even to the extent where it overflowed the adjoining estate.

As to the extent of use open to owners of a loch, they are generally entitled to exercise any act of property over the area opposite their respective lands up to the *medium filum*, or to limits as might have been laid down by the Courts provided only that no action destroys or deteriorates the loch. The position here is similar to the right of an owner whose boundary is the sea or a tidal river. It is different in the case of a riparian owner on a non-tidal river where he is not entitled to do anything on his portion of the river bed which may prejudicially affect the common interest of the flowing water.

The general law of lakes in England and Wales and in Ireland and in America is the same as in Scotland, although some doubt has been expressed in England as to whether the rule of the ownership of the lake being with the riparian owners applies to large lakes where there is a right of public navigation. This right of navigation can be established by proof of use over several years and becomes a right of way. Cases in England on this point arose in connection with Ullswater, and in one of these it was observed that whether the soils of lakes, like that of freshwater rivers *prima facie* belong to the owners of the bed of the lake or of the manors on either side, or whether it belongs to the Crown in right of royal prerogative is not necessary to determine — that is the Court in England left the question open. In Scotland this doubt does not arise. Lochs irrespective of size and whether or not open to public navigation are the property of the owners abutting the loch. An example of such a large loch in Scotland open to public navigation and still owned by the surrounding landowners is Loch Lomond.

The foregoing summary of the general land law relating to lochs and lakes has been presented because the various rules mentioned have to be kept in view when dealing with the land requirements in a reservoir area. Most reservoirs of any size are now works authorised by statute, mostly Private Acts, giving the Undertakers compulsory rights to acquire the lands required for reservoirs. The general law has to be kept in view when such statutory rights fall to be exercised. It is normally the case that the Undertaker obtains the land title for the reservoir by agreement without recourse to compulsory acquisition procedures and thereby saving time and expense, to the benefit of the Undertaker and of the landowners. Whether or not the Undertaker's title is obtained by agreement or compulsory procedure the Undertaker is involved in the general land law, as entitlement to compensation for the land required is related to the extent of ownership. When, for instance, the Caledonian Canal was constructed the owner of Glengarry Estates claimed an exclusive right of navigation on Loch Oich (used for the Canal) with the object of obtaining more compensation. He did succeed in establishing that he had the right to Loch Oich and its bed but the Court held there was no entitlement to compensation under the Canal Act simply for navigational use by the Canal Commissioners. This approach to compensation entitlement is, incidentally, analogous to the principle established under the Railway Clauses Acts whereby while there is entitlement to compensation for damage suffered by the actual construction of the railway, there is no compensation entitlement for loss of damage arising simply from the use of the railway. The same principle has applied with other public works.

One practical measure in the acquisition of a reservoir site is the defining of the land boundaries. The North of Scotland Hydro-Electric Board early in its history adopted the policy of acquiring by agreement all the land for a reservoir site where a new reservoir or loch was being created by the damming of a river - for example Loch Faskally at Pitlochry. Where, however, it was a case of damming the outlet of an existing loch and raising the level for a reservoir, as at Loch Tummel, the extent of land acquisition was limited to the sites of the permanent works, in particular the dam, and the title of the ownership of the areas to be flooded was left with the adjoining landowners. The compensation was thereby restricted to the creation of the burden of the lands becoming subject to the flooding required for the reservoir.

STATUTE LAW

The Statute Law in relation to reservoirs did not have much bearing until the enactment of the Reservoirs (Safety Provisions) Act in 1930. This Act and the new Act of 1975 are the subject of other papers and only incidental points are touched on in this paper.

It is to be noted that reservoir legislation is essentially machinery for imposing a system of safety checks on reservoirs and does not materially alter the general law as affecting reservoirs. It is perhaps a question whether the Reservoir Act procedures would have come about if the failures at Dolgarrog and Skelmorlie had not occurred in 1925.

In the operation of the 1930 Reservoirs Act the experience of the North of Scotland Hydro-Electric Board has been that little difficulty has been met.

One of the stated purposes of the new legislation is to set out a system of enforcement of the Reservoir Act provisions, basically by employing the local authorities. When this proposal was first mooted in 1970 it was suggested for the North of Scotland Hydro-Electric Board that the average local authority was not likely to have staff either qualified or experienced in this special branch of civil engineering.

A departure from the 1930 Act provisions in the 1975 Act is the omission from the latter legislation of the requirement to publish in the Press the receipt of a certificate and report. While admittedly such requirement is of doubtful significance in any enforcement procedure the omission seems hardly justified, particularly if the individuals resident in or interested in property in any area likely to be affected by the escape of water from a reservoir are not in future to have the right to call on the Undertaker for information about certificates and reports.

A welcome provision in the new legislation from the public point of view is that the 'Inspecting Engineer' shall be independent. This point was put to the Department concerned by the North of Scotland Hydro-Electric Board in 1970 in light of the Board's policy to appoint for the periodic examination of their reservoirs Engineers who, and whose partners had had, no association with the Engineers who had issued preliminary or final certificates.. The wording of the new provisions, however, is to the effect that the Inspecting Engineer shall not be in the employment of the Undertakers otherwise than in a consultative capacity, and that he was not the Engineer for the reservoir as construction Engineer nor is connected with any such Engineer as his partner, employer, employee or fellow employee in a civil engineering business. Not excluded is an Engineer who *had* been so connected, the present tense only applying in the wording.

It is also satisfactory that in the new legislation the minimum period of three years between date of issue of a preliminary certificate and a final certificate shall date from the issue of a first preliminary certificate. This was one of the points made for the North of Scotland Hydro-Electric Board in 1970 to cover the obvious case of there often being more than one preliminary certificate. In 1970 the intention was, if the author's recollection is right, that this period should be three years minimum and five years maximum. Instead, the provision in the new legislation is that the construction Engineer has to explain to the Undertakers why, if at the end of five years, he has not been able to issue the final certificate. The five year period might therefore be extended almost indefinitely if a difficulty arose, for example in the works contract, and under the new legislation an Inspecting Engineer cannot inspect when a construction Engineer is supervising the reservoir. In these circumstances problems might arise which could only be resolved by new appointments, but it seems open to question whether Undertakers under the new legislation can change an appointment of a construction Engineer once made. In the new legislation there is provision that the reservoir shall be under the supervision of a construction Engineer, employed to design and supervise the construction or alteration until he, the construction Engineer, gives his final certificate. In 1970 it was pointed out by the Board that to cover the case, which occurred more than once, of an Engineer who gave a preliminary certificate and who died before the issue of a final certificate, the reference in the Act should be that the certificates be obtained from *a qualified Engineer.*

The arbitration provisions in the new legislation, which are essentially a re-enactment of the 1930 Act provisions, only apply in cases of dispute between the Undertakers and the Engineer where measures are recommended for safety purposes or as to the time of the next inspection by the Inspecting Engineer. A delayed final certificate by the construction Engineer does not seem to be covered by arbitration.

A final incidental comment on the new legislation is that the proposed Preamble to the Act *'to make further provision against escapes of water from large reservoirs or from lakes or lochs artificially created or enlarged'* seems rather misleading. One would rather expect there would follow a series of technical provisions for actual works. The Preamble to the 1930 Act heading *'An Act to impose, in the interests of safety, precautions to be observed in the construction alteration and use of reservoirs'* — seems more appropriate.

THIRD PARTY LIABILITY

In the following final sections of this paper reference is made in general terms to the position of third party liability in relation to reservoirs.

In this connection to revert first to the legislation, it is of interest that in the Preamble to the 1975 Act no mention is made, as with the 1930 Act, to its purpose being *inter alia* to amend the law with respect to liability for damage or injury caused by the escape of water from reservoirs. The relative section of the 1930 Act is section 7 in which it is provided that where damage or injury is caused by the escape of water from a reservoir constructed after the passing of the Act under powers granted after that date, the fact that a reservoir was so constructed is not to exonerate the Undertakers from any indictment or action to which they otherwise would have been liable.

This section is re-enacted in the new Act — under Schedule 2 — and while its provisions may seem rather curious the reason for this is simply the following of precedence to have excluded from Private and similar Acts the defence of statutory authority in any legal action for claims in respect of escape of water from a reservoir built under statutory authority. The principle involved is that no action can lie for any act done under the authority of the Legislature although the act, if it had not been authorised under statute, would have given a right to damages for loss.

Under both the law of Reparation in Scotland and the English Law of Tort a distinction is made where there has been an unnatural use of property. Reservoirs obviously fall into such category. Under English Law there is the rule of absolute liability, that is liability for loss produced without fault. This rule is generally known as the Rylands v Fletcher rule from a case in 1865 where liability was upheld in respect of leakage of water from a dam into underground workings, of whose existence the defender was justifiably ignorant. Under the Scots Law of Reparation this doctrine of absolute liability has not been accepted as such. The basic rule in Scots Law is still that negligence must be proved to establish liability. In cases arising from the unnatural use of property the degree of care required under Scots Law is, however, exceptionally high, and while this might be said to amount in effect to insurance the doctrine of absolute liability is strictly not part of Scots Law. This was illustrated in a recent case in Scotland concerned with damage from an escape not of water but of gas during plumbing operations. It was proved by the defenders that the usual methods of working had been employed, and it was held they were not liable for damage from an unforeseeable accident.

With a reservoir being an unnatural use of property and so involving absolute liability under English Law for damage from escape of water from a reservoir, and in Scots Law a very high degree of care, this exclusion of the defence of statutory authority under the Reservoir Act is accordingly of no great significance. Note also that the defence of statutory authority is only open where the statutory operation is carried out without negligence.

The case in Scotland corresponding to Rylands v Fletcher in England was that of Kerr v Earl of Orkney in 1857. Here the defender had erected a dam on a stream to collect water and the dam was breached by flood. It was held that proof of care in the construction of the work did not relieve the defender from liability for resultant loss. The only exception would have been if the loss had been attributed to an Act of God, but in law unprecedented rainfall is not an Act of God. Incidentally the term 'Act of God' while commonly known and used, is rather a misnomer - a shower of rain can be said to be an Act of God as much as a flood. A more appropriate term is *Vis Major* meaning an event against which no human foresight can provide and of which human prudence is not bound to recognise the possibility.

While the foregoing is the position with respect to third party liability in relation to reservoirs in general terms it is more or less obvious that any failure on the part of operators to comply with the provisions of the Reservoir Act legislation would be a strong point in a claimant's case for loss or damage resulting from escape of water. This would still be the position notwithstanding that in the new Act there is a sub-section to the effect that the Act is not to be taken as conferring on any person a claim to damages in respect of a breach by the Undertakers of their obligations under the Act. It seems doubtful in view of the general absolute liability or equivalent in the case of escape of water from reservoirs whether this sub-section is worth much.

The main provision in the new legislation for breach of obligations under the Act is that for the penalties set out much more fully than in the 1930 Act. The heading of the relative section in the new Act is rather forbidding — *'Criminal Liability of Undertakers and their Employees'*, so those concerned are duly warned! The author looks forward to the day when similar provision is made for civil servants in breach of statutory provisions.

It may, however, be of some consolation that liability for injury or damage in respect of the escape of water from a reservoir still depends upon proof that the injury or damage did in fact result from such escape, or in other words that the cause was in fact the reservoir. The liability does not extend to the complete insurance in effect against all escapes of water through a reservoir. This was illustrated in a case involving the North of Scotland Hydro-Electric Board, Stirling v the Board, in 1965.

The circumstances were, briefly, that in 1962 the Claimant's farms lying below the Board's Torr Achilty reservoir were innundated by heavy floods which exceeded the capacity of the reservoir, Loch Achonachie, and which on discharge through the flood gates overtopped flood embankments erected along the River Conon many years before the hydro-electric scheme, the area having a history of flooding back to before 1839. In light of this history of flooding the claimant had lodged objections to the hydro-electric scheme when it was promoted in 1951, and eventually these objections were withdrawn on the Board giving assurance that they would take remedial measures or pay compensation should additional water enter the River Conon from the Scheme causing damage to the embankments. The significant word here is *additional.*

Also in the statutory Scheme there was included a usual section providing that where the Board cause the water in any of the works of the Scheme to be discharged into any available stream or watercourse the Board shall do as little damage as may be, and shall make full compensation for any damage sustained in consequence of such discharge.

At first sight the Claimant appeared to have a case for the damage caused by the water coming through the reservoir. However the legal authorities had to be checked and the facts of the flooding had to be investigated.

Taking first the section of the Scheme with respect to compensation for damage caused by discharge of water from the Board's works, it was brought out for the Board that the legal effects of such provision in a statutory scheme had been established in various authorities. Lord Avonside referred to various earlier cases and particularly to that of the Metropolitan Board of Works v McCarthy in 1874, where Lord Penzance had said *'It may reasonably be inferred the Legislature in authorising works and thus taking away rights of action which the owner of the land would have had if the works had been constructed by his neighbour, intended to confer on such owner a right to compensation co-extensive with the rights of action of which the statute had deprived him. But on no reasonable ground, as it seems to me, can it be inferred that the Legislature intended to do more and actually improve the position of the person injured by the passing of the Act'.*
Another more recent case cited by Lord Avonside was Marriage v East Norfolk Rivers Catchment Board (1950). The judge stated that to maintain the action at all it was necessary that the plaintiff showed that the deposit by the Catchment Board of soil raised in dredging the river above the plaintiff's mill was in the nature of an actionable wrong, which if done by a person without statutory powers would have entitled him to sue. A much earlier case was also cited, that of Rhodes v Airedale Drainage Commissioners (1876), where the statutory provisions for compensation were similar to the section in the North of Scotland Board Scheme. It was held that damage which would but for the Act be actionable is alone the subject of compensation under the Act.

In the North of Scotland Board case the attempt to base the claim on the compensation section in the Scheme was accordingly rejected, but it required this check on the past authorities. It may well be a question whether the meaning of the section as judicially brought out in these earlier cases since 1874 was appreciated, and one can understand that to the man in the street the provision for compensation for damage caused by discharge of water from the Board's works meant just that. But the question really was had the Board caused the discharge, and it was against this question that the Board's defences were prepared to try to prove that the flood damage would have been sustained even if the Board's works had not been there, that is that the damage was caused by natural phenomena. Strictly there was no onus in law on the Board to produce such proof, it being the Claimant's obligation to prove his case but while this was maintained in the Board's defences it was thought advisable that the Board, as a public body, try to provide a technical appraisal of the flood conditions. Although this involved considerable investigation it proved to be justified. In the result Lord Avonside accepted the technical evidence produced on the Board's side that the presence of their works seemed to have modified the incidence of flooding and that the flood conditions which had occurred were not exceptional in conditions of nature.

It also assisted in the Board's successful defence that at an early stage in the case witnesses for the Claimant tried to maintain that the flood gate operations had created a standing wave — an unnatural cause. It was not too difficult for the Board to provide expert evidence demonstrating that such a phenomenon had been fully allowed for in design, and this attempt was abandoned by the Claimant

The obvious lessons from this case are that continuous adequate records of flood control be maintained, that there be always available for application a comprehensive procedure for dealing with flood conditions, that all the equipment is available in working order, and last but not least the control staff have the necessary instructions and are able to cope. It may be noted in conclusion that no question was raised by the claimant of any negligence on the part of the Board or their staff. If there had been any doubt on this point then it might well have been difficult to have had the claim rejected. A reasonable standard of skill is required in law in professional and technical matters and failure involves the risk of claim for damages. It may be taken for granted that this legal requirement would be strictly applied in the event of any discharge of water from a reservoir causing injury or damage.

INSPECTION OF THE RESERVOIRS OF THE NORTH OF SCOTLAND HYDRO-ELECTRIC BOARD

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SYNOPSIS

The Paper discusses the policy developed for statutory, overseeing and maintenance inspections of the 76 reservoirs of the Board. Fairly extensive instrumentation of the dams is carried out and the necessity, frequency and value of these readings, are reviewed and related to Inspection Policy.

INTRODUCTION

The Board own 76 reservoirs certified under the Reservoirs (Safety Provisions) Act 1930. There is a wide variety of types including gravity, buttress, arch and prestressed concrete dams, as well as earthen and rockfill embankment dams although, due to the nature of the terrain and economics at the time of construction, concrete gravity and buttress dams predominate. A comprehensive system of inspection, including instrumentation, has been developed and this is complemented by the maintenance service which is operated by Board staff.

After some ten years' experience of dam instrumentation, the opportunity was taken in 1971 to assess carefully the results and discuss the value of this instrumentation with nearly all the Panel Engineers employed on statutory inspections of the Board's reservoirs. In general the Board's conclusions were closely in accord with the reactions and recommendations of those Engineers. This review led to the adoption of an inspection and instrumentation policy for the future, the objectives of which were:

a) A stricter and more frequent safety inspection of the reservoirs by the Board's own experienced civil engineers. In 1971 this was expected to be a requirement of the proposed revision of the Reservoirs (Safety Provisions) Act 1930.

b) Over the long term, to obtain a better knowledge of the condition and behaviour of all the Board's dams and thereby to identify and monitor any dams where there were defects or uncertainties with respect to safety.

c) To deploy more efficiently and effectively the limited staff available for safety work, as well as to make the optimum use of their skills and experience.

PREVIOUS INSPECTION PROCEDURES

Up to 1971, three main safety procedures were undertaken with respect to Board reservoirs viz visual inspections, leakage readings and dam instrumentation.

VISUAL INSPECTIONS

During the first 23 years of the Board's operations, until 1971, in addition to the statutory inspections carried out by Panel Engineers, a system of inspection by the Hydro Group personnel (mechanical and electrical engineers and waterman) had been developed which was supplemented with inspections by experienced civil engineers of the Civil Division as occasion required

LEAKAGE MEASUREMENTS

Leakage measurements at the dam and immediately downstream are a key criterion for detecting short term deterioration in reservoirs. They are still regarded as of first importance in the duties of a waterman and were a statutory requirement under the Reservoirs (Safety Provisions) Act 1930.

DAM INSTRUMENTATION

Up till 1961, only minimal instrumentation had been installed on the Board's dams although instrumentation was being seriously considered for the Lednock Dam which is sited in an active seismic region and for the Monar and Chliostair arch dams which were in the design stages during the late fifties. After the Fréjus and Malpassat disasters comprehensive inspections were undertaken by Panel Engineers of the Consultants engaged on development of the Board's Schemes in the Highlands. In addition, geological re-appraisals were made in many cases and safety procedures reviewed. As a result of these inspections, level and collimation instrumentation was generally installed on dams with heights above 23m (75 ft.). With new dams full instrumentation was specified, including temperature and stress measurements and the extensive use of pendulums. By 1971 all this instrumentation had been installed and up to ten years of readings had been obtained and analysed. The extent of this instrumentation is described by Curtis and Provan (1).

ASPECTS AFFECTING FREQUENCY OF OVERSEEING INSPECTIONS AND INSTRUMENTATION CYCLES

In developing the policy for inspection and instrumentation, the following aspects were taken into account in defining the frequency and extent of inspection and instrumentation. In addition, during recent years it has been the practice to discuss with and ask Panel Engineers to give their recommendations on the frequency and extent of instrumentation in making their Reports on dam inspections.

HEIGHT OF DAM

Views on the height at which instrumentation becomes necessary range from 15 to 30m above general foundation level, the need for instrumentation normally increasing with the height of the dam. Previously a figure of 23m had been used in the Board. To date, some twenty dams over 23m in height and sixteen dams less than 23m have been instrumented.

FOUNDATIONS AND GEOLOGICAL CONDITIONS

Foundations, including abutments, are of primary importance and such features as the type and condition of the rock, faults, intrusions etc are carefully studied in determining the frequency of inspections.

TYPE OF DAM

The type of design has a major influence on the surveillance and instrumentation necessary to maintain safety standards, eg arch dams and embankment dams need particular attention.

ECONOMY OF DESIGN (OR SAFETY FACTOR)

In any particular type of dam there can be significant differences with respect to safety in the margins which have been incorporated in the designs. It would be invaluable if the critical design parameters and assumptions could be defined in the Reservoir Record Book held for each dam.

AGE OF DAM

The age of a dam can affect the materials used (e g deterioration of concrete), construction techniques (degree of mechanisation for earthen and rockfill dams) and design technology. There would seem to be merit in considering several categories viz :

a) Old dams greater than 75 years old - fairly important developments in design and construction of dams took place around the turn of the century

b) Intermediate dams - less than 75 and more than 30 years old

c) Recent dams - less than 30 and more than 10 years old - dams built since the end of the last war have incorporated important advances in technology.

d) New dams - less than 10 years old - again more sophisticated methods of analysis and design and advances in construction have taken place during the last decade.

PURPOSE OF THE RESERVOIR

A reservoir of a pumped storage scheme is subjected to more onerous conditions than a reservoir used for water supply or straight hydro-electric generation. This is particularly the case with earthen embankments and rockfilled dams where a continuous 'fill and draw' action is imposed.

RISKS PRESENTED BY THE RESERVOIR TO POPULATIONS DOWNSTREAM

The size, situation and distribution of population downstream of a reservoir all affect the assessment of the potential risk presented by a dam. Although this factor was not mentioned in previous British legislation it has now been recognised in the Discussion Paper on Reservoir Floods Standards (2).

ECONOMIC VALUE

Although the economic value of a reservoir is not referred to in the Act, this could increase the frequency of inspection for commercial reasons.

CURRENT INSPECTION POLICY

Following careful assessment of the above aspects, the following inspection and instrumentation policies were put forward for consideration in 1971 and subsequently adopted. Since that time the policies have been in force; the instrumentation results have been informative and valuable and the inspections very worth while.

VISUAL INSPECTIONS

Six types of visual inspection are carried out :

a) Statutory inspections at not more than ten year intervals by Panel Engineers. These Engineers are selected such that they have had no association with the design or construction of the particular reservoir and are changed every ten years so that a completely independent and fresh assessment of the reservoir(s) can be made where the reservoirs act in cascade. The Engineers are appointed for the inspection of all the reservoirs in a valley or basin. This policy has led to fairly radical appraisals being made and the bringing to light of a few aspects which were not suspected as being unsatisfactory.

b) Overseeing inspections carried out by experienced members of the Civil Division at intervals of two to five years; this period varies according to the conditions and behaviour of the dams and their importance with respect to third party and Board interests. The Engineers employed are all Chartered with not less than 10 years aggregated dam experience.

   The inspections are reported on standard format and are made available to Panel Engineers for their statutory inspections, along with instrumentation results. In general, the aspects reviewed and detail of these reports are the same as those covered by the Panel Engineers. For normal reservoirs, it is customery to carry out two overseeing inspections between statutory inspections; for reservoirs with abnormal behaviour more frequent inspections are undertaken.

c) Maintenance inspections - by members of Civil Division

d) Inspections at three to twelve month intervals by Engineers (generally electrical or mechanical) in the Generation Groups with guidance from Civil Division. These are more connected with maintenance aspects, but any important safety matters which arise are highlighted in the reports back to Civil Division.

e) Weekly inspections by Watermen as far as is practicable.

f) Regular exercising and inspection of all gates and valves by Generation Group staff at nominally not more than yearly intervals - these inspections are recorded in writing.

Overseeing inspections and instrumentation are undertaken after major floods, earth tremors or unusual events. For arch dams and large dams in pumped storage schemes subject to rapid variations in head, annual inspections are generally carried out.

The results of this policy for statutory and overseeing inspections have broadened the surveillance of the reservoirs, with an improved reporting procedure, defined more clearly and regularly maintenance requirements and, in addition, brought to light a number of aspects capable of improvement. This latter result, along with the extensive operating experience now available from schemes, has led to the setting up of a Floods Study Group with the objectives of improving the operating regimes and safety aspects of the schemes. An outline of the work of the Group and the results of its studies are described by Reynolds(3), Jarvis (4), and the Author (5).

LEAKAGE

There has been no change in the Board's policy with respect to leakage measurements.

INSTRUMENTATION

The desirable ultimate objective is to instrument all dams of any significance with the most modern and best instruments available and to maintain all instrumentation facilities in first class order so that they may be commissioned immediately as and when required.

Several of the Board's dams, although certified, control the level in existing lochs over a range of only a few metres and as such the structures are very simple and do not require instrumentation. The reading of the instruments and assessment of results can be very heavy in skilled labour, particularly where the dams are remote and situated in areas of inclement climate. Dams shoud therefore not be instrumented unless the need can be substantiated. A very careful scrutiny was carried out of each of the Board's dams to select those dams where instrumentation was required and would be of value in monitoring their safety.

New dams should be fully instrumented as part of construction and regular readings taken over a sufficiently long period of time to gain confidence in the dam's behaviour (eg five years). Thereafter, if the instrument readings are consistent and the dam behaviour predictable, the number of instrument readings can be reduced and the period between readings increased. Typically, readings would be reduced from an annual cycle to,say, one cycle every three to five years, and from reading all instruments to reading selected instruments, but with the visual inspections maintained as outlined above. As the dam becomes older and/or deteriorates it may well be necessary to increase the frequency of the instrumentation cycle. For dams in uncertain conditions or with erratic behaviour more extensive instrumentation is installed and the readings taken more frequently.

It is advantageous to carry out instrument readings over a one year cycle, at the same seasonal times in the cycle, since the readings are affected significantly by water level and, in the case of concrete dams, by concrete temperature. Instrumentation cycles are therefore carried out on a 4 x three monthly basis or 2 x six monthly basis over the particular year selected as occasion requires and conditions permit. The general policy for the frequency and type of instrumentation adopted in the Board is as follows : shown on Table 1.

Classification	Condition and Behaviour	Type of Dam	Age of Dam	Instrumentation
1	Normal	Arch Large	First five years of life	Full instrumentation quarterly
2	Normal	Arch Small	After five years	Full instrumentation twice a year
3	Normal	Gravity & buttress	First five years of life	Full instrumentation twice a year
4	Normal	Gravity & buttress	After five years	Full instrumentation for one cycle every fifth year
5	Normal	Embankment (Earth or rockfill)	First five years	Levelling twice a year, pore pressures monthly (earthfill only)
6	Normal	Embankment (Earth or rockfill)	After five years	Levelling once a year, pore pressures twice a year (earthfill only)
7	Unexpected or uncertain behaviour or where special circumstances apply	Gravity & buttress	First five years of life	Full instrumentation twice a year. Pendulums four times a year (if installed)
8	as above	Gravity & buttress	After five years	Pendulums four times a year (if installed) otherwise instrumentations of key stations twice a year. Full instrumentation for one cycle every 3/5 years or as considered necessary.
9	as above	Embankment (Earth or rockfill)		As per classification 5

Table 1. Frequency and type of instrumentation.

Full instrumentation (where installed) comprises vertical and horizontal movement measured by collimator, level, pendulum and joint gauges, as appropriate, with determination of supporting parameters (eg temperatures and strains).

Where leakage occurs, weekly measurement is required along with an observation of the condition of the flow (clear, muddy etc). This leakage measurement is a statutory requirement.

The policy is modified as and where necessary in the light of the characteristics and behaviour of each particular dam. Where instrumentation has been installed after the dam has been brought into service, frequent readings are taken until the behaviour pattern of the dam has been established.

Details of the instruments used and results obtained over the last fifteen years are described by Curtis (1) (6).

The inspection and instrumentation section is responsible to the Senior Hydro Engineer of the Division and is run by an experienced Chartered Civil Engineer with two surveyors and assistance in the summer from vacation staff.

MAINTENANCE

At the same time as an overseeing inspection is undertaken for safety purposes, the Engineer assesses the condition of the works of the reservoir and makes recommendations for any maintenance work that may be required. This maintenance work is categorised into priorities and the body to undertake the work defined :

E = Essential — must be carried out immediately

I = Important — must be carried out during the season. (Maintenance work in the Highlands is much affected by climatic conditions and is planned on a seasonal basis from March to October inclusive)

D = Desirable — to be carried out as soon as convenient

As far as possible, as much work is carried out by the Civil squad of each Hydro Group (responsible for one or two major schemes, on average, consisting of up to twelve reservoirs and fifteen power stations in total) since this is the most effective and economic way of undertaking the work. Each squad of six to eight men comprises typically foreman, mason or bricklayer, carpenter, painter(s) and usually three or four labourers/watermen, and is paid on a productivity basis. Where the tasks are too large or complicated to be tackled by these squads, the work is put out to contract to a number of selected contractors. The Civil Division is responsible for the overall direction of this work and, in the case of the larger and more complicated projects, for the design, preparation of documents and drawings and supervision of the work.

A maintenance programme of work is drawn up in collaboration with the Groups in the autumn of each year, cost assessed, budget approved and the design and contract stage carried out in the winter in readiness for carrying out the work in the following season. The work in Civil Division is serviced by a small section comprising one experienced and Chartered Engineer, one Graduate Engineer, one Technician Engineer and a Technician, supplemented as necessary by effort from a general pool when work demands. The section is responsible to the Senior Hydro Engineer of the Division. Maintenance inspections of reservoirs and all hydraulic structures are undertaken by the staff of this section. A more detailed description of the maintenance procedures in force is given by Curtis and Provan (1).

CONCLUSIONS

The proposed revisions of the Reservoirs (Safety Provisions) Act 1930 in 1971 introduced an overseeing inspection requirement. In anticipation of this new requirement, the Board accepted this requirement and for the past four years has been undertaking these inspections on their reservoirs with considerable and worth while advantages in terms of safety and maintenance. At the same time, a careful review of their experience on dam instrumentation over the previous ten years or so led to the definition of a new policy linking dam instrumentation and reservoir inspection. Greater emphasis has been placed on the inspections by qualified engineers with a somewhat reduced emphasis on instrumentation, but with no decrease in staff employed on these aspects. Overseeing inspections are undertaken by experienced and Chartered Civil Engineers generally at two to three year intervals and are complementary to the statutory inspections, as well as being integrated with the more frequent and routine inspections carried out by other engineers and watermen in the Generation Groups.

Where dams have behaved consistently and predictably over five years or more, the frequency of instrument readings on the dams has been reduced and the available effort re-deployed on dams not yet covered by instrumentation. The ultimate aim is to have all dams of significance and all uncertain dams covered by instrumentation which is always available and in good condition. Instrument readings are concentrated on those dams with unexpected or irregular behaviour or which are subject to unusual service conditions, with only the occasional instrument cycle being undertaken between statutory inspections on the remaining dams.

By implementation of this policy it is believed that a more effective and thorough safety inspection of the reservoirs is being achieved and more efficient use made of the experienced staff available in the Division for this vital work.

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EFFECTIVE OPERATION OF RESERVOIR LEGISLATION

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SYNOPSIS

The effective operation of reservoir legislation is discussed in the context of British circumstances and the provisions of the Reservoirs Act 1975. A brief summary of the historical background to reservoir legislation in Britain is followed by consideration of those factors which are particularly relevant to British legislation and leads to a comparative study of the rate of occurrence of recorded incidents involving reservoirs.

In the concluding section of the Paper proposals for strengthening the Act by statutory provisions are put forward.

An Appendix lists details of recorded major incidents since 1800.

INTRODUCTION

Provision for proper control over the design, construction and subsequent safe operation of reservoirs is today acknowledged as a responsibility of the State. Legislation to this end therefore operates in most industrialised countries and also, to a lesser extent, in those developing countries carrying through large water resource development programmes.

The nature of the appropriate legislation differs greatly, reflecting national circumstances and, in many instances, the approach to the philosophy of government of the country concerned. Legislative provisions range across a broad spectrum extending from the centralised authoritarian system, with all powers in the hands of the State, through systems which delegate control to duly constituted corporate authorities, to the British system, where only the fundamental objective and the minimum of essential provisions are defined.

British reservoir legislation is unique in that the Reservoirs Act 1975 is not written in terms of any central authority bearing other than indirect responsibility for operation and enforcement. Ultimate responsibility is vested in the individual panel engineer, duly appointed in terms of the Act. A feature of British practice is thus the commendable degree of flexibility inherent in the system, the independence and authority of the individual panel engineer being highly prized. The British approach thus evades the two extremes of decision by remote State official on the one hand and indecision by committee — with all that that can entail in the context of reservoir safety — on the other.

To be properly effective, reservoir legislation must satisfy the following fundamental requirements:

1 Scope and objectives must be clearly defined

2 The detailed provisions must be unambiguous

3 Adequate provision must be made for policing and enforcement

The single most valuable feature of British legislation is the principle of individual and undivided responsibility devolving upon the panel engineer. Matched against the three requirements detailed above, however, it is arguable whether the Reservoirs Act 1975 is, as it stands, entirely adequate.

RESERVOIR LEGISLATION IN THE UNITED KINGDOM

HISTORICAL BACKGROUND

The Bradfield (or Dale Dyke) Dam, one of several under construction or planned to supply Sheffield, failed in 1864 shortly after completion, with very heavy loss of life and damage to property. At the conclusion of the inquest on the 245 victims the Coroner's jury added a rider to the effect that all dams and reservoirs should, by Act of Parliament, be subject to 'frequent, sufficient and regular' inspection. This recommendation was endorsed in the Report and, the following year, by a Parliamentary Select Committee. No other incidents involving loss of life occurred for a further 60 years, however, and no Parliamentary action was taken.

April 1925 saw the failure of a small reservoir at Skelmorlie, and a cascade failure of the Eigiau and Coedty dams above the village of Dolgarrog followed in November of the same year. Both incidents involved loss of life, the figure of 16 deaths attending the latter incident being less than might otherwise have been expected due to the fortuitous attendance of most inhabitants of the village at a cinema located on higher ground.

The Skelmorlie and Dolgarrog incidents following each other in rapid succession compelled Mr Edward Sandeman to express his serious professional concern over reservoir safety in a letter to *The Times*, in December 1925. In this letter Mr Sandeman drew attention to the essential recommendations of the Select Committee of 1865, which included, *inter alia*, requirements for deposition of plans and records and for periodic inspection by competent persons.

Under the impetus of these events and spurred, no doubt, by the St Francis disaster of 1928, the Reservoirs (Safety Provisions) Act duly appeared in 1930. Enacted after a somewhat protracted gestation period, the 1930 Act met the most essential of those principles enunciated by the 1865 Select Committee.

REPORT OF THE ICE COMMITTEE 1966

In the period 1960 — 1965 increasing international concern over reservoir safety became apparent in the wake of disasters at Malpasset, Vajont and elsewhere. The concern was shared by many, if not the majority, of those British engineers charged with operation of the 1930 Act, the deficiencies of which were by then all too apparent. Council of the ICE therefore appointed a Committee to review, and to make recommendations about, operation of the Act. The Report of the ICE Committee (1), published in 1966, adhered to the more valuable principles underlying the 1930 Act but included some 12 major recommendations requiring changes in the legislation. Of particular interest was one recommendation to the effect that certain detailed records should be sent to 'an appropriate Government Department', which would be charged with policing operation of the provisions of the Act regarding inspection and maintenance with the assistance of the then statutory River Authorities.

THE RESERVOIRS ACT 1975 (2)

Following discussion the revised Reservoirs Act, after suffering from low Parliamentary priority rating, has now appeared on the Statute Book. Certain of the points made in the ICE Committee Report have been met in part or in full, but other important and far-reaching recommendations, including the following, have not been implemented:

1 Raising the size of reservoir covered to 45.5 t.c.m. (10 x 10^6 gallons) (The figure in the 1975 Act is now 25 t.c.m.)

2 Design details, information and criteria to be submitted with Certificates.

3 Copies of all Certificates and Reports to be held in a central repository and to be available for inspection.

4 Extension of the Act to embrace storage of fluids other than water (e.g. PFA slurry lagoons).

One significant step taken with the 1975 Act, however, is creation of the statutory appointment of 'supervising engineer' interposed between inspecting engineer and undertaker and charged with routine supervision between statutory inspections. Other significant developments which can be traced to the suggestions of the ICE Committee include those provisions limiting tenure of panel appointment by engineers to five year terms, specifying the independence of the inspecting engineer from the undertaker, and revised requirements as to submission of Certificates and Reports.

The Act provides (Section 2) that responsibility for registration of large reservoirs and enforcement of the Act shall devolve upon the recently reorganised local authorities, which number some 46 in England, 12 in Scotland and eight in Wales - total, 66 enforcement agencies.

LEGISLATION IN THE CONTEXT OF BRITISH CIRCUMSTANCES

GENERAL CIRCUMSTANCES - NUMBERS AND AGES OF DAMS

The circumstances regarding prevailing numbers, age and type of dams, and density of population must have a bearing upon the framework of the reservoir legislation deemed appropriate to any country.

Major problems arise in Britain, stemming not only from the fact that there exists a large stock of dams within a small, industrialised and densely populated country, but also from the particularly high incidence of elderly and old dams within that stock. In this context it is also relevant that the vast majority of British dams, probably 85% of the total stock, are earthfill embankments.

One failing of the 1930 Act in practice was that the number of reservoirs to which its provisions applied was not - and is not - known. Informed opinion places the total at some 2000, and a pilot sample census suggests this to be a not unreasonable estimate. A proportion of those 2000 reservoirs have not yet been inspected in terms of the 1930 Act, and in view of their antiquity and type it can only be surmised that the condition of many is likely to be less than satisfactory.

The only reliable published statistics on British dams relate to those 'large dams' listed on the ICOLD Register (3), i.e. to those dams exceeding 15 m in height, or between 10 m and 15 m high and noteworthy for other reasons. The present (1975) total of dams in this category is approximately 470. Of this total over 65% predate World War II, while 40% date back to the nineteenth century or even earlier. It is reasonable to assume, in view of the early influence of the Industrial Revolution and other factors on the rate of reservoir construction, that an estimate of the proportion of the total stock of dams built pre-1900 is more likely to approach 65%. Figure 1 indicates the steady growth in actual numbers of large dams over the period since 1800. Superimposed on this plot is what can only be an 'inspired guess' as to the likely growth in total number of dams of size subject to the Reservoirs Act over the same period.

It can thus be seen that the large majority of British dams must date from periods when knowledge of dam engineering and of the related sciences of geotechnics and hydrology was minimal. The accountant may assess the life of a dam at 80 years for financial purposes. What, however, is a realistic assessment of the structural life of any dam, elderly or modern? Safe service life is clearly not indefinitely long, and it is thus evident that a very considerable effort will be required to maintain in sound condition a stock of dams whose average age is climbing inexorably upwards.

SERIOUS INCIDENTS INVOLVING DAMS

It is appropriate to review the incidence of recorded serious incidents involving British dams and, by implication, the safety of reservoirs. It has been argued that the 1930 Act proved adequate to its task by virtue of the fact that no incidents involving loss of life occurred during the 45 years it was in force. That no disasters occurred may have been more fortuitous than a tribute to the efficiency of the Act, as past events at Lluest Wen (4) and elsewhere have revealed. Many incidents, some potentially very serious, have occurred, but specific information is in many instances lacking - due, in part at least, to lack of a central agency responsible for monitoring records of reservoir performance. At least an equal number of lesser incidents have probably gone totally unrecorded.

One list of British incidents dating back to 1810 has been published in the United States (5). It is necessarily incomplete, but an updated and extended summary of incidents based on that list and on 'Lessons from Dam Incidents' (6) is presented as an Appendix. Forty-six distinct incidents of sufficient gravity to have been recorded are listed, of which at least 15 were actual failures of dams and six resulted in loss of life. Detailed statistical analysis is clearly impossible on grounds of sample size and due to lack of accurate information on total number of dams 'at risk'. On the assumption that the estimated curve of total numbers plotted on Figure 1 is at least approximately correct, an admittedly crude comparative analysis of incident frequency can be derived. Incident frequency, calculated on the basis of data contained in Figure 1 and in the Appendix and expressed in terms of 'reported incidents/1000 dams at risk/decade', is plotted in Figure 2 for the years between 1840 and 1970.

Examination of Figure 2 shows, not unexpectedly, a very high incident rate in the middle-to-late nineteenth century, a period when the engineer grew bolder than the state of Victorian technology warranted.. The decline after 1870 may reflect greater conservatism and also the known tendency for failure to occur either in the first few years after construction or else after many years. After 1890 the incident rate is sensibly constant prior to commencing a further marked upward trend to 1940. Subsequent to 1950 the rate is once again relatively steady, albeit at a level above that existing pre-1920. It is necessary to bear in mind, however, that many incidents since 1930 onwards have occurred during construction of earthfill embankments, in a period when geotechnical science was in its infancy and rapidity of construction increasing.

No positive inference can be drawn regarding the efficacy of the 1930 Act, the issue being blurred by the influences of increasing average age of dams at risk on the one hand, balanced by the probable benefits of improved understanding of geotechnics and hydrology on the other. As long-term behaviour and stability of embankments is only now beginning to be properly understood, it deserves to be noted that a very large number of dams date from within that period of peak incident rate in the nineteenth century shown in Figure 2.

STEPS TO EFFICIENT OPERATION OF THE 1975 ACT

The new Reservoirs Act is likely to be the subject of debate among interested engineers for some considerable time. Few engineers are likely to argue that the Act matches the ideal in reservoir legislation but equally, few engineers would agree in all respects as to what does constitute that ideal, either in format or in respect of detailed provisions.

The Act is open to possible criticism on several grounds other than its failure to adopt those suggestions of the ICE Committee Report outlined earlier. The Act is not, for example, selective in its application. The criterion of 25 t.c.m. capacity has a certain appeal on grounds of obvious simplicity. It is suggested that some other factors, including those of potential hazard or of risk, could logically have been considered also, perhaps along the lines of the 'Proposed Guidelines for Safety Inspection of Dams' published by the Corps of Engineers (7).

A further criticism can be levelled at the Act in respect of its selection of the new local authorities to be the enforcement agencies. It is submitted that the likelihood of widely varying standards of enforcement arising from the involvement of 66 different authorities apart, few of those authorities are likely to have either resources, or indeed sufficient sense of priority, to ensure adequate policing of reservoirs under their jurisdiction.

Such issues, however, can now only be considered in terms of achieving optimum results within the new Act as it now exists on the Statute Book. It is therefore necessary to consider how efficient operation of the 1975 Act can best be assured within the framework provided.

In the Author's view two significant improvements would be achieved if additional statutory provision were made with respect to:

a) establishing a central registry, and

b) policing operation of the Act.

It is considered that the Act may leave scope, as it stands, for additional steps to be taken to achieve the above aims under the provisions of Section 5 (Section 5: 'The Secretary of State may by statutory instrument make regulations for prescribing anything which is under this Act to be prescribed (and in this act 'prescribed' means prescribed by regulations so made)')(1)

THE CENTRAL REGISTRY

It is submitted as desirable that *all* available essential records of those dams subject to the Act be held on file by a central agency. Information held on microfilm or otherwise should if possible, include all available record drawings; Certificates; geological, geotechnical and hydrologic appraisals; Reports of inspections, and as detailed a history of each reservoir as can be compiled from the available information. A potential hazard index (8) should also be derived and detailed in an 'incident pack' alongside copies of the most fundamental drawings and records presented in standard format.

The creation of the machinery for such a registry and the establishment of the necessary register of information itself is clearly a fairly major undertaking when starting from scratch. Once organised, however, the registry could be maintained with comparatively little ongoing effort. This would suggest that preparation of the initial data bank could be handled by appropriate consultants under contract. The registry itself should obviously be housed within the Department of the Environment with, in the case of Scotland, a subsidiary registry within the Scottish Development Department.

EFFECTIVE POLICING OF OPERATION OF THE ACT

Attention has been drawn to problems likely to arise from 66 authorities being the appointed enforcement agencies. It is suggested that policing operation of the Act should be the function of one central authority as an arm of central government. The local authorities could then be left, under the provisions of Section 2, as the enforcement agency in a purely legal sense, rather than as policeman and prosecutor combined.

The role of the central authority would be one of supervision, the primary goal being the maintenance of a uniform standard of compliance with the intentions of the Act. To this end, the central authority would be responsible for collating and checking receipt of all statutory returns from local authorities and from inspecting engineers.

The form which such a central authority could best take is open to debate, but in the Author's view a case can be made for the creation of a small central Inspectorate of Reservoirs. At this point it must be clearly stated that such an Inspectorate is not seen as impinging in any way on the authority or the independence of the panel engineer appointed for a particular reservoir. The position of the panel engineer must, and could, be safeguarded in all respects.

It must also be made clear that far from advocating the creation of yet another vast bureaucracy, the proposed Inspectorate could, within the context of reservoir legislation and the one Act applicable, be a compact body of suitably qualified personnel.

ORGANIZATION

Carrying the two suggestions to their logical conclusion, it is obvious that control of the central registry should be vested in the proposed Inspectorate. The Inspectorate and registry would form a unified and independent department answerable direct to the Secretary of State for all matters pertaining to the safe operation of reservoirs. Various additional roles for the department can be foreseen, including the possibility that particular problems could be identified from a study of the statutory data submitted to the Inspectorate and, if felt to be of sufficient importance, could be the subject of sponsored research in the Universities [(9)]. The effect of this, in terms of ensuring a positive long-term feedback of research findings to panel engineers operating the Act, is potentially most valuable.

CONCLUSIONS

It is considered that while the Reservoirs Act 1975 represents a step forward on previous legislation covering reservoir safety its value could be enhanced by the creation of some form of Inspectorate incorporating a central registry for all statutory information relating to reservoir safety. The optimum form for such provision is clearly open to debate. It is the underlying concept which is suggested as being of importance. The suggestions are therefore put forward with a view to stimulating debate and opening discussion on ways in which the intentions underlying the new Act can best be realised. The advantages likely to accrue from strengthening the Act along the lines indicated are considered to heavily outweigh foreseeable objections.

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APPENDIX

RECORDED MAJOR INCIDENTS: DAMS IN THE UNITED KINGDOM (1800 - 1975)

No	Year	Dam	Type	Height	Years After Completion	Nature of Incident
	1800					
1	1810	Driggle	E	--	---	+F
2	1810	Blackbrook	E	--	---	F (1st dam F 1799)
3	1815	Whinhill	E	--	19	F (and 1835)
4	1819	Plymouth (nr)	E?	--	---	+F
5	1835	Whinhill	E	13	14	F
6	1843	Glanderston	E	--	---	F?
7	1850	Woodhead	E	30	1st impound.	I
8	1852	Rhodeswood	E	29	UC	I
9	1852	Holmfirth (Bilberry)	E	--	---	F
10	1854	Torside	E	38	1	I
11	1864	Bradfield (Dale Dyke)	E	30	--	+F
12	1864	Harrington	E	--	---	I?
13	1866	Oldham (nr?)	E	--	---	F?
14	1867	Kelso (nr?)	E?	--	---	F?
15	1870	Lliw (Lower)	E	12	3	F (and 1894)
16	1870	Rishton	E	--	---	+F
17	1877	Edgbaston (nr?)	E?	--	---	F?
18	1878	Clackmannan (nr)	E?	--	---	F?
19	1879	Swansea (nr)	E	25	12	F
20	1889	Radcliffe	E	--	---	I?
21	1894	Lliw (Lower)	E	14	27	I (and 1897)
22	1897	Lliw (Lower)	E	14	30	I
	1900					
23	1905	Kettering	E	14	UC	I
24	1915	Walshaw Dean	E	24	1st impound.	I (and 1940)
25	1923	Belmont	E	25	96	I?
26	1924	Cowlyd	E	14	2	I
27	1925	Skelmorlie	E	5	---	+F
28	1925	Eigiau	CG	11	17	F cascade failure
	1925	Coedty	E	11	1	+F
29	1927	Bartley	E	--	UC	I
30	1927	Alston No 1	E	>8	UC	I
31	1929	Broomhead	E	31	1	I
32	1930	Bottoms	E	12	80	I
33	1936	Blain y Cwm	E	18	1st impound.	I
34	1937	Chingford	E	10	UC	I
35	1937	Abberton	E	17	UC	I
36	1943	Muirhead	E	23	UC	I
37	1948	Knockendon	E	28	UC	I
	1950					
38	1951	Harrogate	E	9	82	I
39	1957	Blackbrook	CG	30	51	I (3rd dam)
40	1957	Glendevon (Upper)	CG	45	2	I
41	1962	Blithfield	E	16	9	I
42	1962	Tittesworth	E	15/31	104/UC	I (during raising)
43	1962	Greenbooth	E	36	UC	I
44	1969	Lluest Wen	E	20	76	I
45	1970	Warmwithens	E	10	---	F
46	1971	Upper Creggan	E	10	---	F

Notes: 1. CG = concrete gravity dam
E = earthfill embankment
UC = under construction
I = incident
F = failure

2. Information regarding precise identity type, height or age not readily available in many instances.

3. + denotes loss of life

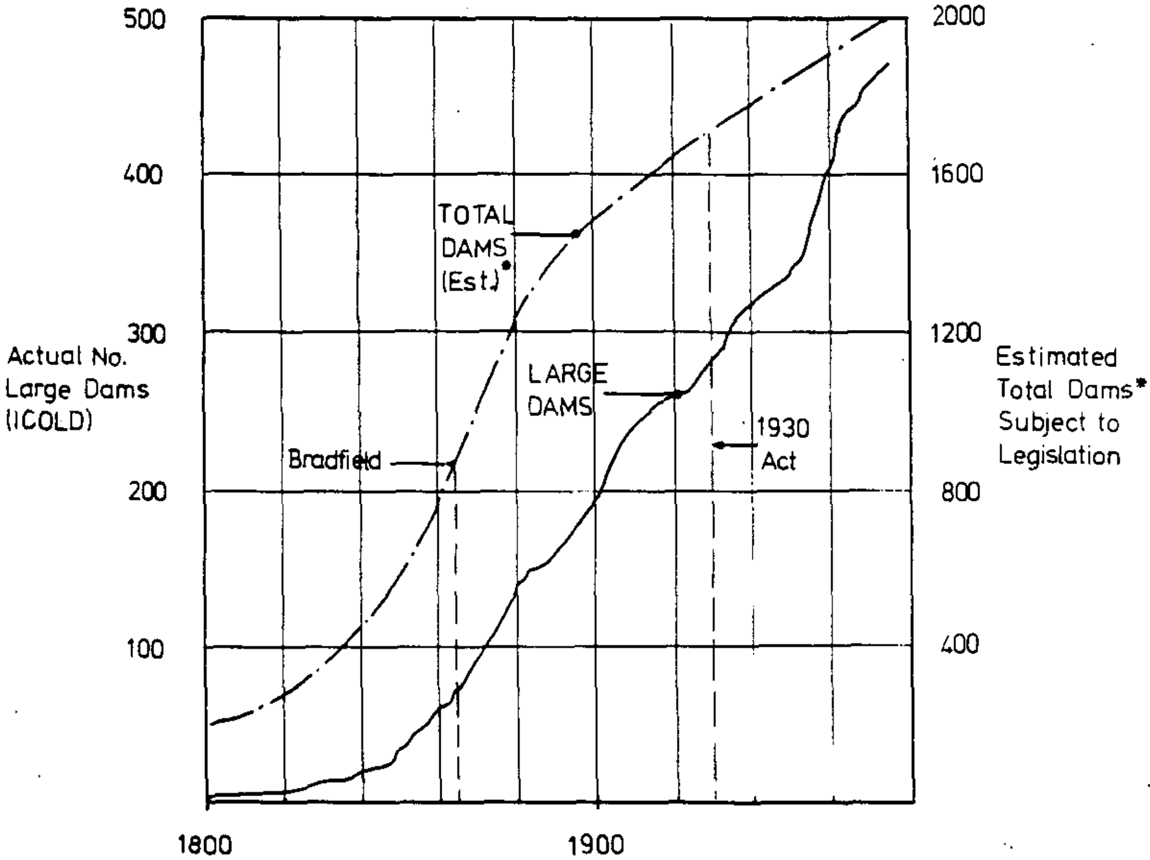

FIGURE 1 : Cumulative Growth of Stock of Dams* in Britain
1800-1975

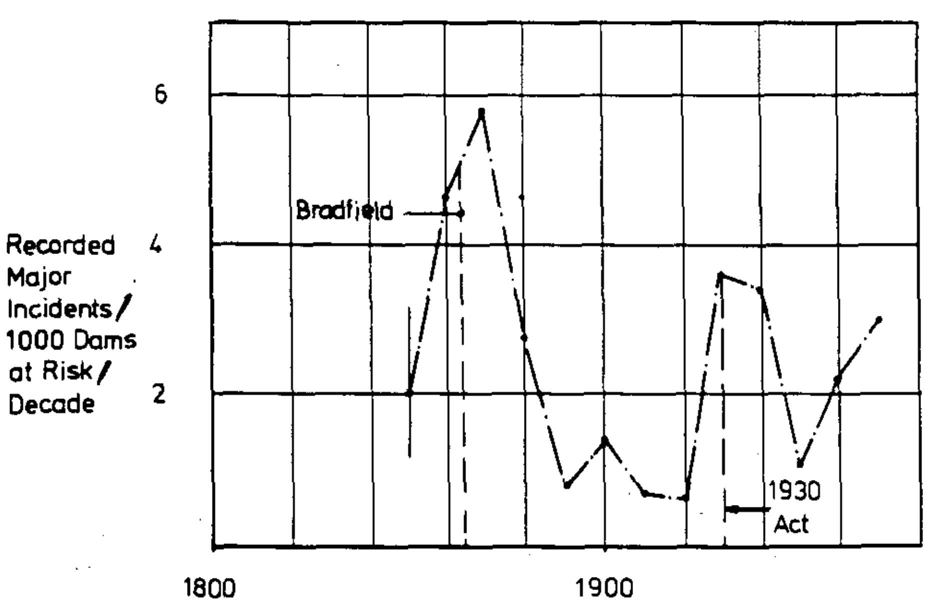

FIGURE 2 : Estimated Frequency of Major Incidents to British Dams*
1840-1970

* Inclusive Non-Impounding and Service Reservoirs over 25 t.c.m. Capacity

FEDERAL LEGISLATION AND ACTIVITIES FOR DAM SAFETY IN THE UNITED STATES

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SYNOPSIS

United States Public Law 92 - 367, enacted 8 August 1972, authorized the Secretary of the Army, through the Chief of Engineers, to undertake a national programme of inspection of dams. It also required that the Chief of Engineers compile an inventory of dams in the United States and prepare for the Congress recommendations for implementing a comprehensive national programme for dam safety, including assignments of responsibilities. Activities of the Corps of Engineers pursuant to this authority have consisted of making a survey of existing practices and capabilities pertaining to dam safety in the United States; preparing guidelines for the safety inspection of dams; and formulating recommendations for implementing a comprehensive national dam safety programme.

INTRODUCTION

PURPOSE AND SCOPE

This paper describes existing federal legislation regarding the safety of dams and reservoirs in the United States and reviews activities of the Chief of Engineers, U.S. Army, under the authority of that legislation. It presents a discussion of the need and justification for a comprehensive national dam safety programme and outlines the essential elements of such a programme. Also included are the conclusions of the Chief of Engineers developed from his study of the dam safety problem, together with his recommendations for consideration by the Congress.

BACKGROUND

Federal legislation concerning dam safety in the United States, Public Law 92 - 367, was enacted on 8 August 1972, following a series of events which focused public concern on hazards posed by water storage dams. These events were the near-failure of the Lower Van Norman Dam during the 9 February 1971 San Fernando, California earthquake; the Buffalo Creek, West Virginia, disaster of 26 February 1972; and the dam failure at Rapid City, South Dakota, in June of 1972. Public concern over these incidents was of such gravity that Congress was moved to react quickly through the enactment of federal legislation.

PUBLIC LAW 92 - 367

Public Law 92 - 367 authorized the Secretary of the Army, acting through the Chief of Engineers, to undertake a national programme of inspection of dams for the purpose of protecting human life and property.

APPLICABILITY

The law applies to all dams, including appurtenant works, which impound or divert water and which are: (1) 7.6 metres (25 ft) or more in height from the natural bed of the stream or watercourse measured at the downstream toe of the barrier, or from the lowest elevation of the outside limit of the barrier, if it is not across a stream channel or watercourse, to the maximum water storage elevation or, (2) have an impounding capacity at maximum water storage elevation of 61.7 t.c.m. (50 acre ft) or more. Excluded are any barriers which are not in excess of 1.8 m (6 ft) in height, regardless of storage capacity, or which have a storage capacity at maximum water storage elevation not in excess of 18.5 t.c.m. (15 acre ft), regardless of height.

INSPECTIONS

The Chief of Engineers was authorised to inspect all dams in the United States with certain exceptions. The inspection provision of the Act has not been fully implemented due to restricted funding. The Corps of Engineers has, however, tried to meet all requests from the states for help relating to dam safety. Since passage of the Act, the Corps has responded to 10 requests for assistance in developing or strengthening state dam safety programmes and to about 130 requests for technical assistance and advice regarding measures to eliminate or mitigate hazardous conditions.

REPORT TO THE CONGRESS

The Act required the Secretary of the Army to submit to the Congress a report containing 1: an inventory of all dams located in the United States, 2: a review of any inspections made, and 3: recommendations for a comprehensive national programme for the inspection and regulation for safety purpose of dams of the Nation, and the respective responsibilities which should be assumed by federal, state, and local governments and by public and private interests.

The Corps' principal efforts pursuant to Public Law 92 - 367 have been devoted to compilation of the inventory and to activities related to developing recommendations for a comprehensive dam safety programme, including a survey of existing practices, capabilities and regulations regarding dam safety and the preparation of recommended guidelines for performing safety inspections of dams.

The Chief of Engineers report was completed and forwarded to the Secretary of the Army by letter dated 16 May 1975.

DAM INVENTORY

The National Dam Inventory compiled under the authority of Public Law 92 - 367 contains data on approximately 49,000 dams.

COMPILATION OF DATA

The inventory data were obtained under direct state supervision in 37 states or territories and in 14 others by private engineering firms under contract with the Corps of Engineers. In three cases the Corps developed the state inventories in-house. Inventory data for dams under the jurisdiction of federal agencies and dams licensed by the Federal Power Commission were compiled by the agency of jurisdiction and furnished to the Corps of Engineers for inclusion in the national inventory. All data have been stored on magnetic tape in a master file for easy access and use by all interested agencies and organizations.

INVENTORY DATA

The inventory data consist of the name of the dam or impoundment; the river or stream on which the dam is located; the type of dam; year completed; purpose of the dam; the height and maximum storage capacity; the name, population and distance from the dam to the nearest downstream city, town or village; the downstream hazard potential; the owner of the dam; and the congressional district in which the dam is located.

The inventory data indicate that only 18% of the dams have been inspected under existing state or federal authority. The downstream hazard potential indicating potential loss of life or property resulting from failure of the dam or mis-operation of facilities has been recorded for 76% of the dams inventoried. Based on these data, estimates indicate that 40% of existing dams (about 20,000) are so located that failure would result in loss of life and important property damage.

SURVEY OF EXISTING LEGISLATION, CAPABILITIES AND PRACTICES

The survey to learn of each state and federal agency's capabilities, practices, and regulations regarding the design, construction, operation and maintenance of dams was made by questionnaire. All 50 states, three territories and 12 federal agencies responded.

STATE AUTHORITIES

The responses of the states and territories indicated that 11 have no laws regarding any aspect of dam supervision. The legislative authority of many of the others is considered inadequate from the standpoint of establishing all activities necessary for dam safety. Twenty-four indicated that their current dam safety regulations do not fully meet present needs and 20 stated that they have active plans to modify existing regulations. Forty-one states and territories require a permit or license to be issued prior to construction of a private dam; 36 require the review of plans and specifications prior to construction; and 23 provide on-site inspection by state personnel during construction. Thirty-two states have authority to perform safety inspections after construction; however, in most cases firm schedules are not maintained. Many perform an inspection only when information is received that a hazardous condition might exist or under other special conditions.

The responses further indicated that 54,195 dams are under state supervision and that $4,371,379 is the determinable approximate annual budget directly related to dam and reservoir supervision by state authorities. This number of dams is larger than that included in the inventory because in some cases state regulations encompass impoundments which do not meet the Public Law 92 - 367 definition of 'dam'.

There are great differences among the states in carrying out their responsibilities to the public for the safety of dams built within their jurisdictions. Many have inadequate statutes and others have inadequate staffs to enforce the statutes. Few states, if any, including those with adequate dam safety regulations, are prosecuting a program comparable to that recommended by the Chief of Engineers in his report to the Congress. Principally this is due to the lack of funds and staff to perform inspection duties.

FEDERAL AGENCIES

Three federal agencies, the Federal Power Commission, the Corps of Engineers and the Mining Enforcement and Safety Administration of the Department of the Interior, have regulatory authority over certain non-federal dams.

The Forest Service, Corps of Engineers, Bureau of Indian Affairs, Bureau of Land Management, Bureau of Reclamation, Bureau of Sport Fisheries and Wildlife, U.S. Geological Survey, Mining Enforcement and Safety Administration, Federal Power Commission, International Boundary and Water Commission and Tennessee Valley Authority have existing authority concerning the responsibility for safety of dams they own or operate or which are located on lands under their supervision. An exception to this is the case of dams located on lands within the National Forest under privately owned easements for which the Forest Services does not have specific jurisdiction. These federal agencies discharge their safety responsibilities with varying capabilities and dissimilar manners. In general on-site inspection by agency personnel is required during construction and periodically thereafter. However, three agencies have no definite schedule for periodic inspections. The dams under the jurisdiction of the Bureau of Reclamation, the Tennessee Valley Authority, the International Boundary and Water Commission and dams licensed by the Federal Power Commission, which are excluded from the inspection provisions of Public Law 92-367, and those owned and operated by the Corps of Engineers are presently being inspected periodically in accordance with existing agency regulations.

The responses to the questionaire further indicate that the determinable annual budget directly related to dam and reservoir safety supervision by federal agencies is approximately $5,617,500 and that 16,473 dams, approximately 11,000 of which are not included in the definition of 'dam' contained in Public Law 92 - 367, are under the jurisdiction of federal agencies. Five federal agencies indicate that their current regulations pertaining to dam safety do not fully meet present requirements and plan to modify existing regulations to improve and strengthen dam safety programmes.

JUSTIFICATION FOR A NATIONAL PROGRAMME

Any artificial barrier which impounds or diverts water creates a potential hazard to human life and property located downstream. The potential hazard is created by the possibility of a sudden release of water as a result of failure of the dam or mis-operation of the discharge facilities. It is, therefore, essential that dams be properly designed, constructed, operated and maintained to reduce the risk of failure or mis-operation throughout the life of the dam.

The United States Committee on Large Dams (USCOLD) conducted a survey of dam failures within the United States, prior to 31 December 1972. Results were recently published jointly by the American Society of Civil Engineers and USCOLD in a report entitled 'Lessons from Dam Incidents, USA,' 1975. The report lists 39 major dam failures which resulted in complete abandonment of the dam, 37 major failures where the damage was successfully repaired and the dam placed in operation again, and some 104 accidents which were prevented from becoming failures by expeditious remedial work or operating measures, such as drawing down the pool. In addition, some 171 other types of accidents or damages were reported which were considered not to affect the immediate public safety.

The protection of the health, safety and welfare of its citizens has long been recognized as a governmental responsibility and, therefore, the protection of human life and property from potential hazards created by dams and the water they impound is deemed a governmental concern. Although the adequacy and safety of dams owned and operated by numerous governmental agencies, public and private organizations, and private individuals are the obligation of the owners, the need for governmental regulation of some type to ensure that the owners' obligations are properly carried out is considered evident. Similar social needs have long been recognized in building codes, elevator inspections, bridge inspections, and many other facets of modern life where governmental regulation has been found necessary to protect the public.

ELEMENTS OF A NATIONAL PROGRAMME

The prosecution of a dam safety programme in the United States would require suitable additional legislation and regulations to define the scope of supervision and authority over dams and to empower designated agencies (regulatory agencies) to carry out regulatory functions. These functions would pertain to the design, construction, operation, maintenance, enlargement, modification, removal or abandonment of dams and reservoirs.

MODEL LAW

USCOLD developed the text of a proposed model law which was distributed in 1970 to the governors of the 50 states and their respective officials responsible for supervision of dams. The published 'Model Law for State Supervision of Safety of Dams and Reservoirs' is considered an excellent example of adequate legislation covering state authority over dams and reservoirs. The model law provides for the safety supervision of dams and reservoirs in all stages of design, construction, operation, maintenance, enlargement, modification, removal or abandonment. It was intended that the model law be modified as required to fit the needs of individual states.

REGULATORY FUNCTIONS

Based upon the provisions of the USCOLD Model Law and the reported experiences of state and federal agencies and private engineering organizations, the regulatory agency should perform the following functions to insure the adequacy of dams and reservoirs:

(1) Review and approve the plans and specifications to construct, enlarge, modify, remove or abandon a dam or reservoir. The plans and specifications should be supported by engineering data, including design analyses, in sufficient detail to permit the regulatory agency to determine the safety, adequacy and suitability of the proposed action before the action is undertaken.

(2) Perform periodic inspections during construction for the purpose of ensuring compliance with the approved plans and specifications .

(3) Upon completion of construction, issues a certificate of approval. The owner should be required to submit to the regulatory agency the "as-built" drawings and other construction records such as foundation data and geological features, properties of embankment and foundation materials, concrete properties and construction history, for review and approval. Upon approval of data and the determination of the adequacy of the structure, the certificate of approval would be issued permitting the owner to store water.

(4) Investigate the dam and reservoir at least every five years to determine their continued safety. The investigations should be detailed, systematic, technical inspections and evaluations to analyze and evaluate the hydraulic and hydrologic capabilities, structural stability and operational adequacy of the project features in order to determine if the dam and reservoir constitute a danger to human life or property.

(5) Issue notices when appropriate to require the owner of the dam and reservoir to perform necessary maintenance or remedial work, revise operating procedures or take other actions including breaching of the dam when deemed necessary. The regulatory agency, under the police power of the state, should enforce these notices and when emergencies exist have the work performed under its direction and supervision if the owner fails to do so.

INITIAL INSPECTIONS

Implementation of a periodic investigative programme for existing dams should be accomplished by an intensive initial inspection effort. Essentially, this would consist of a programme (Phase I inspections) to identify expeditiously those dams which may pose a hazard to human life or property. It is believed that a five year period would be a reasonable interval in which the estimated 20,000 dams in the significant and high hazard potential categories could be inspected effectively. Where results of the initial inspection indicate the need for additional investigations, the owner should be required, under supervision of the regulatory agency, to have the additional in-depth investigations (Phase II) performed by qualified personnel.

INSPECTION GUIDELINES

Recommended technical guidelines for performing safety inspections of dams were drafted to outline principal factors to be weighed in identifying deficiencies and hazardous conditions. They were developed with the assistance of state agencies, other federal agencies, professional engineering organizations, and private engineers.

The guidelines provide for two phases of investigation. Phase I would be an inspection to assess the general condition of the dam and determine the need for any additional engineering investigations and analyses. It would consist of a visual examination of the dam and a review of available engineering data, including operating records.

Phase II investigations would be performed where the results of the Phase I inspection indicate the need for additional investigations and studies. Phase II would include, as required, all additional visual examinations, measurements, foundation exploration and testing, materials testing, hydraulic and hydrologic analyses, and structural stability analyses deemed essential to evaluate the safety of the dam.

The inspection guidelines do not establish rigid criteria or standards but rather provide guidance on the scope of investigations, both Phase I and Phase II, and present reasonable evaluation factors with which to compare existing conditions. Safety must be evaluated in light of peculiarities and local conditions at a particular dam and in recognition of the many factors involved, some of which may not be precisely known. This can only be done by competent, experienced engineering judgement, which the guidelines are intended to supplement and not supplant. Conditions found which do not meet guideline recommendations are to be assessed by the investigator as to their importance from the standpoint of the involved degree of hazard. Many deviations will not compromise project safety while others will involve various degrees of risk, the proper evaluation of which will afford a basis for priority of subsequent attention and possible remedial action. As experience is gained with use of the guidelines the need for revisions will become evident and a continuing updating effort is foreseen.

SUPERVISION OF A NATIONAL PROGRAMME

The division of jurisdiction of the federal and state governments over the water resources of the United States has been fairly well defined by existing laws and precedent and, therefore, discussion of dam supervision is simplified by placing the dams in two broad categories, i.e., Federal and Non-Federal Dams.

FEDERAL DAMS

There are approximately 5,500 dams conforming to the height and capacity requirements of Public Law 92 - 367 in this category.

While various alternatives regarding the responsibility for supervising the safety of federal dams are available, only one alternative is deemed acceptable and practicable. This alternative would require the federal agency owning or operating the dam, owning the land on which the dam is located, or which has existing regulatory jurisdiction over the dam to assume the responsibility. Those federal agencies currently lacking technical expertise and capabilities to conduct the programme could utilize upon request the existing expertise and capabilities of the Corps of Engineers, Bureau of Reclamation or others to assist in initiation and implementation of a dam safety programme.

NON-FEDERAL DAMS

The non-federal dam category includes approximately 43,500 dams and reservoirs conforming to the height and capacity requirements of Public Law 92 - 367. These dams are owned and operated by numerous state and local governmental agencies, public and private organizations and private individuals and are not under the jurisdiction of the federal government. In general the protection of the health, safety and welfare of its citizens has been recognized as a state responsibility in the United States. Under this concept, as has been noted, the majority of the states and territories have enacted Laws recognizing the police powers of the state over the regulations of dams. Hence, a national programme for dam safety should recognize the primacy of state authority in regulating non-federal dams and seek to strengthen, not supplant, existing state efforts.

While some local governmental agencies and public and private organizations may have a technical capability adequate for supervising dam safety, it appears appropriate that the final authority and responsibility rest with the State agency designated by the Governor. Thus the responsibility of local government and public and private interests will be primarily dictated by the legislation of each state. All dam owners, whether public or private interests, have the ultimate responsibility for safe structures and such responsibility needs to be adequately defined by state legislation.

CONCLUSIONS OF THE CHIEF OF ENGINEERS

The conclusions of the Chief of Engineers as reported to Congress may be summarized as follows:

(1) A dam is a complex structure whose safety and continuing adequacy involve the ability of the structure to interact with its foundation in withstanding applied forces which are dependent upon many variable conditions. A high degree of professional engineering performance is required to ensure adequacy of design and construction. To further reduce the risk of failure or mis-operation, continued surveillance of the dam and appurtenant works is necessary to detect conditions of significant structural distress or operational inadequacy and to provide a basis for timely initiation of restorative and remedial measures if necessary.

(2) Based on past history, recommendations of states and technical societies, and the general public's apprehension as recognized by Public Law 92 - 367, it is evident that a comprehensive national dam safety programme is needed in the United States to provide for consistent regulation of design, construction, operation and maintenance of dams.

(3) Authorities should be established to regulate the design, construction, operation, maintenance, enlargement, modification, removal or abandonment of dams and reservoirs, and such authorities should be provided with adequate personnel, financing and powers to enforce their rules and regulations and accomplish their regulatory functions. The jurisdiction of regulatory agencies should cover all existing and future dams as defined by Public Law 92 - 367.

(4) The adequate design, construction and surveillance of dams require professional engineering services. To insure that competent, professional attention is directed toward the question of dam safety, supervision of the recommended dam safety programme should be placed under the jurisdiction of federal and state agencies.

RECOMMENDATIONS OF THE CHIEF OF ENGINEERS

In his report to Congress, the Chief of Engineers submitted the following recommendations:

(1) A comprehensive National Dam Safety Program should be implemented. State responsibility, under the police powers of the state, to protect the health, safety and welfare of its citizens should be recognized and all states and territories should be encouraged to prosecute dam safety programmes encompassing all dams not under federal authority.

(2) Implementation of a National Dam Safety Programme should be followed immediately by the inspection over a reasonable and practicable time period of all existing dams which have a hazard potential of high or significant level.

(3) Those federal agencies possessing technical expertise and capabilities in the field of dam design and construction should be authorized to furnish technical assistance and guidance to the states, upon request, concerning state programmes and the elimination or mitigation of any hazardous conditions which may be found by the states.

(4) Federal agencies owning and operating dams, owning the land on which dams are located, and the agencies having existing regulatory jurisdiction over dams should prosecute the recommended dam safety programme for the dams under their jurisdiction.

(5) The Chief of Engineers, U.S. Army should be provided authority and funds to maintain current the National Dam Inventory.

OVERSEAS PRACTICE ON DAM SAFETY LAWS AND INSPECTION PROCEDURES

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SYNOPSIS

The Author reviews overseas practice on dam safety laws and inspection procedures. The empowering legislation is described together with the constitution and responsibilities of those bodies, statutory and otherwise, charged with implementing the acts. The influence of this legislation on project appraisal, supervision of design and construction, final certification and subsequent inspection of new dams and on the discovery, classification, rehabilitation or demolition of old dams is discussed. The efficiency of the legislation to achieve its objectives is assessed with regard to the penalties for non-compliance and the funding of statutory inspectorates. Attention is focussed on those areas such as the U.S.A. and elsewhere which have similar problems to those of the U.K. due to their high population densities and a long industrial heritage, which has resulted in a legacy of old dams,very often of rudimentary construction.

INTRODUCTION

In industrialized countries where the demand for water and hence the number of dams has increased dramatically over the last century, technological advances and population growth and redistribution have made apparent the need for increased dam safety surveillance. With our increased understanding of geotechnical engineering, hydrology, and seismology, we are now better able to determine whether such structures constitute a hazard to human life or property. The design of many existing dams requires to be re-evaluated in the light of this new knowledge, in particular with regard to spillway capacity. In parallel with this technological evolution, sociological changes have resulted in the State taking greater responsibility for the health, safety, and welfare of its citizens. Together these developments have resulted in increasing attention being given to the establishment and strengthening of dam safety programmes. Even now the existing practice on dam safety laws and inspection procedures covers the entire spectrum from microscopic examination to benign neglect. This paper reviews experience in those countries such as the U.S.A. and elsewhere which have high population densities and a long industrial heritage which have resulted in a large number of new dams and a legacy of old dams, very often of rudimentary construction. Special attention has been given to recent American legislation and to inspection procedures for existing dams.

EMPOWERING LEGISLATION

Legislation governing the responsibility for design, construction, operation and where necessary rehabilitation of dams, falls into three categories:

A) Legislation prescribing the role of dam building and operating agencies, such as the Federal Power Commission and the Bureau of Reclamation in the United States.

B) Public Statutes of provincial or state legislatures.

C) Public laws of national application enacted by Central Governments.

In most instances laws falling into categories A and B have provided the precedent for national legislation. The enactment by central government of public statutes with national application is a recent phenomenon in many countries. The Reservoir (Safety Provisions) Act, 1930, in Britain was one of the earliest attempts to regularize dam safety procedures. In France, interministerial circulars were in existence dating from 1927 and 1928, but it was only with the creation of the Standing Technical Committee of Dams by the Ordinance of June 13, 1966 and subsequently that of May 16, 1968, that central administration was achieved, (Carlier (1)). In Sweden provision for licensing of new dam construction projects has been laid down by central government since 1918 in the Water Rights Act of that year, (Scherman and Nylander (2)). In South Africa central control is exercised through the Water Act, No. 54, 1956 and subsequent amendments No. 77, 1969 and No. 36, 1971, (DuPlessis (3)).

In the United States legislation regulating licensing and inspection procedures has traditionally varied widely from state to state. The supervision and inspection responsibilities of individual states has been summarized in the U S Register of Large Dams (4). These responsibilities are usually exercised through state organizations such as the Department of Water Resources in California or the Water Rights Commission in Texas. An attempt has recently been made to rationalize the differing requirements of each state with the passing of the National Dam Safety Act, Public Law 92-367, in 1972. This statute constitutes a comprehensive dam safety programme and empowers the Department of Defence through the Corps of Engineers (5) to provide guidelines for the safety inspection of all non-federally owned dams. Government dam building agencies such as the Bureau of Reclamation and the Federal Power Commission which have established monitoring procedures are outside the jurisdiction of this statute. It is also recognized that individual states may wish to define the requirements of the national programme in administrative rules or regulations rather than in the basic law. In this manner the new Federal legislation may be better integrated with the existing practice of different states and at the same time facilitate any possible modifications in the future.

The three main features of all central government legislation reviewed are :

1 LICENSING POWER - The approval of the site and the proposed design and method of construction.

2 REGULATION AND INSPECTION PROCEDURES - The power to enter and inspect directly, or through agents, any dam within the term of the act or to require periodic reports from accredited engineers.

3 ENFORCEMENT ROLE - Through the police powers of the central authority to order any repair or alteration as may be necessary, and in default of any rehabilitation to execute such repairs and to recover the costs incurred thereby.

STATUTORY AUTHORITIES

The definition of some body charged with implementation of legal provisions is implicit in all the national legislation reviewed, although the nature of this agency is varied by the extent to which this responsibility may be delegated. In the United States, while the Dam Safety Act, Public Law 92-367, 1972, authorized the Secretary of the Army, through the Corps of Engineers to initiate the programme, it seems not improbable that the inspection responsibility will be delegated to the engineering staffs of the respective states, (Buehler (6)).

In France, the Standing Technical Committee instituted by the Ordinance of June 13, 1966, consists of eight members, six of them civil servants from the three ministries concerned with dam safety, namely the Ministry of Industrial and Scientific Development, the Ministry of Town and Country Planning and the Ministry of Agriculture, and two technical experts from outside the public sector. The design of all new dams must be approved by this central body. The inspection of existing dams falls to regional technical staff of the individual ministries depending on the purpose of the structure.

In South Africa, the Water Act, No. 36, 1971, makes the Minister of Water Affairs, through the Department of Water Affairs, responsible for the certification of all dams. No continuing programme of inspection has been required for non-governmental dams. However, more recent regulations propose mandatory inspections and closer controls on dam operations. To this end three Standing Committees of experienced engineers from both public and private sectors have been suggested. These comprise a permanent technical committee, an advisory committee and a reviewing board.

In Sweden, the statutory authority created by its Water Rights Act, 1918, resides in a special Water Rights Court which constitutionally includes highly qualified civil engineers as adjudicators and may co-opt others as required. This authority must specifically include an economic and environmental

TABLE 1 LICENSING AND SUPERVISION

Authority	Licensing	Actual Dam Design	Supervision of Construction	Final Certification	Post Construction Inspection	Supervision And Approval of Remedial Measures
State of California, U.S.A. Division of Dam Safety, Department of Water Resources	Yes	Outside Consultants	Permanent presence	Yes	Yes	Yes
Federal Power Commission, U.S.A.	Yes	Outside Consultants	Occasional. Approval of outside R.E.	Yes	Every 5 yrs.	Yes
Tennessee Valley Authority U.S.A. Division of Engineering Design	Yes	Outside Consultants	Permanent presence	Yes	Every 5 yrs.	Yes
Bureau of Reclamation, U.S.A.	Yes	Yes	Periodic	Yes	Every 6 yrs.	Yes
Corps of Engineers, U.S.A.	Yes	Yes	Permanent presence	Yes	Every 5 yrs.	Yes
Electricité de France	Central Agency	Outside Consultants	Permanent presence	Central Agency	Every year by Central Agency	Central Agency
Republic of South Africa Department of Water Affairs	Yes	Yes of Outside Consultants	Approval of Outside R.E,	Yes	Every 5 yrs.	Yes
Swedish State Power Board	Central Agency	Yes	Permanent presence	Central Agency	Every 3 yrs.	Yes

review of the consequences of the construction when weighing the approval of the site and the design. A summary of the general responsibilities of some overseas dam licensing and inspection authorities is shown in Table 1.

GENERAL REQUIREMENTS OF OVERSEAS DAM SAFETY LAWS

The Model Law proposed by the United States Committee on Large Dams (7) is a useful example of the format of dam safety legislation. The first requirement is the definition of a dam for purposes of the Act, the jurisdiction usually applying to any artificial barrier or appurtenant works which does or may impound or divert water, and which falls into a size classification defined by one of two criteria:

(i) Height of the dam from lowest point of the natural stream bed, measured at the downstream face, to the crest level.

(ii) Impounding capacity, usually defined as the total storage space in a reservoir below the maximum water surface elevation, including surcharge storage.

In many, but not all countries, levees, railway and highway fills, small conservation dams used for agriculture, and obstructions in canals used to raise or lower water therein are excluded. Also, early legislation was often restricted to on-stream dams but recent disasters such as the Baldwin Hills resevoir failure in Los Angeles, California, U.S.A. in 1963 have caused off-stream reservoirs, settling ponds and tailings lagoons to be included in the definition of a dam within the terms of most overseas safety legislation.

A further description of dams in the recent U.S. Legislation, into categories of hazard potential, is proving useful in establishing priorities for their national dam inspection programme (Table 2).

TABLE 2 HAZARD POTENTIAL CLASSIFICATION

Category	Loss of Life	Economic Loss
Low	None expected (No permanent residential structure would be affected)	Minimal (Occasional structure or agricultural)
Significant	Few (No rural communities or urban developments)	Appreciable (Notable agriculture, industry or structures)
High	More than a few	Excessive (Extensive communities, industry or structures)

The initial U.S. inventory reported 49,000 dams (April, 1975). The inventory was undertaken by the State in 37 states or territories, by private consulting firms in 14 states and by the Corps of Engineers in 3 states. Of the 49,000 dams, 29,000 were found to have low hazard potential, 11,000 significant and 9,000 high hazard potential, (Corns (8)). This qualitative approach has an attractive simplicity, although, in assessing possible loss of life how few is few could be the source of theological debate. An arbitrary value of five has been quoted informally by the Corps of Engineers.

INSPECTION PROCEDURES

The main burden of dam inspection is to preserve the safety of the dam during initial impounding and to ensure the maintenance and good repair of existing dams. Overseas legislation generally places the responsibility firmly on the dam owner/operator, whether this is a public body, a private person or a State agency. Where a State agency is the owner, inspection is invariably carried out within the organization. The construction, operation and maintenance of such dams may or may not be subject to a higher central authority. In general, where there is a unitary system of government it is, and where there is a federal system, it is not. The propriety of agency/owners inspecting their own dams has occasionally been questioned, but this review has shown that these bodies have been in the forefront of enlightened practice.

The 1930 Reservoir Act in Britain has ensured a measure of surveillance of existing dams which compares favourably with overseas practice. However, improved prediction of design basis events and concern about many structures hitherto excluded from the legislation will require a reappraisal of many existing dams. In this situation, in order to distinguish those dams presenting a safety hazard, and to enable their prompt rehabilitation within the existing framework, a system of priorities is essential. Faced with precisely this problem, the American legislators have opted for a two tier inspection system (Table 3). The Phase 1 investigation consists of visual inspection of the dam and a review of available engineering data without expensive explorations or analysis. Phase 2 would only be commissioned where the need was shown and would consist of such additional studies as were deemed necessary. There is general agreement abroad about the field inspection procedures to be adopted. The regulations following recent U.S. Legislation again provide a convenient checklist (Table 4a-c).

PROBLEMS OF IMPLEMENTATION

Legislation itself is not sufficient to ensure dam safety unless there are adequate funds and a corpus of engineering knowledge and experience which can be mobilized to undertake the task. This is the central problem of the kind of regulatory legislation described in this paper.

It is anticipated that the new American dam legislation will cost 30 million U.S. dollars per annum for Phase 1 inspection over the next five years and a further 275 million U.S. dollars for Phase 2 inspections (Corns (8)). Other estimates of total expenditure forecasts are 340 million U.S. dollars for Phase 1 and 700 million U.S. dollars for Phase 2 Wahler [9]. To put the scale of expenditure more in perspective the State of California, with a population of 30 million, land area of 156 000 square miles (cf. United Kingdom 55 million, 93 000 square miles) has some 1300 dams under its jurisdiction. The dam safety division functions satisfactorily with a staff of 60 and an annual budget approaching 2 million U.S. dollars. The rate at which active dam safety programmes have developed is largely a reflection of the level of funding. Adequate expertise has been shown to be available if the resources of both the public and private sectors of the profession are exploited.

CONCLUSION

The hazards posed by dams, and old dams in particular where the dangers are compounded by inferior methods of construction and outdated design criteria, is recognised internationally. This concern has been reflected in increased legislative activity. At present few countries, if any, are conducting dam safety programmes with the intensity of effort considered essential for public safety. This is a challenge to both the engineering community and to those holding the purse strings of central government. Clearly a system of priorities is essential, which will weigh the total social and economic costs against the attendant risks in a rational manner.

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TABLE 3 : PROPOSED REPORT ITEMS FOR TWO TIER INSPECTION after REFERENCES (5)(8)

PHASE I	PHASE II
1 Description of dam including regional vicinity map showing location and plans, elevations and sections showing essential project features, and size and hazard potential classifications.	1 Summary of additional engineering data obtained to determine the hydrologic capabilities and/or structural stability.
2 Summary of existing engineering data.	2 Results of all additional studies, investigations and analyses performed.
3 Results of visual inspection, including photographs and drawings.	3 Technical assessment of dam's safety including deficiencies and hazardous conditions found to exist.
4 Evaluation of operational adequacy of its reservoir regulation plan and maintenance of operating facilities pertinent to dam safety.	
5 Evaluation of hydraulic and hydrologic assumptions and structural stability.	
6 Assessment of the general conditions of the dam with respect to safety. Any further studies necessary should be listed together with an opinion about the urgency of such additional work.	
7 Indicate alternative remedial measures in operating and maintenance procedures which would correct deficiencies.	

TABLE 4(a) : TYPICAL FIELD INSPECTION ITEMS, after CORPS OF ENGINEERS (5)

Feature	Item	Detail	Feature	Item	Detail
Concrete dams	Concrete Surfaces	Determine Deterioration and serviceability	Embankment dams	Settlement	Localized or overall settlement
	Structural Cracking	Record and estimate cause of overstressing (i) Applied loads (ii) Shrinkage, temperature (iii) Differential settlement		Slope stability	Irregularities in alignment of slope and crest. Surface cracks.
	Horizontal and Vertical Movement	Record abnormal settlement, heave, deflection or lateral movement		Seepage	D/S face of abutment, embankment slopes and toes, embankment/structure contacts. D/S valley areas determine source of seepage and effect on dam's stability
	Joints	Conditions at abutments or embankment and construction joints			
	Drains	Serviceability of foundation joint or face drains		Drainage System	Check serviceability. Check embankment or foundation material is not being washed out. Functioning of monitoring systems.
	Water Passageways	Check erosion, cavitations, obstructions or leakage			
	Seepage	Determine source. Examine face, abutments, toes.		Slope Protection	General conditions of U/S face. Note erosion formed gullies and wave notches
	Foundations	Damage due to undercutting of downstream toe.			

TABLE 4(b) : TYPICAL FIELD INSPECTION, after CORPS OF ENGINEERS (5)

FEATURE	ITEM	DETAIL
Instrumentation	Head and Tailwater Gauges	Determine relationship with stream flow, uplift pressures, alignment and drainage system discharge.
	Horizontal and Vertical Alignment (Concrete Surface).	Examine and comment.
	Horizontal and Vertical Alignment, Consolidation, Piezometric observations and Uplift gauges (Embankments)	Examine and comment.
	Drainage Monitoring System	Establish normal relationship between reservoir elevations and discharge quantities.
	Seismic Recorders	Examine and comment.
Outlet Works	As spillway structures plus emergency drawdown facilities	Any constraints on operation to be recorded.
	Pipelines, Aqueducts, etc.	Where possible interior to be examined for erosion, corrosion, cavitation, cracks, joint separation and leakage.

TABLE 4(c) : TYPICAL FIELD INSPECTION ITEMS, after CORPS OF ENGINEERS (5)

FEATURE	ITEM	DETAIL
Reservoir Area	Shore Line	Examine for indications of major active or inactive landslide areas. Determine susceptibility of bedrock stratigraphy to massive landslides.
	Sedimentation	Check for any sudden increase in sediment load.
	Upstream Hazards	Check possible effects of backwater flooding.
	Run-off Potential	Examine drainage for altered run off characteristics, plus any U/S projects influencing dam safety.
	Downstream Area	Check urban or industrial developments have not altered hazard category.
Operation and Maintenance	Reservoir Regulation	Check actual operating practices under normal and emergency conditions do not endanger dam or human life or property.
Spillway Structures	Maintenance	Check efficiency.
	Control gates and operating machinery	Examine structural members, connections, hoists, cables and operating machinery under normal and emergency power.
	Approach and outlet channels	Check operations efficiency.
	Stilling basin	Report any condition impinging on efficiency. Record condition of D/S channel.

DISCUSSION : TECHNICAL SESSION 1

RESERVOIR LEGISLATION AND ITS IMPLEMENTATION

Session Chairman : Sir ANGUS PATON CMG BSc FRS CEng FICE FASCE
Senior Partner
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General Reporter : S F WHITE BSc(Eng) CEng FICE MIWES
Director, Water Engineering
Department of the Environment

CHAIRMAN : Sir ANGUS PATON

Ladies and Gentlemen, this is the first Technical Session, on 'Reservoir Legislation and its Implementation'.

My name is Angus Paton, and I am a partner in Sir Alexander Gibb and Partners. I have been involved with dams since the twenties and thirties, in North Wales, Scotland and Overseas, and I am a member of Panel 1.

I would also introduce the General Reporter, Stephen White, who is Director of Water Engineering, Department of the Environment, he is also a member of the Institution of Civil Engineers' Committee on the Reservoirs Act.

REPORTER : S F WHITE

I am going to deal principally with United Kingdom practice, and I propose to deal with the papers under a few headings. The headings are the *Ambit* of the legislation; the *Responsibility; Qualified Engineers; Enforcement*, followed by *Inspection* and by *Overseas Practice.*

With respect to *Ambit*, United Kingdom legislation applies to reservoirs of 25 tcm capacity of water held above the level of adjoining land and including such things as artificial lakes. Lagoons holding slurries, wastes and liquids other then water are excluded, and this is one of the points of dispute. In the United States, legislation is proposed for reservoirs nearly three times as large and the height of structure can also be taken into account.

Mr Moffat points out in Paper 1.4 that the 1975 Reservoirs Act is not selective in operation and fails to take account of potential hazards. He also finds the exclusion of lagoons unfortunate, and I would be interested to know if he can suggest any form of wording which would not bring in works which would be better dealt with under the Mines and Quarries Act.

Mr Ellis could perhaps say whether the new Act will in fact bring many more reservoirs under control than at present.

Secondly, the *Responsibility*. In the UK this rests firmly with the owner-operator, and failure to comply with the 1975 Act is now a criminal offence. Paper 1.6 confirms that this principle is general practice overseas. Simpson (Paper 1.2) is very interesting on this point and shows that fulfilment of statutory duties under the Act does not diminish liability of the reservoir owner. In English law 'unnatural' use of a property, that is the construction of a reservoir, leads to absolute liability. In Scotland, negligence must be proved, but the expected degree of care is exceptionally high and failure by an undertaker to have complied with the 1975 Act strengthens the case of the claimant. In this connection Simpson draws attention to the value to the undertaker of good records of his reservoir performance, and I want to particularly stress this point - I think it is most important.

What has not emerged in the papers is whether certification and statutory inspection by central bodies overseas affects the principle of owner liability. It seems to me one of the strengths of our system that it is the undertaker who is responsible for ensuring that inspection is carried out and certificates obtained. I would be most interested if Mr Willis or anyone else can enlighten us whether government or local authority overseas takes on any liability under inspection certification procedures. Mr Ellis would perhaps be willing to comment on the possibility of insurance of reservoirs to cover third party liability.

Turning to *Qualified Engineers*, Ellis sets out the different categories of engineers mentioned in the 1975 Act and deals with the certification procedures. In essence the Construction Engineer designs and supervises the construction or alteration of a reservoir and issues a Preliminary Certificate or Certificates to allow the reservoir to be filled to a prescribed level. If after three years he is satisfied the reservoir is sound, he shall give a Final Certificate specifying the level up to which the reservoir may be filled and any conditions under which it may be filled. If at the end of five years from the first Preliminary Certificate he has not issued the Final Certificate, he must give a written explanation for deferring issue. Simpson points out that difficulties can arise in such cases; presumably an Inspecting Engineer cannot be appointed when the Construction Engineer is still supervising. Another point is that the Inspecting Engineer must be completely independent of the Construction Engineer and of the undertaker. The first inspection must take place within two years of the date of the Final Certificate, and there must be further inspections at not less than ten-year intervals as before. There will now be two Inspection Certificates in each case, one by the Inspecting Engineer to state what needs doing and one by a 'qualified civil engineer', who may be an employee of the undertaker, that the necessary measures have been carried out. The 1975 Act also introduces a new category of statutory engineer - the Supervising Engineer - whose task is to carry out day by day oversight of the reservoir guided by the instructions of the Construction and Inspecting Engineers. He may call for periodic inspection and recommend action.

The Secretary of State for the Environment appoints engineers to the Panels related to different categories of reservoirs and the Act specifically requires him to consult the Institution of Civil Engineers. Engineers will be appointed for five years, after which appointments will be reviewed. Nothing in the new Act detracts from the principle that, on behalf of the Undertaker, or in some cases the Enforcement Authority, the heavy responsibility for reservoir safety rests wholly on the individual engineer. The work of the Construction Engineer is, however, checked by the Inspecting Engineer soon after completion.

As far as I can judge from the papers presented the UK system is uncommon and I would be interested to hear what other countries, if any, have adopted it. I would also like to hear whether in the United States, France or elsewhere there is any supervision of the qualifications and experience of engineers responsible for inspecting, constructing or certifying reservoirs and,if not,whether there should be.

The authorities responsible for *Enforcement* will be the County Councils and the Greater London Council in England and Wales, and the Regional and Island Councils in Scotland. Ellis points out that local authorities have other funtions concerned with public safety, and it is appropriate that an Act solely concerned with public safety is their concern. They will keep a register of all large raised reservoirs and information will be prescribed by Regulations by the Secretary of State. Registers will be open to inspection by the public and will include such information as the appointments of engineers, copies of Certificates and notices of intention to carry out works. In the event of non-compliance with the Act by undertakers enforcement authorities will have considerable powers to appoint engineers and take emergency action, provided these are in accordance with the recommendations of a 'qualified civil engineer'. No enforcement provision will breach the principle that technical matters are the sole responsibility of the qualified civil engineer. The function of the local authority is therefore administrative, and they will not need to employ technical staff.

Once a reservoir has been placed on the register it will be subject to the full provisions of the Act. It can only be removed from the register on a certificate by the appropriate qualified civil engineer that it has been so altered that it is no longer capable of holding 25 tcm. There are also provisions in the Act to cover the case where the owner proposed to discontinue the use of a reservoir, perhaps with the intention of bringing it back into use at a later date.

Moffat points out that the UK enforcement system is open to the criticism that with 66 different enforcing authorities there could be widely varying standards of enforcement. He suggests there should be a central authority, perhaps an Inspectorate of Reservoirs, responsible for holding the register and for overseeing enforcement arrangements. Ellis may wish to comment on this suggestion - would it require additional legislation for instance? Also, is it generally agreed with Moffat that such a body would not impinge on the independence of the Panel Engineers?

Kilkenny points out in Paper 1.6 that, though the UK favours the separation of the gamekeeper and poacher, his review has shown that countries that combine the roles tend to be in the forefront of good practice. He also gives his opinion that a committee system of enforcement such as that in France leads to indecision and confusion. I would be interested to hear from anyone who has had experience of this and similar systems.

On *Inspection* of dams, it is particularly interesting to have a paper written primarily from the undertaker's point of view. In many ways the North of Scotland Hydro-Electric Board have anticipated the Act in designing their inspection procedures, and my Department have appreciated the Board's advice in connection with the 1975 Act. In Paper 1.3 Johnson describes the Board's system of inspection. Inspecting Engineers are asked to recommend frequency of reading and extent of instrumentation having regard to height and type of dam etc. The Board have formal overseeing inspections by their own experienced engineers every two to five years, routine maintenance inspections by civil engineers, and weekly inspection by watermen if practicable - I am not sure what that means. This is supplemented by regular inspections by engineers of the Generation Groups and by formal regular exercising of all gates and valves. There are also *ad hoc* inspections after unusual events. Johnson stresses the importance of instrumentation; but he warns that reading and maintenance is expensive and instrumentation must be justified by needs and must be kept up to date. The Board have arranged that work recommended after inspection is classified into three groups :

1 *Essential*, that is to be carried out immediately;

2 *Important*, to be carried out that same season; and

3 *Desirable*, as soon as convenient.

I would be glad if he could give some examples of the work actually recommended under these categories. The Board's system seems to provide a full supervisory system, and it would be interesting to have Johnson's view about cost. Paper 1.6 gives us some figures of cost of implementation in the United States, mentioning California, not dissimilar in size of number of reservoirs to England. In California the State Dams Safety Division has 60 staff and an annual budget approaching $2 million. Perhaps it could be explained how much of this is attributable to the supervisory system as opposed to the oversight of design and construction and formal inspection.

The Department of the Environment is at present consulting the Institution of Civil Engineers about the Regulations which must be made under the Act. The views of the Symposium on the role of the Supervising Engineer, the form in which records should be kept, instructions that could be given by the Inspecting Engineer to the Supervising Engineer and any other related matters would be most helpful.

Finally, on *Overseas Practice*, I find Papers 1.5 and 1.6 complement each other. Morris sets out the present position in the United States: out of 53 States and Territories 11 have no legislation and 23 affirm that their powers are inadequate. Nevertheless 41 require permits or licences to construct a dam, 34 require to review plans and 23 provide on-site inspection. Thirty-two States have authority to inspect after construction, but the application of this authority is not uniform. The responsible Minister is the Secretary of the Army, and the US is in the process of formulating a comprehensive national safety programme. A law passed in 1972 required the Secretary to report to Congress giving an inventory of dams, to review existing procedures and to make recommendations for inspection and regulation. A report was submitted in May this year - a most commendable effort considering they have 49 000 dams. The United States Committee on Large Dams have developed the text of a Model Law which is considered to be an excellent example of suitable legislation. It is based on the principle that safety of reservoirs is analogous to social needs that are met by Building Codes etc. Federal and State agencies are responsible for inspection and enforcement. In this it is analogous to the philosphy of the present UK legislation in that enforcement is in the hands of local authority.

Proposals for the United States are :

1 A regulatory agency should review and approve plans supported by design analyses and data;

2 The agency should inspect during construction and issue certificates of approval;

3 Regular inspection should be carried out at least every five years;

4 Notice must be given to undertakers to carry out necessary works.

Partly because of the immense task ahead the data showed that only 18% of dams had been inspected under State or Federal authority – a two-phase system of inspection is recommended. The first phase would include assessment of the general condition of the dam and determine the need for additional data. The second would evaluate safety when all the information was available – this is a considerable change from what the UK has proposed.

Morris also points out that it is convenient in the USA to categorise reservoirs into Federal and non-Federal. The former could be the responsibility of the Federal agency owning the dam, assisted by expertise from, say, the Corps of Engineers. Non-Federal dams would be the responsibility of a State agency. I do not think this system would suit this country very well, but I would be interested to hear Mr Willis' views on this.

I have already mentioned my interest in third party liability. Would certification of a dam under the USA system absolve the owner in any way or transfer some of the liability to the state? Did the United States consider the British system as a possibility? If time permits, can anyone say more about the French system in particular, and what are the benefits and disadvantages of undertakers having to work under the supervision of a committee of six civil servants and two technical experts?

F G JOHNSON (North of Scotland Hydro-Electric Board) :

I congratulate our Reporter on a very perceptive and stimulating review of the essence of the papers of the first Session. I think the first question he raised in his Report was the Board's classification of maintenance and safety work into *essential*, *important* and *desirable*, for which he requested examples.

Essential — at the last Statutory Inspection of our Mullardoch Dam the emergency roller gate on the ground sluice, which is roughly 3 m square, failed to close when tested. This was immediately classified as essential, to be rectified immediately, and the following week divers were sent down, to find a 250 mm dia. tree trunk jammed across the guides of the gate.

Important — at the last overseeing inspection which we carried out at our Quoich Dam, a rockfill dam with a concrete upstream membrane, it was found that the bitumen filling between the joints in the concrete membrane was springing and becoming denaturised, and the bitumen surface coating of the surface of the concrete was breaking down under attack from the acid water. This work was classified as important and was carried out that season. Other examples of this category are the undermining of pitching, which must be rectified quickly otherwise there is a more rapid breakdown, or the rodding of uplift drains.

Desirable — these are often cosmetic jobs such as repair of superficial frost damage on bridge parapets, painting of upstream face of dams, chip and tar spray of access roads etc.

The second point raised was the cost of running our reservoir safety section. The section is run by an experienced Chartered Civil Engineer with two mining surveyors. We have assistance in the summer, usually from student vacation staff who we find very useful in this work, and it is complemented by a further five experienced Chartered Civil Engineers who undertake overseeing inspections. Typically these five Engineers would carry out three or four inspections a year. The total cost of running this section, including the effort on overseeing inspections, I guess to be about £17,000 a year net, to which must be added overheads. We also have a Flood Studies Group, which undertakes safety work as well as flood assessment. This Group would add another £6,000 a year to net salaries. This reservoir safety section is responsible for 76 dams, of which 36 are instrumented.

The last point - what is meant by 'as far as is practicable for the watermen taking readings'? Many of our dams are very remotely situated in the hills. Between December and March access is often impossible, and we therefore cannot take these readings..

Coming now to the other papers in the Session, I would like to take up four points in Paper 1.1 by Ellis. I very much enjoyed reading this paper, I thought it was well written and took us very gently and carefully through the new Act.

Ellis states in his paper that the Reservoirs Act is solely concerned with public safety. If this is the case I find it most difficult to understand why canals have been excluded from the Act, when many stretches of them contain very much larger volumes of water than the 25 tcm which is the new threshold for certification of reservoirs, more particularly as many of these canals are very old structures in doubtful states of repair. I believe that these present just as great, if not a greater, threat to the public than most of our reservoirs.

Secondly, I fully agree with Moffat in his views, as expressed in Paper 1.4, on the provisions for enforcement on the Act by the 66 County and Regional Authorities. I believe there will be wide variations in the manner in which the Act is enforced. I also feel that many of the smaller authorities, for example the Scottish Islands Regions, have neither the staff nor the experience to undertake these duties, and I also believe that the system will be an inefficient and ineffective way of policing the Act. We have excellent examples of Inspectorates in this country who are dealing with very complex safety problems — I think in particular of the Nuclear Installations and Alkali Inspectorates. They do an excellent job and, from what I have heard abroad, they are regarded with esteem throughout the world. I cannot understand why a similar system should not have been provided in this country for reservoirs.

My third and last point is that, in carrying out our Statutory Inspections, we find that in many cases much technical effort is needed by Panel Engineers who are new to a reservoir to check out the design, often requiring them to go back to first principles. I feel that this is very wasteful effort when it could so easily have been very much reduced if important design parameters, the key design criteria and the major design approaches had all been specified in the Reservoir Records Book, just as the drawings are provided. I also believe that it would be advantageous, when modifications have to be carried out to a reservoir, for Panel Engineers to record the basis on which the design of these modifications have been undertaken.

Lastly I wonder if Mr Ellis would like to hazard a personal view of what qualifications might be required for appointment to the Panel of Supervising Engineers.

S NYLANDER (Swedish State Power Board) :

I work for the Power Board in Sweden and am responsible for dam inspections there. I have the same point of view as many British engineers about legal aspects of dam inspection.

In Sweden the State Power Board produces about half of the hydro-electric power and I carry out dam inspections for that board. We have no Federal Act or federal legislation for dam inspection or dam safety, so each owner is responsible for his own dam. In recent years there have been some accidents involving very small and old dams, private dams. Two years ago a woman was drowned by a dam flood, and that started activity in Parliament. The Government is now working on the problem of legislation. As a start all dams have now to be listed, so that each owner knows which dams he has under his control. The dams can be anything up to 100 years old, or used for fishing or sailing purposes for instance, and they can too easily be forgotten. It is very important to know which dams one has responsibility for. There can be accidents even to small dams – that is another very important point.

M F KENNARD (Rofe, Kennard and Lapworth) :

There are two points I would like to make. One has already been mentioned, and that is the question of enforcement authorities. Mr Johnson has mentioned this, Mr Moffat has in Paper 1.4, and I wish to do so as well. The fact that we are all three members of the Symposium Steering Group is purely coincidental, it is not a concerted attack on the Act, it is just that we hold such views about the problems which we think will arise through a total of about 66 enforcement authorities.

The Institution, in their 1966 Report, recommend a central registry for reports. The Department of the Environment (DoE), have decided against the central body, and records and reports will not be available to interested engineers or to researchers. I am not even sure if they will be available to the Department itself. This has come about through deciding to do things regionally rather than centrally, and it will mean that certain owners and undertakers such as the Severn Trent Water Authority, Central Electricity Generating Board (CEGB) and others will each have 10 or more county councils to deal with for their reservoirs. These county councils, as has already been said, may interpret the Act in different ways. They may keep their records and registers in different ways, and they may staff the set-ups differently. Some may be under a technical branch of the county council, some perhaps under a legal branch, but nevertheless they will all be bureaucratic.

Mr Moffat in Paper 1.4 mentions a total of about 2 000 reservoirs coming under the Act. I think this is probably an underestimate, but nobody knows. In the discussion of the 1966 ICE Report I mentioned this aspect and said it was surprising at that time that even the Institution had not considered how many reservoirs came within the scope of the Act on which they were reporting. I think there are many more than 2 000 reservoirs - one hears quite frequently of reservoirs which have never been inspected or are being inspected for the first time, and even the British section of the new World Register of Large Dams now has 30 older dams included which had not been listed before. Even if it is 2,000 reservoirs, this is an average of about 30 per county council, so the latter are setting up organisations for, in some cases, a very small number of reservoirs – there may only be a handful in a county area. The personnel responsible will have no knowledge of what is involved. They will not know of such things as Terzaghi's Laws or Darcy's Law, but they will know Parkinson's Law.

Why could there not have been a central body? It would have been very much better for the Water Authorities, it would be very much better for the engineers concerned with this work. If, in time, the politicians ever become serious about reducing central and local government staffs and excessive costs, then such a change could occur and reservoir owners, undertakers and engineers would benefit. It would only need a very small central body, headed by an engineer - I am not talking along the lines of Mr Moffat's proposal, where he suggested an Inspectorate in addition to the enforcement authorities. I think there is no need for the enforcement authorities, and I would be interested to have Mr White's views on this, reminding him of his Minister's statement in the House of Commons that the functions of the local authority are purely administrative.

If I could leave that point and turn to the question of lagoons, I do not think that the Department understand what is involved with some lagoons such as the very large ones built by CEGB for disposal of fly ash, where fly ash is pumped in as a mixture of water and slurry. They do not come under the Mines and Quarries Act, because that only refers to lagoons adjacent to mines and quarries. There is one case I can mention where three adjacent lagoons/reservoirs each hold 1 800 tcm or more and contain water for very long periods of time. Eventually they will be filled with a mixture of ash and water, but at the present time they are purely water reservoirs. Mr White has mentioned that he does not think they come within the new Act. The owner, in this case the CEGB, considers they do, and they are looking after them as though they do, but there is the problem that the enforcement authority may take a different view. They may say, and they may be advised by the DoE, that they do not come within the Act. What would happen then? We may have something which starts off as a reservoir because it is holding water, subsequently it is considered not to be a reservoir because it is not holding water as such, but it will still be a reservoir because somebody may say that therefore it is not under the Act and therefore there has been no abandonment. We may be left forever with a reservoir which is not a reservoir.

H B WILLIS (Corps of Engineers, United States Army) :

I would comment on a number of points in relation to our activities in the matter of regulation and inspection of dams in the United States. I would say that we tend to think in different terms in regard to such things as regulation, responsibility for inspection and so on.

I note a tendency in the United Kingdom to put the responsibility on individuals or on boards. In the United States such responsibility is most frequently put on agencies, and that is a distinction I think that you would observe if you compare Paper 1.5 with other papers. We tend to look towards agency responsibilities. There is also a gimmick peculiar to the United States in that we have what we roughly refer to as a balance of power, that is the balance of power between the legislative, judicial and executive of the departments in our Federal Government. There is also a balance of power between the Federal Government and the State Governments, and we find that this is quite evident in Canada also. In the United States police powers are in the hands of the State, so the responsibility for protection of the citizens of a given area is generally the responsibility of the local government and the State government in that area, and not of the Federal government. Some of the things that you say about your arrangements for inspection of dams or responsibility for inspection of dams therefore do not quite fit the political climate in which we work in the United States. These are differences which undoubtedly have something to do with the type of system you are proposing. We have, as a result of the actions described by General Morris in Paper 1.5, developed recommended standards for inspection. This is a rather revolutionary thing for the United States, because it represents the concensus of a great number of different groups on what ought to be the way dams should be inspected. It does not specify the standards by which dams will be judged, but it does indicate to the public and to the rest of the profession what these various groups feel is important in the matter of dam safety.

On the matter of standards, we immediately run into considerable differences of opinion. There are, for example, considerable differences of opinion about the requirements for the adequacy of spillways on a dam. The Corps of Engineers tends to favour, in most situations, the use of the maximum possible rainfall and the resulting flood as the test of adequacy of a spillway. Others have advocated an economic analysis to determine the appropriate capacity for a spillway. These are fairly divergent viewpoints, and it is hard to reconcile them. Recommendations reported to Congress say that we should adopt a national dam safety programme in the United States. We make the Federal agencies responsible for dams over which they have some sort of control. In the case of the Corps of Engineers, who are in a sense custodians for the nation we would have full responsibility for the supervision of the condition of dams and for inspection. In the case of the other Federal agencies, some are owners of the dams and others have regulatory powers over them, but in each case where there was some type of Federal responsibility already that agency would be made responsible for the programme of inspection and compliance with required standards. The bulk of the 49000 dams would be under the supervision of State agencies under the recommendations of the Chief of Engineers. We do not yet know just how that will work out, because there are people who feel that we need a fairly massive Federal programme for a central registry etc. There are others who are just as vigorously pushing for responsibility to be vested in the States while recognising that our States have limited resources, and that for an effective programme financial help will be required from Federal sources.

Touching on one or two other points, the general tendency in the United States in regard to the question of liability under the licence procedure is that the licensing agency assumes no responsibility. In the Eastern states the water laws tend to follow English common law. If you have built what a prudent man would build you are not liable for any type of criminal activity or criminal responsibility for the failure of that project. However, that does not relieve you from liability for personal damages. In the Western states we have a different basis, based upon the old Spanish law. Juries would probably find, in the case of a personal damage suit in the Western states, that the man who was responsible for building the dam is liable for any personal damages.

Regarding qualification of engineers, we have a system of licensing engineers designed to protect public safety. The licensing is carried out by the individual States and with somewhat varying requirements. We would assume with the enactment of any type of Federal law that the broad requirements for licensing of engineers would be the only ones applicable. We have no provision for licensing of dam engineers.

Some States have attempted to license on individual subdivisions within the field of civil engineering, for example the State of Kentucky at one time did license some twenty elements of civil engineering. The trend in recent years has been against this. We generally license people as civil engineers and presumably anybody who is licensed as a civil engineer could carry out the activities required under our contemplated inspection system. In practice, however, people tend to look at the qualifications of the individual beyond the bare requirements of licensing, so we would hope this would ensure that competent people are involved in inspection and licensing of dams.

L E ELLIS (Department of the Environment) :

I am not a reservoir engineer and I am not a lawyer, but I was the clay pigeon who was told to talk about the new Reservoirs Act because I have certain responsibilities for the professional input.

The Act has taken 10 years to prepare since the ICE Report was produced. We have not been wasting our time, because responsibility for the Reservoirs Act did not come across to Housing and Local Government, as the Ministry then was, until 1969. It lay previously with the Home Office, and indeed a year after that we were already attempting to get a place in the legislative programme. We tried on a number of occasions, but governments have been plagued by things other than reservoir safety, e.g. the falling pound, or the European Economic Community, and all of these have taken priority. Also, of course, each time governments change Bills are lost on Dissolution. This happened to the Reservoirs Bill twice. Fortunately on the last occasion Parliament was dissolved the Bill was picked up immediately, we got it straight through and it had a fairly quick passage through the House of Lords and the House of Commons.

The first question the Reporter raised was on the number of reservoirs which were to be newly caught by the 1975 Act. We do not think there will be very many. We tried some three years ago to obtain the actual number of reservoirs over the old 22.5 tcm capacity from River Authorities and we were told then that of some 5000 reservoirs in England, Scotland and Wales about 2000 were within the ambit of the 1930 Act, and quite a number of these were not reservoirs for public water supply.

Of the 2000 that would come wtihin the ambit of the 1975 Act, we reckon that 574 have between 22.5 tcm and 45 tcm capacity and that about 1400 have over 45 tcm. Over half of the 2000 reservoirs are owned by public water supply authorities.

We looked very carefully into insurance, that being one of the recommendations of the ICE Report which is not included in the Act. Some statutory water undertakings have for many years had established insurance cover, but it was by no means certain when we took advice that new insurance business would be taken on at the old rates. The point about insurance is that risk cannot be spread for reservoirs in the same way as can car insurance, where there are millions of cars and there are good, bad and indifferent drivers - or like third party risk for house owners where one can cover fire or third party liability with fire and burglary. With a reservoir there is not the spread of risk and there are not sufficient reservoirs for premiums to be reasonable. We were advised by insurance people that it could only be profitable if the risk could be spread sufficiently wide or that they could select their risks, and in the latter case the premiums would be quite prohibitive. We thus decided that, on the professional advice we had, compulsory insurance was not realistic. Insurance was not realistic. Insurance would certainly have involved the Government in partly underwriting the cover, and Government at the moment is very antiseptic towards taking on any additional financial commitment.

On Mr Moffat's point in paper 1.4, i.e. an Inspectorate of Reservoirs, most of us in the Directorate General of Water Engineering have a great deal of fellow feeling for the concept, but we lent our support to the dispassionate advice to Ministers that the Government had neither the expertise nor the resources to take on a task of this kind. The Reservoirs Act is an Act aimed at securing public safety only, and public safety is a function and a responsibility of local authorities – they have this particular function spelt out to them in the Local Government Act. They are responsible for public safety, the Government is not. A change in enforcement from local authorities to central government would also require new legislation, it could not be done by prescription.I think Mr. Moffat is under a misapprehension here, the Secretary of State's powers of prescription are exceedingly limited by the Act and only apply to those items which can be prescribed, and he certainly cannot prescribe who shall enforce the Act. We do not think that there will be a great problem with 66 different authorities acting as enforcement authorities. They are required to report annually to the Secretary of State on the steps they have taken to ensure that Undertakers comply with the Act, and I am quite certain that the Secretary of State has sufficient powers if it is thought necessary to inspect all the registers which the enforcement authorities will be required to keep.

Regarding Mr Moffat's proposed central registry, we have a great deal of sympathy with this idea, and I think it is something that we shall be taking on board and examining – I am not saying that we can possibly set one up. We looked at it whilst the Act was in preparation and we said that it was going to cost too much money, but we now have the facilities in our Water Data Unit for data collection, and I am quite certain that if the need was justified – and probably we would look to the Institution for that lead – then we could certainly take this on and get all the information we require from the registers which local authorities will be required to keep.

On the point about Supervising Engineers, I think this is probably the most important addition in the new Act. That there was not continual supervision was seen as a major gap in the 1930 Act, and I think the new requirement recognises that the reliance on ten-yearly inspections is in itself not enough to ensure public safety. The point is that if failure is going to occur it must not be left until the stage when it then results in panic action, and this has happened on reservoir failures in the past. The provision and the requirement for Supervising Engineers also recognises that although instrumentation is increasing, and very properly so, it is absolutely no substitute for the trained eye. Further, it is quite pointless installing masses of instruments and collecting all data unless that data is made available to a competent man who can interpret it, recognise warning signs, and then alert a Panel Engineer. I would stress at this point that there is a difference between the Supervising Engineer and the Inspecting Engineer or the Construction Engineer. The Supervising Engineer is the policeman – he must always go to a Panel Engineer for advice. He plays a very important role, however, and I think that he should be a man who knows reservoirs day in and day out, who is used to the management of a reservoir rather than the design of a reservoir, and who is used to operational procedures connected with a reservoir. He should be an engineer with a trained eye, to detect the warning signs and then take the necessary action.

As to why we did not raise the limit to 45 tcm, the brief answer is that it was just felt not to be logical.

Regarding storage of liquids other than water, it was decided that this required quite a separate expertise from water. The hazards are different, the design is different and the experience is different, and it would require different legislation. If we had taken that on board we would not have had the Reservoirs Act in 1975.

On the question of canals, I am dodging that issue because canals simply are not reservoirs. This was the advice we received from our Parliamentary Counsel. It is legal advice that you could not class a canal as a reservoir and it thus could not be included in the Reservoirs Bill.

J L ADALID (Ministry of Public Works, Spain) :

Since 1963 a supervision system on the lines suggested by Mr Moffat has been operative in Spain.

A compact group in the Ministry of Public Works shares supervision of the design, construction, behaviour and operation of dams. The objective is to persuade designer, constructor and operator to improve techniques and methods in order to obtain modern and safe dams, and to supervise old ones and assist the owners to follow and control the behaviour of their dams. Substantial results have been achieved, especially in the area of design.

A I B MOFFAT (University of Newcastle upon Tyne) :

S The United Kingdom has, since the catastrophic failure of Dale Dyke Dam near Sheffield in 1864, been spared the horrors of a major dam disaster on the scale of those at Vega de Tera, Malpassat or Vajont. We have had a limited number of lesser failures in this country, however, recent examples — unpublicised because there was no loss of life or major damage — include total failures at Warmwithens in 1970 and Upper Creggan in 1971, with the Lluest Wen incident of 1969 having been uncomfortably near failure.

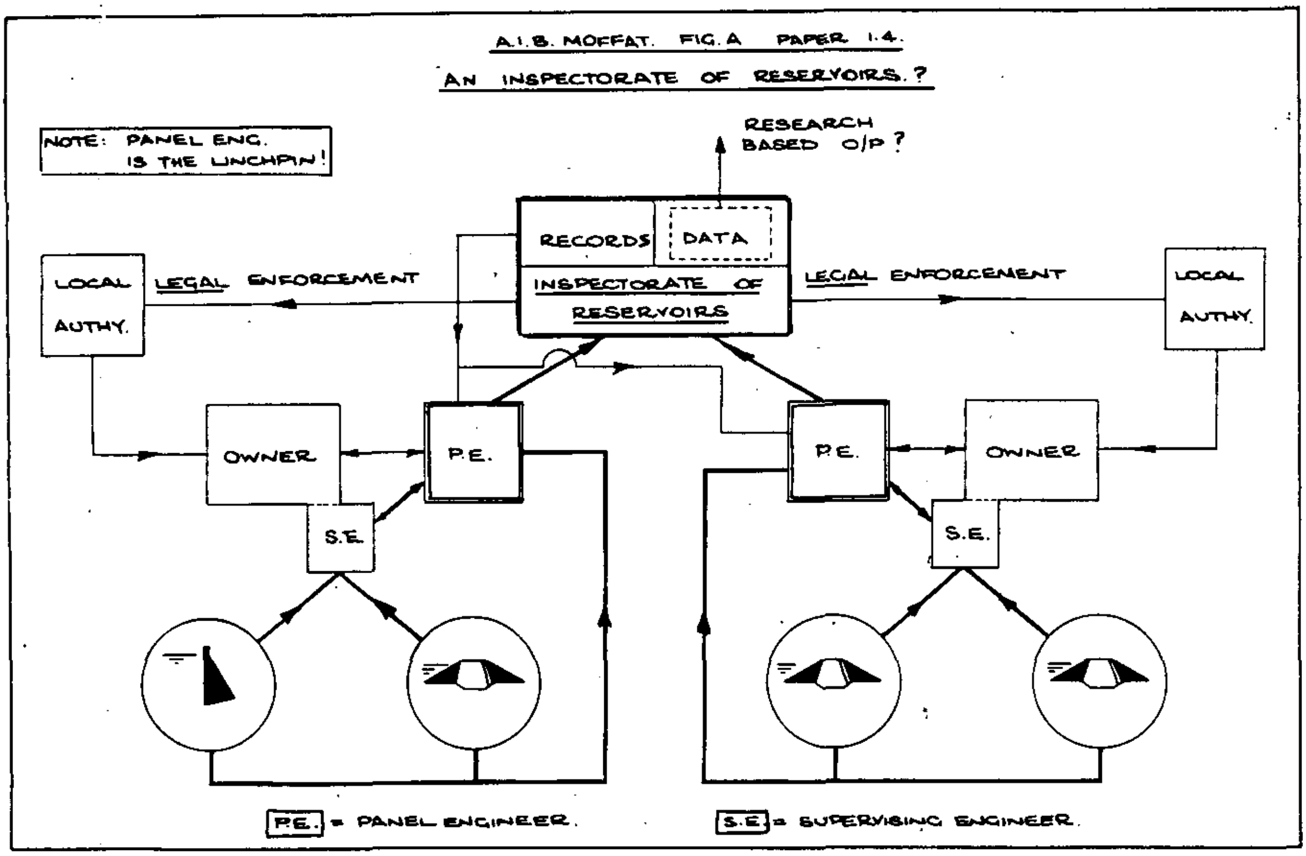
Examples of older, and generally smaller, dams where deterioration has progressed to a significant extent can be found in many parts of this country including, to my knowledge, one abandoned embankment with a miniscule spillway under imminent threat of being blocked by a slip.

British circumstances are such that we have a preponderence of embankment dams. It is a known characteristic of embankment dams in particular that the risk of trouble is highest within the first five years after completion, following which the general pattern is one of relatively good behaviour for very long periods. At some more advanced age, say 70 or 100 years, we must logically expect an increase in the frequency with which incidents will occur due to ageing and time-dependent effects.

The point of Paper 1.4 is that we have very many old embankment dams, large and small, dating from the 19th century. The incident frequency relationship illustrated in Fig.2 of Paper 1.4 is admittedly, and of necessity, a very crude index. It is nevertheless interesting, in that the highest incident frequency plotted coincides with the period of intensive dam-building around 1850 to 1875. Later peaks, e.g. around 1930, and the more recent rise in recorded frequency, are perhaps partly attributable to improved media communication, but they must also reflect the influence of ageing on many of the 19th century dams.

In Paper 1.4 I put forward the controversial idea of a central Inspectorate of Reservoirs fully expecting to be taken to task, and I was not disappointed. The comments of earlier speakers notwithstanding I believe the concept merits consideration. A block diagram indicating the manner in which such an Inspectorate might function within our legislation is set out in Fig.A.

It is important to note that the key man would remain the Panel Engineer. The concept of the link between Supervising Engineer and Panel Engineer being extended to feed information back to a small central Inspectorate charged with policing the Act and maintainting a data or record unit is an attractive one. A great deal is lost to engineers by virtue of the present lack of such a unit housed in a Government Department or, indeed, in a University, where returns from inspections, instrument records etc. could be filed and studied. The possibility of a useful research output feeding information back into the system would also be a significant point in favour of such a unit.



As a final point, the idea of a Risk Index or Hazard Index for reservoirs was suggested by an American engineer, G S Sarkaria, some years ago. It is convenient, but hardly logical, to define legislation in terms of reservoir capacity, with a lower limit of 25 tcm. In at least one case of which I am aware a tiny dam failed and released some 100 times that quantity of water down a remote river without it even being noticed — risk had obviously been minimal. In complete contrast to this we have instances of several old reservoirs in series above heavily populated areas, e.g. Longdendale. It would be valuable, therefore, to have a Reservoir Hazard Index based on capacity, population at risk, head and type of dam etc. — in effect applying a series of standards as a much extended version of what the Americans are doing in their legislation.

With respect to Paper 1.6, Mr White asked what advantages might accrue from owners having to work under the supervision of a committee of six civil servants and two technical experts as with the French system. My personal view is that control by committee is a prescription for disaster, at least in the metaphorical sense. Guidance from an *advisory* committee, however, is quite a different matter, so long as ultimate responsibility remains with one individual.

J B MILLER (Health and Safety Executive — Inspectorate of Mines and Quarries):

I am one of Her Majesty's Inspectors of Mines and Quarries. I look at tips, mines and quarries, and I was taken on particularly to do this job after the tragedy at Aberfan. We work under the 1969 Tips Act, to give it a very short title, which was basically an enabling Act. It states in Part 1, which I look after, that every tip shall be made and kept secure. It continues with a number of definitions, and allows the Secretary of State to make certain regulations and does place one specific duty on the owners and managers of all mines and quarries, to the effect that they must obtain information as to what they must do to keep their tips secure, and all the lagoons at mines and quarries are basically tips consisting of some sort of mineral refuse.

In 1971 we produced some regulations which took care of all those things in the Act which the Minister could prescribe. These particular regulations work on the basis that all of the larger tips, where the sizes are prescribed, must be inspected weekly by pit staff - basically a waterman. In fact, because these tips are growing the whole time from their inception until they are complete and are abandoned, most of the lagoons are actually inspected daily for water levels, clearance of overflow and so on. On top of this every tip must be inspected once every two years by a competent person, whom we normally regard to be a civil engineer experienced in this type of work, and as far as lagoons are concerned many of them are Panel Engineers .

A Certificate has to be produced which says that the tip is secure, what changes in its design and specification are taking place, what exploratory boreholes or measurements have been made during the year and, finally, the most important thing, what inspection and safety measures must be carried on in the future (which is only another two years) to ensure that the tip is made and kept secure. I think that this is a fairly good system, and it is looked after by the Inspectorate. Basically we of course keep no records, as such, of the construction of lagoons. We can always ask for information and obtain it, and if a lagoon is finally abandoned all the plans and specifications do have to be deposited either with us or with the National Coal Board's Records Office,which maintains records for all abandoned mine workings.

One of the aspects on which our system is particularly tight is that every mine and quarry is inspected more than twice a year by one of my colleagues. The owners and managers are required by law to notify the district Inspector that there is a tip on the premises, and these notifications do form a register which is now maintained in London. The result is that if my colleagues in the field see something fairly large they advise me of it. I normally then arrange for one of the engineers who would be making a report on lagoons to walk around such a tip and give it a thorough inspection, and if there are any modifications required this is amicably arranged. An important thing is that there are plans and records kept of these tips when they are abandoned, and the managers and owners keep plans and records on their premises.

I can fully understand Mr Kennard's concern about other lagoons. I obviously do not look at these at the moment, and I am under the impression that in actual fact Panel Engineers look at the majority of them, but there may of course be other factories which have lagoons which at the moment are not covered by any specific section of the law. The new Health and Safety Act, which has actually changed the law as far as factories is concerned, does of course contain a very important clause saying that everybody who has a factory must ensure that he is not going to endanger the general public. This was not in the old Act, nor in our old Mines and Quarries Act, but this new provision will in due course allow factory inspectors to look at any other dangerous structures outside the factory premises. To the best of my knowledge nothing is being done about this at the moment, and perhaps the Institution might like in due course to raise this with the Chairman of the Health and Safety Committee.

D J KNIGHT (*Sir Alexander Gibb and Partners*) :

Before making my main comment I would enquire, following a previous speaker's question on the omission of canals from the ambit of the new Act, whether any consideration was given to the matter of river embankments, estuary embankments and sea-walls, which in South-East Essex retain a large body of water known as the North Sea? As the main object of the Act is to promote the greater safety of water-retaining structures it seems strange that similar safeguards do not exist for those structures. I wonder what Dutch engineers would think of this omission.

My main comment. however, concerns the independence of the'Inspecting Engineer' who, as pointed out by Mr Ellis in Paper 1.1 'cannot be the engineer who acted as the Construction Engineer'. But why not? Whilst, politically, independence may seem the right provision to make I suggest that in technical practice it will mean forfeiture by the inspection process of really detailed knowledge of the structure, since it will be impossible for *all* relevant information to be passed on to the inspecting engineer by the constructruction engineer. This applies particularly to embankment dams, for example, full details of (i) the original and subsequent series of exploratory borehole records; (ii) grouting records; (iii) foundation excavation records; and (iv) complete construction records.

Mr Ellis states that 'the inspecting engineer would pick up those matters notes by the construction engineer'. What tonnage is the former expected to pick up? It seems not only unfortunate that the '*construction engineer*' *cannot* in due course become the 'inspecting engineer'; it must inevitably involve both a waste of invaluable detailed knowledge and unnecessary duplication of effort, at a time when the country can ill afford to dissipate its not unlimited resources.

C C PARKMAN (Ward, Ashcroft and Parkman) :

I would like to ask whether we can operate this new Act with the number of Panel Engineers we have. If one looks at the Panel, it consists of a fairly limited number of men, a lot of whom are quite senior. Has any thought been given to a different method or to a different form of training?

In this country we have had few dams constructed over the last few years due to the environmentalists and to lack of money. With the additional Supervising Engineer requirement, therefore, can we in fact provide the necessary experienced engineers? Perhaps some thought has been given to different procedures or methods of qualification.

T M HYDE (British Waterways Board) :

The question was raised about canals, and my Board is obviously extremely conscious of the risk they represent. When I asked our solicitor some years ago whether there was any risk that the old Act could be extended to cover canals, having advised him that a new Act was pending, it was with a consciousness of the damage which impounded water can do — I was with Essex River Board during the 1953 flooding.

There are 2200 km of canals belonging to the British Waterways Board alone, and only about 1.5 km of narrow canal will provide the prescribed 25 tcm volume. One really cannot seriously consider that a major part of the canal system should be treated as a very considerable number of additional reservoirs. It would make an enormous percentage increase to the 2000 reservoirs quoted earlier. Moreover, a breach in a canal cannot be compared with the breach of an equivalent volume stored in a conventional reservoir. Picking up Mr Moffat's point about a very large area of very shallow depth in the North of Scotland, a narrow canal is perhaps 1.6 m deep with a cross-section of perhaps 12 m^2. If a breach does occur — and fortunately major breaches do not occur too frequently — it takes a considerable time for that volume of water to flow, say, 1 km or more and there is really no comparison with conventional reservoirs. As far as I know the last occasion on which there was loss of life following a breach was in 1944, when the resulting flood washed away a railway line and a railway engine went into the washout, killing the driver.

When one considers the risk on the one hand and the complexity of the task on the other, bearing in mind that the definition of a catchment can be exceptionally difficult to establish, I do feel that the omission of canals from the Act was sensible.

R E COXON (Engineering and Power Development Consultants Ltd):

Mr White asked in his Report whether he could be advised of any governments which were in fact carrying responsibility for risks to dams. I am currently serving on a committee of ICOLD which is reviewing the risks to third parties from dams, but I have not come with all the information that we have because I would not like to pre-empt the Committee's report, but it will be published in Mexico in March 1976 and will contain a lot of information that can be of great value to all of us here.

What I can say is that we sent out a questionnaire to all of the members of our organisation and received replies from 27 countries which indicated the standard of inspection and control which those countries maintained over their dam construction. The replies covered an enormous range, from virtually nothing up to most elaborate programmes. What I think was most interesting was that in the developed countries we again found a very wide range, with no uniform approach to the problem. Even in Australia and the United States the individual States, at the time when we started our committee work, had great differences of opinion, and it is quite interesting to hear from Mr Willis today that some of those differences are now being resolved. I know in Australia that they are attempting some better standardisation; and we have an Australian representative here who may be able to comment further.

On insurance, I do not think I entirely agree with Mr Ellis. I think that the insurance market as such would be prepared to give cover for dams providing that there was a reasonable level of excess which would be carried by the owner, and providing also that governments would in fact be responsible for catastrophe. Governments do usually pick up catastrophes anyway, so this would be possible. Insurers do not appear to think that it would be internationally possible, because of varying national standards of control, but it ought to be possible to set up some kind of system within any country with a reasonable number of dams.

I think one might say that the recommendations of the Committee will, in fact, include minimum levels of inspection which the Committee thinks should be carried out and the areas in which that inspection should be made. Finally, I think that if Mr Grøner would agree, Mr White perhaps could have advance access to some of the Committee information which might help the Department of the Environment in formulating their proposals.

J D HUMPHREYS (Mander, Raikes and Marshall) :

I want to make a few remarks, first of all about responsibility. To me the passage in Paper 1.1 which needs underlining is the following: 'Ultimate responsibility for the safety of a river must rest with the Undertaker'. In a brief discussion one has to over-simplify, but as I see it the Inspecting Engineer fulfills much the same role as the garage man who inspects the car and tells the owner that if he does not have the tyres fixed there will be an accident. I would, at the risk of sticking my neck out, question whether the actual responsibility for a failure in this sort of philosophical set-up rests with the Inspecting Engineer.

I tried out this view on a very senior member of a new Regional Water Authority in a general way not long ago, and he suggested it would be an evasion of responsibility by the Inspecting Engineer. When a senior officer in a new Authority reacts so strongly I tend to get a bit neurotic about it and feel that this is the time for me to see how many people agree with me. An earlier speaker, incidentally, spoke as if the Act was already in force, which it is not, and I would add that the interesting bit, namely the Regulations, have not yet been published. They will provide much more ground for meaningful discussion.

As I understand it, the new Act was primarily aimed at large reservoirs such as those that one would expect to see owned normally by a Regional Water Authority. I assume, and I welcome comment on this, that the Supervising Engineer is envisaged in general as being a responsible employee of that owning Authority. It is on this assumption that I feel that the Inspecting Engineer, with the new Act, owes it to the Authority to discuss with that Authority and, if he can be identified, with the potential Supervising Engineer, the recommendations which he proposes to put into his Report.

It seems to me that if I inspect a reservoir tomorrow and report on it, and if it should fail in eight years' time, that I should resist very strongly any suggestion that I was responsible for that dam failure. I am prepared to accept a situation, if this is clearly defined, in which I accept that responsibility for the next 10 years, or my firm does, but in that case I think that I would then want some separate advice from Mr Coxon on the subject of insurance so that I could properly re-estimate my fee!

There is a second point I would like to make, namely that the smaller dams swept up with the Act include, of course, a number of small privately owned dams for trout and so forth. I would just comment that I am a little baffled to know where such owners are going to find Supervising Engineers. I wonder whether this means, in fact, that Brigadier Parkman was right, and that there may be scope for even the most junior or the most senior of us in the future in this respect.

R D ROBINSON (G H Hill and Sons):

I think Supervising Engineers are going to go some way towards filling a gap which has been growing over the last few years. I have to inspect a number of reservoirs, particularly urban ones, and time was when every reservoir or every chain of reservoirs in one valley had at least one man who lived on the job and who knew everything that went on in his area. When you went to inspect a reservoir he could tell you any differences that were noted from day to day. Over the last few years those men have been disappearing. Reservoir maintenance is carried out by a mobile gang which visits a site as necessary. The gang of men go out by vehicle, do what is required, and then retire. As I inspect many reservoirs I am accompanied by a man who is supposedly in charge of them, but he is also in charge of a whole lot more, and I find that there is an increasing ignorance on the part of such men with regard to knowing what is going on at a particular reservoir.

Embankments have the virtue that if they are going to fail they generally begin to show signs of it in a preliminary way. First of all one gets some leakage or evidence of leakage around the mitres and downstream, or even in the embankment itself. Secondly, one may get some subsidence, some drop in level on the crest or on the downstream slope. If there is nobody there who knows that such occurrences have just taken place a great deal of information is lost.

I wonder how far a Supervising Engineer, who in his turn is going to be responsible for a number of dams, is going to be able to substitute for that waterman of old who knew the reservoir intimately and if anything was different from the day before. I know of one water undertaking, which is now part of another larger Authority, where they sought to fill this gap by making one man responsible for a limited number of reservoirs, two or three, and it was his job to visit them at least twice a week. He had to submit a report to his Head Office every month in which he recorded every detail that was worthy of notice, every difference of circumstance, every change of flow of water, every subsidence and every crack. I wonder how far the Supervising Engineer is going to make good the very manifest deficiency that I now notice from day to day?

C GROBBELAAR (Department of Water Affairs, South Africa):

I think there can be quite a difference in the definition of terms. In South Africa we differ substantially with respect to safety inspection of dams. Normal events, such as operating problems, are taken care of by the supervisory people who actually operate the gates and look after the general condition of the dam. For example, the appearance of a gate is not relevant to safety inspection as such.

What we term safety inspection of dams relate to things like adequacy and safety of the design. Given, for example, a 50 m high arch dam, on what principles was that dam designed? Is the concrete still of the same quality as when constructed? Is the cut-off or grout curtain still effective? What possible earthquakes can occur? Is the spillway compatible with the latest information on floods? This grade of information is included in a safety inspection.

For this type of safety inspection I think the idea of licensing a limited number of people is very appropriate, because, I do not see that an engineer who has been licensed to build a sewage purification works can adequately comment on the reaction line of an arch abutment. We have, unfortunately, not got legislation promulgated yet in South Africa, but we are currently working on that. At the moment we are in a very simple position. The relevant Act simply says that the Secretary of the Department of Water Affairs is responsible for seeing that reservoirs are safe, and that makes things easy, because he simply writes to say 'producea report to say that your reservoir is safe,' which is rather effective.

P COOLEY (Thames Water Authority) :

On Supervising Engineers I find myself very much in agreement with what Mr Ellis has said, but it seems to me not necessarily the case that a man suitable for one of the Panels under the existing Act is a man for day-to-day supervision of reservoirs, and an employee of the Water Authority. In my view he should be a man with direct responsibility for works including reservoirs, or perhaps only for reservoirs. He should be a Chartered Engineer with managerial capabilities and managerial function, and the reservoir attendant should be a man who is in his charge. On these sort of lines I imagine that within the Water Authorities in England and Wales, where there are I suppose about 800 or 900 reservoirs which come within the Act, about 200 Supervising Engineers would be required.

I do not think that the function of monitoring instrumentation is for this man except, perhaps, in a simplified form as, for example, Mr Rofe mentions in Paper 2.9 in a later Session. Possibly it is for the Construction Engineer in the first place, or the Inspecting Engineer, to specify who should do this – either the design department or the consultant to the Authority.

To turn to the question of sludge lagoons Thames Water Authority, besides its reservoirs, is responsible for sludge lagoons and also for coastal and estuary defences, which one of the speakers mentioned. A Water Authority might find it a safe course to adopt similar routines of inspection for the latter categories as it does for the reservoirs that certainly come within the Act. Does the Act really exclude such lagoons? Legal advice says that it does. Which reminds me that there was a movement some time ago to classify a particular sludge lagoon as holding water for the purpose, I think, of exemption from rating, on the grounds that the sludge it contained was 90% water. This argument was demolished in Court by opposing Counsel on the basis that the same could be said of a cucumber!

WRITTEN CONTRIBUTIONS

R M ARAH (Binnie and Partners):

Speakers commented on the unsatisfactory use of stored volume as a measure of the potential danger of a reservoir, and on the practical difficulty of checking volume independently. It would seem better in both respects if the threshold were set in terms of energy stored. For this purpose a simple and adequately valid measure would be the product of the maximum surface area and the square of the maximum depth. The statutory limit of the Act could then be set at about 500,000 m^4; approximately equivalent to a 25 tcm volume with a maximum depth of 10 m.

An equally simple alternative would be to measure the potential danger as the maximum rate of release of energy if the dam were to be instantaneously removed from a full reservoir. This criterion would only require knowledge of the longitudinal section of the dam, and might be shown to be sufficiently sensitive if put simply as crest length, L, times the power function of height, $H^{5/2}$.

Section 7(4) of the new Act envisages the Final Certificate of the Construction Engineer as normally being given within five years of the Preliminary Certificate. With reservoir operation moving towards longer regulating cycles it is unlikely that a significant drawdown will have occurred in this order of time, and possible that the reservoir may not even have filled in some cases. A Construction Engineer may thus be caught between the embarrassment of withholding his Final Certificate or giving it before his design assumptions have been fully tested.

K T BASS (Rofe, Kennard and Lapworth):

I am in full agreement with Mr Johnson in that canals, which appear to have been excluded from the Reservoirs Act 1975, constitute a serious danger. The reasons put forward for their exclusion by Mr Hyde do not hold water. The large number of canals can surely only be a reason for special care, and the suggestion that a human fatality has not yet occured is pure good fortune and cannot be relied upon indefinitely.

The essential facts about canals are that they were mostly built 150 or more years ago, frequently with only 225 mm of puddle clay lining to retain the water. After all this time the lining must be suspect. A canal cannot be drawn down to improve safety, as the lining will dry out. In any case an incoming flood must cause the canal to run full in order for the flood to reach a suitable overflow. Unlike a dam which, if it failed, would cause a flood in a natural valley where it would not be entirely unexpected, the failure of a canal might well cause a flood in areas which would otherwise not be susceptible to flooding.

B L DAVIES (South Yorkshire County Council):

I specifically wish to refer to the newly created role of the Supervising Engineer under the 1975 legislation. Ellis in Paper 1.1 has inferred that this function should be undertaken by a civil engineer who:

(a) is able to keep the reservoir under 'continual' surveillance;

(b) is capable of interpreting operation data and records; and

(c) has the trained eye to notice and assess the effects of unexpected events on reservoir safety.

It is noted that the Supervising Engineer is required to report annually to the Undertaker, but not to either the Enforcement Authority or the Construction/Inspecting Engineer. It is only when, at his discretion a periodical inspection is required or implementation of recommended safety measures is undertaken that there is a statutory duty to inform parties other than the Undertaker.

Clause 10(4) of the Act states that the Inspecting Engineer may include in his Report matters which may require the special attention of the Supervising Engineer, but again no specific channels for feedback of information are given.

It would, therefore, seem to be a weakness of the Act that the Inspecting Engineer, who would appear to have a high 'Panel grading', is dependent very much on the adequacy and competence of the Supervising Engineer between statutory inspections. This, of course, places a high burden of responsibility on the Supervising Engineer, and the mechanism for appointment to the appropriate Panel must be devised with this in mind.

It is regretted that the Act has not devised a better balance between the two grades of 'qualified civil engineers', where the role of the Supervising Engineer is seen much more as a day-to-day continuation of the work of the Inspecting Engineer. The Inspecting Engineer, in his more senior role, should have some part in the selection of the Supervising Engineer, in his briefing for the detailed supervision, and in ensuring that this is undertaken in a satisfactory manner.

A I B MOFFAT (University of Newcastle upon Tyne):

The Appendix to Paper 1.4 lists 46 recorded major incidents, including failures, involving British dams and occurring in the period since 1800. It was emphasised in the paper that the list was inevitably far from complete in view of the inadequacy of records and statistics of British dams, and that many incidents have probably gone unrecorded. The question of correct identification adds to the difficulty of completing a definitive list, as names and/or dates are, at times, incorrectly quoted in such references as are available.

Research since Paper 1.4 was written has revealed further incidents which deserve to be recorded. The definition of 'incident' is that of a failure, or an event resulting in damage and calling for immediate remedial action,or restriction on TWL, to avert possible failure. A supplementary list of incidents is appended, including those affecting reservoirs of over 25 tcm capacity referred to in other papers presented to this Symposium.

The deficiencies of a list of this type are fully recognised. It is considered, however, that if a definitive list could be compiled it would be of value not merely as a historical record, but as a useful basis for an analysis of the risk and potential hazard represented by different types and ages of dams. Work on this and on related topics is in progress at the University of Newcastle upon Tyne, and the writer would appreciate any further information from engineers regarding incidents not listed,or expanding information already held in confidence regarding incidents recorded in Paper 1.4 and on the apended supplementary list.

No	Year	Dam	Type	Height (m)	Yrs after Completion	Nature of Incident
47	1804	Aldenham	E	8	9	I
48	1851	Hollingsworth	E	21	—	F
49	1858	Dunford	E	24	UC	I (slip — re-aligned)
50	1859	Baxter	E (?)	—	—	F
51	1870(?)	Henbury Hall	E	—	—	I [o'top (?)]
52	1875	Stubden	E	20	18	I
53	1894	Rishton	E	—	—	F (?) (2nd Fail - see 16)
54	1908	Upper Creggan	E	10	57	F (o'top — 1st Fail, see 46)
55	1944	Holmfirth (Bilberry)	E	—	—	I
56	1946 (?)	Holden Wood	E	>11	105	I (int. erosion)
57	1948	Spott Lake	E	9	48	F (o'top — abandoned)
58	* 1948	Stobshiel	CG	10(?)	—	I (o'top)
59	1948	Thorters	E	15	48	I (o'top)
60	1950	Lower Lliw	E	14	83	I (3rd — see 21)
61	1957	Lairg	CG/E	20	UC	I (slip)
62	1959	Aldenham	E	8	164	I (2nd — see 47)
63	1960	Breaclaich	R	27	1st impound	I (leakage)
64	1960	Cowlyd	E	14	38	I (2nd — see 26)
65	1961	Altnaheglish	CG	42	27	I (deterioration)
66	1967	Balderhead	E	51	2	I (int. erosion)
67	1968	Bristol (nr)	MG	10	>170	I (o'top, deterioration, demolished)
68	1968	Chew Magna	E	8	120	I (o'top)
69	1968	Ashford	E	8	—	I (o'top)
70	1968	Auchendores	E	10	88	I (wave o'top — slip)
71	1968	Durleigh	E	—	—	I (o'top)
72	1969	Glendevon (Upper)	CG	45	14	I (2nd — see 40)
73	1970	Buckieburn	E	26	68	I (slip)
74	1971	Val de la Mare	CG	32	9	I (AAR deterioration)
75	1975	Aldenham	E	8	180	I (3rd — see 62)
76	1976	Coombs	E	16	182	I (slip)
77	?	Slaughan (?)	E (?)	—	—	(deteriorated — demolished?)

* 57, 58, 59 all the result of abnormal flood, S.E. Scotland, 12 Aug. 1948

Additional key : MG = masonry gravity dam

R = rockfill dam

AAR = alkali-aggregate reaction

PROCEEDINGS : TECHNICAL SESSION 2

OPERATION OF RESERVOIRS, INCLUDING MAINTENANCE AND INSTRUMENTATION

Cont.

OPERATION OF THE METROPOLITAN RESERVOIRS OF THE THAMES WATER AUTHORITY

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THAMES WATER AUTHORITY

SYNOPSIS

The method of use of about forty storage reservoirs in the Thames and Lea Valleys is described as to filling, use, control of quality, yield, maintenance and surveillance.

THAMES VALLEY RESERVOIRS (Figure 1)

The first storage reservoirs to be built for London in the Thames Valley were the Lambeth Reservoirs (1 and 2) at Walton built one hundred and one years ago by the Lambeth Waterworks Company in its ninetieth year, and the last built was Datchet Reservoir built by the Metropolitan Water Board, where filling started in January this year. The two mentioned limit an era of progressive construction of two dozen pumped storage reservoirs. Standing in flat land in the flood plain, all have circumscribing earth embankments with central core walls of puddled or compacted clay. Treatment works, mostly founded before the turn of the century by various companies, have developed over the years at Surbiton, Walton, Hampton, Kempton Park (and Hanworth Road) and Ashford Common (1958), but to a large extent it has been possible to interconnect, so that the best stored water can be used and the system is flexible. From alternative sources good water can always be found and it has never been the policy, nor judged economic, to invest in a very large reservoir that will provide far into the future.

It follows that facilities exist, and are used, for week by week changes in the use of the Thames Valley reservoirs. All reservoirs are if possible full to capacity by the 1st of April every year after a winter period when water levels are lowered to make economies in fuel. The unoccupied volume has been used since reorganisation to reduce flooding at its peak. It is also policy to pass all water through storage before treatment. The decay of bacteria in stored water has been known since the 1890's; there is recent confirmation[1] of this, except in some cases where there is carriage of salmonella from refuse tips by gulls, and of the decay of enteroviruses also. The question of water quality will come later, but it follows that continuous replenishment is the rule. The smaller reservoirs at Walton, Kempton Park and Hampton, the medium size reservoirs at Molesey (Island Barn) and the Staines North and South are all rather shallower and grow algae in greater profusion than average. These stand normally in reserve, taking no part in the purification process but counting for storage against deficiency or pollution of the river flow.

Starting at the upstream end, the newest reservoirs Datchet (37 000 t.c.m.) and Wraysbury (33 500 t.c.m.) are filled from an intake and pumping station above Old Windsor Lock, with a maximum pumping capacity of 2250 t.c.m./d. The public electricity supplies are used, with no provision for standby power. Draw-off towers in these reservoirs are away from the embankment to obtain a more consistent water, and the operation of penstocks at different levels as may be necessary, together with all intake and pump control, is managed from a control room at Staines Pumping Station.

The 2.5m dia. tunnel 12 km long drawing water from the above reservoirs was driven in London Clay with wedge block lining. It leads to treatment works at Ashford Common and Kempton Park. There are connections at Staines from the King George VI and the North and South Reservoirs into the tunnel, but the Staines Aqueduct (1902) also parallels this route from Staines and terminates in the Hampton distributing reservoir. The three reservoirs at Staines depend on the intake above Bell Weir. For this pumping capacity of 540 t.c.m./d is used, utilising either purchased electricity or the output of standby generation plant at Ashford Common. 5 km further down river, above Chertsey Lock, is the river intake for Queen Mary Reservoir, 30400 t.c.m. capacity, maximum depth 11.5m. The present pumping capacity is 660 t.c.m./d. Water from this reservoir passes to Ashford Common Treatment Works by a short tunnel (1952), or into the Staines Aqueduct or into a parallel 1800 mm dia. r.c. conduit (1926) to Kempton Park, with a 1200mm extension into Hampton.

On the south side of the river, 12.5 km below Laleham intake and above Molesey Lock, if the intake at Walton Pumping Station from which is filled the Queen Elizabeth II Reservoir, 20 000 t.c.m. capacity, depth 17.5m, supplying the treatment works at Surbiton and at Hampton. The nearby Knight and Bessborough Reservoirs, holding 5 100 t.c.m. together with a depth of 11.6m, supply the treatment works at Walton.

Summarising the normal use of those reservoirs which are in the line of supply, the reservoirs at Datchet, Wraysbury and Staines provide about 520 t.c.m./d to Hampton and Kempton Park Works, Queen Mary Reservoir provides about 330 t.c.m./d to Ashford Common Works, and Queen Elizabeth II about 360 t.c.m./d to Surbiton and Hampton Works.

There is, of course, deviation from this routine as operational exigencies and the selection of water require. Due to a combination of electricity supply tariff and pump efficiency, the regular use of Wraysbury is favoured and that of Datchet when it comes into operation. This may supplant the load on Queen Mary Reservoir, which is also the scene of ballast winning. An agreement made between the Authority's predecessor and a consortium brings royalties and, over 20 years, will deepen the water by 3m.

THE QUALITY OF WATER IN THE THAMES RESERVOIRS

Given that there is no problem in finding water in the river, the most important operational matter is the condition of the water as it may be drawn off. The river water is typical of a lowland river and densely populated country. A substantial proportion of the catchment area is intensively farmed, and above the intakes are contributions from sewage treatment works for about three million people. Average values of silicate (SiO_2), nitrate (N) and phosphate (PO_4) for 1971-72 are 11, 7 and 3 mg/1 (in the River Lea the last two are higher). This eutrophic water brings its own algae into the reservoirs but, when the retention period is more than about ten days, the reservoirs follow their characteristic cycle of successive blooms of green and blue-green algae, the bloom being determined in the end by the exhaustion of silica or other nutrient. Periodic crises struck most acutely in the early years of Queen Mary Reservoir, when growths of algae almost brought filtration to a standstill. In addition the depth of 11.5 m, though shallow by today's standards, allowed thermal stratification to develop in some summers. The quality of water extracted was improved in the 1930's and since, by decreasing the retention period, siphoning out water from stagnant zones, and preventing the growth of weeds on the perimeter banks above water level. Today the throughput is reduced again, but the construction of a mixing jet as an extension of one inlet has now made an improvement, albeit one dependent on continued filling.

It may be thought that the greener water near the surface and anaerobic water lying below the thermocline could have been avoided by careful selection of draw-off penstock in the middle depths, but changes in wind direction and force cause internal seiches and downwind accumulations of algae that make layer selection a very chancy matter.

Constructing the last three reservoirs to greater depth, between 16.5 and 25 m, has required certain steps to overcome thermal stratification. It would otherwise be difficult to avoid the de-oxygenation of half the reservoir contents in the summer months, and this water would be useless for supply. Inflowing water is therefore introduced at appropriate periods in spring and summer, a series of inlets operating as horizontal and inclined jets from the reservoir floor. The mixing induced by normal discharges at no higher velocity than 3 m/s is enough to make the temperature and oxygen content almost uniform from top to bottom.

At Queen Elizabeth Reservoir, low velocity inlets are used during autumn and winter so that, incoming energy being locally dissipated, the turbidity and bacterial level of the stored water is thereby reduced. In order to avoid thermal stratification it is necessary to employ the jet inlets from early Spring. It has been found [2] that the timing affects the intensity of spring blooms, and if the changeover is made at early stages of the diatom growth this appears to be discouraged although nutrients are there in the water.

The first filling in 1971 of Wraysbury Reservoir (22.5 m deep), being a back-charging from King George VI Reservoir up to 16 m depth, proved a demonstration [2] of seasonal development under natural forces only. A stable thermocline allowed a temperature difference to grow to 9.5 °C between top and bottom water. Hydrogen sulphide and ammonia were present in the bottom water by August, while successive heavy algal blooms in the top water would have led to severe problems if any of this water had been required at the time for filtration. When the inlet jets came into use (March 1972) to complete filling, the reservoir became virtually isothermal, with 90 to 100% oxygen saturation throughout and only minor blooms developing. It is the objective to keep the entire body of water in the range of 60 to 100% saturation so that good stored water can go for treatment, notwithstanding the periodic growth of algae which buoyancy may keep in the upper water.

To further prevent stratification at Wraysbury and at Datchet, recirculating pumps (225 t.c.m./d) are available for times when river water cannot be taken. These draw on middle water which is then ejected from low levels to effect the mixing required. A careful watch is kept on all reservoirs in order that they may be managed to the best advantage. At the last three to be constructed, remote recording of temperature and oxygen content at all depths at three observation towers plus regular sampling helps the biologist to advise on the means of control. Laboratories are maintained for this purpose by Scientific Services staff at Walton and Wraysbury, but sampling for general analysis and measurement of filterability are universal at 'in line' reservoirs.

The management of reservoirs should be carried out for economic total system cost [3], and the burden on filtration works is an important element in this. The variables to be governed so far as possible include the quantity and type of imported algae and silt, reservoir species in incipient development, the nutrients stored with the water, the amount of turbulence or turnover, rate of sedimentation, light penetration, temperature gradient and oxygen content. The means of control lie in pumping rate, types of inlet to be used according to degree and perhaps altitude of mixing advisable, or use of recirculating pumps and selection of draw-off level and, ultimately, a switch to another reservoir if available. Dosing with copper sulphate or other algicide is not now used, because of high cost and transient effect.

King George VI Reservoir (15.5m deep) is left for long periods as a standing reservoir. There is seasonal stratification and blooms of algae take place from time to time. An experimental recirculating pump is pontoon mounted in this reservoir, arranged to pump water at 90 t.c.m./d from a variable low level (below the thermocline) for discharge and mixing about 5m below the surface. It is used to lower the level of the thermocline, and at the same time increases its gradient and stability. This is another approach to ensuring that good water is available.

LEA VALLEY RESERVOIRS (Figure 2)

In this valley the abstraction of water is entirely in the hands of the Division, for the whole flow of the River Lea may be abstracted for water supply — which is invariably taken through the reservoirs — except for 20 t.c.m./d reserved for lockage of the Navigation. The group of smaller reservoirs at Walthamstow built over 100 years ago hold 2150 t.c.m., are not more than 6m deep, and are normally fed from an intake at Chingford Mill through the 11 m and 6 m Aqueducts, altogether about 3 km of channel. No pumping is required for this route so its use is only limited by the sufficiency of water in the river and by the favoured ten-day period of retention for the supply required by Coppermills Treatment Works.

The construction of improved links in tunnel, about five years ago, allows the water to travel from the head reservoir through two alternative pairs, but the capacity here is insufficient for retention for the 290 t.c.m./d required by Coppermills and the 40 t.c.m./d pumped untreated to Stoke Newington. The large reservoirs higher up the valley provide extra capacity for retention, a reserve against low flow, and also alternative sources of stored water if that in the Lea may not be taken or the Walthamstow water is too green. In recent years there have been periods when the nitrate content of Lea water has been above the W H O limit of 11 mg/l nitrate (N). The intakes have been closed, and Thames water imported as described below.

Lockwood and Banbury Reservoirs are of medium size, 2200 and 2800 t.c.m., not more than 10.4m deep, connected by tunnel and filled when necessary at Banbury by pumping from the 11m Aqueduct mentioned earlier. In practice Banbury Reservoir is left as a standing reserve. Greaves Pumping Station is rarely used, its value lying in its contribution to the capability of extraction from the Lea. The Lockwood Reservoir is used and refilled to a limited extent, mainly by Thames water from the Thames-Lea tunnel. It discharges either into the head Walthamstow reservoir or into the Coppermill Stream, an old channel about 1.2 km long leading to Coppermills Works.

Two large storage reservoirs stand at the head of the chain. The King George's and William Girling Reservoirs respectively hold 12400 and 15800 t.c.m. and are 9m and 12.5m deep. The famous Humphrey gas pumps (1913) were sited at the former and both can draw water either from the diverted channel of the Lea or from the Navigation.

The William Girling Reservoir can also be filled by pumping from the Thames-Lea tunnel. The greater depth of this reservoir and the jet-like arrangements of the inlet from Chingford South Pumping Station combine to give a better quality water, so that King George's Reservoir is normally shut-in as a long term reserve. A long outlet channel (3km) leads from King George's Reservoir into an older double brick culvert about 3km long, part of which lies below the 6m Aqueduct, and there is a confluence at the head of the Walthamstow Reservoirs. An alternative connection exists between this outlet channel and the 4km long 2135mm conduit which carries water from the William Girling Reservoir to the Coppermill Stream. From a connection off the 2135mm conduit a bulk supply of 72 t.c.m./d is pumped to the Essex Water Company at Lower Hall 'A'.

DROUGHT

The flow of the Thames and the Lea is such as to make only small demand on storage from year to year because of deficiency, and provision of the reservoirs has been dictated by and calculated upon the rarer years of drought as in 1921, 1933, 1944, 1949 etc. The drought year of 1943-4 has been adopted as a standard. Once the Thames-Lea tunnel had been driven, in 1960, from its Hampton intake on the Thames to discharge into the Lea Valley reservoirs, it became possible to consider the Thames and Lea flow and storage in a single calculation. In ordinary times changes in the rate of filling from the Thames are made by daily arrangement between the Metropolitan Water and the Thames Conservancy Divisions. When Thames flows are very much reduced, abstraction must by statute allow 765 t.c.m./d to flow over Teddington Weir, and at this time there may be hour to hour communication to trim the abstraction.

The flow over Teddington Weir may be reduced below 765 t.c.m./d after certain depletion of storage reservoirs has taken place. A statutory Order relates a prescribed minimum flow over Teddington Weir with the total amount of water left in storage for each day of the season from May to the end of the year. For example, if the amount of water in storage (both Thames and Lea basins) is 135000 t.c.m on August 30, at least 765 t.c.m./d must flow at Teddington. But if storage has fallen to this volume before July 2, the flow may be reduced to 225 t.c.m./d and at intermediate dates in proportion.

An operation plan exists for the balanced drawdown of storage in the Thames Valley, (1) North and (2) South bank, and (3) in the Lea Valley, in such a manner that the amount left in storage is proportional to the demand from those three systems. This routine may be weighted to advantage the Lea reservoirs by thirty days storage if the quality of the Lea, or the use of the Thames-Lea Tunnel, is in any doubt. The first deficiency is met by a general small drawdown in all areas, which is to allow 22500 t.c.m room to pick up any river flushes that sudden rainfall may bring. This was in fact all usefully taken up in 1971. The Lea and the South Thames reservoir levels are then stabilised by directing water preferentially, including use of the Thames-Lea Tunnel. This also has the merit of maintaining higher river flows down to the lowest Thames intakes. When the remaining volumes reach proportionality with the demand, a depletion in all areas follows to keep it so.

Certain pumping arrangements have to be initiated once reservoirs reach particular levels. For example the raw water pumps at Ashford Common are available to boost water from Queen Mary Reservoir and the Wraysbury and Datchet Reservoirs. The Walthamstow Reservoirs must be kept full as long as possible, and the obligatory supply to the Essex Water Co. can only be met from the William Girling Reservoir.

ADMINISTRATION AND SURVEILLANCE

Grassed reservoir banks and surrounding land amount to many square kilometres. A large proportion is let for sheep grazing and thus reduces the annual bill for maintenance. An unwelcome feature of the reservoir banks in the spring and summer is the Chironomid midge, whose adult form swarms above the embankment grass forming dense and annoying clouds of harmless insects. The remedy, on call, is the fog of pyrethin carried on a mineral oil vapour, an effective cheap and impersistent insecticide which is harmless to mammals.

More acceptable inhabitants of the water surface are sailing clubs, now well established in the Lea and Thames reservoirs. The Lea Valley Regional Park Authority and the Inner London Education Authority operate a sailing club and training centre respectively, on King George's Reservoir and Banbury Reservoir. Island Barn Reservoir sailing is leased to a local sailing club. A club has been formed to sail Queen Mary Reservoir and another is in embryo for Datchet. The Authority's policy is to provide club houses and lease on a rent which is as near the economic level as is consistent with ensuring maximum public use.

There has been coarse fishing for many years. Trout are now reared in disused filter-beds adjoining the Coppermill Stream and fly-fished nearby in the low level Walthamstow Reservoirs. Day tickets are issued at £1.00 and the take limited. Trout are also reared in a reservoir at Kempton Park, and it is proposed to stock Datchet Reservoir in due course.

Almost all the tunnel connections to reservoirs are wet tunnels, and are emptied for inspection once every five years. Annual chlorination of draw-off tunnels is conducted to destroy the fresh-water mussles that otherwise accumulate and reduce the carrying capacity. As unfiltered water may become foul while standing in the dark, a small flow is always maintained in the longer tunnels.

Since construction started on Queen Elizabeth Reservoir, in 1957, gauges of various kinds have been built into the foundation and embankments of the last three storage reservoirs to be built. So long as these continue to function and be useful quarterly observations are required to be made. At most, however, these installations can only give the strains, pressures, etc characteristic of the sections where they are installed, and in an earth embankment five km long this cannot amount to ubiquitous monitoring. At Wraysbury and Datchet and elsewhere, therefore, a trial is being made of observation points spaced 100m apart in connected alignments round the toe of the bank, where ground water level, ground level and alignment can be tested quickly and regularly, at least in a comparative way, to reveal local aberrations.

The Thames Valley and the Lea Valley reservoirs are in the charge respectively of two Senior Resident Engineers, with a Reservoir Foreman under each. The largest units are each in the hands of a Reservoir Attendant, while the smaller ones are grouped.

Besides his work in operating valves, checking gates and fences etc., the attendant has a basic role in surveillance for safety. He makes a daily inspection of his reservoir, noting all events: work being carried out, grass cutting, trespass, and any observation of damp patches, cracks, movement. He will report any anomaly immediately. A concrete wave-wall or fence line, for example, may form an alignment which he can readily check when he scrutinises the embankments.

The reservoir foreman makes a weekly inspection, examines and signs the reservoir book at the attendant's office. At least twice yearly the Resident Engineer makes an inspection, examines and signs the book.

The foregoing inspections are additional to statutory inspections and records. The Reservoir (Safety Provisions) Act, 1930, required that statutory inspections shall be carried out by an independent qualified engineer, except that the undertaker's engineer who is responsible for maintenance, if qualified, may inspect. This exception was inserted by Parliament during the passage of the Bill in order to meet objections raised by the Metropolitan Water Board, who valued the experience of their engineers. For the remaining 44 years of the Board's life, statutory inspections were carried out by its successive Chief Engineers, and it is fair to say that the Board's record of attention to its reservoirs was second to none. The reorganisation of the water industry ensures that most of its reservoirs can be in the hands of qualified engineers and in the ownership of corporate bodies of substance, but the new Reservoirs Act will place inspection in other quarters. Time will tell whether this has been a wise change in legislation.

ACKNOWLEDGEMENTS

The Director of Operations, Mr. E C Reed is thanked for permission to give this paper and Mr. G F Mugele and Mr. A Birtles for their advice in its preparation.

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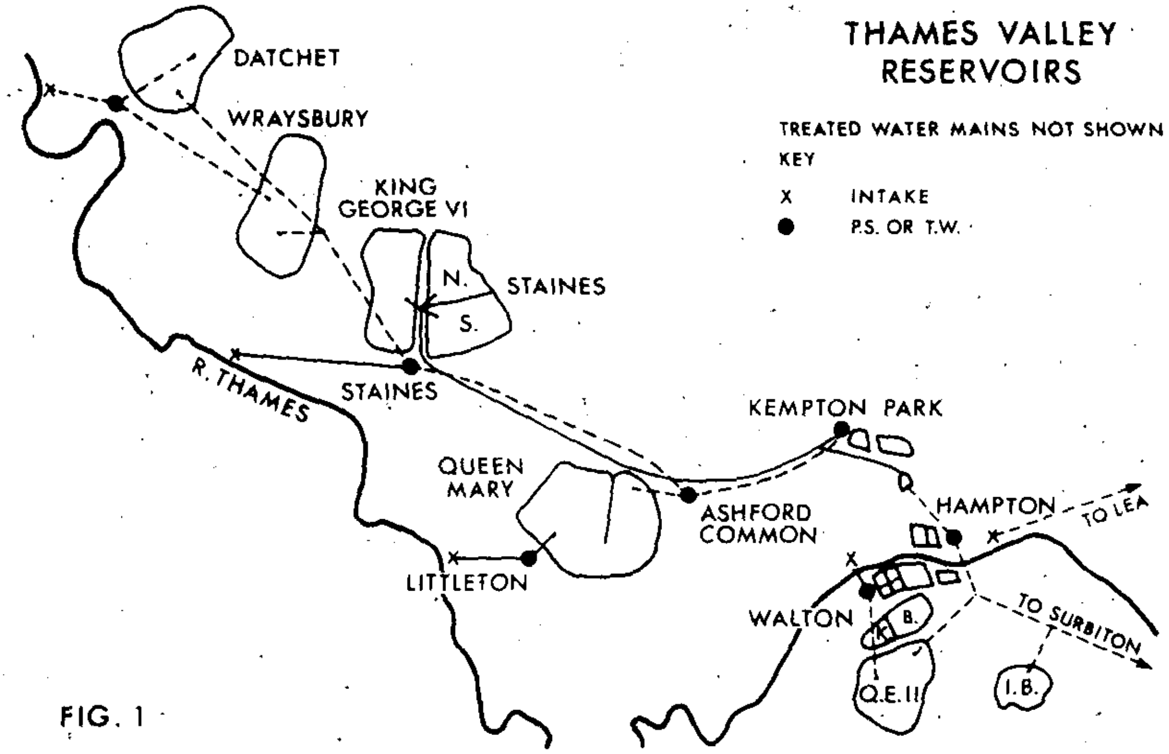
THAMES VALLEY
RESERVOIRS
TREATED WATER MAINS NOT SHOWN
KEY
X INTAKE
P.S. OR T.W.
DATCHET
WRAYSBURY
KING
GEORGE VI
N.
S.
STAINES
STAINES
R. THAMES
KEMPTON PARK
QUEEN
MARY
ASHFORD
COMMON
LITTLETON
HAMPTON
TO LEA
WALTON
B.
TO SURBITON
Q.E.II
I.B.

FIG. 1

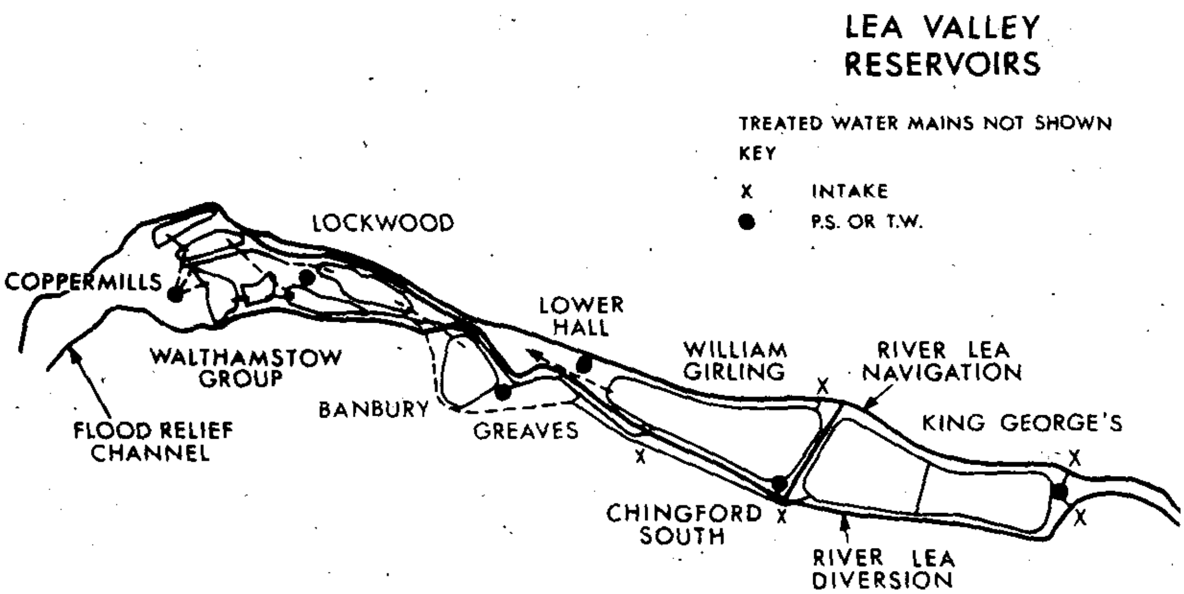
LEA VALLEY
RESERVOIRS
TREATED WATER MAINS NOT SHOWN
KEY
X INTAKE
P.S. OR T.W.
LOCKWOOD
COPPERMILLS
WALTHAMSTOW
GROUP
FLOOD RELIEF
CHANNEL
LOWER
HALL
BANBURY
GREAVES
WILLIAM
GIRLING
RIVER LEA
NAVIGATION
KING GEORGE'S
CHINGFORD
SOUTH
RIVER LEA
DIVERSION

FIG. 2

INSTRUMENTATION OF BOOTHWOOD DAM

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SYNOPSIS

The Boothwood Dam, completed in 1971, is a mass concrete gravity structure. Vibrating wire strain gauges were used in its instrumentation and temperatures were measured by electrical resistance elements incorporated in the strain gauges. The deflections and movements of the dam were measured by three methods : triangulation, plumb-bob and levelling and aligning the crest. The stresses derived from the strain meters in the dam indicate that the stresses are eventually generally compressive at all locations. Slight tensile stresses were indicated in most of the early measurements. The dam was found to rotate in a downstream direction during impounding.

INTRODUCTION

THE HISTORY OF BOOTHWOOD DAM

The Boothwood Dam is in Rishworth near Ripponden. It was commissioned in 1964 by Wakefield and District Water Board Authority and in July 1967 the Contractors commenced excavating for foundations. A gritstone quarry, which was to supply the aggregate for the concrete, was opened out one mile from the site. The reservoir was filled in October 1970.

DESCRIPTION AND CONSTRUCTION OF DAM

Boothwood Dam is a mass concrete gravity dam, consisting of 25 monoliths, each 12.2m wide; eleven to the South of the centre line and fourteen to the North. The four central monoliths are of equal height and one of these, No. Sl, was used for the principal instrumentation investigation. The maximum height of the dam is 61.2m, the crest length is 352.5m, and the maximum thickness at the base is 45.7m. The length of the spillway is 50m (see Fig. 1.) The capacity of the reservoir is 3637Ml. In order to exercise some control over uplift pressure, boreholes were drilled down from foundation level and these have been kept open with extension pipes to relieve any water pressure into the lower gallery in the dam.

DESCRIPTION OF SOIL BENEATH THE DAM

The strata underlying the dam consist of alternating gritstones and shales of the Millstone Grit Series. The foundation for the dam was excavated to natural beds and joints stepping up in a downstream direction from the base of the upstream heel. The depth of excavation, because of disturbed strata beneath monolith S6 and parts of monoliths S5 and S7, was deeper than initially assumed. The disturbed strata were proved by excavation to consist of a series of exceptionally wide fissures containing irregular angular blocks of gritstone, rounded cobbles of liminated micaceous sandstone and in places some layered silt, sand and clay.

INSTRUMENTATION OF BOOTHWOOD DAM

THE STRAIN METER

During the construction of the dam instruments were embedded in the concrete to determine the conditions of stress and temperature developed during the period of construction and reservoir filling. Figure 2 shows a central section of the dam and the layouts of structural behaviour measurement systems. The instruments comprise twelve vibrating wire gauges. These gauges are installed at two different levels in monoliths S1, (see also Fig. 3.) Some gauges are installed close to the foundation level at the upstream toe and downstream heel of monolith S1, where maximum tensile and compressive stresses can be expected. At 11.5m above the foundation level other gauges are installed at different positions of width and length of the dam to indicate any stress differentials that might exist at these positions. Laboratory tests were made on cores from the concrete to determine values of Modulus of Elasticity and enable stresses to be computed from the measured strains.

RESISTANCE THERMOMETERS

An electrical resistance element, incorporated in each of the vibrating wire strain gauges, had its resistance read with a potentiometer incorporated in the portable comparator (receiver) to record temperature. Temperatures were measured at various points in monolith S1 to assist in the interpretation of stresses and strains from the strain measurements. At the same time shade temperatures of the atmosphere were taken corresponding to the internal temperatures.

DETERMINING DAM DEFORMATION

Notation

u - Displacement in y-direction (mm)

v - Displacement in x-direction (mm)

w - Displacement in z-direction (mm)

L - Length between two stations on horizontal plan - (mm)

β, β', α and α' - Horizontal angles (radians)

θ, ϕ and ϕ' - Vertical angles (radians)

ε_t - Strain

C - Thermal coefficient of expansion

ΔT - Change in temperature from the initial temperature (^{o}C.)

a) GEODETIC METHOD

One system for determining deformations of the dam comprised a grid pattern of 25 observation targets installed on the downstream face of the dam. Figure 3 shows the location of the targets. Theodolite stations are located downstream from the dam. They were rigidly fixed to hard ground so that their positions remained unchanged during the tests. The respective locations of suitable points on the dam were established by intersection, by sighting from both ends of a base line of known length, and each end of the base line was a rigid theodolite station. The vertical and horizontal deflections of the dam at any point are obtained from equation 1 and 2 The vertical deflection of the point in the z-direction is obtained from equation 3 These equations are developed in the Reference. The deflections were determined at intervals over four years. Computer programming was used to obtain these deflections from the measured angles.

$$u = L\left(\frac{\sin\beta\sin(\alpha-\theta)}{\sin(\pi-\alpha-\beta)} - \frac{\sin\beta'\sin(\alpha'-\theta)}{\sin(\pi-\alpha'-\beta')}\right) \quad \ldots\ldots \quad (1)$$

$$v = L\left(\frac{\sin\beta\cos(\alpha-\theta)}{\sin(\pi-\alpha-\beta)} - \frac{\sin\beta'\cos(\alpha'-\theta)}{\sin(\pi-\alpha'-\beta')}\right) \quad \ldots\ldots \quad (2)$$

$$w = L\left(\frac{\sin\beta\tan\phi}{\sin(\pi-\alpha-\beta)} - \frac{\sin\beta'\tan\phi'}{\sin(\pi-\alpha'-\beta')}\right) \quad \ldots\ldots \quad (3)$$

b) PLUMB-BOB METHOD

The second system for determining dam deformation measured the deformation of the top of the dam with respect to points near the foundation. Shafts for the plumb-lines were formed in the concrete between the foundation gallery and the top of the dam. The plumb-lines are located as shown in Figure 2.

DISCUSSION OF THE RESULTS

STRESSES

Readings of strain gauges in the dam have continued regularly since installation, at daily intervals for the first month then about once each week for one year, and thereafter every fortnight. Approximately 150 readings were taken of each gauge over a period of about four years.

The strains indicated by each gauge can be split into two components :

1. Strains resulting from stresses in the concrete,
2. Strains caused by thermal movement.

Strains caused by change in temperature can be determined from the equation :

$$\varepsilon_t = C.\Delta T \qquad \text{(4)}$$

In general the vertical strains increase from the upstream to the downstream face. The vertical strains continue to increase with time, except for some variation due to water level (see Figure 4). The maximum stress indicated was 0.765 MN/m^2 compression in November 1973. The core concrete used was specifiedto have a 152 mm cube strength of 10.5 MN/m^2 at 28 days.
It consisted of a 9:1 mix with 30% of the cement replaced with pulverised fuel ash (fly ash).
In general the shapes of the longitudinal strain curves indicated compressive stresses. In December 1968, tension was indicated longitudinally near the heel in the upstream part of the section, 3m above the foundation level, for the duration of a few weeks during impounding. The longitudinal stresses generally continue to increase with time. Stresses increase towards a downstream direction then decrease near the downstream face. Some examples of stresses are now given. The maximum stress indicated at 27m in downstream direction from S.O.L. (Setting out line, see Figure 2) was about 0.455 MN/m^2 compression at the end of May 1973.

Gauges D3 and D12 are at the same level of approximately 11.5m above the foundation and are 28.5m apart, East to West. D12 had an early temperature of 14°C in April 1969, whereas D3 had its early peak temperature of 33.6°C in September 1969. Both gauges had the same temperature of 23°C in late September 1969 and they both settled down to approximately 10°C in October 1972. Gauges D9 and D10 are both approximately 26.5m East of S.O.L. and they are 9.2m apart vertically. In November 1969 the temperature recorded by D9 was 9°C and D10 recorded 11°C. In September 1972 D9 recorded 7°C and D10 recorded 11°C. These differences in the internal temperature have an effect upon internal stresses. Comparing observed stresses with those determined by the geometric analysis method(1), the magnitude of the calculated stresses are more similar to the measured stresses in the upstream than in the downstream portion of the section of the dam. An explanation of these differences could be that they are due to temperature changes. For example, the measured stresses given by Gauge D6, which is near the downstream face, and the corresponding calculated stresses generally seemed to follow the same law, but the magnitude of the measured stresses was generally lower than the calculated stresses by approximately 25% (see Fig.4)

RESULTS OF DEFLECTIONS

CREST LEVEL

Triangulation on the stations to points on the downstream face of the monoliths and levelling and lining the crests of the monoliths commenced in November 1970, shortly after impounding commenced. The A.O.D. level of the crests for monoliths designated N decreased during the Summer of 1971 by a maximum of approximately 12 to 13mm for the central monoliths and then increased by about 25mm during the winter of 1971/1972. Observations continued during 1972, showing the same pattern. The A.O.D. level of the crests for monoliths designated S increased during the Summer of 1971, decreased during the winter of 1971/1972 and continued to increase during 1973. The difference in performance between monoliths designated N and those designated S is probably due to the soil beneath the dam and the pore pressure in that soil.
The crest level for monoliths designated S seemed to be affected by water level, in that greater water levels lifted the dam to some extent. The maximum lift was between January and February 1972 for monoliths S3 and S6. The observation bores showed that little seepage was taking place around or beneath the southern portion of the dam. The indication is that there is a higher uplift pressure beneath the northern than the southern monoliths. This may account for the cause of the rise in crest level. In all cases the crest settled to begin with and future settlement was not very much greater. The maximum settlement was about 2l to 22mm.

MOVEMENT DOWNSTREAM

Deflections were obtained from trriangulation measurements on the stations on ten monolitl
The rotation of the monoliths in the direction of the stream are obtained from measurements on the plumb-bob lines. Initial readings showed a slight tilt in a downstream dierection during

winter 1972/1973. By April 1973, when the water level of the reservoir dropped rapidly, the readings showed a slight change towards an upstream direction. When the reservoir was full in March 1973 the readings indicated a downstream deflection of about 12 to 13mm at the central monoliths (Figure 5 shows the variation of deflection with respect to time and reservoir water level).
The deflections of the crest of the dam essentially have two components:

1. The movement upstream due to uneven settlement of the dam when there is no water pressure.
2. The movement downstream due to the water pressure.

The relationship between movement downstream and time seems similar for similarly positioned N and S monoliths. The slight exception is in the case of N7, S7, N8 and S8. The differences here may be due to the differences in foundation levels.
The agreement of calculated deflections with those determined by triangulation is very good except for those monoliths in the middle of the dam and the monoliths S6 and N3, where the calculated and measured deflections were of opposite sign. (Fig.5). This may be due to the presence of weak rock in this area of the dam, or a layer of weak material at some distance beneath the area of contact of the dam with the rock. Subsequently to finishing the construction of the dam, holes were drilled from the galleries into the rock and also at the heel, and the fissures in the rock filled with cement grout.

FUTURE INVESTIGATION AND IMPROVEMENTS FOR STUDYING THE STRUCTURAL BEHAVIOUR OF DAMS

SOME POINTS WHICH MAY BE OF USE TO FUTURE INVESTIGATORS

1. A good quality theodolite accurate to at lease one half second is essential for this type of triangulation.
2. The best times for taking theodolite observations are early in a morning or late afternoon.
3. The vibrating wire gauges used for measuring internal stresses are most satisfactory.
4. Many of the gauges used incorporated means of measuring temperature electrically and this was not satisfactory.
5. The original sum allocated for instrumentation was decided without consultation with the Authors, being included as a nominal amount compared to the cost of the dam - some dams in the U.K. appeared to have had no instrumentation and this was a great handicap throughout the whole conduct of the work. Also, the dam was partially constructed before consultation. This limited the places where strain measuring instruments could be incorporated.
6. It would have been nice to have had the luxury of more internal strain and temperature measuring gauges in various extra positions, and of a special laboratory programme of work for investigating the properties, such as, for example, stress/strain relationships, Poisson's Ratio, creep, shrinkage, and temperature coefficient of expansion, of the very large aggregate concrete. The 100m aggregate is too large for the use of normal concrete laboratory test specimens.
7. Finally, one could perhaps say that instrumenting dams is necessary for two reasons: 1) to be happy that a disaster is not commencing and this means a minimum amount of instrumentation readings being continually taken, considered and acted upon if necessary throughout the whole life of the dam, and 2) to see how real stresses and deflections depart from existing theories in order to throw more light on future more economic design of dams.

ACKNOWLEDGEMENTS

Acknowledgements are due to the late Mr. T E S White, Engineer and Manager, Wakefield and District Water Board, a much liked and respected man in his profession, for initially engaging the senior Author to help decide upon the minimum instrumentation considered necessary for the dam, and subsequently to guide the instrumentation. The writers also acknowledge the co-operation and enthusiasm of his successor Mr P J Gadd, who is now Water Manager, South Western Division, Yorkshire Water Authority. During the period of the instrumentation the writers acknowledge the considerable help of the following staff of the former Wakefield and District Water Board: Mr E Haydock, Resident Engineer, Mr J A Readman, Mr D Fox and Mr J Finn. Reference has been made occasionally via Messrs White and Gadd to their soils advisers, Edgar Morton of Manchester.

REFERENCE

1 Mohsen, M N (1974) *The Structural Behaviour of Boothwood Dam* MSc Thesis. Univ. of Bradford.

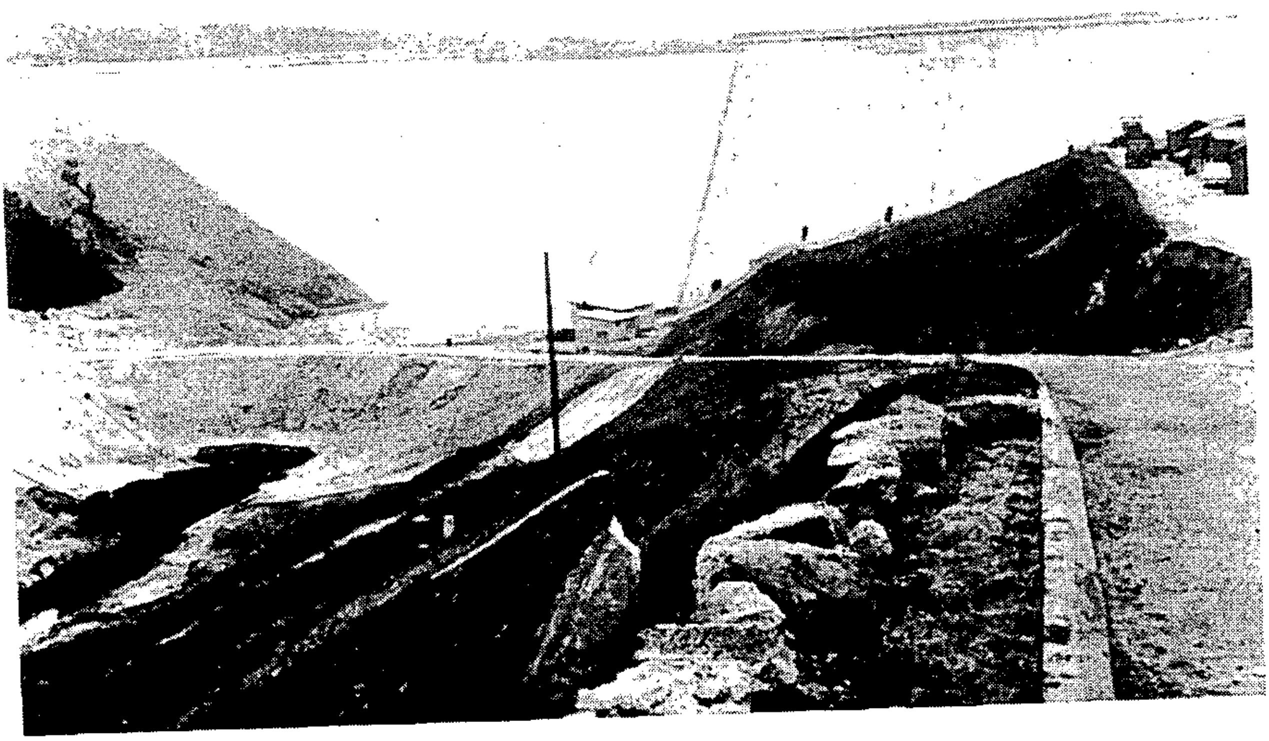


Fig. 1 Boothwood Dam

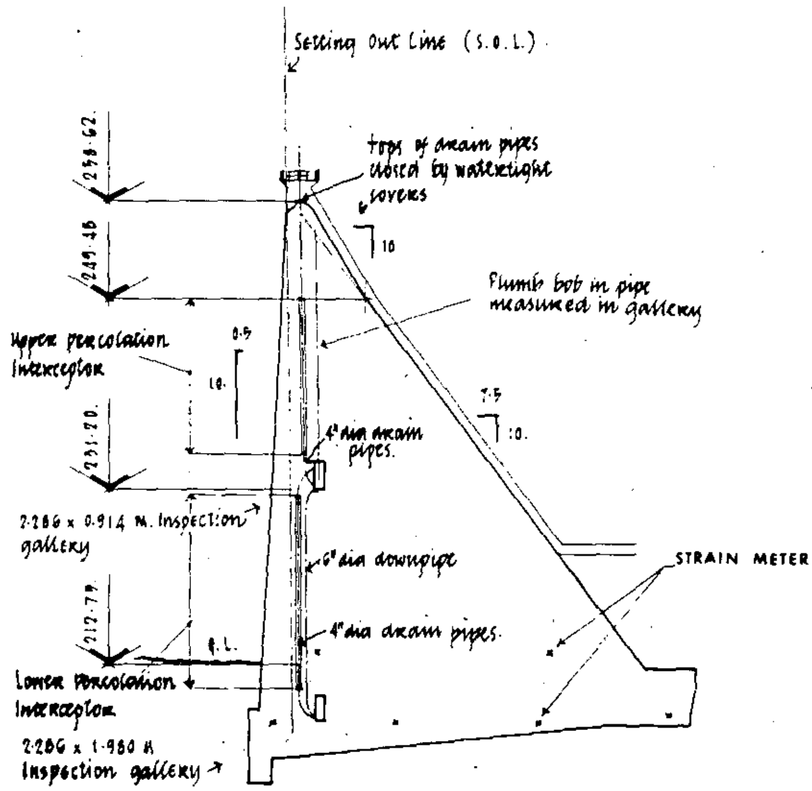


Fig. 2 Construction section

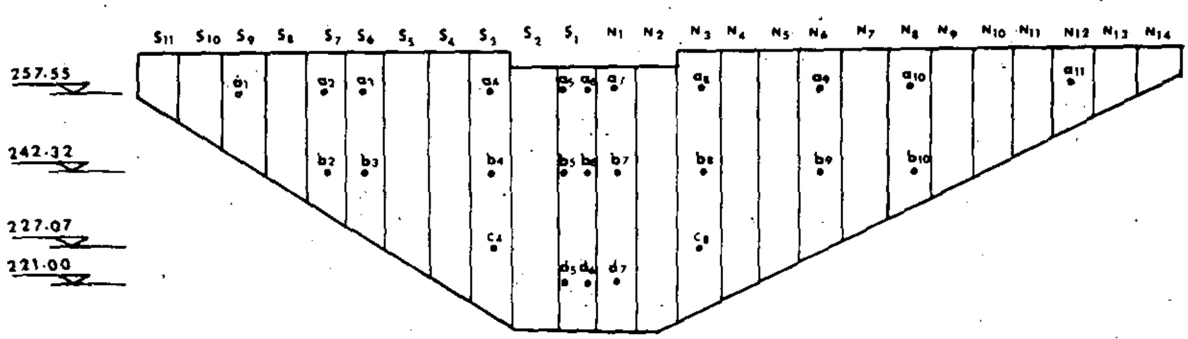


Fig. 3 Position of targets on downstream face

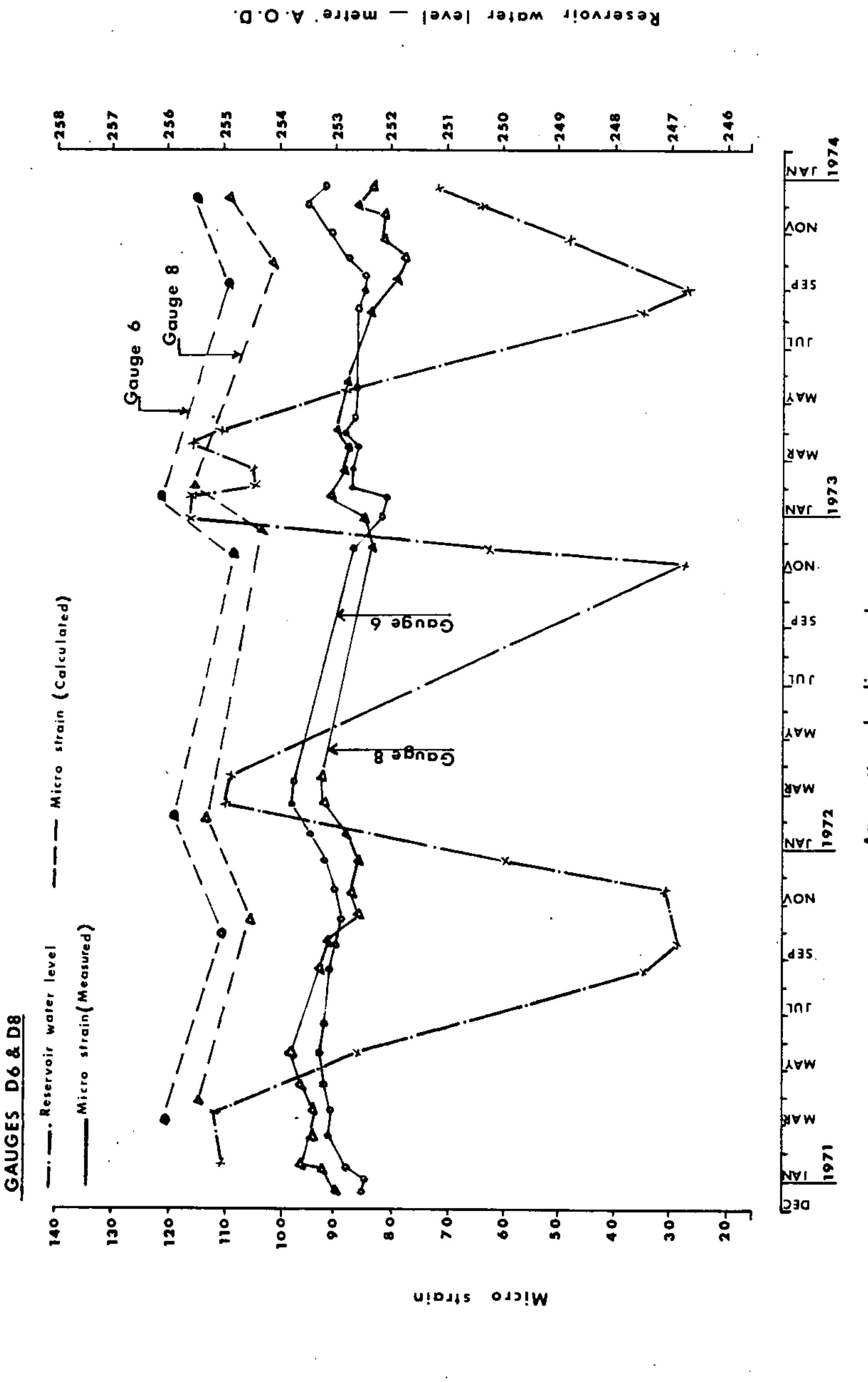
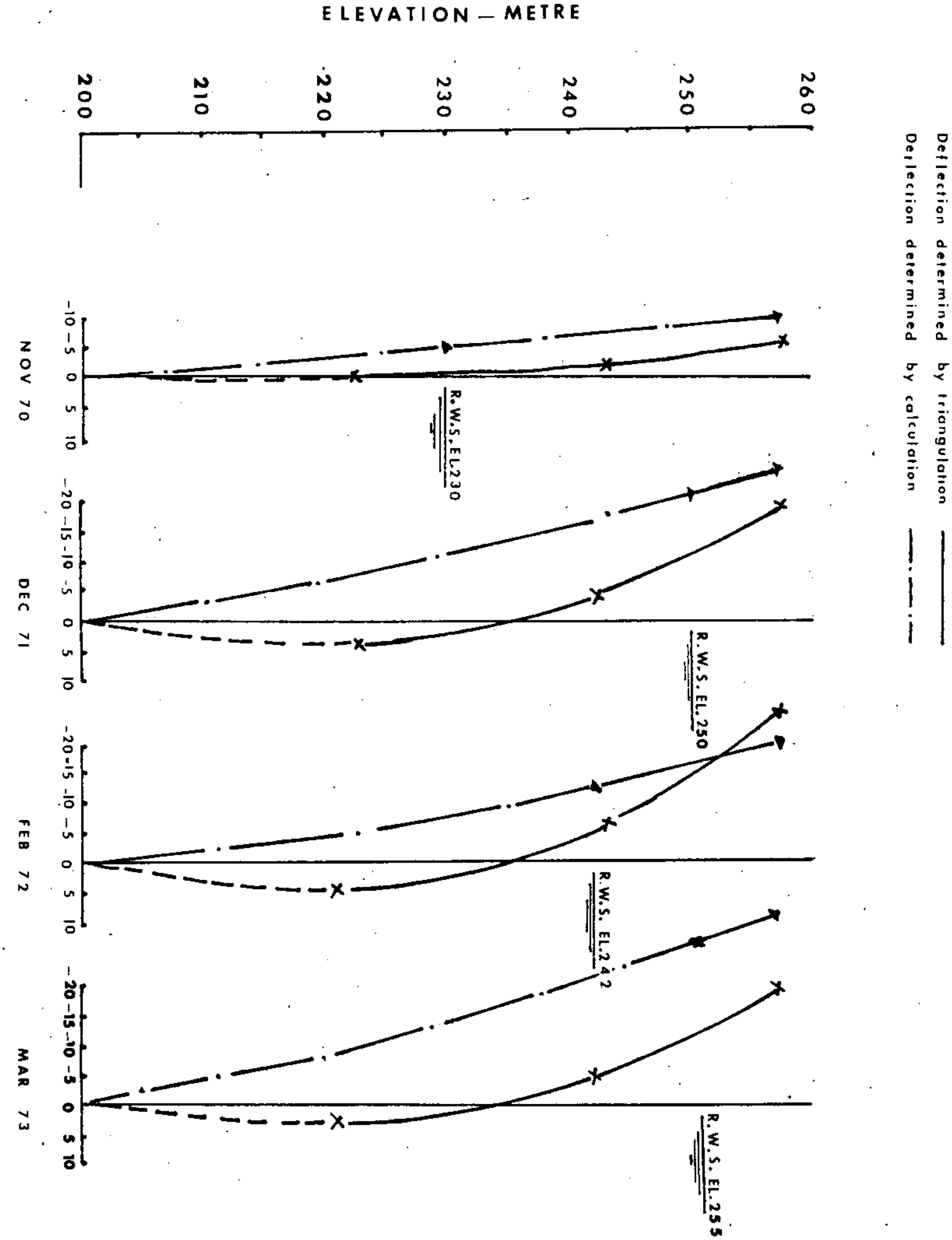


Fig.4 STRAIN FROM ISOLATED GAUGE AFTER PERIOD OF STORAGE

BOOTH WOOD DAM
MONOLITH S1



DOWNSTREAM DEFLECTION — MILLIMETRE

Fig 5

OPERATION OF THE RESERVOIRS OF THE NORTH OF SCOTLAND HYDRO-ELECTRIC BOARD

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NORTH OF SCOTLAND HYDRO-ELECTRIC BOARD

SYNOPSIS

Hydro-electric power supplies approximately 40% of the Board's power requirements and this is derived from 33 schemes comprising 54 power stations and 76 reservoirs which were certified under the Reservoirs (Safety Provisions) Act 1930. The operating regimes for these reservoirs have been designed to meet two major operating conditions, viz. normal operation to meet the electrical supply requirements and flood conditions where local conditions dictate requirements for each reservoir. The bases and development of these regimes are described.

PART 1 - NORMAL OPERATING REGIMES

INTRODUCTION

The Board operate 33 conventional hydro schemes comprising 76 reservoirs and 54 power stations with a total installed capacity of 1052 MW and an average annual output of about 2900 million units (32% annual load factor). Maximum reservoir storage capacity is 942 million units or about 32% of average annual output.

The Board also operate two pumped storage schemes, Cruachan (400 MW) and Foyers (300MW). Operation of these plants is based on fairly straight-forward principles and consequently most of the first part of the paper is concerned with conventional hydro schemes with only a short section on pumped storage.

Some of the hydro plants are located in hydraulically isolated sites but many are in river valleys with main storage reservoirs discharging in cascade to one or more reservoirs downstream. Figure 1 shows the layout of the Conon Valley, a typical cascade group. Here, Lochs Fannich and Glascarnoch, with storage capacities of 101 and 33 million units respectively, provide the main storage.

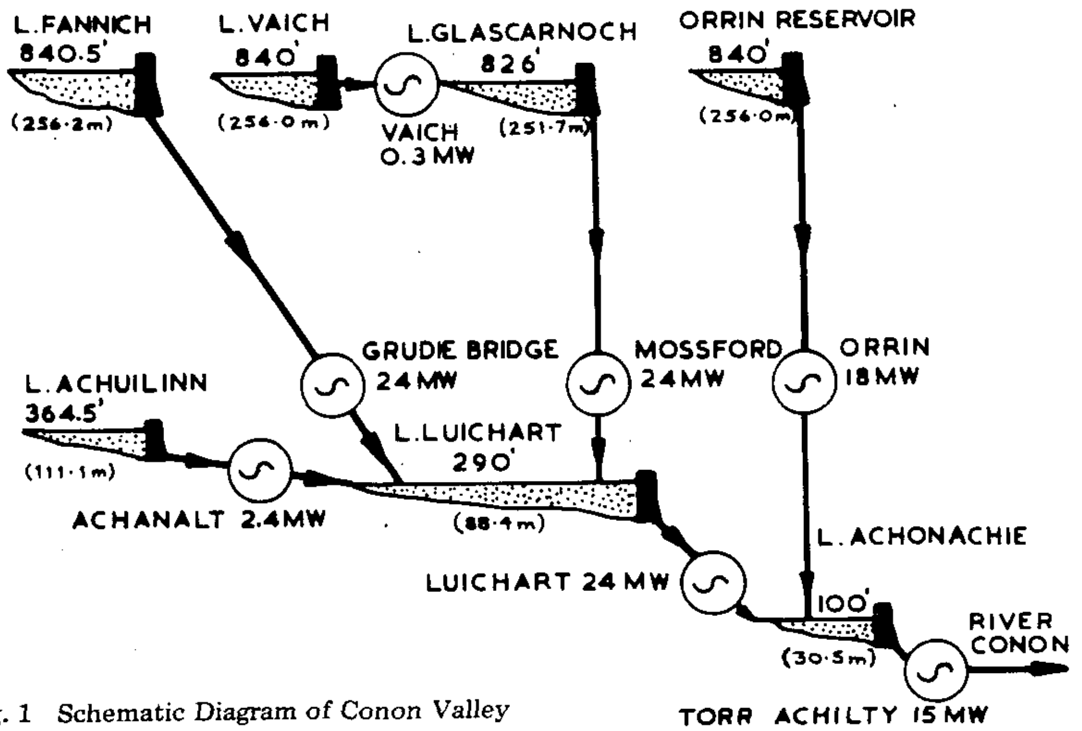

Fig. 1 Schematic Diagram of Conon Valley

Hydro schemes now supply only about 40% of system energy requirements, the remainder being largely provided by thermal generation at the Carolina Port oil-fired station in Dundee, and import from the South of Scotland Electricity Board. To achieve minimum generating cost for Scotland the Board's generating plant is operated in close co-ordination with the mainly thermal system of the South of Scotland Electricity Board to meet a total Scottish load which in 1973/74 had a peak demand of about 5500 MW and annual energy requirements of approximately 26,300 million units. The Board's hydro plant is thus in effect part of a much larger, predominantly thermal, system.

Control of the hydro stations is undertaken by seven Group Control Centres which allocate duties to individual plants to meet the group output required by the Board's Central Control at Pitlochry.

The following sections describe how duties are allocated to conventional hydro schemes throughout the year and on a day-to-day basis.

LONG TERM PLANNING

The average annual distribution of run-off in the Board's catchments follows the load pattern fairly closely so that with average run-off high hydro output is available in the winter heavy load period, when it is most required, without large variations in storage.

However, the climate is such that unpredictable departures from average are common and the methods of reservoir control are to a large extent influenced by this. Three control curves are used as guidelines for the operation of each main storage reservoir throughout the year, viz. Maximum Output Level (or Full Generation Level), Minimum Duty Level and Economic Average Duty Level. In deriving the first two, use is made of long term run-off records for the River Moriston going back to 1929 which are periodically updated.

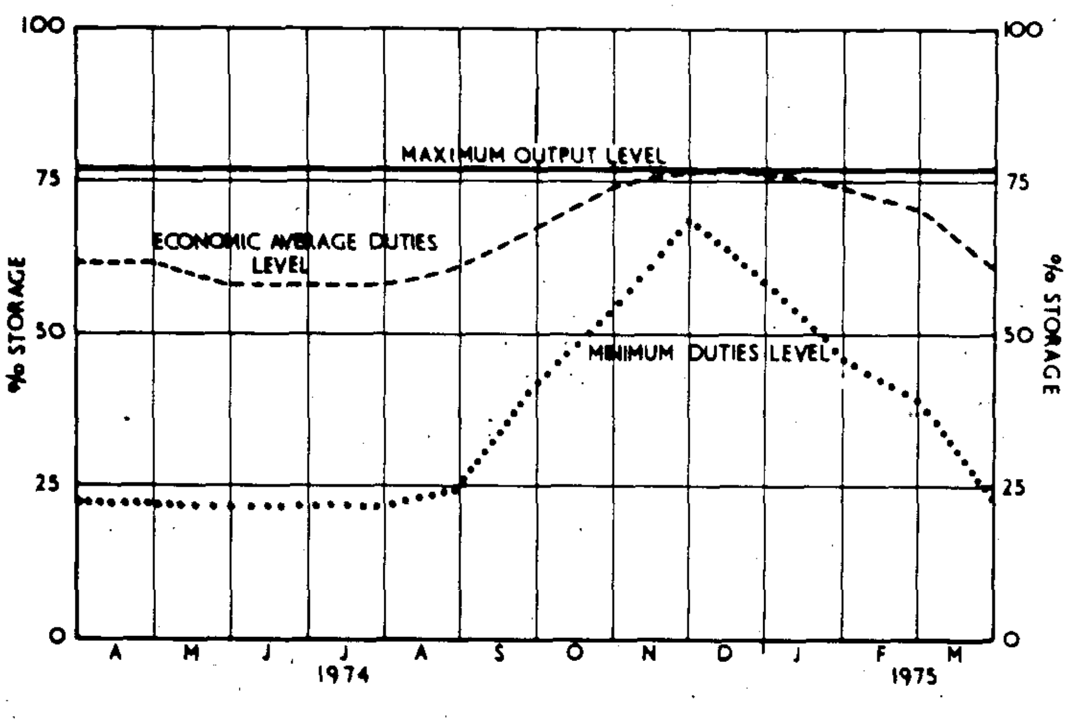

Fig. 2 Loch Lyon Control Curves 1974/75

While there is diversity between the incidence of run-off in the different catchment areas, statistical tests have shown that this catchment is reasonably typical of Board catchments in general. Figure 2 shows the three control curves for Loch Lyon for 1974/75.

MAXIMUM OUTPUT LEVEL

As the title suggests this is the reservoir level above which the plant should normally be generating continuously at maximum output. The level for each reservoir was based initially on calculations of spill probability using River Moriston records of previous high run-off periods and assuming maximum generation except in the case of upper cascade reservoirs, which provide the major storage. Here an allowance is made for reducing generation in the early stages of a flood when downstream reservoirs are spilling from local run-off and there may still be appreciable freeboard at the upper reservoirs. The Maximum Output Levels have been modified when necessary in the light of practical experience.

MINIMUM DUTY LEVEL

Monthly minimum duties are calculated for each main reservoir for periods up to two years ahead. The duties depend upon four factors :

(1) The Scottish load remaining to be supplied after all available thermal and pumped storage plant is at maximum output. This load must be supplied by hydro and can be allocated to individual reservoirs.

(2) Generation for security to limit import into any particular section of the system to a value which can safely be carried on the transmission system following the loss of major circuits on fault.

(3) Generation to provide river flow requirements, mainly for fishery purposes.

(4) Allowance for additional duties not covered in the previous sections. This applies particularly to peak load reservoirs such as Sloy whose duties cannot be accurately forecast.

The duties derived from the above are the minimum acceptable and reservoir storages must be controlled so that each station is capable of meeting this load for a period of up to 52 weeks even with a repetition of the lowest run-off experienced in the past.

Computer calculations based on the minimum duties and on the lowest recorded run-off for periods up to 52 weeks result in a series of levels for each reservoir over the year known as the Minimum Duty Curve. If generation is reduced to the minimum duty whenever storage falls to the Minimum Duty Level then storage should not fail at the end of any dry spell lasting up to 52 weeks provided the run-off is not less than that previously recorded.

ECONOMIC AVERAGE DUTIES LEVEL

Within the limits described in the previous section, the reservoirs are operated to make the best economic use of the run-off. This entails high output in winter when thermal generation costs are high and lower output in summer.

Knowing the maintenance programmes of the thermal, pumped storage and hydro plant on the system and the expected system load, and assuming average monthly run-off, the most advantageous level for each reservoir is calculated for each month. This is known as the Economic Average Duties Level and is used for budgeting and guiding operation throughout the year. It is highest in late autumn in preparation for the winter heavy load period, falling in the spring and building up again in the autumn.

DAY TO DAY OPERATION

As stated previously the three control curves and associated duties are for guidance. In the day to day allocation of duties to individual reservoirs they are used in conjunction with the actual conditions at the time. The procedure is as follows.

Each day an estimate is made of the hydro output which can be made available for the following day.

It is based upon three factors:

(1) Actual levels of individual reservoirs

(2) Estimated run-off depending upon the weather and weather outlook

(3) Generation requirements for economic operation and security.

The output of individual hydro plants is calculated according to the storage relative to the three control curves so that outputs could vary from maximum, where the reservoir is above the Maximum Output Level, to minimum for reservoirs below the Minimum Duties Level.

The aggregate hydro megawatt and energy outputs are compared with estimated Scottish daily load curves and adjusted if necessary so that the hydro and pumped storage plant cater for as much as possible of the day period load fluctuations, leaving the thermal plant with as constant an output as possible throughout the day. Every endeavour is also made to keep down the amount of older, high fuel cost thermal plant required.

During the day the hydro programme may be altered at short notices because of differences between actual and estimated conditions (e.g. breakdown of thermal or pump storage plant, high run-off, weather colder or warmer than estimated). Even if the storage is below the Minimum Duty Level additional output will be provided for short periods if required and relief given at a later date when conditions may be easier than estimated.

PUMPED STORAGE

Cruachan was commissioned in 1966 and Foyers in 1974. Since Cruachan came into service the differential between night and day fuel costs of Scottish thermal plant has been high enough to make it economic to pump at night on all but a few occasions.

These stations pump for about seven hours each night and day generation varies according to system requirements. When Cruachan was planned it was anticipated that no generation would be needed during the week-ends and the plant would operate to a weekly cycle but, due to the growth in load at the week-end, some generation is required on Saturday and Sunday.

PART 2 - FLOOD REGIMES

INTRODUCTION

For most of the Board's Schemes, there is now 10 to 20 years' valuable experience of operating the reservoirs and routing floods. During this period, most of the Schemes have experienced one or two major floods which have highlighted specific problems and the potential dangers of these floods.

On many of the Projects there was a paucity of meteorological and hydrological data on which to design, but many of these unknowns can now be defined from operating data. Moreover, the Interim Report of 'Floods in Relation to Reservoir Practice' was limited, particularly with respect to large catchments.

It has been the Board's recent policy to appoint Panel Engineers for Statutory Inspections who have not been associated with the design or construction of the reservoir so that a completely independent and fresh assessment can be made [1]. This resulted in fairly searching re-appraisals being made and the bringing to light of aspects which were not suspected as being unsatisfactory and which required detailed investigation.

The above factors led to a Floods Study Group being formed within the Board in 1972 to undertake detailed studies of a number of Schemes where problems had arisen. The Group comprised the Generation Engineer of the particular hydro scheme, Meteorologist, Hydraulic Engineer and reservoir safety Engineer, under the Chief Civil Engineer. Its terms of reference were:

(1) To develop general methods for analysing floods

(2) To re-assess the magnitude of floods in particular catchments and basins.

(3) To establish safety criteria for each reservoir and basin

(4) To revise, as necessary, the normal operating and flood regimes for schemes.

BASIS OF STUDIES

The aspects which were taken into account by the Group were:

(a) Operational data, meteorological conditions, catchment characteristics, gate performances etc.

(b) Experience of flood routing in the basins or valleys since the Schemes became operational

(c) Potential improvements to normal operating regimes

(d) The proposed method of the Floods Studies Team of the Institute of Hydrology for prediction of extreme rainfall and run-off. The Group's initial studies were undertaken before the Institute of Hydrology Report became available, but during the past two years generous help and advice has been given by the Institute which has allowed this modern method to be generally adopted in the studies.

In addition the rapid development of computers over the past decade has permitted flood routing proposals for complex cases to be analysed much more easily than in the past.

It was also felt that some of the provisions in the Interim Report were unsatisfactory for the following reasons:

(1) Efforts were made to attach a return period to normal maximum and catastrophic floods, as defined by the Floods Report, but it was found that the range of return periods arising was too great to be of use. Analysing and extrapolating the data did not give a consistent return period for the catastrophic floods, the return period decreasing with catchment area.

(2) The Floods Report only defines the hydrograph shape for catchments up to 100 km^2. When dealing with catchments of 200-1000 km^2 extrapolation is excessive.

(3) Particularly in cases where the reservoir area is a relatively high percentage of the catchment area, the stipulated '50 cusecs/1,000 acres' initial discharge is very severe and may represent in itself a flood with a return period of the order of 10 years.

(4) Multiple reservoir systems are not adequately treated.

To ensure consistent criteria were applied to all reservoirs, the Group drew up tentative safety criteria. Much difficulty was encounted in determining appropriate return periods for the flood event to be adopted for analysis. The questionnaires sent out by ICOLD (2) in 1973 show wide divergencies in standards adopted for maximum design floods from 100 - 10,000 year return periods. Moreover, many countries adopt different return periods for new and existing dams, for different types of dams,and for varying degrees of risk. After much consideration, a 1,000 year return period (1 in 1,000 chance of occurring in any year) was adopted for existing dams, based upon subjective judgement with the following criteria as objectives :

(1) The dam structure must remain safe.

(2) Peak river flow and/or reservoir level under flood conditions should be no greater than pre-scheme where the flow or level is critical.

(3) Hazard to life should be avoided.

(4) Some material damage to plant, structures and property may have to be accepted. On specific schemes there may be a further requirement of no material damage with a flood of a specified return period of less than 1,000 years.

(5) Normal operating regimes should be improved where possible to give a better utilisation of the catchment area and reservoir storage.

With regard to item (2), particularly where gated spillways are adopted and the scheme involves raising an existing loch, the post-schemes flow can be greater than pre-scheme flow if the flood control procedures are not carefully formulated.

At the present time, where the Interim Report can be reasonably applied the dam must also remain safe under catastrophic flood conditions if this event should be more severe than the 1,000 year event. With the three schemes analysed to date, certification of the reservoir under the Reservoirs (Safety Provisions) Act 1930 has been related to the more severe event, but it is expected that this provision will be removed once the new ICE recommendations become applicable.

HYDROLOGICAL ANALYSIS

In analysing schemes, the first objective is to obtain an estimate of the run-off from a storm event with a 1,000 year return period. The resultant run-off is a combination of several components of varying importance; rainfall intensity, duration and profile, antecedent river flow and soil moisture deficit.

Ranges of probabilities for the above components were considered in combination to give 1,000 year storm events. As a result of this exercise, it has been found sensible to set the antecedent conditions to the most likely (median) condition and consider only combinations of rainfall volume, duration and profile. The assessment of each of these components and the way in which they are combined to produce the most onerous 1,000 year flood event are discussed in detail by Jarvis (3) and Reynolds (4) in papers to this Symposium.

Computer programs are written for each basin containing built-in equations covering hydrological parameters, reservoir storage, operation of machines and aqueducts for each reservoir for both pre-scheme and post-scheme conditions. This enables standard reservoir lag calculations to be undertaken for specified inflow hydrographs for each section. In cases where valley storage is significant, equations of valley storage/discharge are also built into the program enabling attenuation calculations to be carried out. Where valley storage is small, attenuation is ignored and a specified delay time based upon field measurement is used.

Data to allow computation of an inflow hydrograph for each section of catchment is specified as input for a run. The upper reservoirs in a basin use these hydrographs without modification, but the lower reservoirs in cascade have the hydrographs modified by the delayed discharge from the reservoir immediately upstream. In post-scheme cases, machine flows and the condition of aqueducts can be included as required and initial reservoir levels can be selected as appropriate. The programmes are typically arranged to give the hourly variation of inflow, outflow and level for each reservoir.

RESULTS OF STUDIES

After development of the computer program for a valley, the Group have endeavoured to feed into the program the parameters of major floods which have been experienced and recorded and to run the program to check whether the actual results recorded could be reproduced. To date, these checks have been reassuring, giving results close to those encountered during the floods, as will be seen from Figure 3 shich shows the actually recorded and computer predicted flows for the 1966 Flood in the Beauly Basin.

The discrepancy between the actual and predicted flows at Beannachran Dam is believed to be due to an error in waterman's readings.

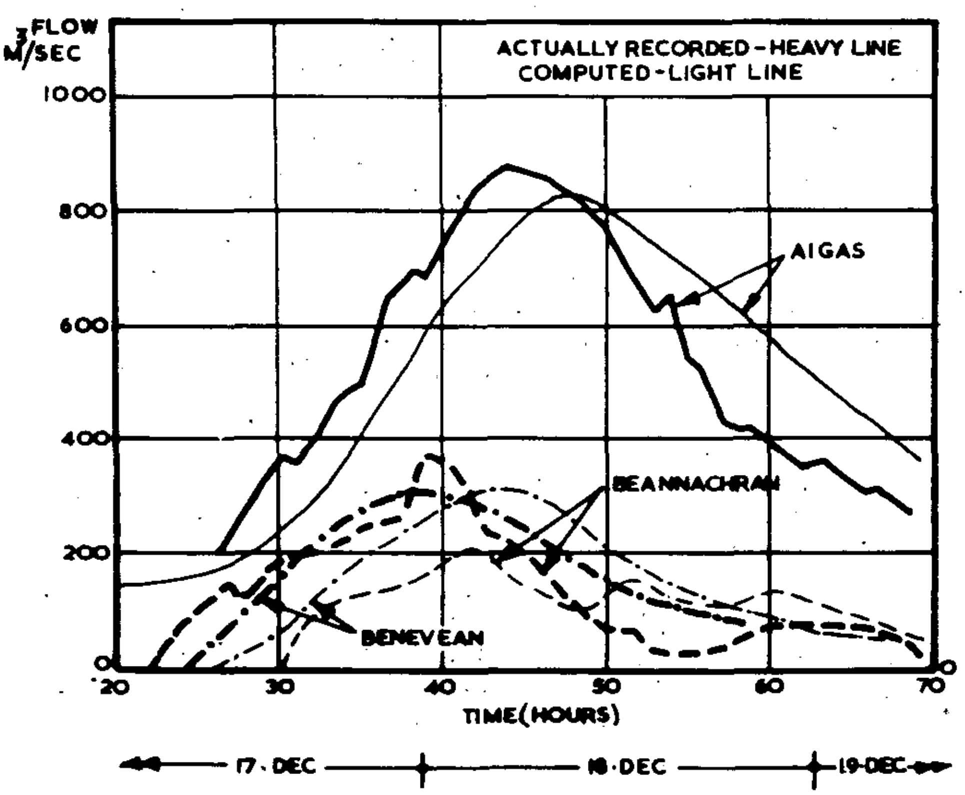

Fig. 3 Comparison of Recorded and Computed Flows in Beauly Basin. 17th-19th December 1966

Three major schemes have been analysed to date viz Awe, Beauly and Shin. The results of the Beauly Study are described in detail by Jarvis [3].

AWE SCHEME

The Awe Scheme utilises loch Awe as a main reservoir with a gated barrage at the outlet from the Loch and a power station at Inverawe near the mouth of the River Awe. The Cruachan Pumped

Storage Scheme utilises the Loch for its lower reservoir and the Nant Hydro-Electric Scheme discharges into it. The Barrage is a concrete gravity structure 18m high which regulates the Loch within its natural range.

The Scheme was designed on the basis of the Interim Report as the only information available at the time, although it was recognised that its catchment area of 840 km^2 was completely outwith the range for which the Report is appropriate (up to 100 km^2). The antecedent flow of '50 cusecs/ 1,000 acres' was also excessively large for a catchment of this area. Some 10 years' operating experience is now available and with the new Institute of Hydrology method of flood estimation it was decided to undertake a re-appraisal of the Scheme and its operating procedures since there were suspicions that the rainfall and run-off data were inaccurate.

The methods previously referred to were used, adopting a 1,000 year storm event as criterion. The results of the Study have led to a greater confidence in handling the whole range of floods with worthwhile reductions in minor flooding at the mouth of the River Awe. In addition, it is estimated that spill losses will be reduced by about £15,000/annum and some £12,000/annum will accrue by additional units generated.

SHIN SCHEME

The Shin Scheme utilises Loch Shin as its main reservoir into which is directed the headwaters of the rivers Cassley and Brora, with a total catchment area of approximately 650 km^2. Because of fishery considerations, a two-stage damming arrangement was adopted with a concrete and embankment dam some 12m in height situated at the narrows dividing Loch Shin from Little Loch Shin. An intake dam about a mile downstream diverts water into the tunnel serving the main power station at Inveran on the Kyle of Sutherland and at the same time regulates the level of Little Loch Shin.

There has been a long history of flooding of the village of Lairg every few decades, including post-Scheme in 1966 due to overflowing of Little Loch Shin. The main objective of the Study was to attempt to ameliorate the flooding of Lairg without impairing the output of the Scheme. The maximum acceptable level at Lairg Village depends upon the level of Little Loch Shin, which in turn is controlled by the discharge past the two dams and the flow to Inveran Station.

The scheme was analysed using the Institute of Hydrology method, modified with local data for various magnitudes of storm and a range of spill levels and flow regimes from Loch Shin. Estimates were made of the flow and levels which would arise in Little Loch Shin with the objective of discharging as much water as possible early in the flood without inundating Lairg. It was found that it was possible to accept a 500 year storm without flooding Lairg by modified gate control and without significant spill.

To check the estimates of level and flow, fullscale tests were carried out last December during a period of heavy rainfall and high loch level. The estimates of levels of Little Loch Shin versus flow were verified and shown to be within a range of 2.5 cm to 4.0 cm of those computed.

The above studies were undertaken prior to the publication of the Flood studies Report (5) and some adjustments will be necessary in the future to comply with its recommendations and with the possible requirements of the new Reservoirs Act,1975. Until this new legislation has been implemented the reservoirs have been assessed so that thev comply as far as practicable with the requirements of the Reservoir (Safety Provisions) Act 1930.

The effort spent by the Board on the three Schemes described has been modest, amounting to a total of just over three man years over the past three years, although this effort is necessarily of high calibre.

CONCLUSIONS

The Board's operational regimes are aimed at maximising the amount of hydro electricity generated during periods when the marginal cost of thermal generation is high while keeping spill losses down in wet weather and avoiding storage failure in dry. They are not rigid rules, but are used in conjunction with the evaluation of actual system conditions at any particular time.

Some 10 to 20 years experience of operating the Board's Schemes has revealed deficiencies in operating regimes and identified specific flood problems. For large catchment areas, over 100 km^2 and upwards, the Interim Report of Floods becomes increasingly inaccurate and inappropriate. With the advent of the new approach developed by the Institute of Hydrology in their Flood Report and the much better computer facilities now available, opportunity has been taken to analyse in detail a number of large schemes in the Highlands. The findings have been illuminating and valuable and have pointed to refinements and further development of the present operating regimes, improved flood management and, in the odd cases, increased output.

Analysis of other Board Schemes is now in hand with the ultimate objective of reviewing all Schemes and refining the operating and flood management procedures. Priority is being given to Schemes where problems have been encountered or deficiencies have become apparent.

On many projects there is often a paucity of meteorological and hydrological data on which to base the design. After the Schemes have operated for a few years many of the uncertainties can be resolved by detailed analysis of the Schemes, leading to improved operating and flood arrangement procedures with a very small expenditure of high calibre and experienced effort.

ACKNOWLEDGEMENTS

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THE OPERATION OF THE WATER RESOURCE SYSTEM OF MANCHESTER

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SYNOPSIS

A review is made of the physical development of the Manchester Water Resource System from the pioneering works in Longdendale to the fully integrated system of reservoirs pumping stations and aqueducts in operation today. An outline is given of the progressive advance that has taken place in the operation and management of the system. Derivation of pumping rules for the pumped augmentation of an impounding reservoir is described and an example given of the method used to check the supply position for a particular group of direct supply reservoirs during low inflow conditions.

INTRODUCTION

There are numerous considerations affecting the operation of a water resource system and these include aspects of hydrology, water quality, treatment, aqueduct management, distribution, amenity considerations and economic and industrial factors.

HISTORICAL DEVELOPMENT

LONGDENDALE

The reservoirs on the River Etherow at Longdendale in the West Pennines are the oldest Manchester source of supply still in use. Construction began in 1847 and at the time the scheme was the most immense and complex ever attempted in Europe. The scheme consists of seven impounding reservoirs, including two which because of their lower elevation are used for storage of compensation water for the River Etherow. Although water from Longdendale was first supplied to Manchester in 1851, work continued for another thirty three years on the dam and aqueducts and, since then, service reservoirs and treatment works have been constructed for this source. The resulting system is complex but the slow development ensured that staff had time to assimilate each new addition into the operating methods.

THIRLMERE

Manchester continued to develop into a major industrial centre and the demand for water soon approached the safe yield of Longdendale; however further potential local supplies become unavailable because of the effects of river pollution. In 1879 Parliament authorised Manchester to develop the Thirlmere Catchment in Cumberland by raising the lake level and building an aqueduct link nearly 160 km long. Under the terms of the Act other Authorities along the route were entitled to a supply of water from the Aqueduct.

HAWESWATER

The affording of bulk supplies from the Thirlmere Aqueduct foreshortened the time when a further major resource would be required. In 1919 Manchester was authorised to acquire Haweswater and adjacent catchment areas, but due to the economic depression it was not until 1941 that this water reached Manchester and then it was via a temporary connection into Thirlmere Aqueduct. Construction of the first stage of the Haweswater Aqueduct was interrupted by the war and was not completed until 1955. In the 1950's, water from the adjacent catchments of Heltondale, Swindale and Wet Sleddale was diverted into Haweswater. Although these diversions nearly doubled the inflow to the reservoir, they added to the complexity of operation.

ULLSWATER AND WINDERMERE

As demand for water grew at an even faster rate, Manchester looked for another major resource in the Lake District, but considerable opposition had grown from the amenity lobby and it took seven years and the rejection of one Parliamentary Bill before a Ministry Order for the Ullswater/Windermere Scheme was made. The scheme uses pumped abstraction from Ullswater and Windermere to augment the Lake District direct supply reservoirs. The Water Order, however, imposes severe limitations on the operation of the pumping stations in order to preserve minimum river flows and amenities.

This augmentation scheme was intended to make use of the greater aqueduct capacity available to the South of the Watchgate Treatment Plant at Kendal and accordingly included for the construction of an additional aqueduct link, down the Longsleddale valley to the North. This vital link, however, was deleted from the Water Order and full benefit of the scheme will not be obtainable until 1977/78 when the Shap Aqueduct, constructed as the alternative to the Longsleddale link, comes into operation.

TREATMENT PLANTS

In the original development of the sources the policy had been to purchase the entire catchment area so that access could be restricted and other activities carefully controlled to prevent pollution of the source. Demands arose for amenity use of the catchment and water space, yet at the same time standards for drinking water rose and the more polluted lakes such as Ullswater and Windermere came into consideration as possible sources. Modern treatment works were therefore constructed on the Longdendale supply in 1962 and, under the Ullswater/Windermere Water Order, at Watchgate, on the Haweswater Aqueduct, in 1973.

PERSPECTIVE

Figure 1 shows the reliable yield of resources at each commissioning date superimposed on the historical record of rising demand. It illustrates that as demand rises more rapidly, new sources have been introduced over a shorter time scale and experience can no longer be relied on as a basis for efficient operation. With the advent of the Ullswater and Windermere Scheme the Lake District sources became interlinked and it was necessary to derive operating rules for the efficient management of the headworks, aqueducts and treatment processes. It was also necessary to introduce more sophisticated methods of assessing the reliable yield of the system as it became more and more vital to use the 'last drop', since possibilites for further major developments were becoming scarce and costly in the light of the worsening energy crisis.

PRESENT RESOURCE SYSTEM

Figure 2 is a diagrammatic representation of the existing system, and the basic operating principles are discussed below :—

THE LONGDENDALE GROUP OF RESERVOIRS

The demand on these reservoirs is now approaching the reliable yield and operating rules based on synthetic drought sequences and specifying a permitted depletion each month for a chosen rate of draw are in use. A monitoring analysis can be introduced to determine the supply position to suit the pattern of demand and operational difficulties being experienced at any particular time.

ASSESSMENT OF OPERATING POLICY DURING DROUGHT

In an average year, more than the reliable yield may de drawn from the reservoirs, but as reservoir level falls below a chosen permitted depletion curve the rate of draw must be cut back to conserve supplies in case of ensuing drought. The level of cut back is generally chosen on operating considerations to balance the distribution system. However, it may happen during a drought that various operating difficulties arise with the result that a cut back in supply cannot be implemented as soon as indicated on the permitted depletion diagram, the reservoir level has to be artificially lowered for maintenance work, or the aqueduct restraints prevent water being drawn at the required rate. In the case of Longdendale a fine balance must be maintained between the storage of water in the compensation and the supply reservoirs in order to be able to meet statutory obligations. The following method of monitoring allows flexibility in operation according to the circumstances and enables an assessment to be made of the need to apply for a Drought Order.

The basis of the monitoring analysis is the 1% synthetic drought (Figure 3). Consider the situation where the reservoirs are becoming depleted well below the permitted depletion curve and a cut back in the rate of draw is indicated. The weather has been dry over the past few months and there is every prospect of it continuing so. It would be too pessimistic to consider that the inflow in the following months will have a 1% probability of occurrence, since if the inflow in the preceding months has been of this severity then the addition of further such low inflows in the following months would result in a total drought inflow with a very low probability of occurrence.

It is necessary, therefore, when planning for the future months, to take the past into consideration This method assumes that a drought of 1% probability of occurrence began at some time (to be estimated) in the past and will continue. The object is to find the critical duration of such a drought as far as the reservoir is concerned, taking into account the past pattern of draw-off, and at what rate water can be safely drawn from the reservoir if a 1% risk of failure is accepted.
The data required for each assessment is :—

a) The 1% synthetic drought, cumulative inflow.

b) The date the reservoir was last effectively full.

c) Weekly or monthly inflows to the reservoir since it was last effectively full.

d) Present water available in store (S).

e) Compensation requirements.

The method is illustrated in Figure 3 :—

1) A first estimate of the start of the drought may be the date at which the reservoir was last effectively full.

2) From that point in time, inflows to the reservoir are summed to the present day, total = I, and this is marked on the diagram.

3) The duration, D, from the start of the drought to the present day is calculated and the point X with co-ordinates (D,I) is plotted. The vertical distance between the line I X W and the drought curve is the expected inflow in the coming months. This will probably be negative in the first few weeks indicating that it is now no longer possible to have a 1% drought of this duration beginning when the reservoir was last full.

4) New perpendicular axes are drawn through the point X and the point Z plotted with co-ordinates (O, —S) with respect to new axes, i.e. X Z represents the present storage S.

5) The gradient of the tangent to the drought curve passing through the point Z is the average rate of draw Y that the reservoirs can sustain should the drought become of 1% severity, but there is an attendant 1% risk that the reservoirs will be completely empty at the point T in time.

From Figure 3 —

Total inflow I	=	0.1 x A.A.R.O. t.c.m.
Duration D	=	3½ months
Present storage	=	0.175 x A.A.R.O. t.c.m.
Duration of critical period	=	16 months, i.e. 12½ months from present day.
Maximum rate of draw Y	=	0.002 x A.A.R.O. t.c.m./day

This analysis may be repeated with the assumption that the drought began at some later date. If the resulting value of Y is lower than the first estimate then this lower value should be adopted.

The vertical distance between the line ZV and the inflow curve is the storage remaining in the reservoir at each point in time and gives an indication of when a Drought Order might be required the usual criterion being when forty days supply is left. An alternative approach is to draw the line ZV at a slope equivalent to a required rate of draw and assess the length of time the chosen rate of draw can be sustained before the 'forty days supply' level is reached.

The basic method was extended to include two separate analyses for the supply reservoirs and the compensation reservoirs at Longdendale, assuming that the respective synthetic droughts would be in the same proportion as the catchment areas.

Variation of the existing compensation water discharge is under consideration and investigations are in hand to determine the additional reliable yield which may be available if the present constant release is replaced by a prescribed flow in the river and if storage at existing terminal reservoirs is used. Figure 4 shows initial estimates of yield derived from simulated operation of the reservoirs during a synthetic drought. Adoption of a prescribed flow of 0·6 m^3/s with a minimum release of 0·265 m^3/s and a maximum release of 0·53 m^3/s would result in a gain in yield of 0·138 m^3/s.

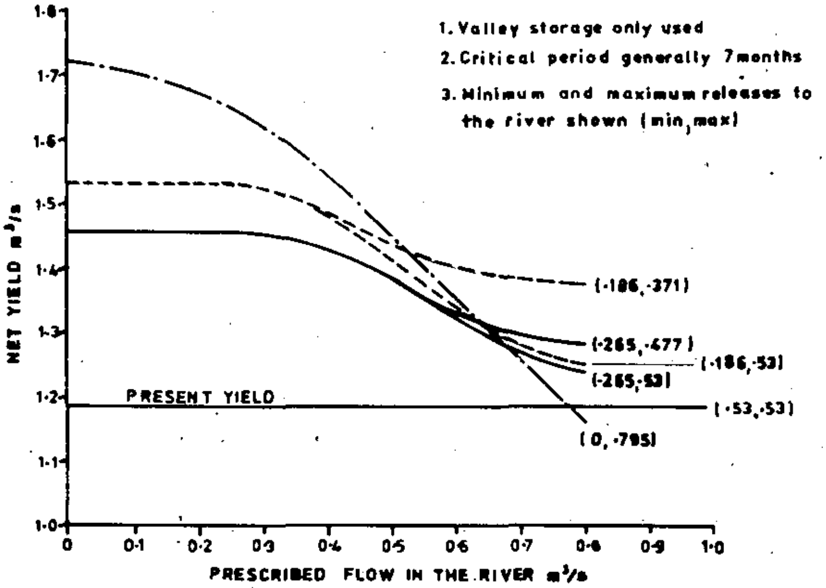


Fig. 4 Net Yield of Longdendale under Various Compensation Requirements.

THE LAKE DISTRICT SOURCES

The pattern of development of these sources has been considered in stages as construction work has progressed from the direct supply reservoirs of Thirlmere and Haweswater to :—

1) Ullswater/Haweswater (without Shap Aqueduct).
2) Windermere/Thirlmere.
3) Conjunctive use of the total Lake District sources with Shap Aqueduct.

The conjunctive use of the Longdendale sources with those of the Lake District is difficult at the moment because of treatment plant restrictions at Longdendale.

In order to maintain the reliability of the augmented system and at the same time keep spillage to a minimum, the augmentation source must be operated in a carefully calculated manner. This can either be in accordance with a design operating rule or in a manner shown by *ad hoc* calculation to maintain reliability in a particular circumstance.

Augmentation by pumping must not only be carried out against the possibility of failure of Thirlmere and Haweswater, but also against the possibility of pumping not being permitted, because of Water Order limitations, at the very time when it is needed. However, as combined pumping and treatment costs of the Lake abstractions are of the order of £7.5/Ml against treatment only costs of £1.2/Ml for the gravity supplies, considerable sums of money are wasted if spillage occurs.

The factors wich determine the form of the optimum pumping rules are therefore :—

a) Hydrological — inflow patterns to various sources.
b) Physical — capacities of pumps, reservoirs and aqueducts.
c) Legal — terms of the Water Order, licences, compensation water releases.
d) Operational — water quality, distribution, amenity.
e) Economic — electrical energy consumed in pumping (terms of the electricity tariff).

These five sets of restraints make the production of rule curves a complex matter.

Two basic methods of producing rule curves for the separate systems of Haweswater/Ullswater and Thrilmere/Windermere have been considered by the former Manchester Corporation Waterworks staff and their Joint Consulting Engineers.

The first is based on synthetic droughts of 1% probability beginning in each month. By daily simulation, the quantity of water available from the pumped source is calculated for one particular pumping rate, taking electricity tariff and water order restrictions into account, and is added to the natural inflow to the reservoir. This total inflow is then compared with the total required to supply a chosen average rate of draw, the difference being the storage required to maintain that rate of draw for that particular pumping rate during a drought beginning in a particular month. The process is repeated for each of the twelve droughts to give twelve points for the rule curve. An example is shown in Figure 5. It is not necessary to pump as soon as the storage has dropped below the no pumping line; the yield can still be maintained without pumping if the storage is allowed to drop to lower levels provided that once the storage reaches the maximum pumping rate curve, the pumps are used to their full capacity. In practice, these types of rules are used conservatively and pumping introduced at a low rate when the corresponding depletion occurred. It was felt that an operating rule designed to allow for a gradual increase in pumping rate as the reservoir depleted would be easier to operate, although more difficult to derive. The second method produces such an operating rule.

This was acheived by simulating operation of the system over the historic record of thirty five years. A rule was postulated for each simulation run, based on the rule curves derived by the first method, but allowing pumping to increase in stages with depletion of the reservoir, and similarly to decrease in stages as storage increased. The maximum depletion in each year was recorded and extrapolated to a 1% probability of occurrence. If this 1% value of depletion was greater than the available storage, the pumping rule was adjusted and the simulation repeated until the 1% value was approximately the available storage. The rule was then further adjusted to reduce pumping costs, if possible, and consideration was given to the proporotion of time that the reservoir was drawn below any chosen 'amenity level'. An example of such a rule is given in Figure 5.

This second method is very flexible as it can accommodate any changes in operation which may become necessary. It could be improved by using a longer data record from data generation. It is proposed to use an adaption of this method to produce operating rules for the more complex system of the Lake District sources used conjunctively when Shap Aqueduct is in operation.

THE FUTURE

The North West Water Authority at present has under construction the first stage of the Lancashire Conjunctive Use Scheme. This scheme involves abstraction of water from the River Lune to meet the increased demands of the Central Lancashire Area. The Manchester Area will benefit from this scheme, however, by a reduction in the quantity of water taken from the Thirlmere Aqueduct en route, thus making better economic use of the Aqueduct carrying capacity. Long term strategy for water resources in England and Wales, proposed by the Water Resources Board in 1973, included proposals for modifications to the use of two of Manchester's major Lakeland sources, and, by implication, the whole Lake District system. One proposal is the enlargement of Haweswater, the other the partial redeployment of Thirlmere to supply West Cumbria. From initial investigations it would appear that the latter can be implemented on a small scale without seriously affecting supplies to Manchester or creating any operating difficulties. However, an enlarged Haweswater could cause a complete change in the economic balance of the operation since water from the catchwaters, as well as that from Ullswater and another augmentation source, would have to be pumped to the higher level. The Manchester Area resource system has been continuously developed and adapted as society and economic values have changed and it is unlikely that an ultimate system will be arrived at in the foresseeable future.

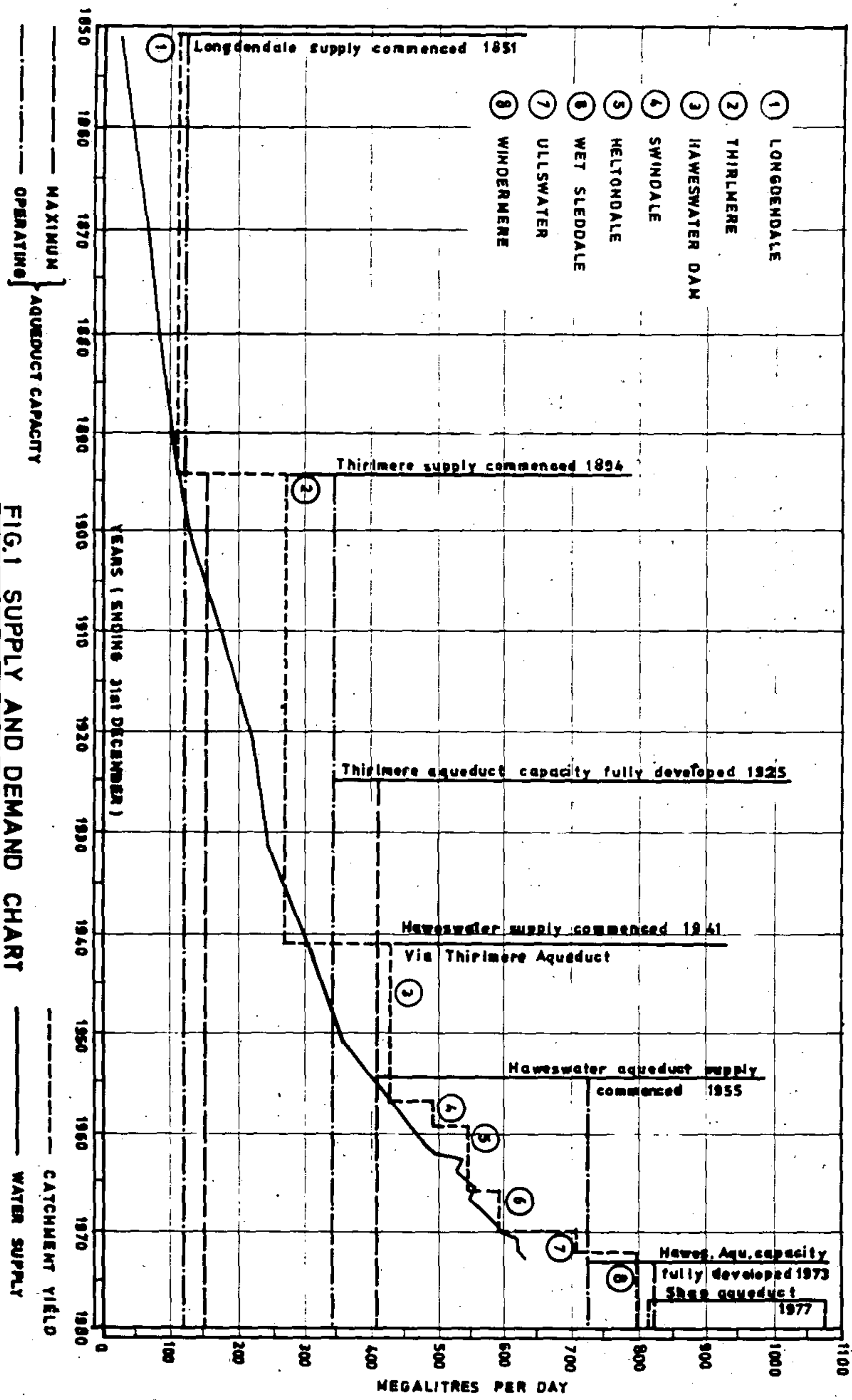
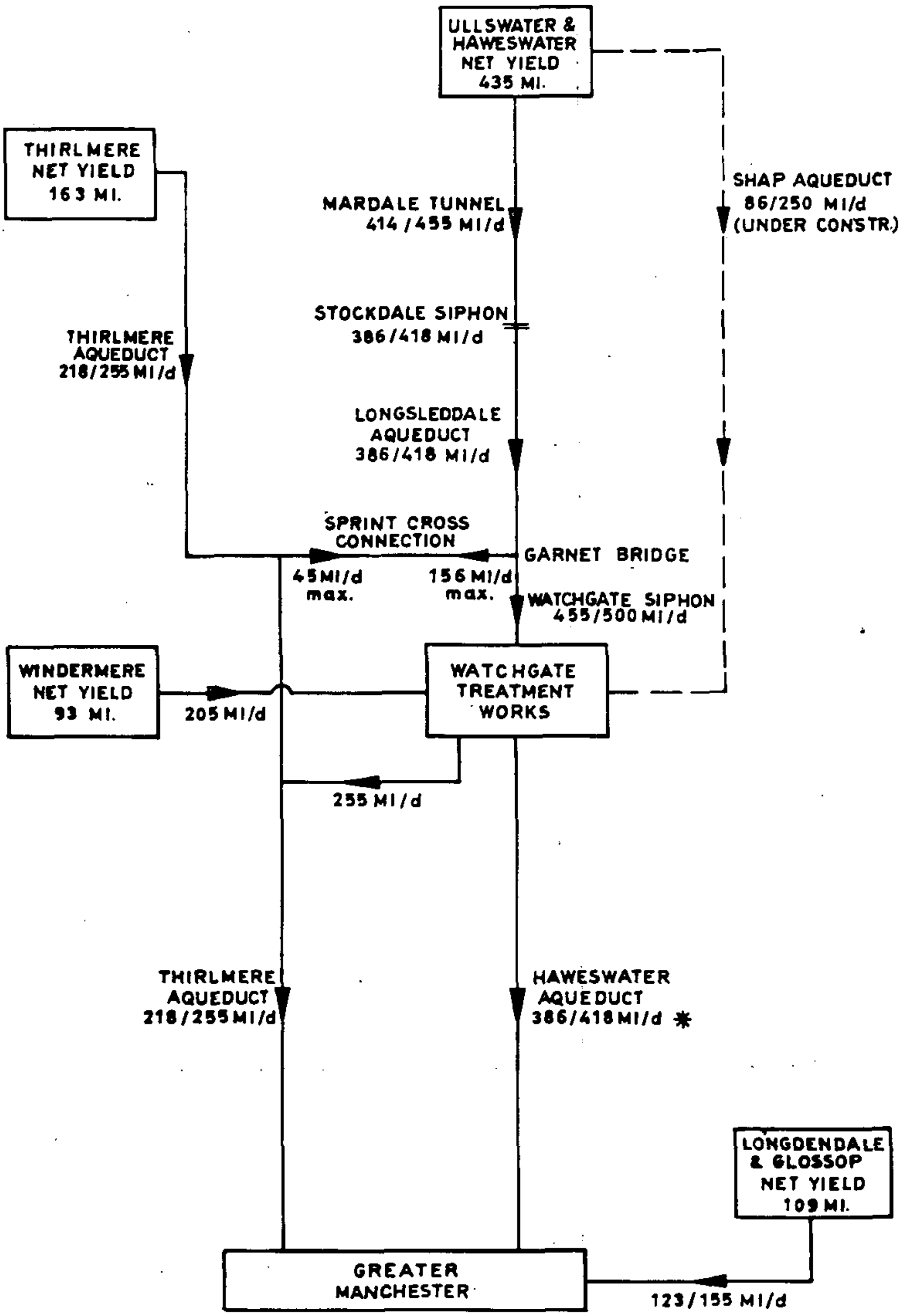


FIG.1 SUPPLY AND DEMAND CHART



NOTE:-
CAPACITY OF AQUEDUCTS SHOWN = NORMAL OPERATING/MAXIMUM
* AQUEDUCT CARRYING CAPACITY INCREASED 86 Ml/d ON COMPLETION OF SHAP AQUEDUCT

FIG. 2 SOURCES AND AQUEDUCTS

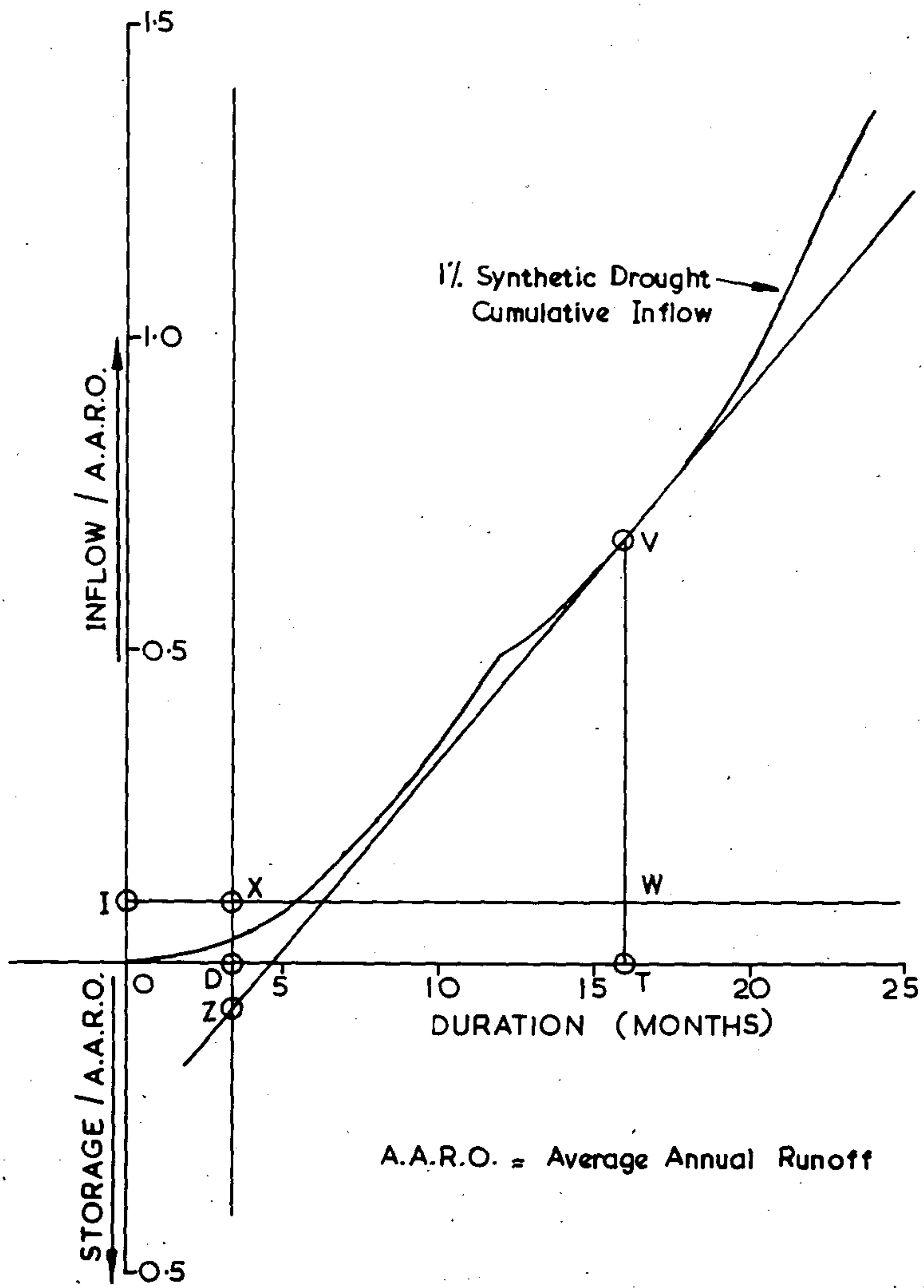
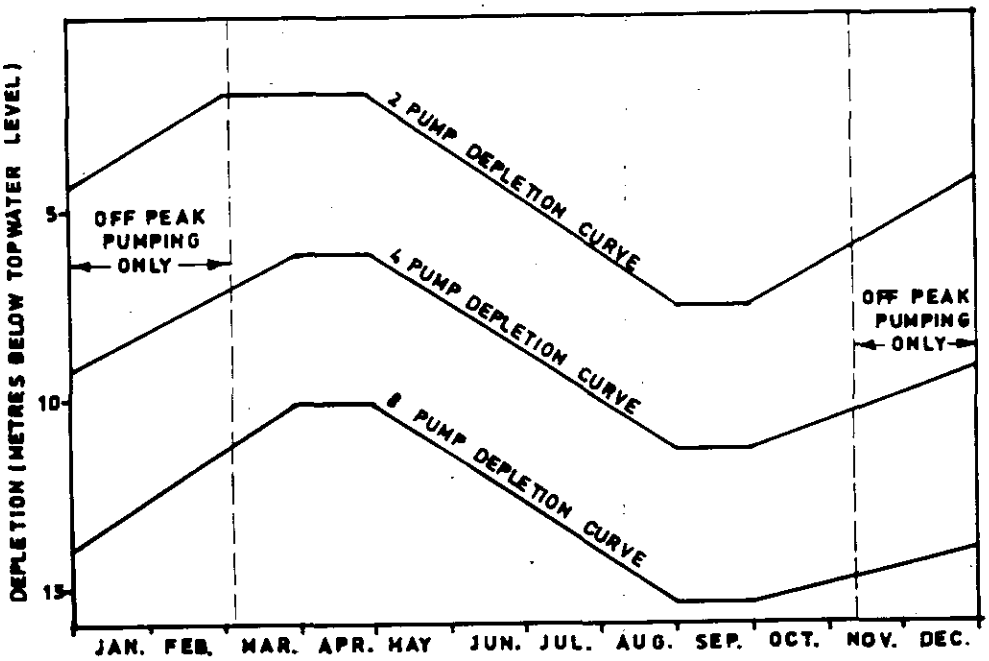


FIG.3 ASSESSMENT OF OPERATING POLICY DURING A DROUGHT

PUMPING RULE TO MAINTAIN X m³/s TO SUPPLY WITH 1% PROBABILITY OF FAILURE (FIRST TYPE)



PUMPING RULE TO MAINTAIN Y m³/s TO SUPPLY WITH 1% PROBABILITY OF FAILURE (SECOND TYPE)

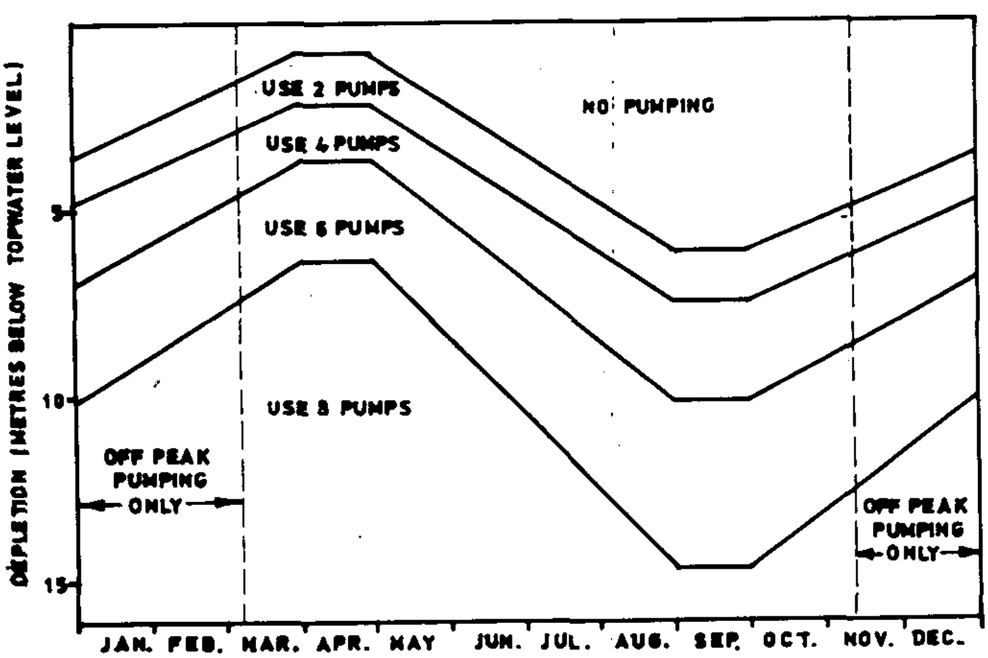


FIG 5. EXAMPLES OF PUMPING RULES

INSTRUMENTATION OF THE DAMS OF THE NORTH OF SCOTLAND HYDRO-ELECTRIC BOARD AND INTERPRETATION OF RESULTS

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SYNOPSIS

Standardised instrumentation was adopted by the Board fifteen years ago using pendula, levelling, collimation, straingauges and specialised techniques. These systems are disposed throughout a dam so as to give optimum correlation and confidence in the results. Readings are made to a prepared timetable and results are presented graphically.

Most dams exhibit regular cyclic movements which reflect seasonal temperature changes, except where hydrostatic forces predominate. From the records an envelope can be drawn to enclose the normal range of movement. This therefore affords a quantitative measure and clearly indicates the onset of undesirable behaviour. If the movement falls outside the envelope abnormal behaviour may be indicated. Instrumentation therefore gives a quantitative measure of behaviour and an early warning of deterioration.

INTRODUCTION

The North of Scotland Hydro-Electric Board has under its authority 56 large dams listed in the World Register of Dams and 76 reservoirs within the ambit of the Reservoirs Act. The oldest dams were taken over from the British Aluminium Company and the Grampian Electricity Supply Company, but a large majority were built by the Board between 1946 and 1964.

Some instrumentation was installed by the Board's Consulting Engineers during construction and about 15 years ago the Board decided, in the light of the increasing importance of instrumentation, to extend instrumentation systematically and to adopt more sensitive methods. The equipment was standardised as far as possible in order that the principal reading instruments could serve at a number of dams and so that results from different dams would be strictly comparable. Some of it was added to existing structures and this inevitably curtailed its full potential. Metric units were adopted at that time.

Readings are made in accordance with a prepared schedule at 36 arch, buttress, gravity and embankment dams, Figure 1. As most of the Board's dams are of concrete construction, and as the majority of the measurements relate to displacements, this paper will concentrate on the deformation behaviour of concrete dams.

TYPES OF INSTRUMENT AND SYSTEMS OF MEASUREMENT

STANDARD PENDULUM

The invar steel wire pendulums are located in protected spaces and are read by a portable laboratory travelling microscope mounted alternately in two orthogonal horizontal directions. The reading accuracy is ± 0.01mm [1]

INVERTED PENDULUM

Each inverted pendulum is mounted in a vertical steel tube and tank filled with oil, Fig. 8. An indicator mark on top of the float projects above the oil and is observed through a hole in the tank lid. The microscope is mounted as for the standard pendulum and the accuracy is again ±0.01mm. [1]

THE LEVEL AND STAFF

A self-setting level with parallel plate micrometer and invar staves is used for the measurement of settlement. Sighting distances are equalised and are limited to 15 to 20 m. Two independent runs of levels are taken which normally produce a closing error of less than 1.0mm.

This is distributed throughout each run, and the results are considered reliable to within ± 0.2 to ± 0.5mm. A lower order of accuracy is accepted for embankment dams.

THE OPTICAL COLLIMATOR

An Italian collimator with associated target equipment is used for measuring deflections of the crest. The telescope of the collimator has a magnification of x60. The mobile targets carry vernier scales reading to 0.1mm mounted traversely to the line of sight. A careful check is kept on the accuracy of the vertical plane of the collimator, for which purpose a special 10m high pendulum has been installed in the enclosed space under the spillway of Errochty Buttress Dam.

The installation at a dam usually consists of a concrete pillar near each end of the dam and founded on rock beyond the influence of the foundation forces, and several stations along the crest for the mobile target.

It is standard practice to read the targets from both abutments, Figure 7. The deflection is calculated from the mean of the readings, at least eight readings being taken from each end. Twice the Standard Error is obtained as a measure of the tolerance at the 95% confidence level. This has been found to vary from 0.15mm for good conditions to 0.4mm for an overall range of about 600 metres, beyond which the operating conditions become more severe due to refraction, weather, communication, etc. These techniques and the calculation of results were based on Italian practice and have been developed by the Author in the light of the Board's experience. [1]

OTHER INSTRUMENTS

Electro-acoustic strain meters with coupled resistance thermometers were installed during construction in Monar Arch Dam and Cruachan Buttress Dam, and all are functioning extremely well. Several other dams have thermometers, and frequent readings are obtained for the purpose of assessing concrete temperature changes.

SPECIAL EQUIPMENT

Certain specialised techniques have been developed to meet particular needs. At Cruachan Dam a camera of the aerial reconnaissance type is used for recording the movement of the inverted pendula. The camera is set up to photograph automatically the float and reference wires at intervals of a few hours. The movement of the buttress in response to the daily filling and emptying of this pumped storage reservoir is thus recorded over a number of weeks.

DISPOSITION OF INSTRUMENTS

The extent of installation required at a dam depends upon a number of factors including size, foundation conditions, type of dam and age. For each case therefore a different layout has been developed to provide satisfactory information. Typical intergrated arrangements for concrete dams have been installed at Lednock Dam, and at Sloy Dam, Figure 2. A principal requirement is that one buttress should be monitored by each of the systems used in order to provide correlation between them and confirmation of the results. A typical rockfill dam will incorporate a series of level stations along the crest, and possibly subsidiary lines of stations along contours on the upstream or downstream faces. More extensive instrumentation has not been necessary.

PROGRAMME OF MEASUREMENT

The general principles leading to the choice of frequencies with which instruments are read have been discussed elsewhere (2). The time of year at which observations are made is important because readings should include the extreme values of each parameter. This implies that readings should be taken in late winter and late summer, the arch dams being visited first as they respond most quickly to change of temperature, followed by buttress dams, gravity dams and embankments. In practice however, observations must be spread over some months due to limited manpower and equipment, dams being visited in the order indicated above.

Based on these requirements a schedule is drawn up each year, allowing the necessary site time at each dam with an allowance for adverse weather. Normally one or two days at a dam is sufficient but four days is required at one particular dam. If a pendulum is the only instrument to be read, this can be done in a half-hour visit.

PRESENTATION OF RESULTS

It is common experience that dam instrumentation produces a vast amount of data and the ensuing problem is in so presenting it that behaviour patterns can be determined, and if possible recognised, at a glance. The horizontal displacements of the crest are calculated from the sets of collimator readings by computer using a simple programme. All vertical movements are determined by computer from the double runs of levelling, a short programme having been written for each dam. The calculation of results from the pendulum and temperature gauges is straight-forward. All results are first recorded numerically in a table, either as relative changes for displacements or as values on standard scales as for temperature etc.

Next, graphical plots are made for selected instruments and for any particular parameter which is to be studied or compared with another parameter, eg deflection with temperature, or leakage with water level.

Only one standard graph sheet is used on transparent copies of which everything is plotted to the same time base (2). The same vertical scale is used for the same parameter at all dams. Each year is on a separate but identical sheet. Additonal curves are accommodated on secondary sheets. All sheets can, of course, be laid over each other, and all years and parameters can be compared. It will be realised that the behaviour of different dams can be equally easily compared. The graphs in this paper have been prepared from the records on these standard sheets.

INTERPRETATION OF RESULTS

LEDNOCK BUTTRESS DAM

At Lednock Dam the installation of equipemnt is extensive, and the systems have been arranged so that they overlap in Buttress 7, which has a height of 41m. At this buttress the normal annual range of horizontal deflection of the crest is between 2 and 3mm, within a total range of 3.90mm, and the range of vertical movement is between 3 and 4mm, within a total of 5.00mm. From the graphs it can be readily adduced that the horizontal and vertical movements of the crest exhibit a repetitive annual rythm or cycle largely corresponding to thermal changes in the concrete. It may be added that mass concrete gravity dams exhibit similar crest changes.

SLOY BUTTRESS DAM

Full instrumentation readings have been taken at all the installations indicated in Figure 2 over a period of five years, and pendulum readings have been continued thereafter. Figure 4 shows the horizontal and vertical movements recorded for Buttress 6, the tallest at 55m, and Buttress 13, the shortest.

Sloy Dam is significantly higher than Lednock Dam and the maximum range of deflections at the crest is greater, as might be expected. At Buttress 6 the normal annual range of deflection is about 4mm within a total range of 6.04mm measured by collimation, or of 7.05mm measured by pendulum. The range of reservoir water level is also greater and the influence of hydrostatic pressures can be seen superimposed on the influence of concrete temperatures, particularly in 1969 when the direction of the annual cyclic pattern of crest deflection was reversed as shown in Curves C6 and C13.

As the horizontal displacements of this buttress are measured by the pendulum, Curve P6, as well as by collimation, Curve C6, an interesting comparison of the results can be made. For this, however, it must be assumed that the foundation of the buttress and the collimator stations on rock remote from the abutments are completely stable, and that the pendulum records the full deflection of the buttress. As it is anchored only 5m below the spillway crest this condition is very nearly satisfied. For easy comparison, only those pendulum readings made on the same dates as the collimator readings have been used. Thus, Curve P6(A) is seen to confirm Curve C6. This gives increased confidence in the results that have been provided only by the pendulum since 1972.

MONAR ARCH DAM

The physical deformations of Monar Dam are monitored by collimator at the crown, by theodolite at 13 crest stations, and by level at five crest stations, Figure 3. There are a number of levelling stations at the foundation, and four inverted pendulums have been constructed in the rock foundation. Concrete temperatures are obtained from resistance thermometers associated with a large number of straingauges.

T36 indicates the temperature of the water at depth and of the concrete face on the upstream side, and T20 indicates the concrete temperature on the downstream face at the same level.

The results of instrumentation are shown in Figure 5 which has been drawn with the same scales as used on the other graphs. Although the change of water level is much less than at Sloy Dam the crest movement is very much greater. During impounding in 1963/64 an initial deflection of about 10mm took place. Since then the additional annual range of movement has been between 10 and 12mm in a total range of 13.74mm.

This movement indicated by the collimator has been largely confirmed by the movement indicated by triangulation from two theodolite stations. These are shown by Curves A and 7 respectively. Levelling of the station M3, also at the crown, indicates a very consistant range of movement of about 4mm similar to those for other concrete dams, clearly reflecting the influence of concrete temperatures. At this dam the thermal effects now appear to dominate the overall behaviour although the influence of a low water level can be seen in the greater upstream movement of the crown in 1971, preceded by the opposite situation in 1970.

The foundations are monitored at four places in three dimensions. Movement in the upstream/downstream direction at the base of Block 8, indicated by Pendulum P8, is about 2mm and is greater than the lateral movement of about 1mm, both of which show reasonably consistent behaviour. The level of station M21, which is adjacent to the pendulum at the base of Block 8, appears to have a downward trend from 1968 to 1974, but the 1974 position has in reality returned to where it was in 1964/65.

LOCH DUBH DAM

There are two collimator stations for crest deflections at Loch Dubh Dam which is a relatively small concrete gravity structure with a maximum height of 20m. Station UL3 is near the right hand abutment where the dam is constructed with ordinary portland cement concrete and is founded on variable calcareous rocks. Station UL5 is at the highest part of the dam. Here it is constructed of grouted concrete, using large aggregate, and is founded on hard quartzite.

From Figure 6 it will be apparent that for at least seven years the annual deflections of the crest were not cyclic but were trending steadily upstream at an average rate of nearly 0.7mm per year at UL5. This progressive movement cannot be due to a general upstream movement of the dam or its foundation, but might signify an expansion of the body of the concrete dam. It does not appear to be due to thermal expansion. Past information on this is insufficient but detailed measurements are being taken.

Since 1969 movement has become cyclic, exhibiting a normal range of less than 2mm which is consistent with the height of the dam, but latterly having a reversal of direction in relation to the seasons. Much the same pattern,though of lesser magnitude,is shown at UL3.

During the 20 years since construction the surfaces of the grouted concrete have shown considerable exfoliation and loss of the surface grout, extending to a depth of at least 50mm in places especially at lift joints. In 1968 in the light of the deterioration and the continuing unusual behaviour as shown by the crest deflections, it was considered that special repairs might be required. To obtain more information 18 piezometers were installed and read for several years partly as research project by the University of Newcastle upon Tyne (Mr. A I B Moffat) and CIRIA. The readings indicated some consistently high pore pressures in the centre of the dam and some very low pressures where the piezometer has presumably had a connection with the downstream face. In 1971 the outlet pipes from the foundation drain situated immediately downstream of the cut-off were tested and some were found to be completely blocked.

It was therefore necessary to reinstate an effective drainage system and this was done by drilling eight new 62mm diameter holes into the structure and providing four new outlets. The water was shown to be ground water in the foundation rock, largely from the sides of the valley, and not from the reservoir. Some pore pressures have since dropped and there has been a visible improvement in the surface condition of the concrete. Also the upstream trend of the crest movements has ceased.

CONCLUSION

The pattern of deflections to be expected can be determined from an analysis of the records of instrumentation taken over a sufficiently long period of time. Five years is considered minimum.

This has been done at 16 of the Board's major concrete dams. For the smaller gravity dams movements due to thermal expansion usually predominate and the pattern is cyclic in relation to the seasons. For the larger dams larger thermal movements take place, but the movements due to hydrostatic pressures are significant and on occasions will predominate, especially during particularly dry summers with consequent low reservoir levels.

For each dam, or for each buttress, an envelope can be drawn to enclose the normal range of movement. The dam then continues to exhibit normal behaviour when the points are found to lie within that envelope. Conversely abnormal behaviour can be defined as occuring when points fall outside those limits and into what could be called the warning zones. Furthermore a reliable quantitative measure of such behaviour is given by the values of those points.

It must be emphasised that a single point falling into the warning zone will not of itself signify danger, as there is often a small proportion of results of doubtful reliability. The first action is to re-check the calculations and compare the results of the various measuring systems installed and if necessary to repeat the observations.

If there has been no demonstrable error and when abnormal behaviour is therefore indicated it will be necessary to search for other, new signs of distress and to repeat instrumentation readings at shorter intervals until satisfied that abnormal readings no longer persist or until the cause has been found. Loch Dubh is a case in point where the envelope would have lines about 2 mm apart but the actual movement has trended over 5 mm during 7 years. Also, two or three cases have occurred where the readings have shown a significant 'jump' on the graph. This has sometimes persisted for the next two or more sets of observations before the readings have returned to their previous value and normal behaviour has been seen to continue.

The installation of a deflection measuring system is thus seen in the nature of an early warning system. This is particularly true when the deterioration is slow and its progressive change will not be obvious to the personnel regularly inspecting or maintaining the structure. It is of course essential that readings are taken and recorded conscientiously, and that the system is of a sufficiently high order of accuracy to detect movements appreciably smaller than the normal annual cycle.

ACKNOWLEDGEMENT

The Author wishes to record his appreciation for the permission given by the Board to use the extensive data on which this paper has been based and to thank the Chief Civil Engineer for his encouragement.

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	LEADING PARTICULARS				INSTRUMENTATION · Showing Number of Measuring Locations										
DAM	TYPE	Year of Completion	Height above Foundation	Length of Crest	Triangulation	Collimation (Alignment)	Levels (Settlement)	Levels (Inclination)	Pendulum	Inverted Pendulum	Concrete Temperature	Strain Meter	Joint Gauge	Uplift Pressure	Leakage (V notch)
GARTHBEG (Foyers)	E	1896	6	325			30							14	1
SLIGNEACH	E	1925	4	39		3									1
ERICHT	G & E	1931	17	404		3	8								2
TROMIE	E	1941	14	109									2		
SLOY	B(1)	1951	55	356		5	11	8	2						
BENEVEAN	G	1951	37	177		1	1		1		24		11	3	1
MULLARDOCH	G	1951	48	727		4	18		1		12		28	6	2
LUSSA	CR	1952	17	81		4	5						6		
TARSAN	B	1953	30	344		3	7		1						1
COL (TARSAN RESERVOIR)	R	1953	16	202			13								1
LOWER SHIRA (East)	G	1954	22	146		3									
LOWER SHIRA (West)	E	1954	18	192		5	9		1						1
LOCH DUBH	G	1955	20	88		2	2							18	
GLUANIELOYNE	G	1956	40	675		1			1		36		6	10	1
LOYNE	G	1956	22	549											3
QUOICH	R	1956	39	320			69		2						2
CRUADHACH (North)	G	1956	22	147		15	30								1
CRUADHACH (South)	G	1956	23	87		7									
ALLT NALAIRIGA	X	1956	24	425		3	7					13	2		
ERROCHTY	B	1957	50	500		3	6		2				27		2
GLASCARNOCH	G & E	1957	43	535		2	41		1				3		1
FANNICH	R	1957	12	745			18								2
VAICH	E	1957	38	257			18								1
LEDNOCK	B	1958	41	290		5	50	12	1		21		7		1
LUBREOCH	B(1)	1958	40	531					1				11		
SHIRA (Main)	B	1959	45	726		4	4		1				48		2
ORRIN (Gravity)	G	1959	51	312		2			1						
ORRIN (Embankment)	E	1959	25	312			11								1
DROMA	R	1959	13	356			11								
BREACLAICH	R	1960	27	433			30						9		3
CHLIOSTAIR	CR	1961	15	113		3	6				9	9	9		
GLASHAN	G	1961	22	378									3		4
NANT	G	1962	28	372											1
LOICHEL (Monar Reservoir)	G	1962	20	170											1
MONAR	C	1963	40	161	11	1	24			4	51	51	5	12	6
GRUACHAN	B(1)	1964	47	316		3	15	2		2	14	14			

TYPE:- E = EARTHFILL, R = ROCKFILL, G = GRAVITY, B = BUTTRESS, B(1) = MASSIVE BUTTRESS, C = CUPOLA, CR = ARCH CONSTANT RADIUS, X = PRESTRESSED

* METRES

Fig. 1 Instrumentation at Hydro-Board Dams

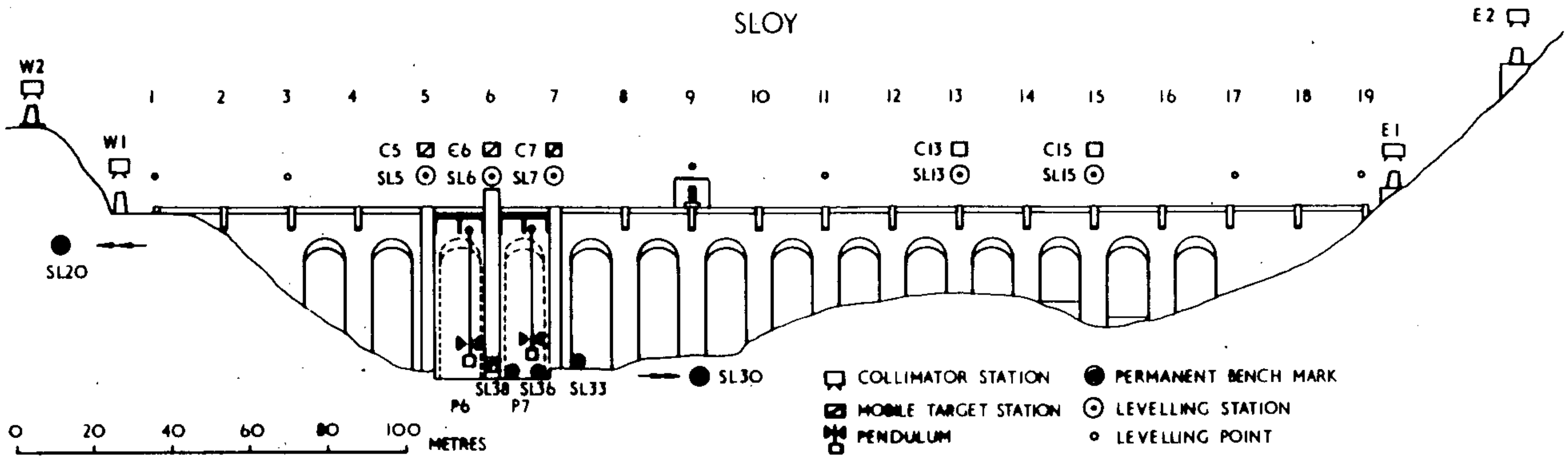

Fig. 2 Sloy Buttress Dam

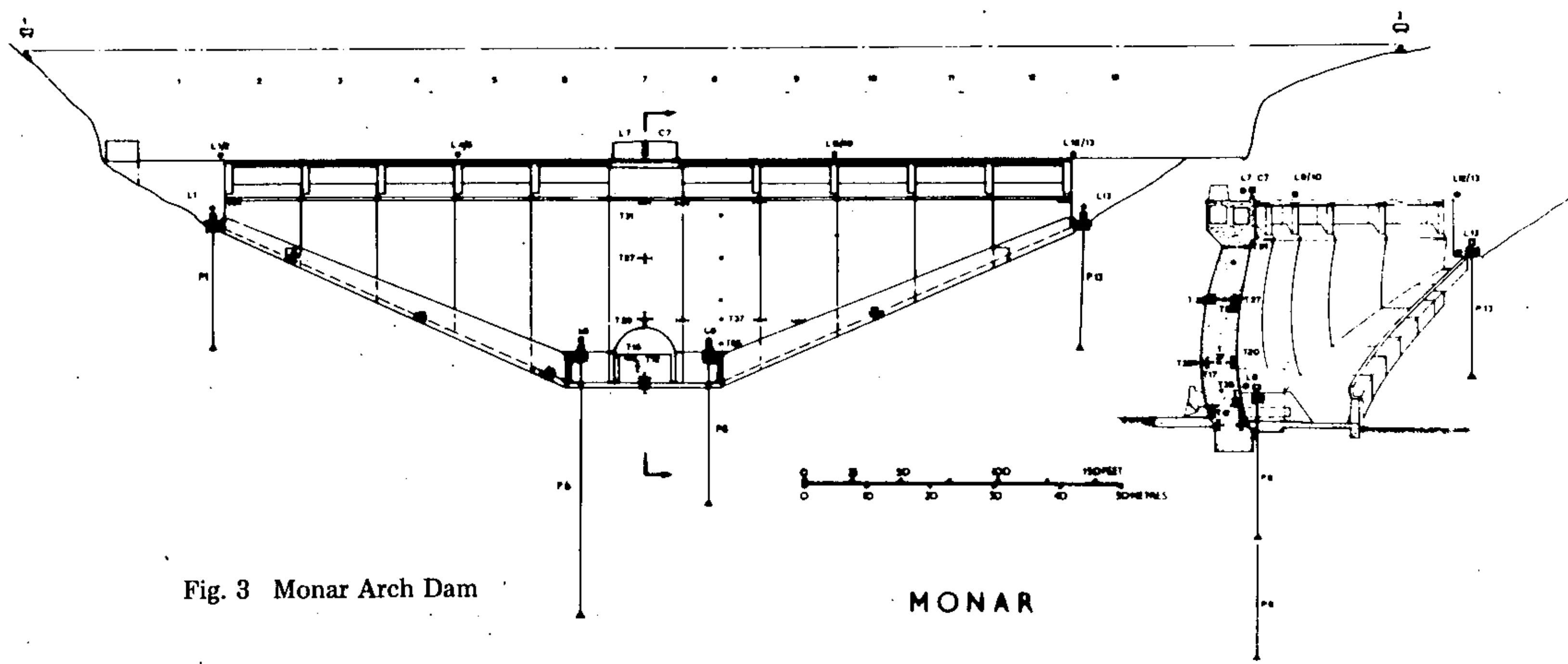

Fig. 3 Monar Arch Dam

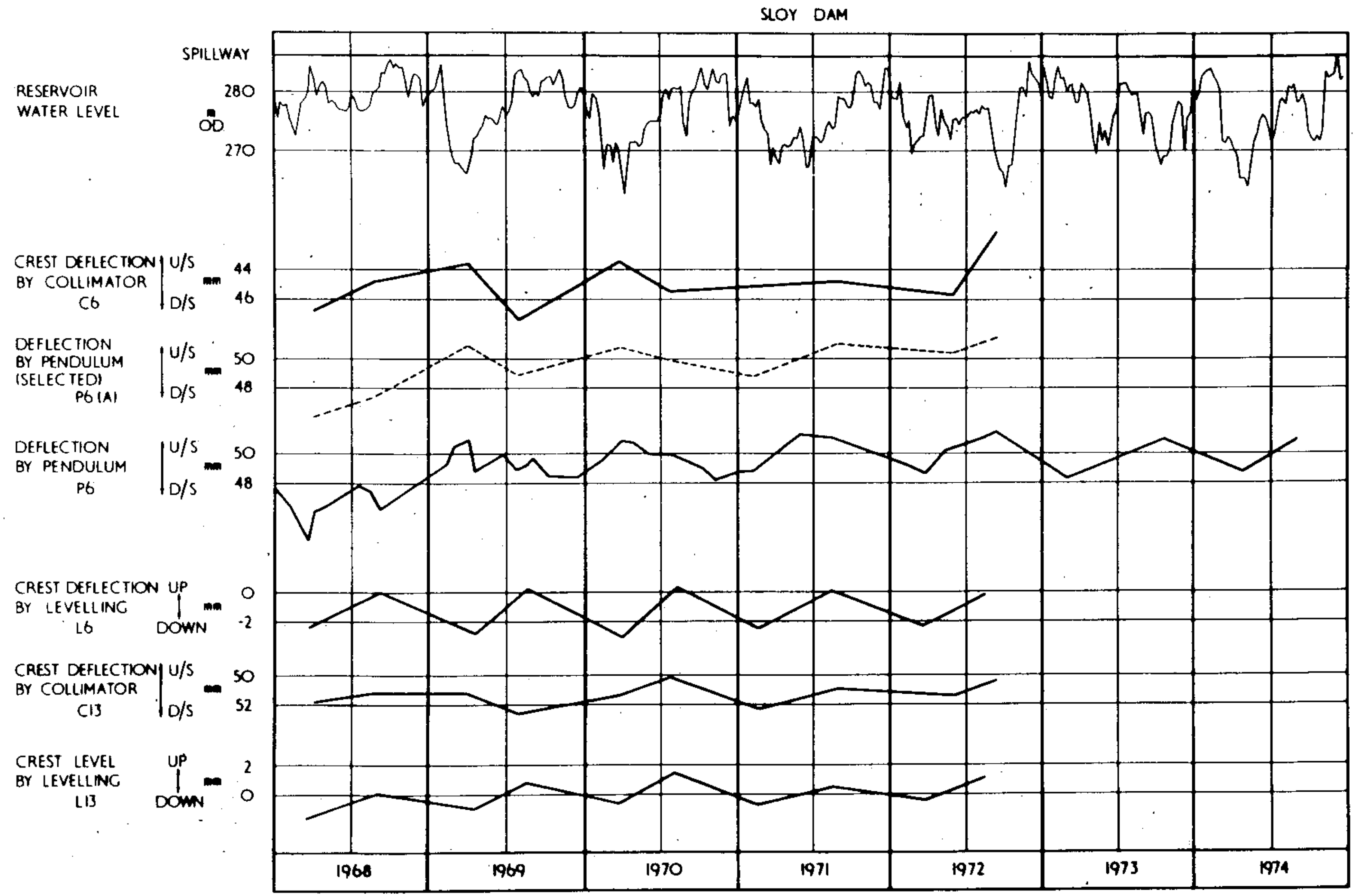

Fig. 4 Instrumentation Results at Sloy Buttress Dam

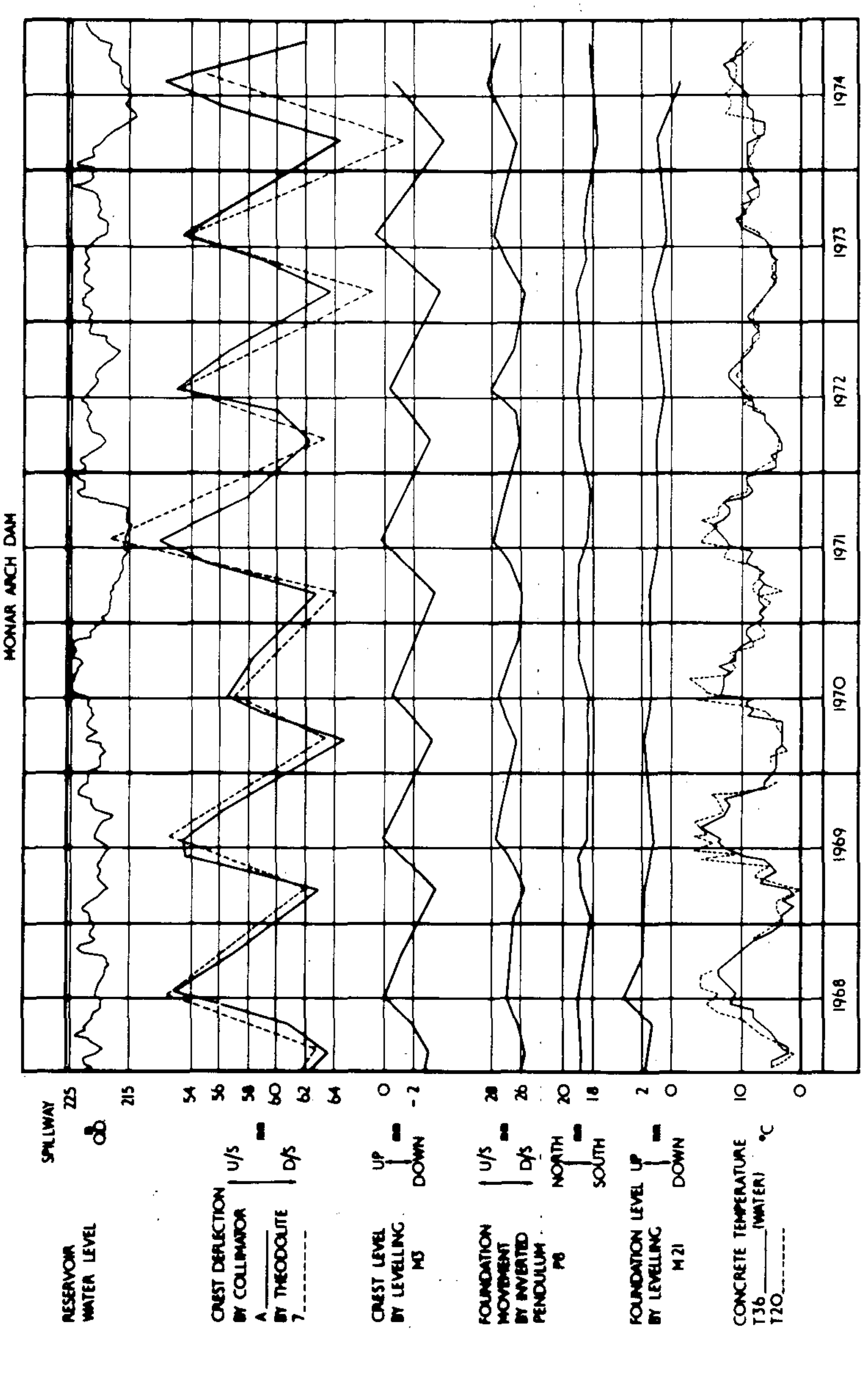

Fig. 5 Instrumentation Results at Monar Arch Dam

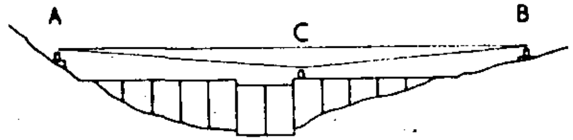

Fig. 7 Double-Ended Collimation

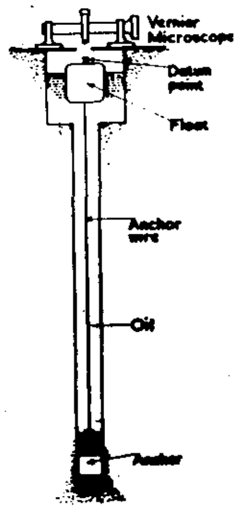

Fig. 8 Inverted Pendulum

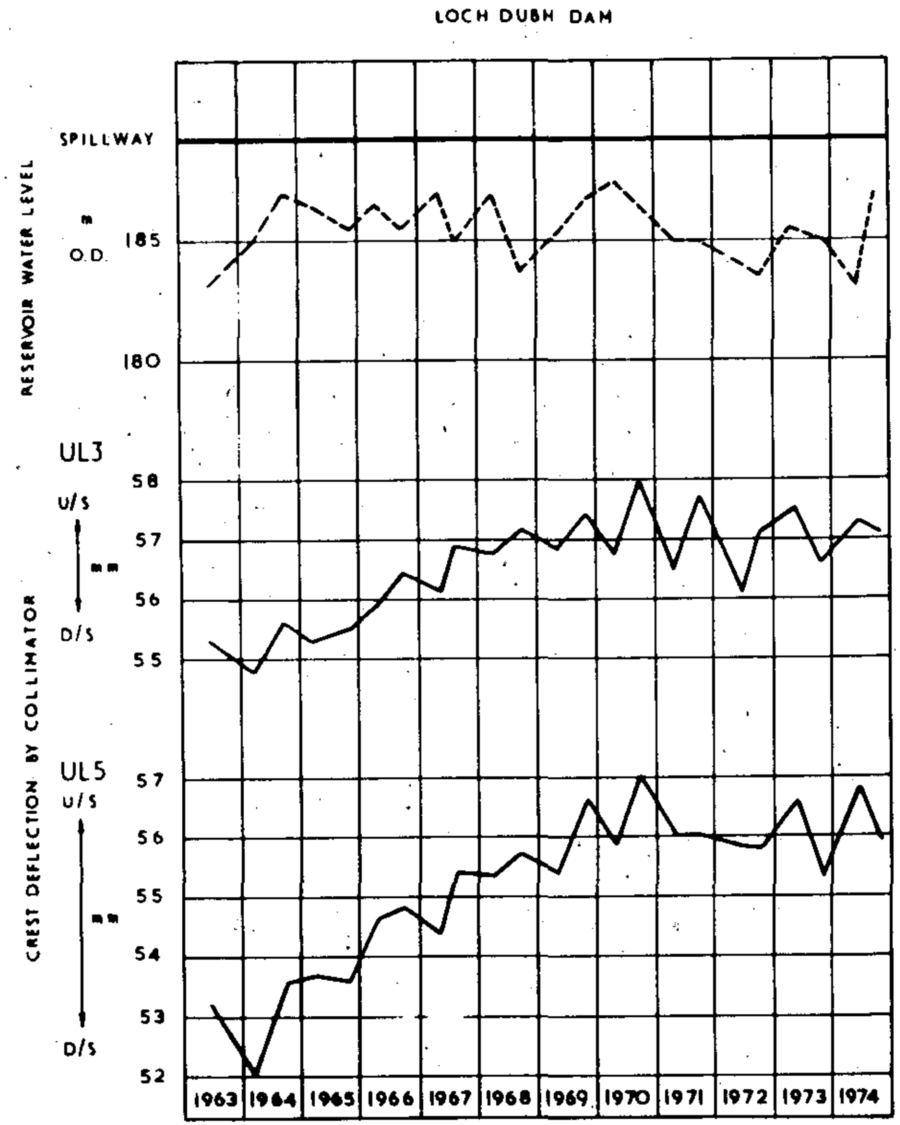

Fig. 6 Instrumentation Results at Loch Dubh Dam

INSTRUMENTATION AND OPERATION OF BRADAN RESERVOIR

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ASSOCIATE
BABTIE SHAW AND MORTON

SYNOPSIS

The paper describes the hydraulic control equipment and the hydraulic instrumentation at Bradan Reservoir and the methods used to monitor the behaviour of Bradan Dam during impounding and subsequently. Higher than anticipated uplift pressures occurred under one section of the dam and action was taken to reduce these pressures.

BRADAN RESERVOIR

Located 26 km south of Ayr at an elevation of 315 m.o.d., Bradan Reservoir is the most recently constructed major reservoir in Scotland. Constructed for the Ayrshire and Bute Water Board it is now the responsibility of Strathclyde Regional Council. It provides storage directly for the Bradan catchment and, via a catchwater aqueduct, for the adjacent Stinchar catchment. The reservoir has a storage capacity of 19,200 Ml at the level of the overflow spillway crest. The estimated reliable gross yield is 106 Ml/day. 15 Ml/day is required for compensation and 91 Ml/day is available for supply.

The establishment of the reservoir required two mass concrete gravity dams, Bradan Dam and Lure Dam. Construction started in mid-1970 and took just over two and a half years. The reservoir filled throughout 1973 and the reservoir level reached the spillway crest for the first time in January, 1974.

BRADAN DAM AND LURE DAM

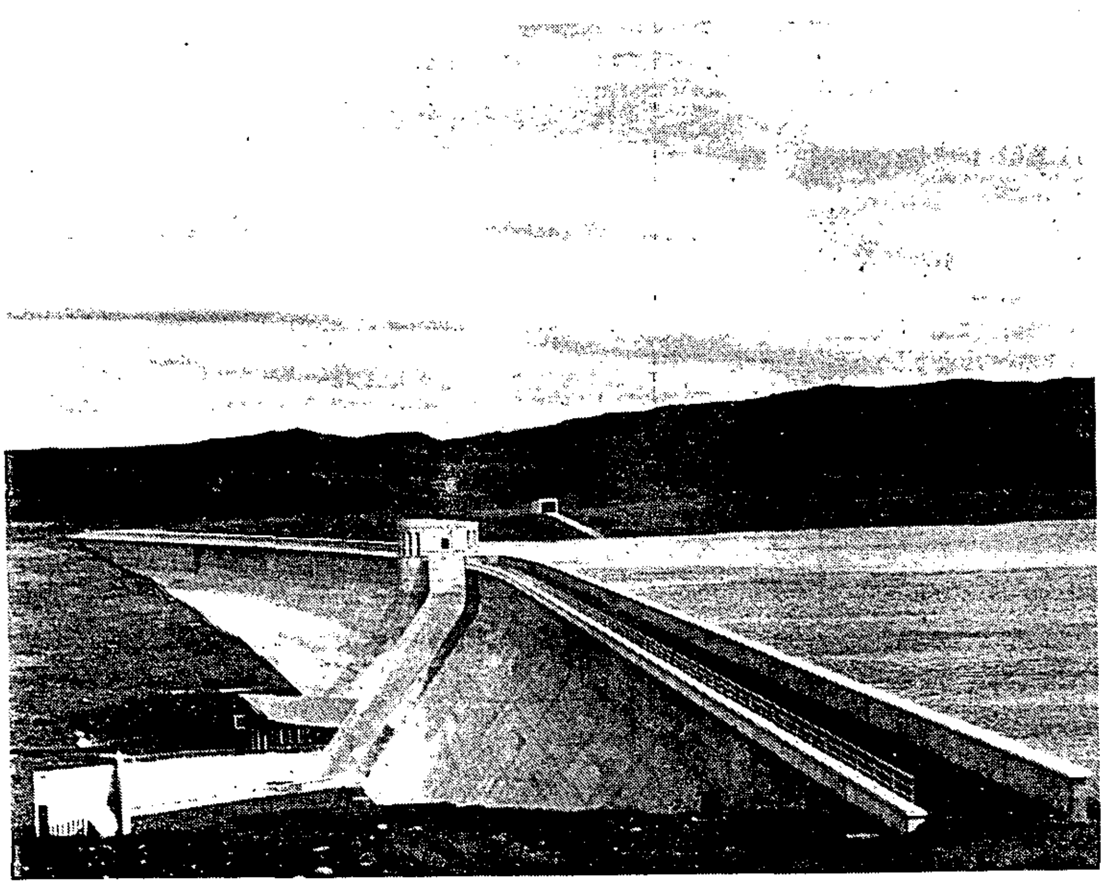

Fig. 1 Bradan Dam

Bradan Dam incorporates the overflow and draw-off works. It is built across the valley of the Water of Girvan and is 438 metres long. The maximum height from foundation level to roadway is 30 metres. Lure Dam is a small, non-overflow dam adjacent to the outfall from the Stinchar Aqueduct. An earlier paper (1) has described the design and construction of Bradan Dam.

BRADAN TREATMENT WORKS

The principal supply from Bradan Reservoir, 86.5 Ml/day, is conveyed by a 1090 mm dia. steel pipe to Bradan Treatment Works, which acts as the control centre for the Loch Bradan Development. A minor supply, 4.5 Ml/day, is maintained to Kerse Treatment Works.

HYDRAULIC CONTROLS AT THE DAM

Draw-off valves and pipework are built into Bradan Dam in the monolith adjacent to the east side of the spillway. Three draw-offs are set in the same vertical plane, each controlled by a 1067mm dia. butterfly valve. The principal control valve on the supply main at the dam includes an automatic closing mechanism which operates in the event of a major leakage in the Bradan main between the dam and treatment works.

The scour pipe is located directly below the three draw-off pipes and, since all four pipes have identical bellmouth entries, the scour can be controlled by the same bulkhead gate as the draw-offs. A 900mm dia. free discharge 'sleeve' valve is mounted on the downstream end of the scour pipe to discharge into the spillway basin.

The valves can be operated by handwheel and by electric push-button at the valve, but they are normally controlled by push-button switches on a control panel at the dam. Provision is also made for operation from the Treatment Works.

HYDRAULIC INSTRUMENTATION

Instruments have been installed to monitor reservoir inflow and outflow and the reservoir level.

The flow in the Stinchar Aqueduct is measured at a trapezoidal flume and transmitted to the Treatment Works. The principal flows to supply and compensation are measured at the Treatment Works by Dall tubes. They are recorded by data-logger which prints out cumulative flows at three-hourly intervals. The reservoir level is measured by pressure bulbs connected to the principal pipework at the dam and transmitted to the Treatment Works.

STRUCTURAL INSTRUMENTATION

It would have been difficult to justify a sophisticated system to monitor Bradan Dam. The foundation rock is sound and capable of withstanding higher loads than occur and many similar dams without instrumentation have functioned satisfactorily. It was decided to install a fairly large number of 'manually-operated' low-cost measuring devices and to accept a lower degree of accuracy for individual results than is offered by the more complex systems and, in the event, the expenditure on gauges, surveying pillars and target stations was £6,000 in a contract valued at £1.5 million. In addition piezometers measure pore pressure in the concrete as part of a research project by the University of Newcastle upon Tyne (Mr. A I B Moffat) financed by C.I.R.I.A.

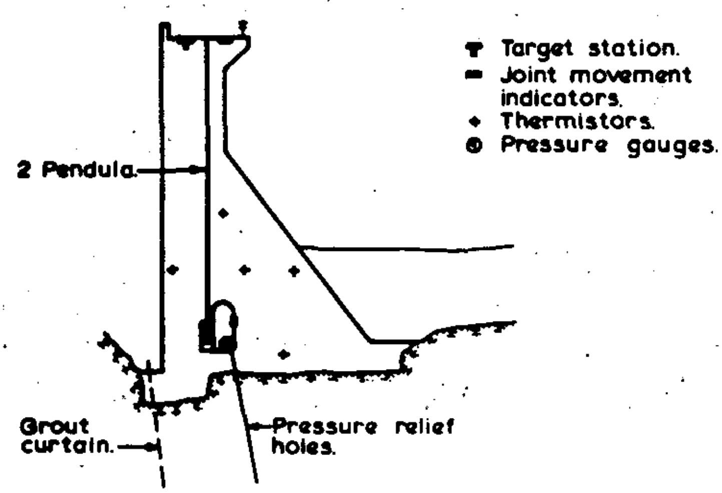

Fig. 2 Bradan Dam instrumentation

In time, it is hoped to establish a pattern of behaviour for the dam so that any departure from the norm can be examined and any necessary action taken.

UPLIFT PRESSURE

The greywacke and shale forming the dam foundation are relatively impermeable and grouting consists of a single row of grout holes at the upstream heel of the dam.

At the shallower sections of Bradan Dam there are no pressure relief holes and the design assumes that uplift pressure varies from the full reservoir head at the upstream face to zero at the toe. The higher sections of Bradan Dam have pressure relief holes and the design assumes a lower uplift pressure, varying from full reservoir head at the upstream face to approximately 50% at the pressure relief holes and then to zero at the toe (or tail-water where appropriate).

50mm in diameter, the pressure relief holes were drilled from the inspection gallery through the foundation concrete and about 3 metres into bed-rock. They were unlined originally, but they gradually filled with small stones and silty material washed from the sides of the hole. Early in 1974 the holes were cleaned out by air-jetting and lined with a perforated P.V.C. pipe inside a porous plastic liner tube.

Normally water flows up the pressure relief holes and discharges into the drain in the gallery and at these holes the pressure can be read using a Bourdon gauge. Where the pressure head is below the level of the gallery the water-level in the holes is measured using a battery-operated dip-meter.

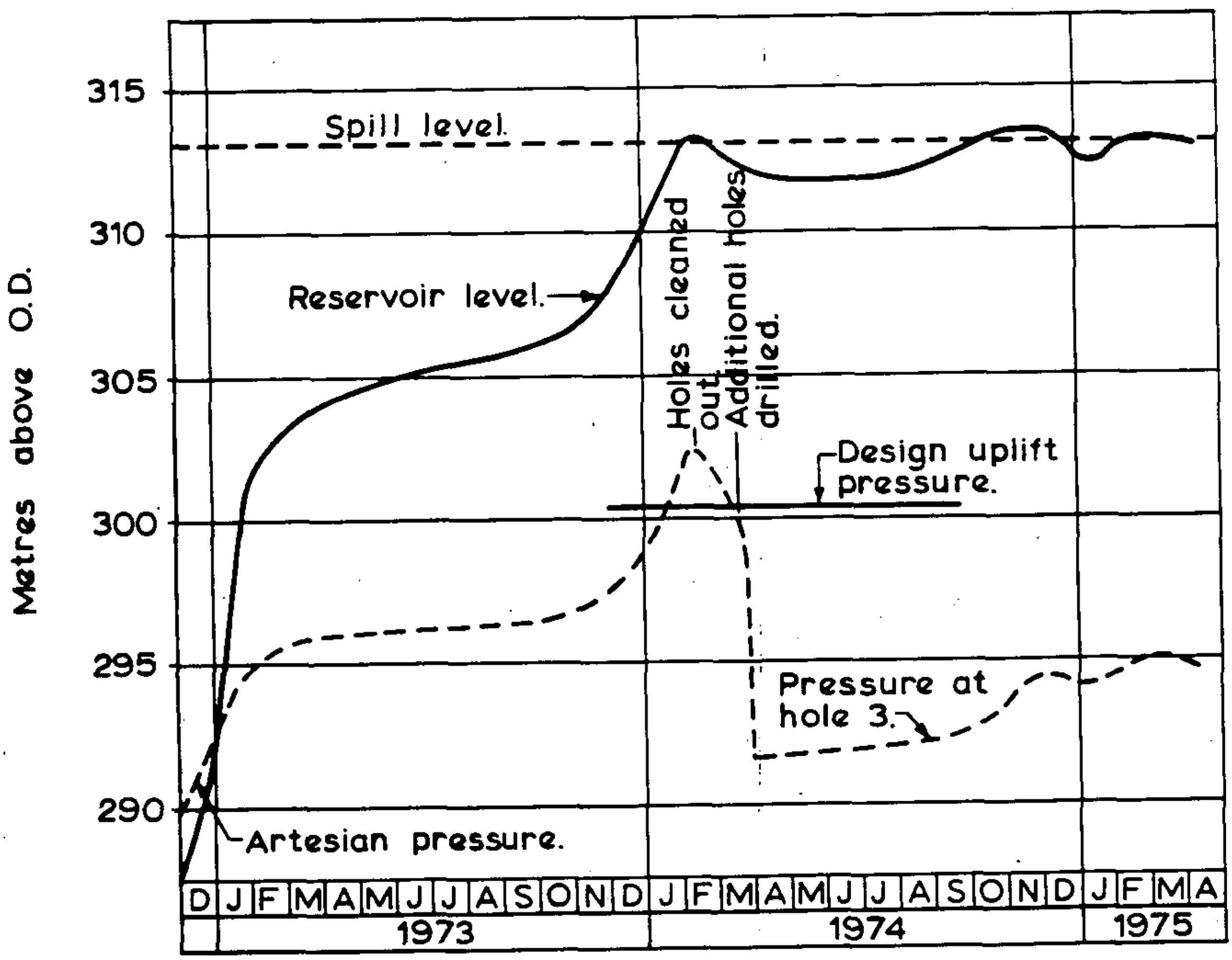

Fig. 3 Uplift pressure at Block 26

Artesian pressures had been observed at the site investigation stage and their existence was confirmed in readings prior to impounding. The artesian pressure influenced readings at low reservoir levels and so also did percolating ground water after heavy rain. The reservoir level itself had the greatest effect on uplift pressures, as exemplified in Fig. 3 which shows variations in uplift pressure at Block 26. In contrast, there are holes in which the pressure has remained constant throughout. The overall situation is represented in Fig. 4 based on average readings at each monolith in April 1975 and with the pressure relief holes functioning normally. At the holes the pressure varies from 11% to 27% of full reservoir head, the calculated average being 20%.

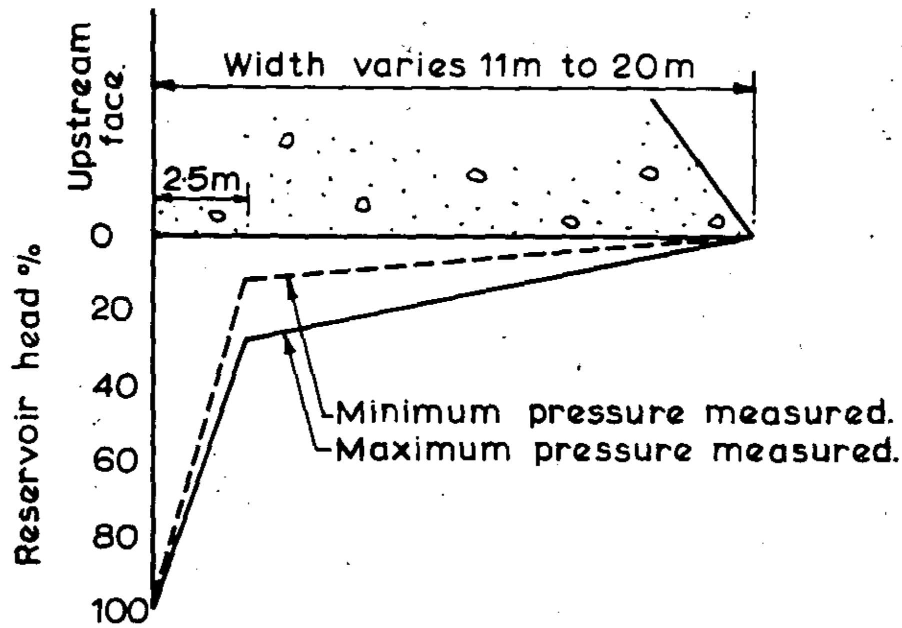

Fig. 4 Uplift pressures (with pressure relief)

Block 26 was particularly responsive to changes in reservoir level. On average the pressures at full reservoir head were just within the design assumptions but at Hole No.3 in the middle of the monolith the pressure in early 1974 exceeded the design uplift (Fig. 3). In March, 1974, six additional holes were drilled and, as drilling proceeded, the pressure drop in the original holes was immediate, the greatest decrease being just over 8.5 metres at Hole No. 3. The joints in the foundation rock are spaced at wider intervals than elsewhere and this seems to have reduced the effectiveness of both the grout curtain and the pressure relief holes at Block 26. An exercise was carried out in April, 1975 to evaluate the benefit of the pressure relief holes. The holes were closed off and pressure readings were obtained corresponding to the 'no pressure relief' situation. The spread of results in the 'no pressure relief' case is much wider, the average pressure on each monolith varying from 15% to 49% with an overall average of 31%. Isolated results exceeded the design assumption, and this exercise emphasised the value of the pressure relief holes and the importance of ensuring that they operate satisfactorily.

The flow of water in the inspection gallery has also been monitored and to date the highest flow recorded has been some 35,000 litres per day, which is regarded as satisfactory.

DEFLECTION (i.e. Movement in an Upstream/Downstream Direction

Precise survey techniques measure movements relative to points beyond the structure and pendula measure movement of the head of the dam relative to inspection gallery.

All readings are referred to readings taken before impounding. These initial readings are influenced by the residual heat of hydration in the concrete and by the absence of reservoir loading.

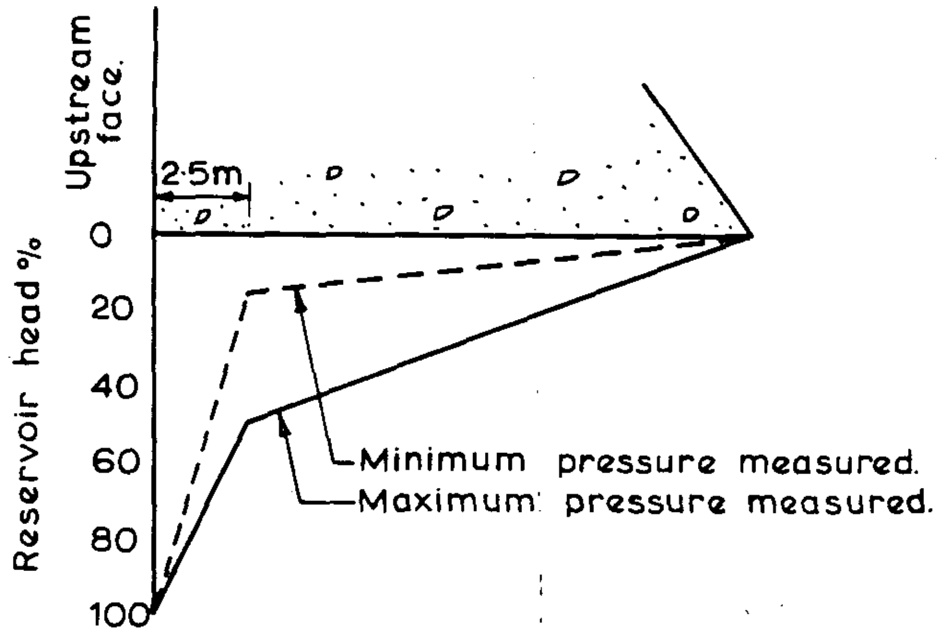

Fig. 5 Uplift pressures (no pressure relief)

Experience on other dams suggested that relatively large movements could be expected during impounding and that this will be followed by a pattern of seasonal movements in which the range of total movement will be smaller than the initial movements.

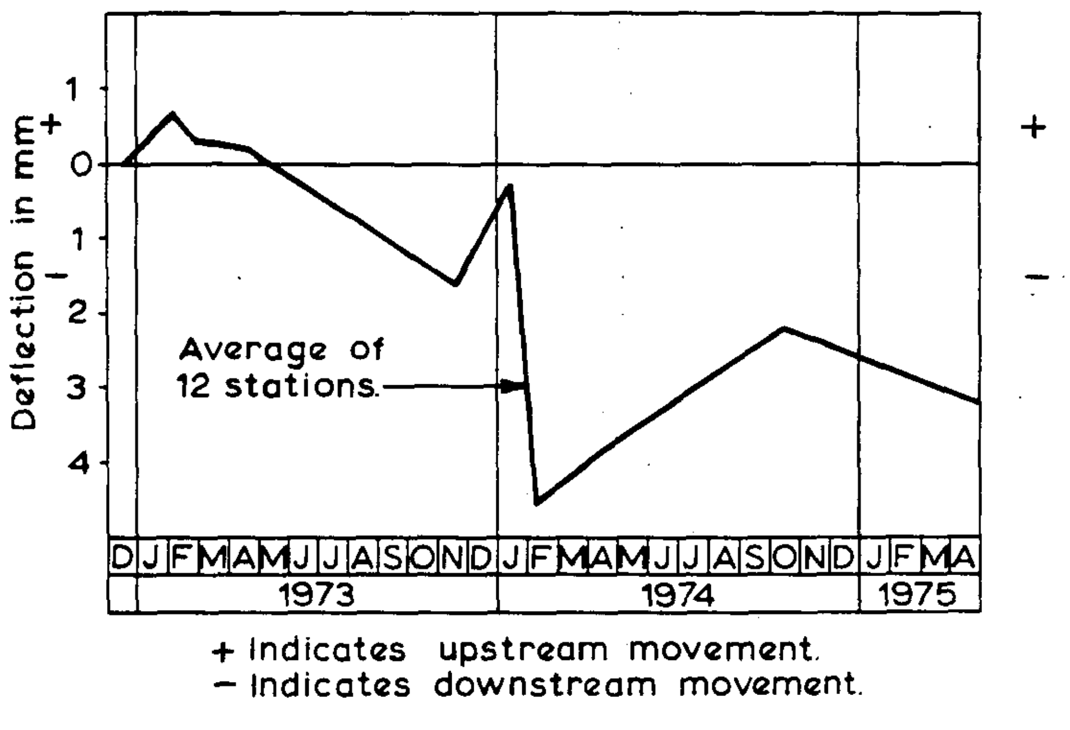

Fig. 6 Horizontal deflections at roadway level

Initially, the head of the dam moved upstream as the reservoir level rose, a maximum movement of 4.5mm being recorded when the reservoir was just over half height. As filling continued the movement reversed and, when the reservoir spilled for the first time, all but one station indicated movement downstream, the maximum movement recorded being 8mm. Since that time movements have been in a narrower range ± 3 mm. Readings have been timed to suit the best surveying conditions and it is possible that greater movements have occurred than have been noted.

The pendula movements are smaller than those indicated by surveying methods, At the end of the initial impounding a tilt of 2.5mm was noted and subsequent figures have been within 0.5mm of this reading. Displacements of the pendula of 1 to 2mm along the axis of the dam have also been observed. Experience on similar dams, described by Curtis(2), has indicated that a cyclical pattern can be expected with an upstream movement in the summer months and a downstream movement in the winter in response to changes in concrete temperature. Indications of this trend can be seen in Fig. 6. There is also evidence of a link between uplift pressure and deflection.

VERTICAL MOVEMENTS

Levelling pins have been cast into the head of the dam and at ground level on the downstream face. To date there has been a general upwards movement at the head of the dam, the maximum rise being about 5.5mm.

Two influences appear to be at work. The structure expands and contracts in response to changes in concrete temperature and movements occur in the foundation. For example, during the winter of 1974-75 one section of the foundation rose by about 7mm and the head of the dam rose just over 4mm, the difference being accounted for by shrinkage of the structure.

A definite pattern has not yet been recognised in the vertical movements and this aspect will be scrutinised with particular interest in the next few years.

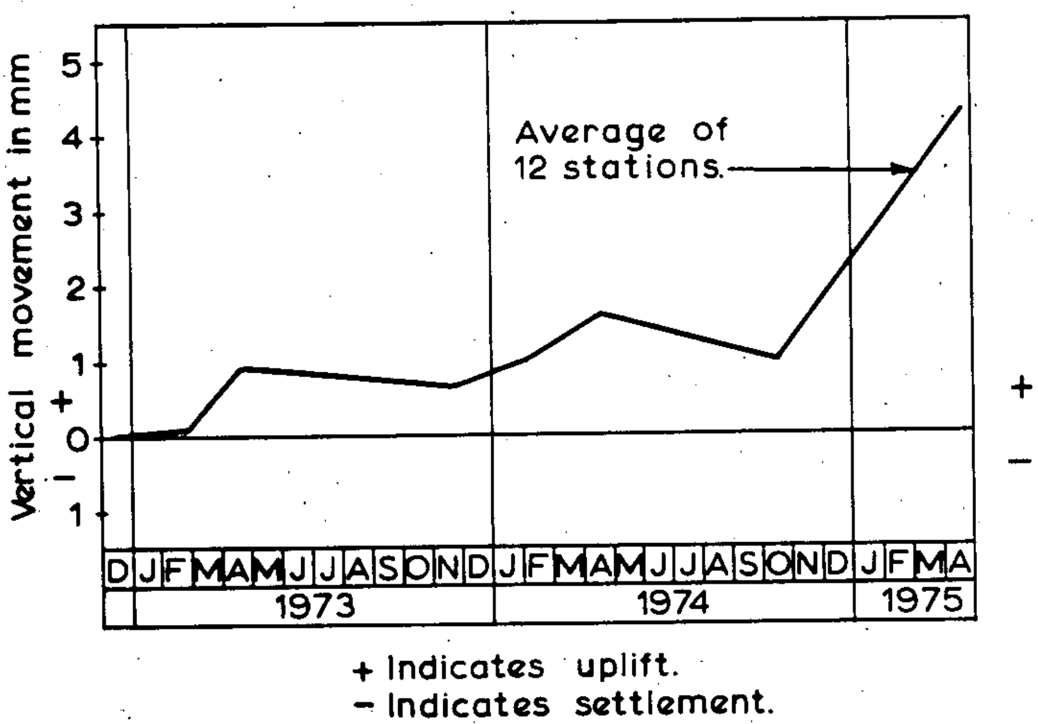

Fig. 7 Vertical movements at roadway level

CONCRETE TEMPERATURES

Temperatures in five monoliths are recorded using thermistors connected to terminal sockets in the gallery.

A seasonal cycle has already shown itself with the extent of the variation dependent on the location of the thermistors. The displacement between the temperature cycle of the dam concrete and ambient temperature can be observed in Fig. 8.

Temperatures near foundation level have varied from 5° C in early spring to 8.5° C in late autumn. The highest thermistors have shown the greatest range, typically from 3° C to 10.5° C. The reservoir overflowing has a marked effect, e.g. with the reservoir overflowing, the highest thermistor in the spillway registered 5° C when 10.5° C was noted for the equivalent thermistors in non-overflow monoliths.

MOVEMENTS AT JOINTS BETWEEN MONOLITHS

The dam is constructed in monoliths (or blocks) with a maximum length of 12 metres. Joint movement indicators have been installed to monitor the relative movement of some of the higher blocks at roadway level and in the inspection gallery. Two different cycles can be observed, as shown in Fig. 8. The joints at the head of the dam respond fairly closely to air temperature while, in the inspection gallery the movements reflect the concrete temperature in the heart of the dam.

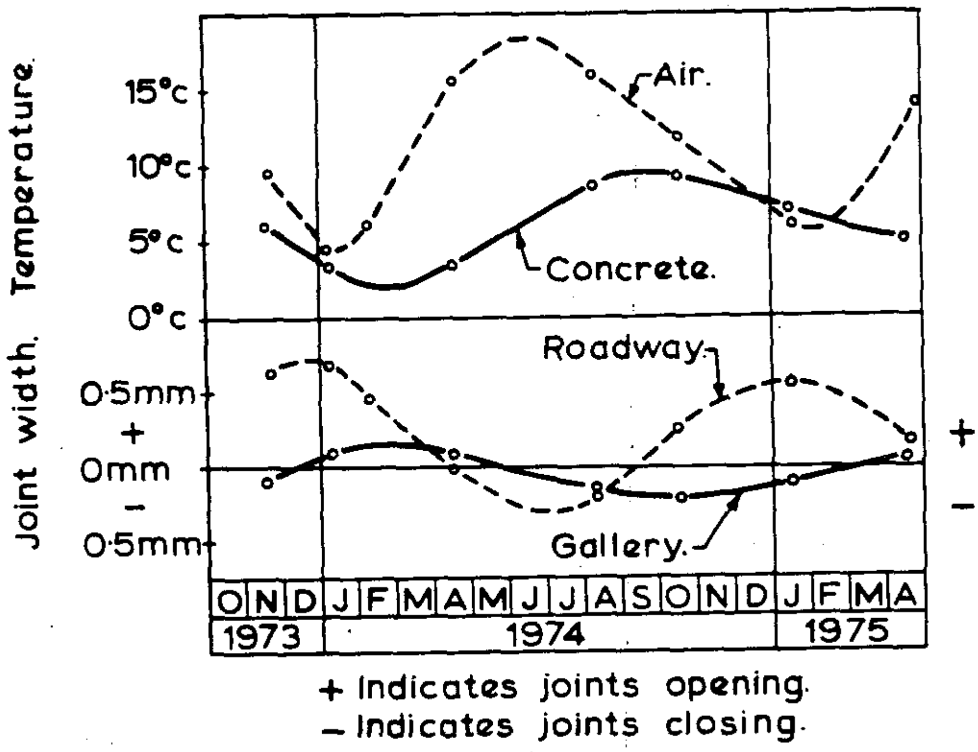

Fig. 8 Concrete temperatures and joint widths between monoliths

ACKNOWLEDGEMENTS

The recording of information at Bradan Dam is a combined operation by the staff of Mr D Mitchell, FICE, Divisional Manager, Ayr Division, Department of Water, Strathclyde Regional Council and the Council's consulting engineers, Babtie Shaw & Morton. This paper is based on the work of both groups of engineers.

The author wishes to express his thanks to the Strathclyde Regional Council and to Mr. W T Devenay, BSc FICE, Director of Water, for permission to present the paper and to acknowledge the assistance of Messrs J Ferguson and R G Menzies, BSc, in the preparation of the diagrams.

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MOVEMENT INSTRUMENTATION AT THE UPPER GLENDEVON DAM AND IMPROVING THE OUTLET ARRANGEMENTS AT THE LOWER GLENDEVON DAM OF THE FIFE REGIONAL AUTHORITY, SCOTLAND

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SYPNOSIS

A method of measuring small horizontal and vertical movements of a concrete dam more quickly and accurately than by optical methods using a laser is described, together with measurements by levelling and across joints.

Alterations to permit the full capacity of the outlet arrangements at an earth embankment dam to be used more safely are also described, together with other improvements.

INTRODUCTION

The Lower Glendevon Dam is a 31m high earth embankment completed about 1922. The Upper Glendevon Dam, in the same valley, is a 46m high (from stilling basin floor to overflow crest) mass concrete gravity dam completed in 1955. The dams are in the Ochil Hills and supply water to the Fife Region which lies to the east. This paper describes some of the measures taken at the dams as a result of recommendations made following statutory inspections.

MOVEMENT INSTRUMENTATION AT THE UPPER GLENDEVON DAM

LASER ALIGNMENT

Traditional methods of measuring small horizontal and vertical movements of a structure in relation to the ends of a reference line which is assumed not to move require skilled observers and are excessively time consuming. The Author was fortunate in obtaining a demonstration of the laser alignment method at a time when a decision on the method to be adopted at the dam was about to be made. The potential for simplicity and dramatic reduction in time of observations by the use of the laser method in comparison with other methods was immediately apparent. The difficulty, as always when promoting something new, was to get someone to make the equipment. After some negotiation with specialist firms an order was placed with Survey & General Instrument Co. Ltd. for the supply of equipment under the guidance of the Division of Optical Metrology of the National Physical Laboratory.

The equipment consists of four units; a 4 mW laser connected with bulldog clips to and powered from the observer's car battery; a 1mm dia. hole (known as the pinhole) mounted immediately in front of the laser and through which its beam is directed; a zone plate holder into which zone plates of the correct focal length for each of the seven movement positions are in turn placed; and a screen onto which the laser beam is focussed by the zone plates to form small bright spots, the horizontal and vertical positions of which are recorded for each zone plate position. The 'ironmongery' of the system, which ensures accurate re-location of the units, comprises spigot mountings for the units which fit into 10 sockets embedded in the dam. Movements are measured relative to and at right angles to the line between the pinhole and the screen, which are set 362m apart.

It is important to appreciate that precise re-location of the laser is not necessary. The zone plates are thin metal into which concentric rings and a central hole have been cut. They act exactly like optical lenses, which could not be used in this case because the focal lengths required are too long. The screen looks like plain squared graph paper.

Spot movements on the screen magnify zone plate movements. For example, a zone plate half way between the pinhole and the screen moves half as much as its spot on the screen.

A sight line only 300mm above the roadway surface has been adopted in order to simplify the equipment and the procedures for using it.

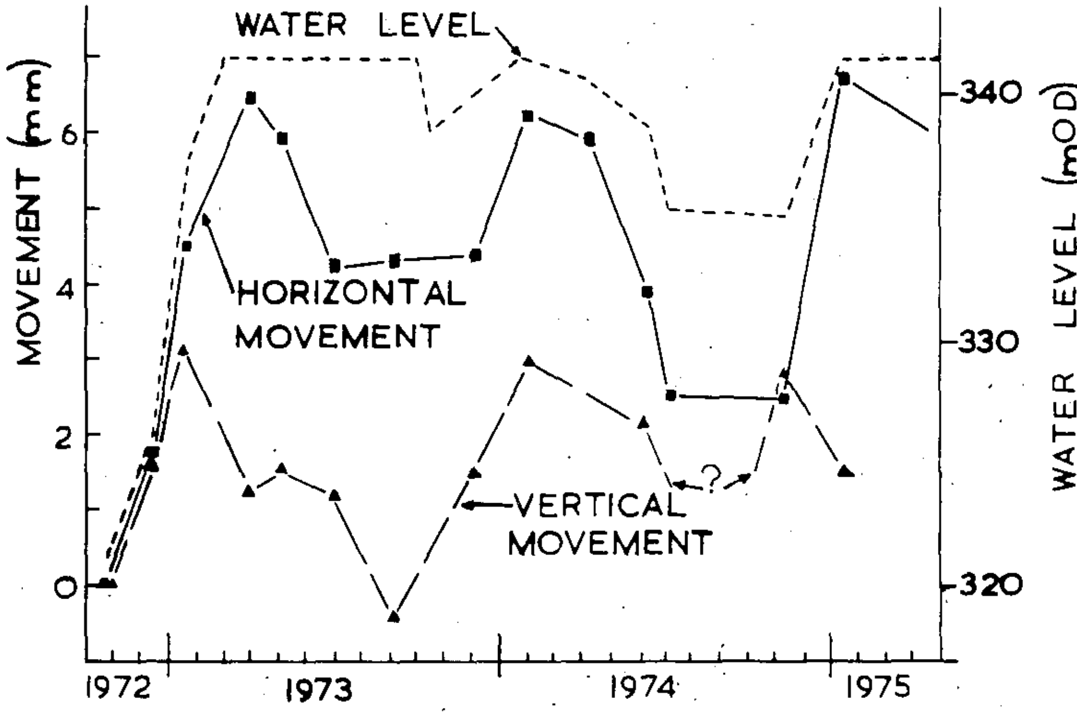

Fig. 1 Movement of Monolith 18

Figure 1 shows the horizontal and vertical movements at the top of the highest part of the dam in relation to the reservoir water level. Because the Upper Glendevon Reservoir feeds into the Lower its water level can be adjusted, within limits, without interfering with water supply requirements, and it has been possible to get a good idea of the movements resulting from changes in temperature and from changes in water load. From Figure 1 it is deduced that due to temperature changes with the reservoir full between February and August the top of the dam moves 2mm upstream and 3.5mm upwards, and that due to increased water load resulting from a water level rise of 20m to overflow level the top of the dam moves 4.5mm downstream and a negligible amount vertically. The pattern of movements of the other monoliths is similar, the amounts varying with their heights.

The following comments are made following 2½ years use of the system:—

The system has proved to be far superior in simplicity, speed and accuracy to alternative methods of which the author has had experience. The attainable accuracy of any method is limited by atmospheric distortions and with sufficient care similar accuracy can probably be attained by most methods. In practice however the simpler and quicker a reliable method is the more likely it is that the best accuracy will be achieved.

There is little difficulty in obtaining satisfactory observations of the horizontal positions even when atmospheric conditions are not good. On a few visits to the dam conditions were too bad to obtain observations for some zone plate positions furthest from the screen.

Satisfactory observations of the vertical positions are only obtained during suitable weather conditions. It would be of great interest to know to what extent raising the sight line would have improved the incidence of suitable conditions; it is thought probable that not much improvement would result.

The ideal weather conditions are overcast with a strong wind. In winter good vertical results are often obtained when the weather is not 'ideal'. In summer it is much harder to get good vertical results and present indications are that they may only be obtained before sunrise in *ideal* weather.

There is no difficulty observing during 'ideal' weather. One demonstration of the equipment in use is enough to make a competent observer.

The magnitude of the oscillations of the spot on the screen provides an excellent guide to the reliability of the observations. The greatest magnitude occurs with the zone plate position furthest from the screen (242m.). This is never better than 0.7mm. and at worst is about 5mm. Quite good vertical results are obtained when this spot oscillates less than about 1.8mm.

Although greater amplification of the movement is obtained the further the zone plate is from the screen, greater accuracy is obtained the nearer it is to the screen because atmospheric distortions are less.

Two observers will usually agree on the mean spot position within 0.3mm. when it is oscillating 3mm.

The spot diameter varies between about 6mm. for the zone plate 242m. from the screen and 1.5mm. for the zone plate 41m. from the screen.

A set of observations takes about 1¼ hours at the dam and can be made by one person.

It seems reasonable to claim that the probable error in the movements shown in Figure 1 is less than 0.5 mm.

LEVELLING

Level points have been established along the crest of the dam and have been levelled using a Watts autoset level (with a parallel plate micrometer) and two invar staves. The results agree well with those made with laser. The levelling procedure takes at least a whole day with three or four people and has to be stopped in windy conditions. More points are covered than with the laser equipment. It is expected that the laser alignment will be part of the regular observations of the dam with the optical levelling in reserve as a means of providing more data in the event of any unusual condition being revealed.

JOINT METERS

Joint meters have been installed at the top of the dam at 13 joints between monoliths. They record relative movements across joints in three orthogonal directions. On one side of each joint there is a metal block having 3 flat faces to suit the 3 directions and mounted on a bar embedded in the dam. A metal sphere, mounted on a bar embedded in the dam on the other side of the joint, is positioned about 4 mm from each of the flat faces. Feeler gauges are used to measure the gaps between the flat faces and the sphere.

The distance between joints is 12.2 m and the average longitudinal movement between February and August is 0.6 mm (Some joints open and close more than others). The extreme horizontal (upstream - downstream) and vertical movements are not more than 0.2 mm except at one end of the dam where the valley is steep sided, and movements up to 0.4 mm have been recorded there.

IMPROVING THE ARRANGEMENTS AT THE LOWER GLENDEVON DAM

ORIGINAL ARRANGEMENTS

The dam has a draw off tower in the reservoir connected to a culvert about 168 m long, running through the dam . The supply pipe is 610 mm dia. and water can be drawn from three levels in the tower. The means of emptying the reservoir is by a separate scour pipe which is 915 mm dia. at its inlet from the reservoir and tapers to 610 mm dia. near its outlet. A 610 mm dia. sluice valve controls the discharge from this pipe. The only guard to this valve was a flap at the upstream end of the pipe and operated by chains. The 381mm dia. compensation water pipe branches from the scour pipe. The supply and compensation pipes extend the full length of the culvert but the scour pipe ends near the base of the tower so that the discharge from it flows in the culvert. A long standing instruction had limited the opening of the 610mm dia. valve to 64mm in view of the risk of the energy from the high velocity discharge dislodging the two pipes which rest on concrete stools in the culvert. It had, in fact, been opened 229mm on one occasion.

ALTERATIONS BETWEEN 1924 AND 1955

The circular tower comprises a 229mm thick brick lining inside with masonry (at high levels) and concrete (at low levels) outside. Between the brick and outer material there is a 75mm thick coal tar pitch water seal. Early in the life of the tower vertical cracks developed in the outer material and steel bands were placed round and near the top of the tower in 1924, 1929 and 1930. Horizontal cracks have also developed and in 1955, as a precaution against collapse of the tower, a 1780mm thick concrete plug was placed in the culvert near the base of the tower and more steel bands were placed below the original sets. All three pipes and an access pipe with blank flange pass through the plug.

RECENT ALTERATIONS

The following alterations, for which reasons are given, were made between 1972 and 1974 :—

The flap was removed and a second sluice valve installed at the downstream end of the scour pipe. The flap had on occasions shut suddenly with undesirable impact and in 1972 the chain broke and was found to have badly rusted. It was likely that, even with a new chain, violent shutting would occur with full bore velocity in the pipe as it was not possible to lift the flap above the horizontal. Installation of the new valve ensures that the flow can be shut off in the event of malfunctioning of one valve in an open position. It is intended that the new valve,which lies immediately downstream of the concrete plug, will normally be shut and the old valve open. In the event of collapse or flooding of the tower it is likely that it would still be possible to drain the reservoir through the scour pipe. The thrust on the new valve is catered for by a special casting, flanged for the valve, fitting over the spigot end of the scour pipe and with four holes for anchor studs. Holes were drilled through the 1780mm thick concrete plug for the stainless steel studs which have bearing plates and nuts at the upstream side of the plug. The studs were tensioned and their holes resin grouted. The valve is electrically operated either remotely or at the valve.

A reinforced concrete collar, well keyed into the brick lining of the culvert, was constructed downstream of the concrete plug to improve the resistance to shear between the plug and the brick, the adequacy of which was in some doubt.

A 6mm thick galvanized steel wall was constructed to separate the high velocity discharge of the scour pipe from the other two pipes in the culvert. For the upstream half of the culvert the concrete invert and brick walls were lined with 6mm thick galvanized steel secured by bolts, where the high velocity flow could possibly cause erosion. The cross section of the culvert before and after the alterations is illustrated in fig. 2. This arrangement has now been tested with both scour valves fully open, when the discharge is about 6m³/s. The steel wall is 915mm high for the upstream half of the culvert and 1372mm high for the downstream half,which allowed a little margin over the calculated depths of flow. It is gratifying that the actual depths accord well with those calculated. The velocity at the valve is 20m/s and at the downstream end of the culvert about 6m/s,which is still super-critical (i.e. there is no standing wave).

The alternative of laying a third pipe the full length of the culvert was not adopted because it would have involved an undesirable reduction in the discharge and would have proved more costly.

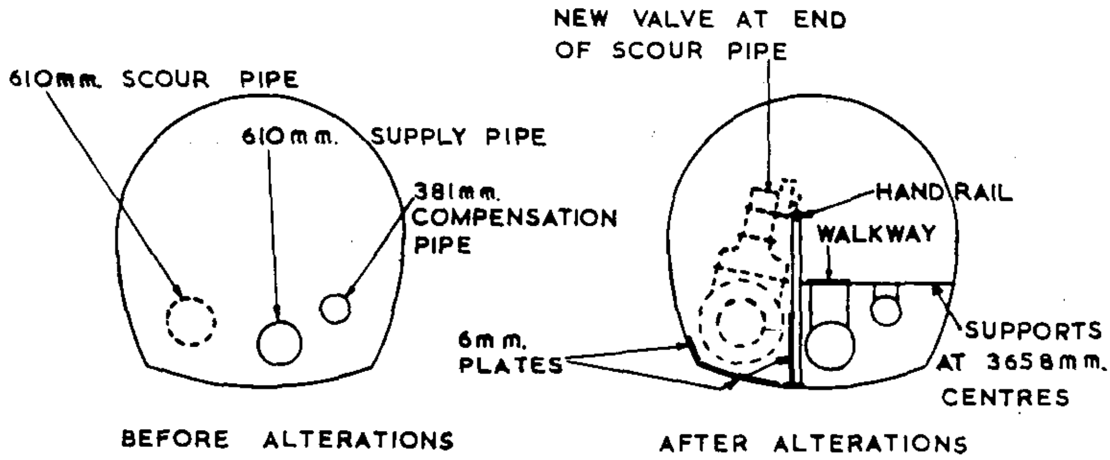

Fig. 2 Cross section of culvert

To improve access and security a steel walkway and handrail was incorporated into the design of the supports for the wall and a steel screen placed across the culvert at its portal. Shaped oroko timber blocks were added between the support steelwork and the pipes to resist flotation of the pipes, which was possible in extreme flood conditions.

The total value of the Contracts for carrying out these recent alterations was about £14,000.

MONITORING RESERVOIR OPERATION FOR INSPECTIONS

H C Parkman, BEng MEng CEng FICE MASCE FIPHE FIWES

MANAGING DIRECTOR,

WARD, ASHCROFT AND PARKMAN

SYNOPSIS

The scope, value and use of instrumentation in the daily operation and statutory inspection of dams is discussed in the light of experience gained in the commissioning and monitoring of the gauges and other instruments installed in a mass concrete gravity dam in Dyfed, South Wales.

The instrumentation embraces vibrating wire strain and temperature gauges, optical stressmeters, piezometers, inclinometers, crest alignment collimator and V-notch weirs for the measurement of seepage flow.

The requirements of instrumentation and frequency of monitoring are discussed.

The paper also describes the alarm and control telemetry system provided to operate the river regulating reservoir impounded by this dam.

INTRODUCTION

Reservoirs should not be managed without up-to-date knowledge of the condition of all items of plant and structures which together comprise the headworks. It is therefore necessary that those in charge of day-to-day operations should receive a reasonable flow of current data on the behaviour of the impounding structure.

The local supervisor must be able to cope with, record, tabulate if necessary, and comprehend the situation depicted by these readings. They must be so selected that they warn him of immediate problems yet they must also be sufficiently detailed and comprehensive to enable the Inspecting Engineer to interpret the long term trends.

The data, therefore, is the basis for all decisions on the performance and safety of the structure, a structure which is usually extremely costly to build, difficult to replace and performs a vital role in the health, prosperity and convenience of the community.

REQUIREMENTS OF DATA PROVISION SYSTEM

It is logical, and easy, to agree with this appreciation of the necessity to provide data. It is usually more difficult to meet all the following basic requirements of a monitoring system :

1) Provision of a reasonable amount of understandable readings which can be easily stored.
2) Easy operation of measurement equipment.
3) Good access to measurement read-out point.
4) Durability of instruments and associated measurement and transmission devices.
5) Simplicity of initial installation.
6) Reasonable cost of equipment.
7) Correct spread and mix of parameters read.

This paper deals with these points with particular reference to the Llysyfran mass concrete gravity dam, 32m high, constructed in Dyfed, South Wales in the period 1968 to 1972.

MAIN DATA TO BE MONITORED

The three main items which need to be monitored on any impounding structure are seepage , uplift or pore pressure and movement. A secondary, though in some cases extremely important, item which should be recorded is internal stress.

It is also, of course, necessary to know the general operational conditions existing at the time of measurement of data, and therefore readings of the water level, rainfall, air and water temperatures, sunshine hours and spillway and regulated discharges (including tailwater level) should form part of the reservoir records.

SEEPAGE

In order to measure seepage, and identify the general location of the flow path, it is necessary to separate the percolation beneath the dam from that through concrete and, of course, isolate the surface water run-off from both.

Figure 1 illustrates a typical monolith of the Llysyfran Dam. Seepage water from the reservoir which does manage to traverse the grout curtain finds its way into the porous concrete pipes, surrounded by gravel, which are laid across the broad foundation and is led into a drain along the downstream toe (see Figure 2). The flow can be measured at manholes located at frequent intervals along this collecting drain. Foundation percolation may also be at sufficient pressure to overflow through the pressure relief drains and can then be measured, also separately, in the drainage gallery channel.

Percolation from the reservoir which passes through the upstream face of the structure is intercepted by 100mm diameter drains constructed at 3metre centres. The higher drains discharge into the inspection gallery and the lower ones into the drainage gallery. The separate flows can therefore be measured in the galleries and their location identified. An alarm is connected to the measurement point in the supply tunnel so that excessive inflow into the drainage gallery, or a burst in the scour pipes, is immediately ascertained.

Surface run-off from the downstream face of the dam and from the abutments is collected in a completely separate drainage system (see Figure 2) and discharged into the stilling basin.

Adequate monitoring on a dam of this type and size can therefore be seen to require a gallery system and an extensive network of separate drainage pipework and manholes. The galleries have, of course, other functions such as access across the spillway, access for future redrilling, if necessary, of the relief wells and access to the strain gauge, inclinometer and piezometer reading points. Indeed only parts of the dual system, and the manholes, are solely installed for flow recording purposes.

UPLIFT PRESSURE

The second main item to be monitored is the distribution and variation, with water level and time, of the uplift pressure across the base of the dam.

At Llysyfran Maihak pore pressure gauges were installed beneath three monoliths. The location of those installed beneath Monolith F is shown in Figure 3 and the distributions compiled from the readings over a period of 2 years shown on Figure 4.

Uplift pressures on the line of the pressure relief wells can also be directly measured by plumbing the wells.

MOVEMENT

Horizontal and vertical deflection of the crest at Llysyfran is measured by micro-alignment telescope and inclination of the monolith by Maihak inclinometers located as illustrated in Figure 3. The movements so far recorded have been compatible with an initial compression of the foundation rock but no significant tilting or flexing of the dam itself.

STRAIN

The comprehensive installation of strain measurement instruments installed at Llysyfran is not necessary for every similar structure. Llysyfran is a particular case as it is intended that it should be raised by 12 metres as soon as the anticipated demand in the area commences to increase. The initial section of the dam was therefore designed to accept additional concrete on the downstream face at the time of raising (see Figure 3). In order that the strain within the concrete mass could be monitored before, during and after the increase in height of the dam in order to assess the homogenity of the extended section, rosettes of vibrating wire strain gauges have been installed in four of the monoliths. The extent and location of these instruments in Monolith F is shown in Figure 3.

It is intended to compute the principle stress at each rosette point and a computer programme has been compiled to ease and accelerate the calculations. Meaningful results cannot as yet be produced as there has not been a large enough cycle of water levels to zero the structure. The reservoir inflow since commissioning has been steady and large enough to satisfy demand therefore there has been little fluctuation from the full state.

In order to convert the strain readings to stress values caused by hydrostatic load alone, the characteristics of the concrete, and variation of those characteristics with time need to be determined. Accordingly a programme of concrete testing has been carried out by Dr. F T Williams at Sheffield University to determine the stress-strain-time characteristics of the hearting and facing concrete used. Information and experimental data has also been obtained on Young's Modulus, Poisson's Ratio and temperature effects on the concrete.

To confirm the stress values obtained from these vibrating wire straingauges, three optical stressmeters supplied by Dr. F T Williams have been installed in Monolith F (see Figure 3)

LLYSYFRAN INSTALLATION EXPERIENCE

From the experience gained at Llysyfran it is evident that seepage monitoring can be easily arranged to satisfy all the seven basic requirements listed earlier in the paper. It may be argued that staff show a reluctance to climb down manholes to measure the head through V-notch weirs. However the alternative method of measurement by electrodes and remote read-out is less reliable.

The piezometers have proved durable. They are not, however, direct reading and require skilled staff to operate the instrument which provides the impulse to pluck the vibrating wire and counts the time period for a fixed number of cycles. Further calculation is then necessary to convert the measurement to a pressure head.

The vibrating wire straingauges also need similar experienced personnel to take the readings and produce the stress values from the measured strains. It has also been found that the instruments have deteriorated with time and a number have ceased to function.

There is a significant installation cost associated with the vibrating wire gauges and their associated cables to the read-out points in the galleries. At Llysyfran this was justified by the need to provide basic data for the raising. For single phase construction it is not advocated that the instrumentation should be so extensive and this point is made in the following section of the Paper.

ADVISED EXTENT OF INSTRUMENTATION

Each particular dam will need monitoring instrumentation specifically designed to measure the parameters most important to the location, foundation, strata, dam shape and type, construction materials, operation procedure, earth tremor incidence or research requirements. The instrumentation installed at Llysyfran is listed in Appendix A. The general advice following is based on UK experience of concrete gravity dam design but the basis for consideration of the scope of instrumentation is for all impounding structures.

Sufficient separate drainage runs and measurement points should be installed to identify the leakage area and to monitor the quantity.

Uplift or pore pressure gauges should be extensively installed not just at the structure/foundation interface but within the foundation strata as well. It is advisable to select two areas subject to the highest heads and also to install gauges where the performance of the cut-off is considered uncertain or liable to deteriorate.

Crest movement must be monitored,and the recent application of laser beams should improve the speed and comfort of the procedure. Inclinometers should also be installed to enable an assessment to be made of foundation movement on impounding.

The extent of strain measurement is less easily determined than the instrumentation for the other parameters. Specific cases such as Llysyfran do warrant a significant number but, besides the initial installation cost, it must be remembered that there is the necessity to assign experienced staff to read, calculate and interpret results. It is therefore recommended that each scheme is considered on its merits, but should the decision be made to use gauges it is prudent to install them at least in three separate locations to allow for installation damage, malfunction and possible deterioration with age.

FREQUENCY OF MONITORING

It is assumed that dams are visited at least once a week by operational staff. Alarm systems connected to the Water Authority's headquarters will warn of excessive seepage ocurring during unmanned periods.

Each week a complete record of the seepage rates should be made and once a month the uplift pressures measured. These readings should be augmented by further sets during flood periods.

Strain gauge reading and movement measurement need not take place at intervals of less than three months.

These basic measurement intervals will, of course, be shortened during impounding or at times when the values show fluctuations inconsistent with the variation in hydraulic loading.

CONTROL OF RESERVOIR OPERATION

In order to obtain the maximum economic benefit from a river regulating scheme the controlled discharge from the reservoir should be so calculated, and the release so timed, that there is just the correct amount of water available for supply at the abstraction point.

In order to arrange such a finely tuned procedure at Llysyfran, it is proposed to install a small computer to calculate the magnitude and timing of the controlled releases from records of river flow and level and current data of the flow rates within the catchment.

The computer will be installed in the Division's Treatment Works and therefore data from the reservoir, river gauging stations and in the catchment and the abstraction station will be relayed over VHF telemetry links to the computer. The necessary instructions will be returned, using the VHF system, to the dam and the electrically motorised discharge valves thus remotely operated.

It is here relevant to stress that it is even more important in remotely operated schemes than manned installations to arrange for frequent regular and planned maintenance of all valves and associated equipment. An acceptable maximum interval between operations is one month.

The telemetry link will also be able to pass intruder and high drainage flow alarms to the Division's system operators at the treatment works.

CONCLUSION

Even in this time of national austerity it is unwise to attempt initial financial saving by reducing the money allocated to monitoring structures in service. Instrumentation, and other data providers and storage systems,are an integral part of the structure and indeed are of prime importance in avoiding dangerous situations arising,and conserve maintenance cost by giving warning of the onset of deterioration.

ACKNOWLEDGEMENTS

The Llysyfran Scheme is now owned and operated by the West Wales Water Division of the Welsh National Water Development Authority — Divisional Engineer and Manager, Mr. H Prothero, MA CEng FICE FIWES.

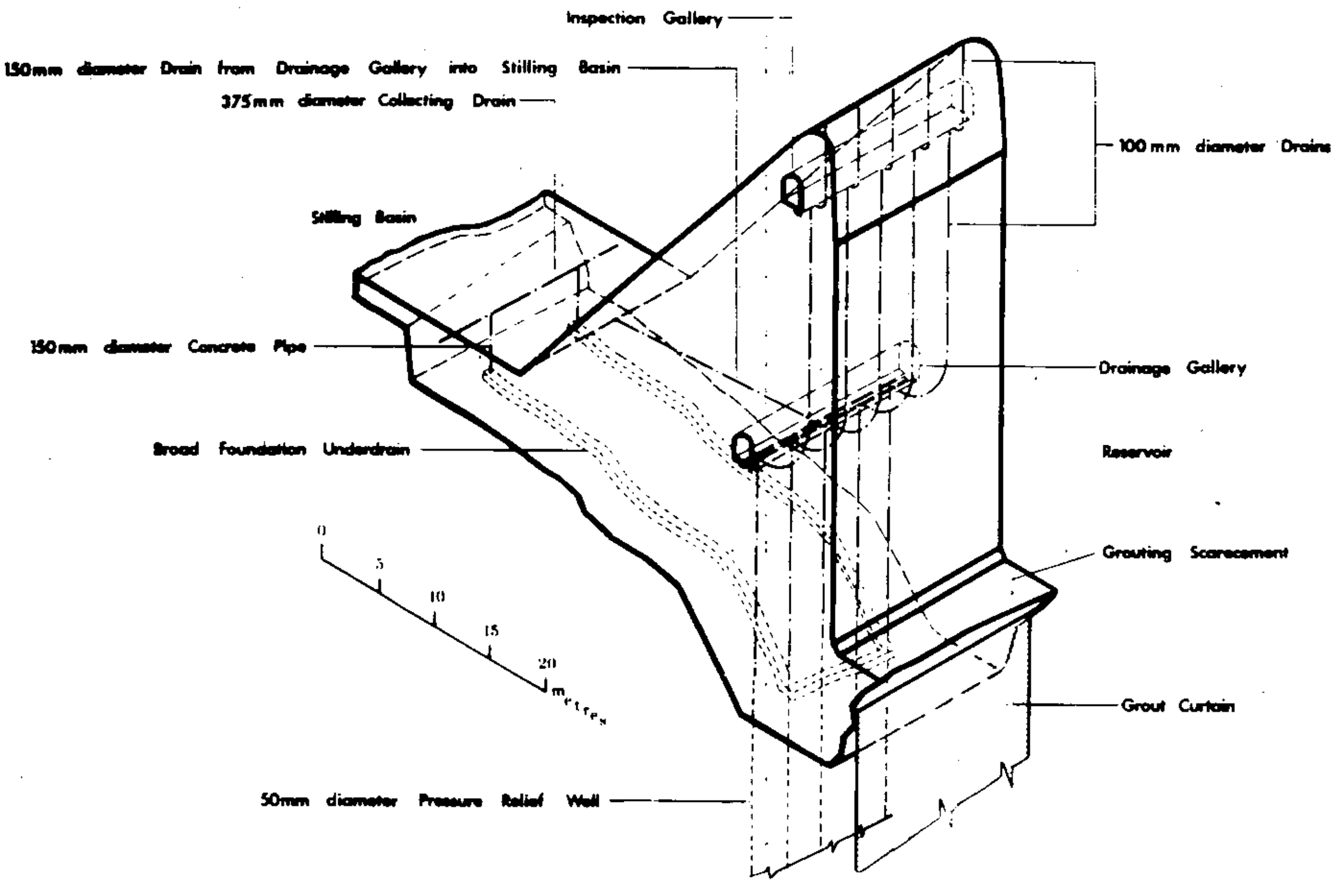

Fig. 1 Drainage of Typical Overflow Monolith — Llysyfran Dam

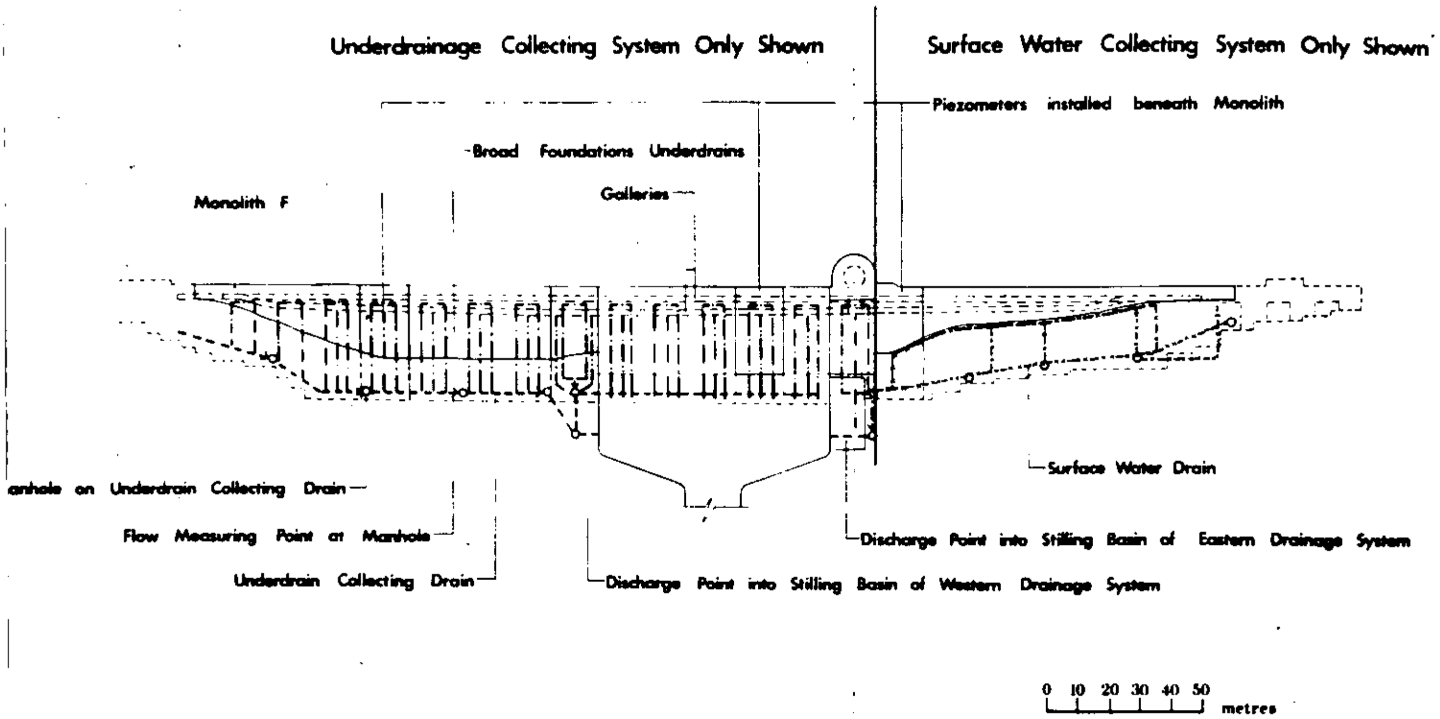

Fig. 2 Drainage System — Llysyfran Dam

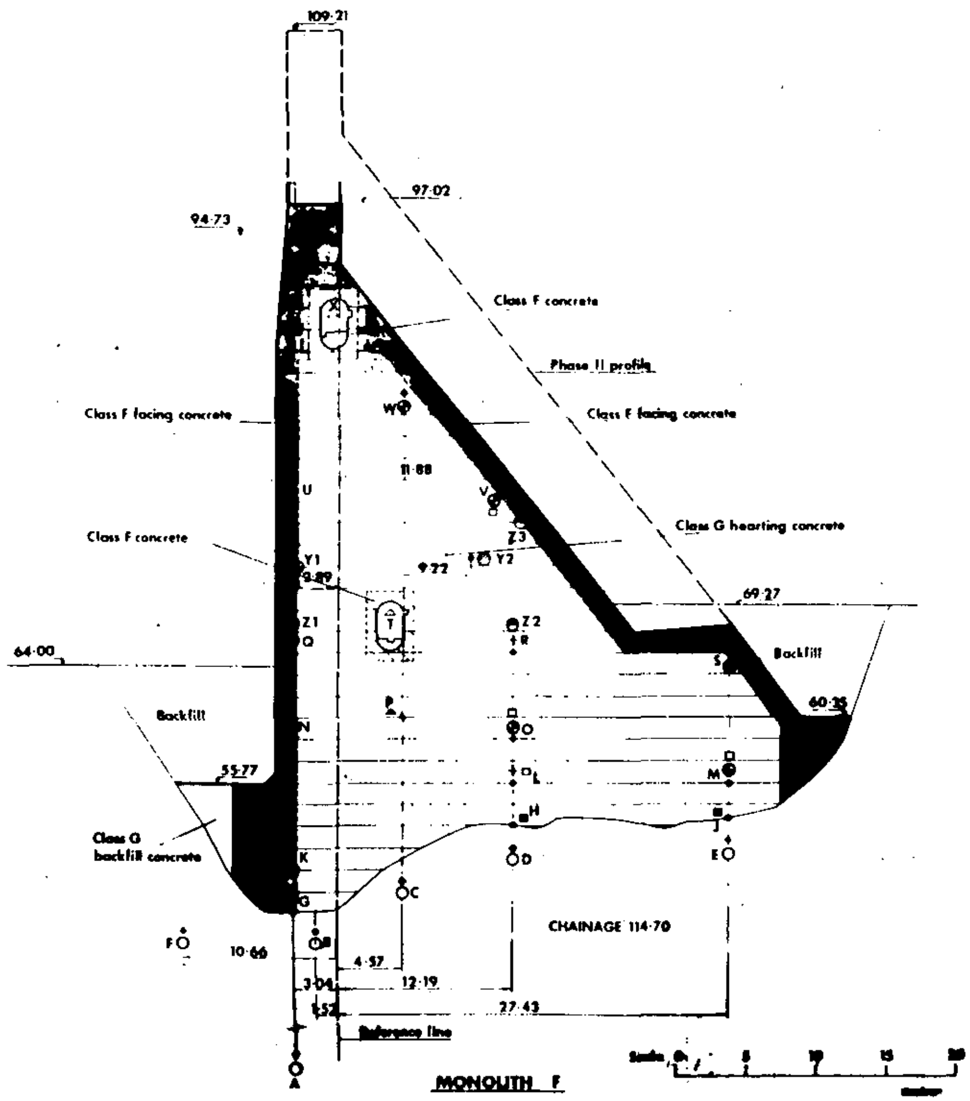

Fig. 3 Instruments in Monolith F — Llysyfran Dam

Key to Symbols in Fig. 3

SYMBOL	PARAMETER TO BE MEASURED	INSTRUMENTS PER LOCATION	REMARKS
+	TOTAL STRAIN IN CONCRETE	4	CAST IN BRIQUETTES
◔	TOTAL STRAIN & TEMPERATURE IN CONCRETE	4	CAST IN BRIQUETTES
□	STRAIN DUE TO SHRINKAGE AND EXPANSION ONLY	1	CONTAINED IN METAL CYLINDER
○	UPLIFT PRESSURE	1	PLACED IN 100 mm. DIA HOLE DRILLED AFTER COMPLETION OF GROUTING
△	INCLINATION	1	FIXED TO VERTICAL CONCRETE IN POCKET IN WALL OF GALLERY
△	INCLINATION	1	EMBEDDED IN CONCRETE
⊠	ROCK PRESSURE	1	PLACED ON CONCRETE/ ROCK INTERFACE
◒	TOTAL STRESS IN CONCRETE	1	SUPPLIED BY DR.F.T. WILLIAMS, SHEFFIELD UNIVERSITY
◍	STRAIN DUE TO SHRINKAGE & EXPANSION ONLY UNDER NO LOAD AND TEMPERATURE	1	CAST IN CONCRETE CELL

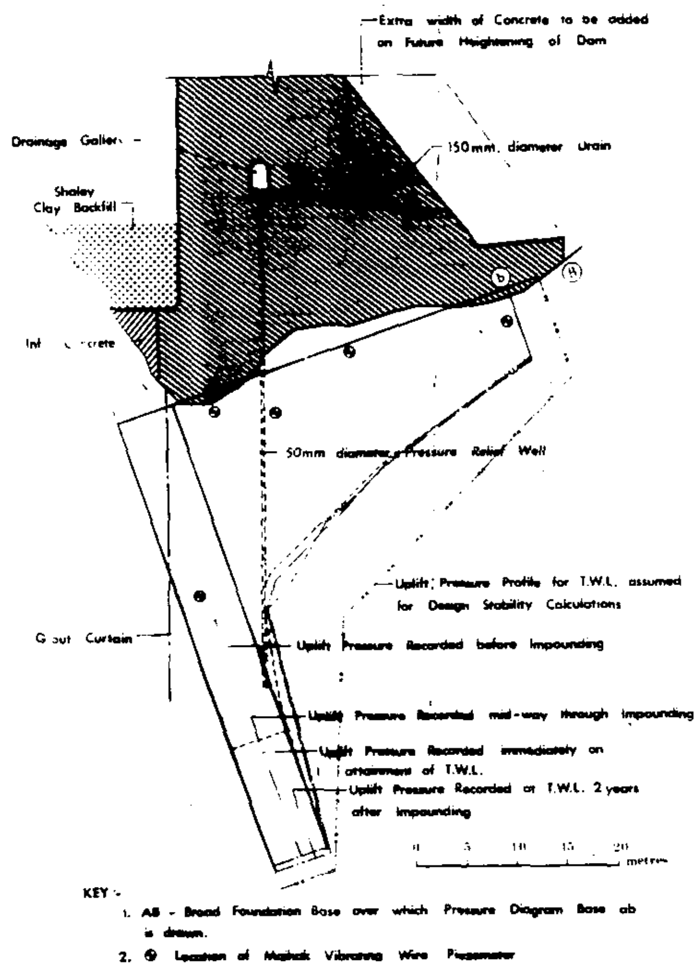

Fig. 4 Uplift Pressure Distribution — Monolith F Llysyfran Dam

APPENDIX A

INSTRUMENTATION INSTALLED IN THE LLYSYFRAN DAM

Parameter Measured	*Instrument Type*	*No.*	*Manufacturer*
Seepage	V-notch weirs	12	-
Uplift Pressure	Vibrating Wire	18	Maihak
Crest Movement	Micro Alignment Telescope	1	Rank Taylor Hobson
Inclination	Vibrating Wire	9	Maihak
Strain	Vibrating Wire	143	Deakin and Maihak
Strain and Temperature	Vibrating Wire & Resistance	18	Deakin
Temperature	Vibrating Wire	4	Maihak
Rock pressure	Vibrating Wire	9	Maihak
Stress	Photoelastic	3	from Dr. F T Williams, Sheffield University.

USE OF EXISTING INSTRUMENTATION IN EMBANKMENT DAMS

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ROFE KENNARD AND LAPWORTH

SYNOPSIS

Many embankment dams built over the last two decades have had an extensive range of instruments installed during construction to monitor pore pressures, settlement and horizontal displacement. These are generally read continuously and accurately during construction, but are often not properly maintained or read once the reservoir is in use.

The paper summarises the instrumentation installed in a series of existing dams and the action that has been taken during the post construction period to read these instruments and use the information. Reasons for retention or disconnection are discussed and proposals put forward for long term strategy in this respect.

INTRODUCTION

Instruments are installed in embankment dams to provide information or monitor performance during three main periods.

a)	Pre-Construction or design stage.	To provide information on foundations and construction materials
b)	During Construction and initial filling	To monitor performance against design predictions and allow for adjustments if required.
c)	During the operating life of the Reservoir.	To determine long term behaviour under operating conditions.

Most instruments are put in for the purpose of periods (a) and (b) and disregarded subsequently. It is the use of instrumentation during the third period which will be considered hereafter.

EXTENT OF INSTRUMENTATION

A review of the application of instrumentation in earth dams was carried out by Rofe and Tye (1) in 1969 which covered all the major earth dams constructed in this country during the previous decade. This indicated that over 1,200 piezometers, 115 vertical settlement gauges, 100 total pressure cells and approximately 50 inclinometers and strain gauges had been installed and the majority of these were still in operation at that time. Since that time several new earth dams have been constructed, so that there are at least 2,000 instruments installed in reservoir embankments built in this country in the last 15 years - quite apart from the large number in road and rail embankments, etc.

Has this considerable expenditure been justified by the use to which these instruments were put during the pre-construction and construction phases? Generally — Yes; but are the instruments still being used to best advantage? Very seldom, and we should consider why.

APPROACHES

After completion of the construction stage, including preferably a complete cycle of filling and emptying, the operation of a reservoir with its embankments and ancillary works is finally handed over from the designer to the Authority's Operation Staff. These staff will probably not be familiar with the reason for installation of the instruments, and hence will not be in a position to analyse the results obtained from reading them.

To meet this situation one of the following actions may be taken:—

(i) Disconnect all instruments and terminate readings.

(ii) Disconnect some instruments and continue to read a representative group.

(iii) Disconnect some instruments and check the remainder against warning marker limits.

(iv) Continue to read all instruments and check against warning marker limits.

(v) Continue to read all instruments and check readings against preset computer programme assessment.

(vi) Continue to read and record all instruments and arrange for regular design check.

REASONS FOR RETENTION OF INSTRUMENTATION AS OPPOSED TO COMPLETE DISCONNECTION

It is very easy to consider disconnection of all instruments and termination of readings when the embankment appears to be performing satisfactorily and well within the factor of safety calculated in accordance with the design assumption. However, it should be remembered that many of these design assumptions are based on parameters obtained from results of tests on small laboratory specimens carried out over a short time scale. For example, little is yet known about the long term behaviour of some fissured clays and alluvial materials, although there have been indications that considerable decreases in strength can be anticipated under certain conditions.

If instruments are disconnected when they are no longer required for the immediate purpose for which they were installed, then there will be no data available from operating conditions to check the design theory. This is, of course, the case in the older dams where no instruments exist and it is often only possible to detect trouble when the embankment shows visible signs of failure on the surface — often too late for effective remedial action.

The taking and recording of instrument readings is subject to human error, and often only by having a complete consecutive set of readings available can these errors be eliminated. Everybody is familiar with the instances of apparent rises in piezometric height of 10m or recorded movements of 1m downstream — enough to strike terror into the heart of any unsuspecting design or inspecting Engineer !

Unless it is completely impossible to continue readings of useful instrumentation, for reasons beyond the control of the owner, it is therefore desirable to keep at least some instruments connected and a system of readings maintained.

PARTIAL DISCONNECTION AND REPRESENTATIVE READING

This action is frequently taken as a compromise solution to the problem to reduce the number of readings and hence the time taken on maintenance. This is obviously valid provided the selection of instruments to be disconnected is determined in a logical way.

The first instruments that can be disconnected and the readings discontinued are those which were put in primarily to monitor the construction and initial filling of the reservoir and will not provide any useful future information as far as can be seen at the time.

They will probably include earth pressure cells which are mainly installed to monitor the distribution of stress during construction and during initial impounding — these frequently require maintenance by the passing of nitrogen through them, which is in itself an expensive operation, and in addition the reading requires the aid of an oscilloscope. Because of these disadvantages it is believed that there are no long-term records of this type available except in limited research projects.

In some instances, particular groups of piezometers were installed to monitor particular events during construction — a typical such event occurred at Covenham Reservoir where a special series of piezometers were installed to check on the dissipation of pore pressures in areas subject to slip due to local foundation ground conditions discovered during construction.

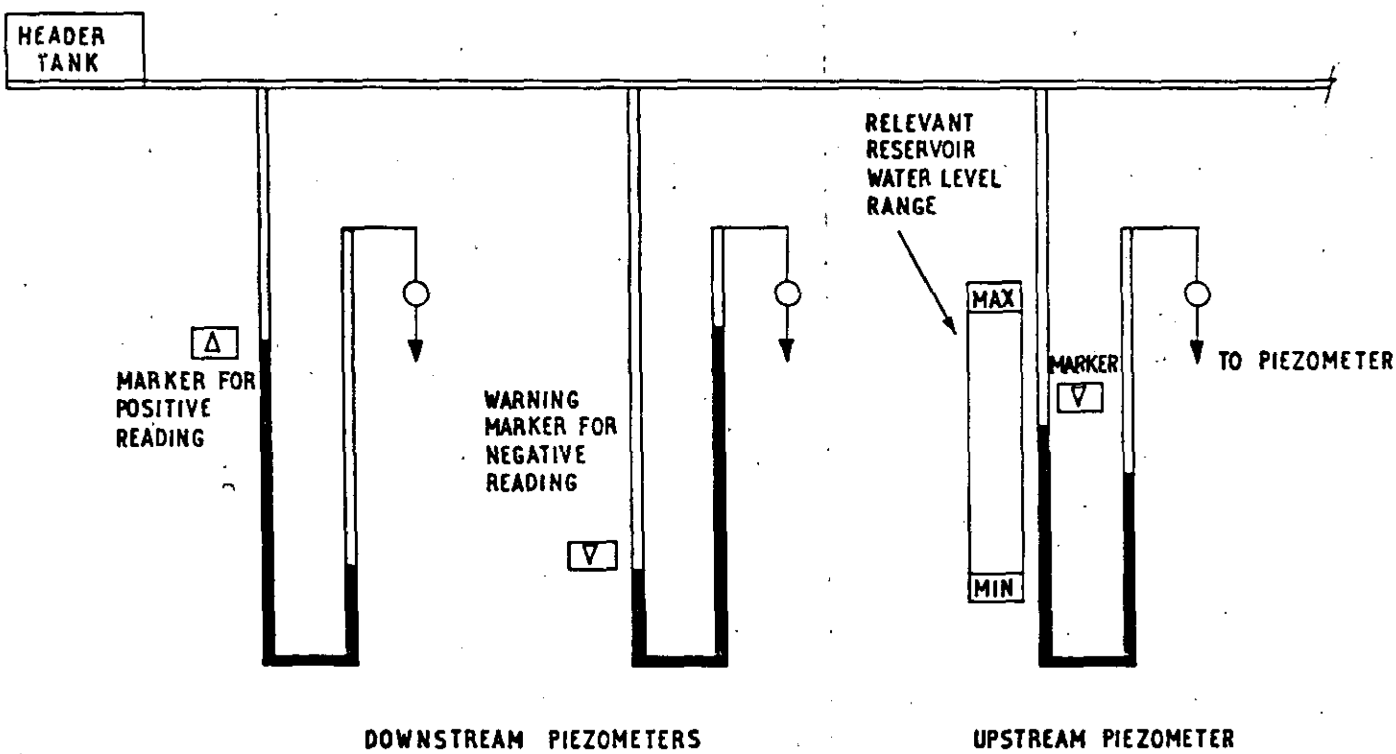
PARTIAL DISCONNECTION AND USE OF WARNING MARKERS

The disadvantage of continuing to read and record a representative group of instruments is that these still require an engineer with sufficient understanding of the particular problem to analyse the readings and check them against the current stability parameters for that bank.

To meet this problem, the Author's Firm have introduced a warning marker system for piezometer readings on three earth embankments (Draycote, Ardleigh and Covenham). In each case the system has been introduced two or three years after the reservoir has been first filled so that a steady pattern of pore pressures within the embankment and its foundation has been established.

For the downstream or outer shoulder of the embankment the markers are set close to the established pore pressure values taking into account any trend of dissipation of these pressures noted over the last two years. Initial settings have been made fairly close to existing pressures in order to receive warning of any sudden rise, but a secondary set of marker positions has been calculated taking into account the higher pressures pertaining at the end of construction so that a known safety limit can be incorporated in a quick analysis of a situation. A typical arrangement of a marker is shown in Fig. 1.

FIG. 1



TYPICAL LAYOUT FOR WARNING MARKERS ON MANOMETER SYSTEM

For the piezometers on the upstream or inner shoulder of the embankment, the markers are designed only to be used when the water level is below the bank level above each piezometer tip: above this point the readings have no real significance.

Once the markers are installed it is only necessary for the reservoir supervising engineer or assistant to check the inner limb of a set of manometers and, provided the mercury is below the marker, no action is required. This marker system has been working satisfactorily on three reservoirs, in one case for over four years, and has made a considerable contribution to ease of operation.

The system could easily be adapted for a transducer system with automatic reading but there does not seem to be any great advantage in this as very little time would be saved and, in the Author's opinion, it would be a disadvantage if the gauge houses were not visited regularly to make sure the recording instruments were in a satisfactory condition.

READING ALL INSTRUMENTS WITH WARNING MARKER CHECKS

One disadvantage of partial disconnection, and of reading only to warning marker limits, is that there is no longer available a continuous set of readings to assess the past history of an instrument as a check against its present accuracy.

If the staff are available to continue reading all instruments so that a complete record of readings is maintained, then if a warning limit marker is exceeded it is easier to check back on the history of that particular instrument to see if there is a particular reason for the present warning. However, provided the instruments with warning limits are within the set bounds then the reservoir keeper may be satisfied that stability is being maintained.

The additional cost of continuing to read all instruments need not be very great once it is accepted that regular visits to gauge houses will be maintained. The time taken actually reading instruments may be less than ten per cent of the time taken to get to and from the gauge house, so where this is the case there is good reason to maintain a full recording programme.

READING ALL INSTRUMENTS AND CHECKING AGAINST PRESET ASSESSMENT

This is an alternative which could be considered for embankments in vulnerable positions or where regular visits are difficult to arrange. The readings can be transmitted to a central control point and fed into a preset programme to assess the current safety factor and trends of the readings. There is however an obvious danger that this work will be done by inexperienced personnel who have not been made aware of the design concepts built into the assessment.

MAINTAIN FULL RECORDS AND ARRANGE FOR REGULAR DESIGN CHECKS

Where the embankment is under the supervision of an Authority with experienced design engineers on its staff and where the embankment is in a location where failure could cause extensive damage and loss of life, then it may be advisable to maintain full readings and recordings of all available instruments and arrange for a regular annual independent design check to ensure the continued safety of the structure. It is suggested, however, that this will only be necessary in a very few instances.

SUMMARY AND CONCLUSIONS

Various alternative solutions to the problem of what to do with instrumentation after a reservoir has been completed and handed over to the operating Authority, have been outlined. The advantages and disadvantages of different levels of intensity of readings from complete disconnection to full continuity have been considered and the solution to a particular problem must be drawn with respect to local circumstances such as availability of staff, design expertise, vulnerability, and accessibility.

Where possible, the Author would recommend continuance of as many readings as possible combined with a check against warning marker limits to ensure the continued safety of the reservoir. The continuous reading of instruments enables a quick check to be made in the event of a warning limit being exceeded and also ensures that valuable and irreplaceable data is not lost to posterity.

REFERENCE

1 . Rofe, B H and Tye, P F (1971) *Application of Instrumentation to Earth Dams* Jnl. Instn. Wat. Engrs. 25 No. 3

DISCUSSION : TECHNICAL SESSION 2

OPERATION OF RESERVOIRS, INCLUDING MAINTENANCE AND INSTRUMENTATION

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Gentlemen, I am very pleased to welcome you to Technical Session No 2, which I hope you will find to be interesting and instructive.

On a personal note I would say what a great pleasure it is for me to occupy this somewhat exalted position and to meet so many of my former contacts and friends. I retired five years ago as Chief Civil Engineer to the North of Scotland Hydro-Electric Board. I have always taken a very active interest in the affairs of ICOLD, and I am a member of Panel 1.

It is my pleasant duty to introduce Mr J P Williamson, who has taken over the onerous task of General Reporter at quite short notice.

REPORTER : J P WILLIAMSON

The title of this Session is 'Operation of Reservoirs, including Maintenance and Instrumentation', and from that I take it that we are dealing not solely with the structure that retains the water but also with what we are doing to utilise the retained water efficiently. Nine papers have been contributed to this Session, and these cover very fully the 'Operation' and 'Instrumentation' of reservoirs. In one or two of the papers there are reports on remedial measures which would certainly come under the heading of maintenance, and one paper does deal in part with routine maintenance.

We do not, however, have a paper devoted entirely to the perhaps rather mundane matter of routine maintenance of dams and reservoirs. This is no doubt because dams are designed to need a minimum of maintenance. Nevertheless I mention this point because, having read the nine papers, there is to my mind a clear indication that the type of staff best appointed to carry out particular parts of the function of reservoir operation and maintenance is a subject worthy of careful study. As the size of reservoirs and the complexity of operation and instrumentation increases, the appointment of staff of the right calibre to various levels of responsibility is most important.

Turning to the papers themselves, those dealing primarily with operation show that there are several different criteria which determine the most effective way to carry out the function for which the reservoir is operated. I particularly did not say for which the reservoir was built, because the purpose and use of a reservoir can change quite significantly over a period of years, as can the mode of operation. The time when a reservoir was operated in isolation from other reservoirs or sources of supply has nearly passed, and the first point which emerges from all the papers on operation is the need for a series of operating rules to maximise the total resource, whether for power generation or for water supply. Some such rules exist in the form of curves and some are, in fact, translated into computer programmes, with the advantage of rapid answers and inclusion of a number of varying parameters.

Unfortunately, this is not always fully practicable due to the restrictions of plant or aqueduct capacity, aqueduct in these circumstances being any pipeline or structure for transporting water. Restrictions on rates and times of abstraction, of course, also have a major influence. A number of papers touch on such restrictions.

Many of the rules mentioned in the papers rely on anticipated rainfall, which introduces the concept of a 'synthetic drought' or, I would submit, 'droughts', if the reservoirs involved are some distance apart. The rules must also allow for the costs which will be incurred if water has to be pumped, or purchased from another authority. Even without the cost incentive there are good reasons for minimising pumping at a time when the conservation of energy is so necessary.

There are major considerations other than yield of water in operating certain reservoirs. The maintenance of the quality of water drawn for supply is dealt with very fully by Mr Cooley in his Paper 2.1, on the Metropolitan Reservoirs of the Thames Water Authority, and in Paper 2.3 on 'Operation of Reservoirs of the North of Scotland Hydro-Electric Board' Mr Johnson and Mr Cooke deal with floods and the need to ensure that hazard to life and property downstream as well as to the structure itself is taken into account. In the latter case a computer has been used to predict flood patterns and the results obtained have agreed with experience.

In Paper 2.4 on the 'Operation of the Water Resource System of Manchester' Messrs Bolton, Milligan and Sankey give us a very clear picture of the factors which govern the optimisation of an integrated system of reservoirs, augmented by pumping and with restrictions on abstractions and restrictions due to plant and aqueduct capacity. The pumping rule curves which they reproduce are, I feel, of particular interest. I was also interested in their 'Supply and demand chart', Fig. 1, and in Fig. 3, 'Assessment of operating policy during a drought'. In Fig.1 the actual quantity of water to supply is plotted and new resources seem to have been related to this line. In my own authority we have tended to draw another line above the anticipated supply line. We call this the 'desirable resources' line. This line is above the supply line by a percentage which varies from 15% at 22 500 m^3/day to 5% at over 360 000 m^3/day. One might call it a factor of ignorance to account for unforeseen circumstances, such as a large consumer wanting to set up.

Turning now to the papers which are primarily concerned with instrumentation, we have some very valuable contributions which have naturally concentrated on the instrumentation of the structure rather than of flow of water. This is a relatively new field for the engineer concerned with reservoirs and dams. My own authority is responsible for some 35 dams, all but one being embankments, and the oldest being over 150 years old. We have piezometers in two only, although weirs to measure seepage or suspected seepage have been installed at a number of locations. We have a large proportion of dams which are relatively old - coinciding with the age-group shown as troublesome on Mr Moffat's chart in Paper 1.4 - and at the moment we really have to rely on external inspection, levelling and alignment.

The papers have tended to concentrate on the instrumentation applicable to concrete structures, although one or two deal with instrumentation in embankment dams with a special reference to the need to continue reading piezometers after the construction period is completed.

The principal instruments used and measurements taken are mentioned in a number of papers, but the use of a laser beam on the Upper Glendevon Dam, as described by Mr Kitching in Paper 2.7 , is of particular interest. I did wonder about checking the results obtained from the laser at regular intervals by conventional methods, and to what extent Mr Kitching thinks this to be necessary?

One of the values of the papers generally is to give the magnitude of the deflections recorded for the crests of concrete dams, both vertically and horizontally. It would seem that acceptable limits should be indicated by the designer of the dam so that the envelope of values of safe deflections is produced as part of the design, and not just as the result of experience. Again, more information on actual deflections at other dams would be helpful to obtain comparisons of experience throughout the profession. Several papers deal with the vitally important matter of uplift pressures below concrete dams and Paper 2.6, by Mr T A Johnston, deals with remedial measures for locally high uplift under Bradan Dam.

Mr Rofe refers in Paper 2.9 to the use of a warning marker system for piezometer readings in earth embankments so that the operator knows immediately when some action is required, and such a system appears to have much to commend it.

Several papers mention the use of V-notches to measure seepage or suspected seepage. I do not believe there is any reference made to chemical analysis of the seepage water, and I wonder if any of the authors have any knowledge of this being used successfully to determine the route or the source of leakage. No mention is made of seismic recording instruments. Admittedly all the papers are about British reservoirs, but this could well be of interest near any areas where earth tremors have been known to occur.

One of the main points stressed by the papers dealing with instrumentation is the need for regular and accurate readings. Experience helps to determine the desirable frequency of readings in each case. Several papers give valuable guides as to the frequency at which readings should be taken, In this connection one wonders if something akin to a 'Code of Practice' could be drawn up, incorporating notes on the advisable extent of instrumentation. It could include notes on suitable types of instruments, the types of measurements needed, frequency of readings and methods of recording readings in readily comparable form, methods of determining what are acceptable deflections or variations in different parameters and the need, and practical use of, alarm systems. The papers which we have before us this afternoon certainly contain much information to provide the basis required for such a code.

I feel that this brief Report has not really done justice to the papers, which contain a very great deal of practical information on how the operation, maintenance and instrumentation of particular reservoirs and dams has been carried out. The papers are not, however, a mere record of what has been done, they also give excellent guide-lines for others involved in similar work, and I feel certain that these papers will be referred to in the future.

I also feel it is difficult to provide an adequate summary of the wide-ranging information contained in the papers. The main points which emerge include, to my mind, the following:

1. The need for suitable staff at each level of responsibility, with a clear indication of those responsibilities. The reservoir keeper or the supervising engineer, for example, must know exactly how he fits into the chain of responsibilities.
2. The value of rules or control curves to maximise operation of the reservoir, no matter what its particular funtion happens to be.
3. The value of regular instrumental checks on dams, and the importance of ensuring that the results are interpreted by suitably qualified staff.
4. The need for the instruments used to combine accuracy with robustness and reliability.
5. By studying the behaviour of existing dams and reservoirs it should be possible to determine how the behaviour of the actual structures follows, or departs from, existing theory, thus influencing future designs.

? A JOHNSTON (Babtie, Shaw and Morton) :

I would like to deal with instrumentation of concrete structures and, at the same time, issue a word of warning to those of you who are not familiar with such instrumentation. You may find that those of us who dabble in this sound terribly confident - in fact we are not very confident at all. We do not know nearly as much as you might imagine from reading our papers, largely because we measure movement, pressures, strains at particular sections of a dam at a particular point in time. Perhaps there are dams where continuous recording methods are used, but I do not know of any in this country. Our readings are therefore limited, and it would be very surprising indeed if there are not greater uplift pressures, greater movements or greater strains than those we are able to measure. This is something which we should bear in mind when recorded information is subsequently used for design purposes.

Uplift pressures comprise the major unpredictable load on most gravity dams, and for this reason it is worthwhile monitoring uplift pressures to ensure that they fall within the design limits.

It was interesting to note that the pressure recorded at Bradan was not always initially a water pressure – indeed quite a number of the holes registered an air pressure. Geologists advise me that there is quite a lot of air trapped in joints in rock, and in due course at Bradan the air was expelled and water pressures were recorded.

For the vast majority of the monoliths at Bradan, the foundation uplift pressures were satisfactory. At one particular block, however, as described in Paper 2.6, (see Fig. 3), we found that uplift pressure did exceed the design value. Simple remedial action was taken by drilling a few additional relief holes at Block 26, the results being quite dramatic. Drilling an additional hole about 1.5 m away from one indicating high pressure reduced the pressure reading by 8.5 m head. I think this confirms what I said initially about the variable nature of pressures under a dam. Even if you have 150 holes throughout the length of the dam, as at Bradan, the worst pressures may not be recorded.

We carried out another interesting exercise earlier this year when we blocked off some holes. Within half an hour the pressure had jumped up from 27% to 49% of the full reservoir head.

Mr Williamson touched on what is a very important question, namely the desirable level of instrumentation. It is unsatisfactory if we say that it is impossible to define an acceptable level. I think we really must try and do better than leave it entirely to the individual designer, and a first step is to decide what tolerances are acceptable for each of the parameters to be measured. If we are trying to measure deflection of the dam, are we trying to measure it to plus or minus 0.5 mm, 0.1 mm or even 5 mm? Once we have answered these questions we will be able to set up a proper instrumentation system to deal with that parameter.

As mentioned in Paper 2.6, at Bradan we adopted a simple alignment system, with a concrete station block at either end of the dam, and using a one-second theodolite. The authors of Paper 2.2, on Boothwood Dam, recommend a half-second theodolite. I agree that is desirable, but it costs a great deal more, and for Bradan it was decided, on balance, to make use of a one-second theodolite.

In all, the cost of the equipment at Bradan in 1971, including supply and installation, was about £6000. We also investigated what else was on the market, and one manufacturer of sophisticated equipment quoted a scheme which cost £24000 for supply only. That installation would have given very accurate data, but at a much smaller number of locations. It is therefore necessary to form some judgement as to whether you want to know a lot about a little or a little about a lot. I would think that probably academics would prefer the former, while practising engineers would accept a lower degree of accuracy of recording but at rather more locations in the dam. In Paper 2.7 Mr Kitching mentioned the problem of getting the instrumentation that one actually wants. To a small extent we had this problem at Bradan. We wanted a movable target, for example, and found that we were going to have to design our own.

It thus seems that the range of options is wide. We have to be able to satisfy ourselves - and probably satisfy the accountants also - on the expenditure that is to be recommended. This is the age of cost-effectiveness, and I wonder just how we go about justifying this expenditure in a fashion that is acceptable to the financial authorities.

F F POSKITT (Ferguson and McIlveen) :

I have set out one or two points, regarding maintenance rather than instrumentation, which I hope may be of interest. I would like, however, to make a quick point regarding instrumentation.

The question raised by Mr Grøner in his Introductory Address regarding the life to be expected from embedded instruments in rigid or embankment dams is very important. If they are to have a more or less indefinite life or give a fair period of service, then instruments might be made more robust than those installed purely for construction purposes and then to be abandoned.

A problem that I am aware has caused a great deal of heart-searching in the past is that of the first few years after construction of a dam, and I would suggest that apart from any Statutory Inspections by Panel Engineers that may be prescribed in future, the following procedures would be desirable for all dams :

1 A particular detailed and rigid monitoring of data during the first five years after construction, which should be emphasized and explained to those responsible for data collection.

2 A clear statement to supervisory staff of their individual responsibilities, and of the immediate action to be taken by particular nominated personnel if certain specified occurrences which may affect safety are detected at any dam. Such events would include new or changing leakage patterns, external deformations, essential control mechanisms becoming defective or inoperable, and malicious damage to any vulnerable areas or controls.

3 A system of routine inspections by responsible staff, with the objective of each inspection clearly defined, at daily, monthly and yearly intervals depending on seniority of the staff concerned and including the 'qualified engineer' in charge, and which should cover all necessary items requiring regular maintenance, painting, lubrication etc. in addition to those which must be checked for safety purposes for each dam.

4 A set of proper log sheets on site for each type of inspection described in 3 above, with five-year summaries of all intermediate results filed centrally with the records prescribed to be maintained for each dam.

The responsible staff obviously have various grades, and it seems again most appropriate that exact inspection content should be defined to them and that the top of the line should finish with the senior engineer or supervising engineer responsible for technical use of the structure. Each of the preceding inspections from lower down the ladder should be summarised for him.

The last point I would like to make is on the question of central records. In the North of Ireland the Department of the Environment is responsible for all of the water undertakings previously vested in the various councils, and so we are perhaps in a fortunate position as regards the collection and processing of data. We are in a less fortunate position in the way that we are in a system whereby the Department of the Environment is the Undertaker or Owner and also the Enforcement Authority, and we anticipate that any legislation that emerges will have to reconcile those differences. One can only hope that within our Act, when we frame it, appropriate steps will be taken to see that if they do anything remiss as Undertaker, they can take themselves properly to task as Enforcement Authority!

G R CURTIS (North of Scotland Hydro-Electric Board) :

S I have been asked to say a few words to illustrate the work done on instrumentation by the North of Scotland Hydro-Electric Board.

We have some 35 dams carrying a greater or lesser degree of instrumentation, and of these some 20 dams carry provision for crest alignment checks by collimation. Movement is also monitored by pendulum in some cases, and there are various forms of inverted pendula. The form we use is simply a vertical steel pipe coupled into the base of a steel can, all filled with oil, with a weight at the bottom and a float at the top. A wire is stretched between float and weight. An optical instrument is used to record movement of the float on top of the tank. Accuracy is fairly high with this instrument, certainly as high as we require.

A previous speaker mentioned a dearth of continuous recording instruments. The Board has adapted a camera designed for aerial reconnaissance purposes and which can be set to take photographs at variable intervals. We have it mounted so as to look down on to the top of an inverted pendulum and photograph the position of the float in the tank at two-hourly or four-hourly intervals.

With respect to collimation, the theodolite is mounted on a rigid pillar set on a concrete base beyond the end of the dam, mobile targets being established at points on the dam. All this equipment is Italian and was selected some fifteen years ago.

Lednock Dam, a buttress dam, has this form of instrumentation and a very rythmic pattern of movement upstream and downstream obtains throughout the years. The movement is quite systematic and can be used to predict what one would expect the following year. Sloy Dam, again a buttress dam, of height 55 m, has a similar form of instrumentation which was added after construction. A moderate amount of movement is recorded, and it does not at this point conform to what one would expect by analysing Lednock Dam. Lednock went down in the winter and up in the summer. Sloy Dam is doing the opposite, probably due to the timing of low water levels. The pendulum readings bear this out.

Turning to Monar Dam, a cupola structure, collimation is supplemented by provision for levelling and for strain gauges etc. The movements are of the order of 10 mm to 12 mm per annum upstream/downstream, the maximum range shown by the collimator being about 13 mm. Earlier measurements of the crest prior to impounding show 10 mm to 11 mm of deflection movement during impounding, due to hydrostatic pressure.

In the case of Loch Dubh gravity dam we have a progression of movement over the years, from 52 mm to 56 mm. If one draws an envelope of normal or expected movement one can obtain a band of probably 6 mm for the first two dams I referred to and possibly 2 mm or 3 mm for this dam, and provided the movement in future years continues within say 3 mm, one could construct a curve which would be the norm, i.e. the dam would be behaving in its normal fashion. On Loch Dubh we had a pattern which went outside that envelope and suggested further detailed study of that dam. Following investigation a new drainage system was drilled into the dam and uplift pressures were reduced. Since then the dam has been moving within a band of about 2 mm.

B W KITCHING (Allen, Gordon and Company) :

Ⓢ The laser unit, as employed on Upper Glendevon Dam to check crest alignment, is powered from a car battery. The laser beam is shot through a pinhole immediately in front of the laser, the pinhole and not the laser being regarded as the accurate reference point at one end of the dam.

A zone plate is mounted at each point at which movement is to be measured, the zone plate being a device that operates like an optical lens. An optical lens could not be used at Upper Glendevon as the focal ranges required are too long.

A target screen is mounted at the far end of the dam and is illuminated by the laser beam to show a small light spot. There are no moving parts in this system, and the only thing that does move is the light spot on the screen. The screen is gridded like graph paper, and the spot position is read both horizontally and vertically on each occasion of reading.

The laser equipment consists of the laser and its mount, a power pack and transformer and, in this case, five zone plates. The components have spigot and socket arrangements for mounting on the dam. Each zone plate is essentially a thin piece of metal with concentric rings etched into it.

As regards joint meters, at Upper Glendevon Dam, one side of the joint has set onto it three machined faces at right angles to each other, and the other side of the joint has a sphere initially set about 4 mm from each of the three machined faces. Feeler gauges are subsequently used to measure the three gaps. Very little movement is measured in the upstream/downstream and the up and down directions, but longitudinally we get a measure of closing of the joints between summer and winter, a movement of rather less than 1 mm.

Turning briefly to the work carried out on Lower Glendevon Dam and also described in Paper 2.6 I should perhaps mention that the purpose of the low steel division wall in the tunnel is to keep the very high velocity flow, initially at some 20 m/sec, away from the pipe stools.

I would like to answer the Reporter's query about whether we checked the laser method against another method. I think anybody who has been involved with instrumentation would probably agree with the point I am going to make, that once you have used a reliable system a bit you develop confidence in it, and you do not feel the need for a check by any other method.

Another way of putting it - if one records the position of the dam one day and then returns the next day one expects the instruments to indicate it has not moved, and if they do so one feels confidence in them.

Six months later, when the equipment shows the sort of movements expected one develops real confidence in it. It is interesting to note that Upper Glendevon Dam is doing the same sort of thing with the same order of movements as was reported in the other papers.

It would be helpful to have more records of the movements that are being recorded. A few have come out of this Symposium, but the sort of thing that the central collection of records could achieve is an easily referred to list of movements of dams in the United Kingdom. This would be very valuable in assessing whether a particular dam is behaving in a 'normal' way. Mr Curtis' records for one dam that has not behaved normally are of great value.

S E H FORD (Binnie and Partners) :

Mr Cooley, at the end of Paper 2.1, makes a rather plaintive remark about the independent Inspecting Engineer who will no longer be a member of the Thames Water Authority, and I think this raises the point of the extent to which the Inspecting Engineer will be expected, or wish, to consult the designer and the Construction Engineer. It may be that on sophisticated dams it would be a very brave Inspecting Engineer who did an entirely independent inspection. It is desirable to have independent Inspecting Engineers, but they must at some stage, having done their preliminary inspection, have a facility for going to see the people who have been on the spot for many years and know about the dam and its idiosyncrasies. Perhaps these inspections will have to be in two stages, getting the benefit of independence *and* the benefit of the people who have been at the sharp end and, in particular, the benefit of consultation with the designers and the construction engineers.

This leads to my second point, which is cost. The 1975 Act is going to cost a lot more than the 1930 Act did. I would suggest a worthwhile subject of research would be to investigate the cost-effectiveness of the organisation that is going to be set up for reservoir safety, including the cost of all the overheads and, perhaps, the cost of Mr Moffat's central government data unit from Session 1. The cost of all the instrumentation and of labour-intensive reading of instruments over indefinite periods should also be evaluated. The idea of this would be to see whether we should perhaps be better designing in a few hundred cubic metres of extra concrete so that uplift was not so critical, or to add a few points to the Factor of Safety by designing a toeweight into embankments. It may be that money spent on putting a little bit more into the safety of the dam at the beginning would save some very sophisticated instrumentation which is not, in any case, going to last forever and which has got to be monitored by people costing more and more over longer and longer times.

These are suggestions for research, possibly by the University of Newcastle upon Tyne with the help of organisations like the Thames Water Authority and the North of Scotland Hydro-Electric Board, both of whom seem to have got a fair idea of what their supervision costs are.

B H ROFE (Rofe, Kennard and Lapworth) :

I would like to comment on the methods used for the production of operating rule curves for the Lake District sources of the former Manchester Corporation Waterworks, described by Mr Sankey and his co-authors in Paper 2.4. I have been associated with this work for several years, from the initial promotion of the Ullswater Scheme to the present construction of the Shap Aqueduct. Of the five factors listed by the authors at the top of page 2.4-5, the hydrological studies were carried out first, and the data derived formed the basic input for one of the earlier resource simulation studies to be put on a computer. The inflow patterns and river flow regimes determined from these studies led to the physical factors, which had to be modified in the light of discussions on the legal, operational and economic factors. This process demanded a constant series of reiterations to assess the effect of such matters as prescribed river flows, provision of freshets, maintenance of lake levels for navigation etc. In particular, it should be noted that the later rule curves utilising the basic rules established by the earlier procedure illustrated the fact that it was not economical under this regime to pump during peak power periods. Also, no allowance had been made for maintaining reservoir or lake levels for amenity purposes, but it could be seen from the examples of the rule curves shown in Paper 2.4 that this could involve a considerable additional cost in pumping to maintain the reservoir levels at a higher level than was required for yield purposes during a drought period.

The latest expression of the operating rule curves was not only a development in complexity, but in the process there had been a substantial shift in thinking from reliance on the old-established principle of the 1% synthetic three-year drought period, using actual known sequences, to the use of a synthetic period derived from historic records over a 35-year period. Extensions can be established by data generation, but nevertheless the tendency is to produce a smoothing of the natural data. It should also be noted that the original selection of the 1% probability situation is dependent on statistical theory involving selection of normal and shared distributions, which can give large variations which must be determined by a subjective judgement by the engineer and hydrologist. This reliance on statistical interpretation is a matter of some concern which may also be highlighted in our consideration of flood studies in a later Session.

Referring to two issues raised earlier in the discussion on instrumentation and relating to my Paper 2.9, I would make the following points:

Firstly, in his Address Mr Grøner asked the very pertinent question: 'How long will an instrument give reliable readings?' Secondly Mr White in Session 1, stated that instruments must be justified by the need. I feel the second point to some extent answers the first. If the need for an instrument of a certain type and in a certain place in a dam is justified, then if it does cease to give reliable readings it must be replaced or alternative means of obtaining the same information be put in hand.

E O ADEWOLE (National Electric Power Authority, Nigeria) :

Various problems have crossed my mind with regard to instrumentation of Kainji Dam, and I am happy that during this discussion some of them are being brought out.

First of all, how much data do we require? We have an instrumentation section which goes round Kainji about twice a day to take the readings in the concrete dam and in the rockfill dam. How much of this data is actually required? It seems engineers have not, as yet, determined what instrumentation is required and how important some of these readings are. How often are these readings to be taken? Should they be taken once a week? How often should we inspect a dam?

Lots of data is collected, but how does one properly interpret this data as far as the safety of the structure or the life of the dam is concerned? It seems that there are no guidelines as we have, for example, with a concrete beam, where we know when it is safe or when it is unsafe. It appears that this is not yet the case with dams.

From the readings taken, what are the guidelines for remedial action? What day to day maintenance should be taking place on a dam?

When I see some of the instruments and how they are being affected by leaching and seepage I wonder how long they will last. For example, Kainji Dam came into operation in 1969. Leaching has now put some instruments out of use, and yet we are expected to operate this dam for over 50 years. Is there any way of replacing these instruments? This is one of the things engineers should think about.

Regarding personnel, it seems in the case of Kainji that we have not developed an adequate technology for instrumentation, because the people who take care of Kainji are people who have come in since construction of the dam. Looking around one cannot easily pick personnel to advise you or to employ and so one has to fall back on specialists and others who perhaps were responsible for the construction of the dam. What is ICOLD, for example, doing to see that a technology is developed for dams as for other engineering structures? Who can actually take care of dams, personnel whom one can call upon at the level of engineers, at the level of technologists and at the level of tradesmen? I hope that ICOLD will take these points into consideration because they are not only to the benefit of developing countries. In a developed country one can go to a University or to someone who has experience, but this is not easy in developing countries. What is ICOLD doing to provide the technology that should be readily available to engineers rather than their having to individually seek advice?

In Kainji Dam recently there was a vertical movement that worried us, but there was nothing to show us whether this movement was something serious; however, it is of such concern that we are taking readings over the years of operation, recording movement with rise and fall of the lake and trying to see whether this falls within presribed limits. Should we rely on empirical data like this, or should someone give us a guideline from the construction stage that will guide us, even if in a small way, throughout the life of the dam? As engineers and hydrologists I think such questions should concern us.

I am not a specialist in hydrology, I am not a civil engineer, I happen to be a mechanical engineer, but because I am Station Manager I have to become involved in this, and I happily do so because I find it most interesting.

R C BRIDLE (T and C Hawksley) :

S I would like to give a brief description of the movement monitoring system, using surveying techniques, which was provided to complement other deformation measuring devices at Empingham Reservoir. It is hoped that this will expand upon, and contrast with, details of survey movement monitoring equipment given in some of the papers presented for discussion and be of use to engineers faced with having to measure movements of large structures and judge their significance.

Monitoring of movements occurring during construction was required, and a surveying approach seemed appropriate. Early efforts revealed the difficulties in analysing the results, especially in determining their accuracy so that real movements could be distinguished from surveying errors. Fortunately we learnt that Dr V Ashkenazi of Nottingham University had developed techniques for the statistical analysis of survey networks which assessed accuracy limits on the results of the observations. On his advice and on that of Dr P R Vaughan of Imperial College, adviser on the soil mechanics aspects of the design of the embankment, a system was designed to give an accuracy of between ± 5 mm and ± 10 mm for the location of any point on the embankment.

Fig. 1 shows the system, which has also been described in the reference, and comprises an outer network of stations around the dam and borrow pits, with two distant points to give an assumed fixed point and fixing bearing. Numerous detail points on the embankment include the tops of the inclinometers and the datum points of rod gauge extensometers. The location of the detail points is determined from the network stations by angle measurement in a manner similar to that used at Boothwood Dam (Paper 2.2). Observations are made with a Wild T3 theodolite and a Tellurometer MA100 electronic distance measuring device.

The main advantages of the system are :

i) Accuracy — accuracy limits are given with the results, so that actual movements can be readily distinguished from errors. Definition of the accuracy limits is achieved by including sufficient 'redundant' observations to allow the location of the stations to be determined by several different computer calculations. The standard deviation of the location of network stations can then be established. If the apparent change in location of a station between successive observations is, for example, one standard deviation, then there is a 68% chance that there has been movement. If the change is, say, three standard deviations, then there is a 95% chance that the change is actual movement.

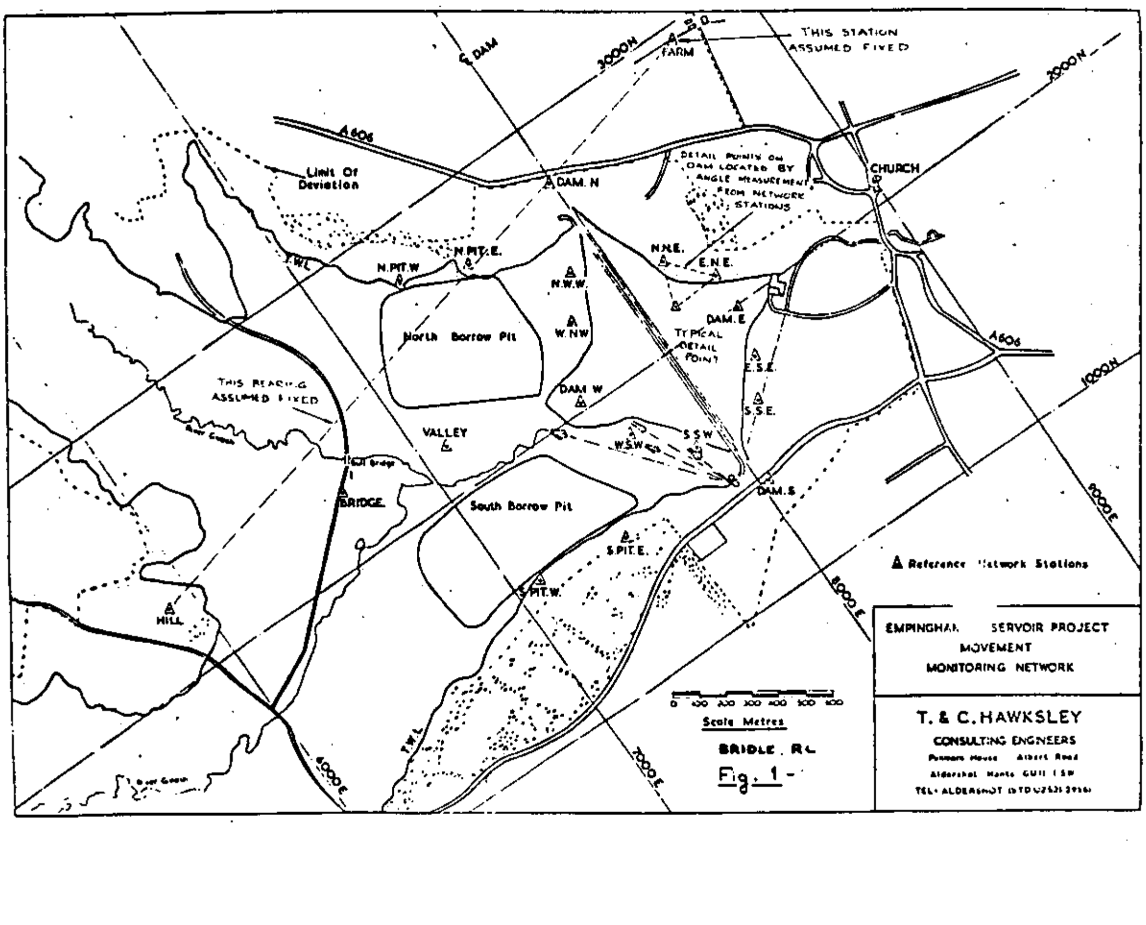
ii) Flexibility — network or detail stations can be added to or removed from the system (e.g. by submersion in the reservoir) without loss of continuity of the monitoring process.

On the debit side, although analysis of the results and location of the detail points is quick, taking the observations for a complete analysis of the network took about three weeks. However, this was only necessary three times during construction and would be required far less often for long-term monitoring.

At Empingham, the absolute horizontal position of the network stations has been established to within 3 mm. This is better than anticipated, and remarkable considering that the network covers an area of about 6 km^2.

Reference :

Ashkenazi, V (1975) *Deformation Measurement at the Empingham Dam Site.* Int. Symp. on Deformation Measurements by Geodetic Methods, Cracow, Poland.



Dr S GIUDICI (Hydro-Electric Commission, Tasmania) :

I came to listen, but I have been emboldened to say something because Mr Curtis is doing the kind of work that we as a Commission are also doing.

I found the papers on the instrumentation side very interesting, firstly because I detect a certain diffidence about talking of instrumentation costs. I am reminded of an English proverb that 'you cannot make a silk purse out of a sow's ear'. If you want good results, I think you should be prepared to pay for them.

There are certain points that I want to emphasise about instrumentation in general. We have in Tasmania something like 25 dams over 50 m height and we investigate, design and construct them all within one organisation. We have a fair amount of experience in instrumentation and fair experience of observing large dams.

We have come to the conclusion that one has to have an underlying philosophy regarding what one wants the instrumentation to achieve. If you want the instrumentation to check design assumptions, or to have the intellectual satisfaction of seeing that the dam moves roughly as calculated, that should prescribe one type of instrumentation. If you want instrumentation from the point of view of observing over a long period to assess the safety of the structure and to make deductions about its operation, that should prescribe a different type of instrumentation. Obviously we want a bit of both, but generally speaking I am talking about strain measuring instruments or deformation measuring instruments for concrete dams.

Strain occurs in structures because of all sorts of things. We have heard of thermal strains; there is also creep, autogenous growth and other factors, and sometimes even strains because of stress. Now if you are going to deduce stress from strain readings you have to have a lot of money, because it means that you have got to do a lengthy exercise establishing the properties of real concrete, not screened 50 mm material when you have 250 mm aggregate in the dam. To do that thoroughly you need to do a lot of tests over a long period of years. The United States Bureau of Reclamation has done this at Flaming Gorge Dam, but their publications indicate that costs are of the order of millions of dollars to carry out the task properly.

We are therefore left with the deformation type of instrumentation if you want something quick, something believable, and something which is not subject to a lot of deduction. I am talking of course about the pendula that have been mentioned. These, to my mind, are the cheapest type of instrumentation in terms of value for money. You can get readings of the order of 0.02 mm without any difficulty, and if they indicate that the dam has moved 10 mm it is hard to ascribe that movement to error of observation or to any other kind of supposition.

One other point I would like to make; we have been talking about instrumenting the dam itself. Failures of concrete dams are associated with failures in the abutments,and I haven't heard mention of instrumentation of abutments other than by piezometers. Piezometers are necessary and drainage and measurement of drainage is also necessary, but it is very simple to measure movements in the abutments using rock compression meters if you install these at time of construction. I am talking here about drilling a hole perhaps 20 m to 25 m into the abutment, anchoring a rod at one end of it and anchoring a measuring device in the dam. Usually one can use drainage galleries for the read-out point. You enter the rod, one end is fixed and one end is movable, and by moving the latter you can thus get very accurate readings. It is a very cheap system and it does tell you what is happening to the valley, especially when impounding. We have measured these abutment movements as the water level has risen in the valley, most recently for a 150 m high arch dam, and one can literally see the valley move, taking up its deformed shape. One can compare it, in fact, to what one calculated and, more importantly, you can establish a cyclic pattern as the years go by. I would refer back to Mr Curtis for this, I think the most important thing is the cyclic pattern. Let the dam tell you what it wishes to tell you rather than impose on it something that you might, with very crude assumptions, have calculated at the design stage.

Mr Curtis also talked about the reduction of results, and I am entirely in favour of what he is doing. I would refer you to a lot of Italian papers on this question which indicate, over a period of 15 years, just what can be done with a statistical approach to instrumentation. We have followed this approach with some success in the sense that early warning systems are automatically set up by studying repeatability of results.

Regarding telemetry, mentioned by Mr Parkman in Paper 2.8, we have recently investigated telemetering of results. Several speakers have said that to observe dams for years will cost a lot of money. It does not take much to show that if you can get a telemetry channel installed the cost reduces significantly. Telemetry channels are usually available from the power stations back to Head Office, and generally it is only a question of getting wiring from the dam to the power station and then borrowing the telemetry channel that is usually provided to monitor other sorts of instruments.

We have found the expense of monitoring dams is 5 to 10 times the initial cost of the instrument. That is a rough figure, and it may only be valid in our conditions, but it seems to me that if you are going to put in any instrumentation at all you are bound to observe it well. If you do not do that, you might as well rationalise in the beginning and not put in the instrumentation.

I would support any thought of a 'Code of Practice' on this subject, because there must be many piles of paper around containing instrumentation readings for some dams over many years. It seems to me that there is a better use that one could make of that paper rather than let records go yellow in some cupboard.

R G SHARP (Severn-Trent Water Authority) :

I am taking it that this particular Session seems to have widened from the general title of the Symposium, in that it includes operation of reservoirs as distinct from operation of dams, and it is on this particular point that I wish to make a few remarks.

The complexity of operation of reservoirs has increased greatly in recent years and has strengthened the need for rule curves for effective, efficient and reliable operation. This has come about for a variety of reasons. Firstly, the increasing use of water supply reservoirs for regulation of rivers as distinct from direct supply, and this of course has led in turn to a much more variable release pattern for the water. Secondly, there are distinct possibilities, where reservoirs are still used for direct supply, for compensation water patterns to be varied. This is particularly the case where reservoirs balance over more than one season and where winter releases can be cut back and that water banked in a winter between two consecutive summers, thus making more water available for higher releases during those summers and gaining some degree of regulation as well as the main purpose of direct supply.

There is also the question of multi-source operation where there are two or more reservoirs regulating a particular river system, and we in Severn-Trent Water Authority have this problem starting now and probably looming larger in the future. The matter gets more interesting when one has reservoirs of widely different characteristics. It may well be that one regulating reservoir is filled by gravity, and fills every season so that it is single-season critical. It may well be that another is pump-filled, balancing over more than one year, and one has to resolve the conflicting needs of economic and environmental considerations. This conflict occurs in many considerations and it does so particularly here in reservoir operations. From purely economic motives the right thing may well be to draw heavily and use as the front-runner the cheaply and quickly filled single-season reservoir, but this may mean that the extent to which the reservoir is drawn down may be unacceptable, particularly for the people who are sailing on it or fishing. Where do you draw that line? At a price one could possibly keep that variation of levels down, but it would then mean a greater use of the more expensively filled reservoir.

There are also considerations of variable yield of a reservoir rather than sticking to the idea of a fixed reliable yield, and having variable yield in relation to storage at a given date in a particular year. In other words, if one has all the water needed to last for the rest of that season, draw more from it. If one is going down to very low levels towards a difficult time of the year one has to cut back. It could be done in discrete steps or it might be almost continuously variable. This seems to be a very fruitful field for operations and rule curves.

Finally there is the question of the addition of further catchment areas or of additional pumping that can very much alter the characteristics, and thus the operation, of a reservoir. It may even change it from a two-season reservoir — we have this very situation with the Derwent Valley reservoirs in our area, where the addition of further catchments has changed from a two-season to a single-season critical situation, with a need for reassessment of yield and operating characteristics.

J D HUMPHREYS (Mander, Raikes and Marshall) :

S To me, one of the most fascinating and rewarding contributions we have listened to was that by Mr Adewole on the Kainji Dam in Nigeria. Let me just say that there are at least three other people here today qualified to comment on Kainji Dam, but I personally was responsible for the design of the rockfill and earthfill sections of the dam. I think most people would agree that it is fascinating to hear from somebody who is trying to operate a scheme one was involved with and is thus in a position to comment on the durability and performance of our instrumentation, the more particularly if he has questions over the performance of the dam and is honest enough to say that he is not a civil engineer. I think that our hearts go out to somebody in that position, and I congratulate Mr Adewole for the way in which he has expressed himself.

I shall quickly outline the nature of the rockfill dam and the relevance of the principal instrumentation. Kainji is a very long dam, the main structure being about 4200 m long and with a separate saddle dam 4400 m long. The main dam consists of a 550 m long concrete section accommodating intakes, spillway, generating equipment etc. and flanked by two rockfill and earthfill embankments of 2440 m and 1220 m length, which particularly concerned me.

The cross section at the highest point of the dam was roughly 80 m in height and, from memory, the embankment shoulders were of rockfill with a zoned internal core. I would suggest that the pore pressures that now matter most would likely be those associated with drawdown.

I am not attempting to answer the question on safety in a few minutes, but I am amazed at the implication that the management at Kainji do not have the operating manuals and the manuals concerned with instrumentation that were meticulously prepared, largely under Mr Coxon. I would have thought that if one could have claimed anything about this scheme it was that the whole concept and design were very thoroughly written up indeed, and a thorough instrumentation manual was prepared and handed over to the Authorities. I think that the lesson that we all have to draw from the contribution by Mr Adewole is that this forms a vital part of the documentation of any new scheme, and of course does require careful attention. It should be demanded by the client when the 'as constructed' drawings are handed over.

Finally, he was modest enough to refer to what he called the developing countries. The problem of building up a technology which is adequate to cope with problems of handling instruments and so forth is, we can assure him, not only a problem in his country but certainly also applies here.

R E COXON (Engineering and Power Development Consultants Ltd.) :

Having, like Mr Humphreys, been involved with Kainji Dam I, too, am grateful to Mr Adewole. We have got a problem, or at least a question, to discuss later on.

Mr Adewole has highlighted the problem of communication, because in fact there was a very detailed operating manual sent out as Mr Humphreys has said — it included both the concrete and the fill dams. I do not remember what the period of reading was, but certainly charts were prepared against which the measurements could be taken. I think that if Mr Adewole now checks his records he will find that there was a full inspection of the dams - taking all those readings into account - by the Canadians a year or so ago, and they said the dam was acting entirely in accordance with the design concept as set out in the manual. I think this is a further problem of communication in that these documents have to be in a form that can be readily assimilated by people. The volumes represent a great tome and perhaps that is their failing. There is so much detail that it is not readily assimilated in the context of the rather rapid changes of staff taking place in developing countries, when people go from one senior position to a more senior position.

C F GRØNER (President, ICOLD; Chr F Groner A/S, Norway) :

I believe that in one of the Italian arch dams they once had so many instruments installed that they had to use 15 civil engineers to study them over a long period. I am concerned that those points which have been mentioned about instrumentation today must be thought about very carefully, or instrumentation may prove too expensive.

In Norway we have about 150 large concrete dams. We have two arch dams which are instrumented. At a lot of the other dams we have only used pendula for a short time and then left the dams to themselves. I am afraid that if we have numbers of instruments installed to be read every half year or so it will cost too much. This is a point I think we should consider.

If, of course, you have a difficult dam then you have to do what you think is right, but if you have a 'normal' dam you have probably to think in a different way. It is popular today to use instrumentation but that is no reason to use it when it is not otherwise necessary. In embankment dams especially there are frequently a lot of instruments. I know that in one of the big embankment dams in Norway they put in 250 measurement points, and I wonder how long they can continue to read these. If it is only for a short time, for first filling or second filling, that is acceptable, but we cannot continue reading indefinitely. This is a matter for careful consideration.

F G JOHNSON (North of Scotland Hydro-Electric Board) :

I think there has been some very good sense talked today, particularly by Mr Adewole and Dr Giudici. I feel they are very much in line with the policy which we have been following in the Hydro-Electric Board.

Some four years ago we found that we had rooms full of instrument records, which were continuing to accumulate. In facing the position we asked ourselves what value are these? How long do we keep taking them? What happens when instruments become defective or fail — do we replace them? What is our policy? With this in mind we discussed the situation very carefully within the Civil Division, with our Generation Engineers, who are responsible for stations or groups of stations and dams, and also with the Panel Engineers who inspect our dams. Over a period of two or three years, after every Statutory

Inspection we asked our Panel Engineers what instrumentation they thought was necessary, how often should it be read and how should the results be interpreted. This led to a change of emphasis in the whole attitude to instrumentation.

The results of our discussions and deliberations were quite clear. The desirable ultimate objective is to instrument all dams of significance with reliable and effective instruments, and to maintain all necessary instrumentation facilities in first-class order so that they may be commissioned immediately as and when required. Exceptions to this objective are the fair number of dams which control existing lochs with only a low head and a very small range of water level variation. As such, their structures are simple and do not require instrumentation.

The reading of instruments and the assessment of results can be very heavy in skilled labour, particularly where the dams are remote and situated in areas of inclement climate such as the Highlands. We therefore believe that dams should not be instrumented unless the need can be fully substantiated. In implementing this policy we carried out a very careful scrutiny of each of the Board dams to select those dams where instrumentation was required and would be of value in monitoring safety. New dams should be fully instrumented as part of the construction procedure and regular readings taken over a sufficiently long period of time to gain confidence in the dam's behaviour. We consider that, in general, a period of around five years is about right. Thereafter, if the instrument readings are consistent and the dam behaviour predictable, as Mr Curtis has shown with a number of the slides he showed, the number of instrument readings can be reduced and the period between each set of readings increased. Typically, we have reduced the readings from say an annual cycle to, say, a cycle every three to five years with, desirably, a cycle phased just before the next Statutory Inspection so that the Panel Engineer can scrutinise the behaviour immediately before his inspection.

We have increased the number of dams on which we have instrumentation so that we cannot be accused of not instrumenting all the important dams, although this task is not quite completed. The instrumentation installed is robust, simple and can be replaced, and it is kept in first-class working order. If we have a major incident, such as a major flood or seismic movement, we can immediately take readings again and check the dam's performance and safety.

Lastly, I would say that this change of emphasis has been coupled with overseeing inspections undertaken by our senior and experienced engineers in the Division, as I described this morning. The instrumentation readings are also carefully scrutinised, but we have found these analyses to be nowhere near as valuable to us as the results of the overseeing inspections. Our effort has not increased in the Division; we have still the same staff we previously had, but we believe we are getting more value from our instrumentation and certainly a lot more value out of our engineers.

J W SEDDON (Severn-Trent Water Authority) :

I have found this a most interesting Session, but it does reinforce my view that we are very quickly losing the tremendous amount of expertise of those who have been responsible for : (a) building; and (b) maintaining and looking after reservoirs throughout the years. One finds that with more rapid methods of building dams you do not get the engineers who worked on the job continuing on as they used to, nor do you get the reservoir keeper or the inspector who had worked on the job and knows exactly what to look for.

It is not easy to put a man onto looking after a reservoir and to tell him exactly what to look for to indicate when things could possibly go wrong. I could give several instances from my own experience, one in particular where a reservoir keeper, through his observation and his own know-how, really did avert what could have been a catastrophe. I know of others where reservoir keepers have told their masters what they thought to be dangerous and nothing has been done about it until an inspection has been made, perhaps five or ten years after.

I think we really do now need to consider carefully who we appoint to the job of looking after reservoirs, particularly as Supervising Engineer. It will not, in my view, be enough to merely put a man there because he happens to be senior, or to give him the job as a status symbol. It must be somebody who is properly trained and knows what he is doing and how to look after reservoirs and he must, of course, be able to supervise. The man appointed must be able to transmit the knowledge he has to the day-to-day personnel, because the whole business really comes down to the man on the spot.

I was very interested to hear Professor Isaac say this morning that his Department is, through Mr Moffat, initiating an MSc course in Reservoir Engineering. I think that represents a step in the right direction.

K A SANKEY (North West Water Authority) :

Following up the last speaker and something that the General Reporter said as regards monitoring the performance of a dam and comparing with the calculated behaviour, I would think that if the Supervising Engineers are appointed at the right time and are the right individuals these sort of people could be a vital link between the Construction Engineer and the Inspecting Engineer.

GENERAL REPORTER : J P WILLIAMSON (Concluding summary) :

Before placing instruments in any dam it is essential to decide the purpose for which the readings will be used, the degree of accuracy required and the length of time that the instruments are to be satisfactorily operated. It can sometimes be more helpful to know a little about a lot rather than a lot about a little. In determining the scheme of instrumentation to be adopted the cost must be taken into account, but if instrumentation is desirable this cost should be faced. Further, if a decision has been taken to insert certain instruments in a dam and if one of these later fails to operate properly it should be replaced if at all possible, otherwise the original decision to insert the instrument must have been superflous.

There was general agreement that exchange of data was most important so that engineers in charge of dams would have an indication of the limits within which variation from the normal readings of a particular parameter could be accepted. It was also considered that the design engineer should specify the limits within which readings could safely fluctuate. The possibility of a data bank with information on the behaviour of dams was suggested, and particular reference was made to information on dams which behaved in an abnormal way.

It was considered important that the design engineer should interpret the results of readings and that the first five years of a reservoir's life were probably the most important as far as those readings were concerned. The point was made that caution should be exercised in assuming that maximum readings recorded were in fact the maximum which had occurred, since in most cases continuous recording of readings is not practiced. The semi-automatic recording of movements of an inverted pendulum by means of a special type of camera was, however, referred to.

The need for ease of taking readings was stressed, and in this connection confidence was expressed in the use of a laser beam to determine horizontal movement of the crest of a concrete dam.

A further interesting contribution made mention of rock compression meters with particular reference to instrumentation of the abutments of arch dams. The use of telemetry channels to reduce the cost of reading instruments was also suggested.

One speaker did wonder whether the money spent on instrumentation would not be better used by increasing the factor of safety of the dam itself so that instrumentation was rendered less necessary, or could be substantially reduced in extent.

The Discussion also stressed the need for all members of staff involved in the operation, maintenance and instrumentation of reservoirs to be fully aware of their respective responsibilities and of their position in the overall scheme of responsibility. The value of having reservoir keepers on site who had a clear knowledge of the points which they should examine regularly and report on was properly stressed.

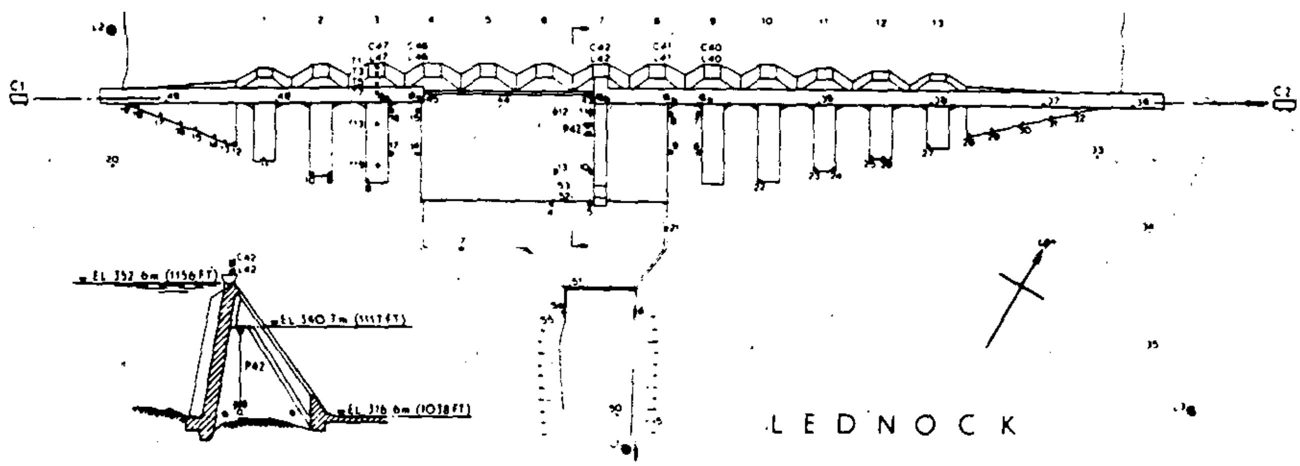
The value of rule curves in the successful operation of reservoirs was also emphasised. This applies whether the reservoir is being used for water supplies, power generation or river regulation, and is particularly important when the reservoir forms part of an integrated system. Two speakers made mention of the maintenance of reservoir levels for amenity purposes, but pointed out that this could usually only be obtained at a price.

WRITTEN CONTRIBUTIONS

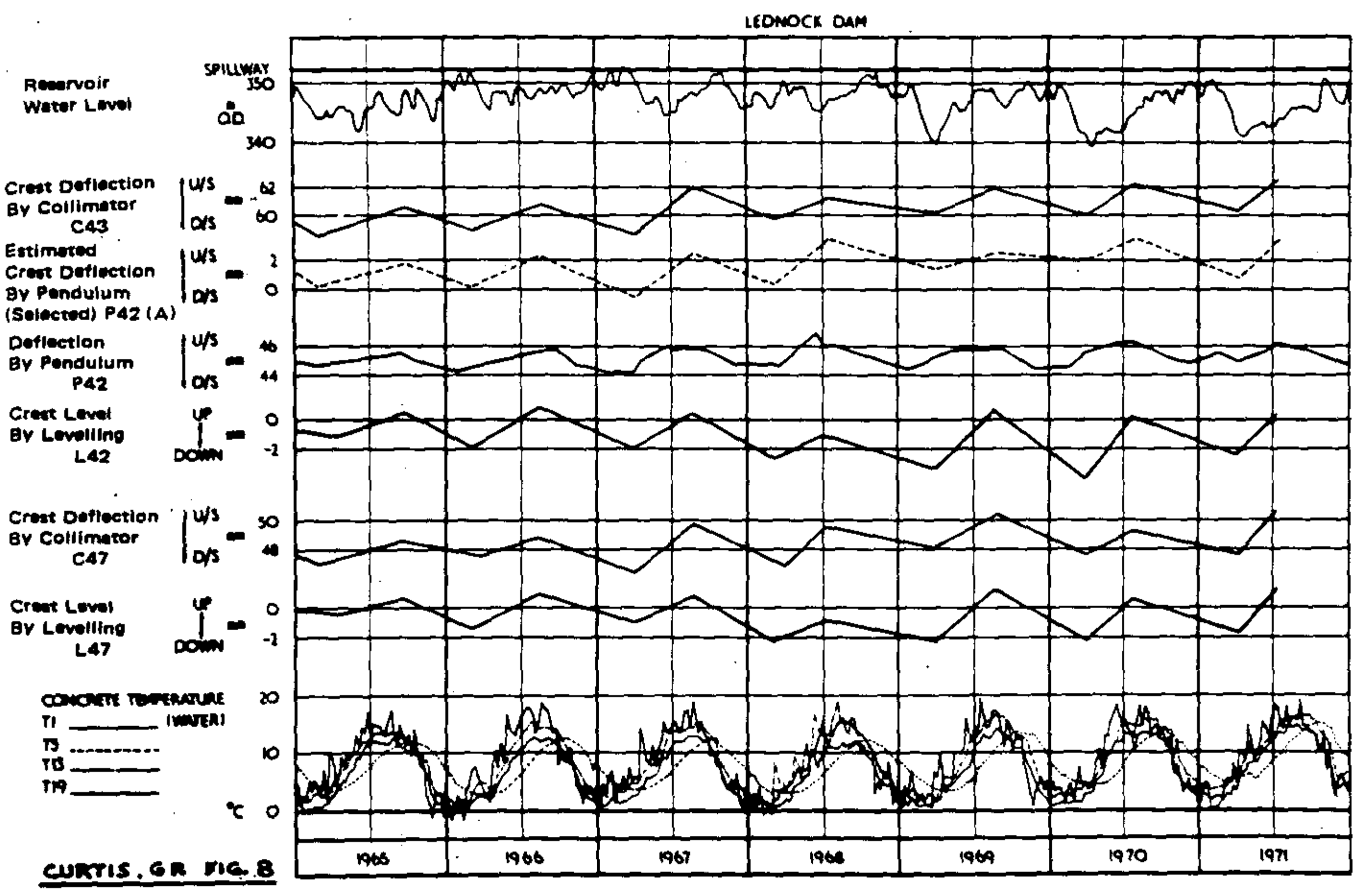
G R CURTIS (North of Scotland Hydro-Electric Board) :

In amplification of Paper 2.5 and subsequent contributions, the instrumentation of Lednock Dam (Fig.A) was designed and installed in conjunction with the Board's Consulting Engineers in 1964. It illustrates a comprehensive arrangement, overlapping in Buttress 7 and extending to three bench marks some distance from the dam.

The movements are shown in Fig. B to be consistently cyclic in response to seasonal temperatures. Curve P 42 shows the movement in Buttress 42 as recorded by the pendulum. The dotted Curve P 42 (A) gives the movement of the crest at this Buttress as estimated from the pendulum, and it can be seen that there is close agreement with Curve C 43 using the collimator. The method of preparing Curve P 42 (A) is similar to that used for Curve P 6 (A) at Sloy Dam (see Paper 2.5), except that the movement given by the selected pendulum readings has been multiplied by an arbitrary factor of two as compensation for that part of the height of the buttress not included in the span of the pendulum.



CURTIS, G R FIG. A



CURTIS, G R FIG. 8

K A SANKEY and C MILLIGAN (North West Water Authority):

Mr Rofe, in his comments, pointed out that when listing the factors considered in producing the rule curve the hydrological study was first. The Authors would agree that this was intentional, the starting point must be the hydrological study. However, the physical factors then have to be considered when interpreting the rule curves. Equal credence will not be given to these factors, and the emphasis will probably vary from time to time as circumstances change.

Mr Cooley, in Paper 2.1, laid great emphasis on water quality factors. These are important, but in the Lake District supplies quality tends to be very similar and changes seasonal, so that all supplies suffer in a similar manner. This is therefore seldom a major factor when considering operation.

Paper 2.4 tried to show that the production of rule curves is a desirable aid to the operation of complex resource systems. These curves must be used intelligently, however, and they must be revised frequently to meet the ever-changing circumstances. As the Manchester sources approach a drought situation the aqueduct flows invariably increase, because other areas drawing supplies from the aqueduct are already nearer the drought condition and need additional water to replace that no longer available from their own sources. Hence the system can only be operated within the broader framework of the North West Water Authority because, under such circumstances, the interests of the Eastern Division could be secondary to those of the region as a whole.

R E COXON (Engineering and Power Development Consultants Ltd.):

The discussion on Kainji stirred some further thoughts relevant to Technical Session 2.

The fill section of Kainji was nearly eight km long, including the long low embankment on the left bank which crossed a saddle. At this section the retained water depth is less than 10 m, but the foundation was such that special measures were taken which included a blanket upstream and some drains on the downstream side. The design for this short length was formulated on site, and because of availability of equipment the drains were simple and of relatively small diameter. A detailed operations manual covered their inspection, maintenance and, if necessary, replacement.

At the time of a routine inspection of this section what was thought to be an upstream deformation of the embankment was noted. Further enquiries revealed this to be excess fill placed during construction and not ordered to be trimmed by the soils engineer on the basis that the additional material was a useful bonus. Subsequently, some recent concern on the reasons for a particular grading of riprap on the low embankment has been simply explained by construction methods which in part involved dozing material up the slope from the toe.

A further point relates to a washout of part of the toe of the dam on the right bank during first filling of the reservoir. For the usual kind of reasons, which seem ludicrous with hindsight, a longitudinal cofferdam was not breached before flooding and when it overtopped the resulting wave scoured along the upstream face of the dam. I do not recall the amount of material involved but it was substantial and had to be replaced from a pontoon. Although the placement could be reasonably controlled, even under water, the section was overfilled to some degree but not accurately surveyed on completion.

The reason for recounting these incidents is that as far as I am aware none of the relatively minor variations were shown on 'as-built' drawings. Certainly the first was not and only came to light on contacting the individual concerned with supervision. On concrete dams it is relatively easy and a standard routine to record 'as-built' details, whereas I suspect that such is not always the case on embankment dams where minor changes are not in themselves of apparent significance at the time. They can, however, be very confusing to an Inspecting Engineer at a later date. The problem will become greater as we develop more difficult sites, in particular those where foundation conditions are complex and especially on long embankments where modifications are carried out as the work proceeds.

I must add my doubts to those raised by earlier speakers on whether the appointment of Supervising Engineers will obviate this particular problem, even if they do participate in the construction. In any case staff turnover will have to be considered and this, as I said previously, has probably been a contributory factor to communication shortcomings at Kainji.

I suggest, therefore, that the Construction Engineer should as a matter of routine be called back to contribute to the early inspection of the dam, whether it be to attend on the Inspecting Engineer and provide back-up knowledge as required or to comment formally on the Inspection Report.

Dr K H M ALI (University of Liverpool) :

Mr Cooley has presented a most interesting and useful Paper (2.1). The writer is well aware of the comprehensive model tests carried out by the Thames Water Authority in connection with the Datchet and Wraysbury reservoirs, where water-jets of various configurations and inclinations were used successfully to improve the mixing.

The writer is involved in a similar experimental study of mixing in service reservoirs. Doubtless water-jets can be very efficient for mixing the contents of reservoirs, if pumping costs are not excessive. Air-jets, however, can be even better mixing agents. Aerators may be very economical and usually require very little maintenance. Their positions can be easily changed, and, unlike water-jets, they do not depend on the reservoir inflows for their performance. The writer does not know of any Water Authority in the United Kingdom using aerators for improving the mixing of their reservoirs, but aerators have been so used in the United States and in the Netherlands.

A floating aerator was installed at the Ossining reservoir in New York State in 1956[1]. A 6 kw blower, connected to the aerator, was sufficient to circulate the entire volume of water (360 tcm) at the rate of 690 tcm/day. The aerator very quickly eliminated reservoir stratification.

At Biesbosch in the Netherlands, the 'Honderd en Dertig' and Petrusplaat reservoirs (two of four reservoirs planned) are in operation with air-mixing systems. Details of the impressive performance of the systems have been published [2]. The operators found that the systems need only be used from April to October since, during the rest of the year, stratification was not a problem. They also found that the costs of mixing were very low.

Owing to the complexity of reservoir circulation, especially where more than one inlet is used or where the reservoir geometry is complicated, it is desirable to compare model and full-scale results. It would be of interest to know if the Thames Water Authority conducted any tests on the actual reservoirs in order to check the circulation patterns predicted by the models and whether consideration was given to wind directions and magnitudes when the locations of inlets and outlets were being decided.

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J D EVANS (South-West Water Authority) :

I would like to comment on the role of the Supervising Engineer in relation to instrumentation installed during construction and subsequently required to be read by the Undertaker.

One often finds that the instruments are forgotten after the Resident Engineer's staff have left or that large numbers of readings are taken regularly and filed away, without comment, never to be seen again. I think that it is here that the role of the new Supervising Engineer lies. He should be in the best position to oversee the taking of readings, the arrangements for them to be plotted, and then to give initial consideration to the results. He must, of course, have had guidance from the Construction Engineer on the extent of the readings required, the relative importance of the various instruments and the limits outside which the readings will indicate a warning that the structure is not behaving as predicted. His knowledge will then become invaluable and form the link with the Inspecting Engineer. Thus it is essential that the Supervising Engineer is able to devote sufficient time to his task so that information is properly studied and the implications fully understood.

H C PARKMAN (Ward, Ashcroft and Parkman) :

I endorse the view expressed by several speakers that, in the cases where extensive instrumentation is necessary, adequate arrangements must be made for the special skills necessary in taking subsequent readings of the gauges, in compilation of the results and in interpretation of the trends.

In the case of the Llysyfran Scheme described in my Paper 2.8, we were fortunate enough to obtain the agreement of our clients that we should be commissioned to instruct staff in the reading of the various instruments, to monitor those readings, and to analyse and where necessary compute them into stresses. We were commissioned to do this for an initial period of three years, giving an annual report to the client on this instrumentation. This has the distinct benefit of ensuring continuity of the instrument readings from the construction stage and covering the first three years or so of operation.

With electrical type instrumentation it is important to take the greatest care, in their original selection to ensure robustness, and also in their subsequent installation. Only by aiming at 100% performance is one likely to achieve any worthwhile results. To install this type of instrument on the grounds of economy for a limited period only could well mean little or no value from embarking on such a scheme.

It is my opinion that many expensive operations and repairs to structures in the past could have been avoided if, firstly, our forefathers had committed to paper more details of the finished structures and, particularly, more details of the supporting strata, and secondly, they had allowed the designer to continue to advise on the performance in operation of the dam.

May I therefore suggest that BNCOLD may wish to recommend the advisability of a continuing consultation role for the Construction Engineer?

A I B MOFFAT (University of Newcastle upon Tyne) :

Reference has been made by the Reporter to the concept of a Code of Practice covering various aspects of instrumentation, while contributors to the Discussion have drawn attention to the quantities of data generated by existing instrument arrays.

With respect to the first point, the danger of a Code of Practice is that it is often interpreted as being mandatory rather than advisory. Certain publications refer to levels of instrumentation, but they do not adequately fulfill the requirement outlined by Mr Williamson. It is suggested that what is required is a text-cum-guide, biased towards British circumstances and based on the considerable experience already available in this country.

On the question of properly utilising the enormous quantities of data generated by instruments, I would make the point that the potential research return is frequently neglected. This is to a degree understandable in that the consulting engineers or others responsible for the instruments are primarily interested in ensuring that structural performance of the dam is satisfactory. Our understanding of the field behaviour of materials and of the structural behaviour of dams could, however, benefit greatly from a distillation of available instrument records. In the absence of any national provision for research projects of this nature the Universities are in a position to make a valuable contribution, given the necessary backing, financial and otherwise. The results would be greatly to the long term benefit of designer and operator alike.

R M ARAH (Binnie and Partners) :

At first sight there is a difference between instrumentation for the purpose of checking design parameters and assumptions and instrumentation for monitoring the safety of the dam, with the inference that the former can reasonably be abandoned once the initial behaviour of the structure has been analysed. However, both purposes have the same need to feed data from instrumentation into an abstract model of the structure in order to predict future behaviour; bearing in mind the rate of development and refinement of such abstract models in relation to the life of a dam it seems unwise to abandon any source of data unnecessarily. It is also difficult to forecast exactly what information from the past may be of help in interpreting the symptons when specific troubles appear to be developing.

Mention was made of the use of half-second theodolites to increase the sensitivity of deflection measurements. As repetition of readings presents no problem it should be possible to produce the required accuracy on a statistical basis by averaging enough repetitions, provided the instrument has facilities for the usual correction of built-in errors.

R V ASH (South West Water Authority) :

Referring to the question of the value of data from instruments against the cost of installation raised by Mr Johnston, from my experience there appears to be more relevance in the installation of instruments in an embankment dam for the purposes outlined in Mr Rofe's Paper 2.9, to which I would add the significance of using instrument data, particularly pore pressure, in the control of construction progress and, subsequently, the control of initial filling.

Referring to the partial disconnection of instruments at Covenham Reservoir described in Mr Rofe's paper, the piezometers which were disconnected were terminated in manhole chambers in such a way that each piezometer could be re-established temporarily whenever required by connecting to a portable or temporary manometer. The warning marker system of reading the remaining piezometers at Covenham simplified the monitoring and obviated the need of a skilled observer. However, it is necessary to periodically de-air the instruments and follow with accurate readings of the manometers to ensure the restoration of proper working order.

As Mr Moffat was able to show that there is a greater risk of failure of an earth dam after 50 years and that most recorded failures have been with earth dams, it is evident that these require more surveillance. In my opinion, the installation of piezometers and inclinometers would be of great advantage in this surveillance, particularly in embankments with measurable seepage and reservoirs where the operating regime has changed to greater level fluctuations.

PROCEEDINGS : TECHNICAL SESSION 3

PROBLEMS OF MASSIVE DAMS AND REMEDIAL MEASURES

PAPERS :

DISCUSSION:

WRITTEN CONTRIBUTIONS:

PORE PRESSURE AND INTERNAL UPLIFT IN MASSIVE CONCRETE DAMS

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SYNOPSIS

The paper presents a brief review of the nature and significance of pore pressure and internal uplift as a primary load to be accounted for in the design of massive dams. The requirement for more quantitative data on pore pressure is emphasised, and a summary of the development under CIRIA contract of a suitable piezometer is presented. An account of the trial installation of the *piezometers* in Bradan Dam is given, together with the pressure profiles gained from two years' observations. Preliminary conclusions drawn from the proving trials are presented, together with references to further work required.

INTRODUCTION

Seepage of retained water through or under a dam under the influence of hydrostatic pressure gives rise to pore pressures and thus to a resultant uplift force. This uplift force is the least determinate of primary loads to be accounted for in design and analysis.

Internal uplift may be defined as 'the force system acting upon any horizontal plane within a massive dam which is attributable to the pressure of seepage water contained in joints, fissures or pores in the parent material' (1). Internal uplift must be clearly differentiated from the more commonly discussed and more familiar foundation uplift, arising from a similar process of seepage through underlying strata or along the dam/rock interface. Foundation uplift will not be discussed further as considerably more published information exists as to its magnitude and significance.

The definition of internal uplift quoted belies the complex nature of the problem, a complexity arising from the nature and behaviour of concrete on the microscale and, as mass concrete, on the macroscale. For this reason considerable debate has centred upon the entire question of uplift since interest was first aroused by the publication of Levy's work (2) following failure of the Bouzey Dam in France in 1895. The complexity of the uplift problem is also masked by the deceptive simplicity of the equation defining uplift force, u, in terms of its two component parameters:

P	: intensity of pore pressure
dA' or A'	: effective area of an element or section of superficial area dA or A

Resultant uplift force, u, on a horizontal element may be defined thus:

$$u = p.dA' \qquad \text{.......... (1)}$$

where $dA' = \eta dA$, i.e. dA' is the proportion of superficial area dA over which pressure can be considered to act η being an appropriate reduction coefficient.

Equation (1) may thus be rewritten as:

$$u = \eta dA.p \qquad \text{.......... (2)}$$

In relation to an entire horizontal section, the resultant internal uplift force, U, on the superficial section area, A, is expressed as :

$$U = \eta \Sigma dA.p = \eta \int p \frac{dA}{A} \qquad \text{.......... (3)}$$

In practical terms, evaluating U for purposes of checking overturning and sliding stability in conventional analysis of massive dams, equation (3) may be amended to:

$$U = m A . P_{avg} \quad \ldots\ldots\ldots\ldots\ldots (4)$$

Pore pressure intensity, p, should also figure in stress analysis, an excellent exposition of the appropriate technique being attributable to Zienkiewicz (3).

As uplift arises from a process involving seepage of a fluid through porous media, however, it must be appreciated that uplift pressure is merely the vertical component of pore pressure. A horizontal component also exists, i.e. the hydrostatic load commonly considered as a surface load is, strictly speaking, an internal body force.

A further point which must be appreciated is that the seepage process giving rise to uplift is not necessarily apparent on visual inspection of a dam, as ambient conditions normally ensure immediate evaporation of seepage water except in the case of the most obvious defects, e.g. poor lift joints.

NATURE OF INTERNAL UPLIFT

Understanding of the nature of uplift has gone through distinct phases, an exhaustive account of which has been published by Leliavsky (4). The various phases are also reviewed and discussed elsewhere (5).

Initially conceived as arising from hydrostatic forces due to water in open fissures cracks or defects in otherwise 'impermeable' concrete or masonry, it was only later appreciated that uplift was an induced phenomenon arising also from seepage through intact masonry or concrete. Consideration of uplift was then centred upon assigning a suitable value to m, the area reduction coefficient, and a considerable amount of experimental effort was expended upon investigation of this factor by Terzaghi, Leliavsky, McHenry, Serafim and others. Experimental evidence pointed to the value of m approaching unity. The key to understanding $m = 1.00$ was supplied rather earlier by Terzaghi, who envisaged concrete as a porous material analogous in structure to an idealised soil and drew the obvious parallel between pore pressures and effective stress in soils and in concrete. Terzaghi's conception of an idealised microstructure for concrete illustrating that $m \rightarrow 1.00$ is shown in Figure 1.

It is now generally accepted in most quarters that $m = 1.00$ is a realistic assumption for analytical purposes.

Advances in the study of the structure of intact concrete led to further developments in the appreciation of pore pressure and thus of the concept of internal uplift. The micropores in intact concrete are sufficiently fine to give rise to surface force effects such that negative pore pressures may prevail. It has also been postulated that the permeability of intact concrete in a 'perfect' dam, with no joints or fissures, would be so low that saturation and the attainment of equilibrium pore pressure distribution would only be achieved on a time-scale measured in centuries (6).

In practice, however, it is suggested that the influence of microcracks, joints and similar inevitable 'defects' is predominant in dictating permeability and therefore time to reach equilibrium pore pressure distribution. Multi-directional access of seepage water via such 'defects' will expose otherwise intact concrete to positive pressure and will result in earlier penetration into pores in the intact concrete. An analogy can thus be drawn between mass concrete in a dam and a saturated rock mass. Equilibrium pore pressure distribution will be a time-dependent function of 'defect' frequency and pattern and of micropermeability. Pore pressure at any time is therefore indeterminate except by actual measurement in the field.

A summary of the position with regard to formulating design criteria for internal uplift is thus that the area reduction coefficient should be taken as unity in equation (4). This is by no means universal in practice. Assumptions regarding pore pressure distribution are in most cases simplified versions of distributions determined from flownet studies of seepage through a homogeneous and ideal dam. Design criteria applied in practice are notable only for their variability and, in some cases, the questionable underlying philosophy.

As regards control of pore pressure and uplift, the benefits of an adequate relief drain system have long been known. A properly designed and detailed relief system, capable of access for reaming-out or cleaning, is the only efficient and reliable method of containing uplift. (1)(5) Such a system can give rise to considerable economies in terms of adequate stability for the dam being provided by a more slender cross-sectional profile.

The situation has not been helped by the lack of reliable quantitative field data on uplift intensity upon which valid criteria could be founded. Early attempts to monitor pore pressure were of doubtful value due to the variety of technical problems encountered. In an attempt to clarify the position the University of Newcastle upon Tyne has been conducting a limited programme of research under the sponsorship of the Construction Industry Research and Information Association (CIRIA) since 1968.

A desk study and review of the uplift question has been completed (5) resulting in the publication of Technical Note 63 (1). In parallel with this work development of a piezometer suitable for installation in concrete was undertaken. As an interim field experiment, standard simple piezometers were installed in Loch Dubh Dam in conjunction with the North of Scotland Hydro-Electric Board in 1968. On completion of the development of a CIRIA piezometer, proving trials were initiated at Bradan Dam in 1973 with the cooperation of the then Ayrshire and Bute Water Board (now Strathclyde Regional Council) and their Consultants, Babtie, Shaw and Morton.

RESEARCH AT UNIVERSITY OF NEWCASTLE UPON TYNE

DEVELOPMENT OF THE CIRIA PIEZOMETER

A piezometer designed to be installed in existing or new construction dams must be sensitive, durable, relatively easy to install and to monitor, and be of reasonable unit cost. Consideration was initially given to an electrical pressure-cell type unit and also to a twin-tube hydraulic piezometer, and preliminary designs were prepared for both types of instrument. Advantages of the electrical type were, however, felt on balance to be outweighed by the inherent disadvantages, those of cost and durability in this instance. Development was therefore concentrated on a twin-tube hydraulic piezometer tip, based in principle upon the well proven Bishop type used successfully in earthfill embankments since about 1960. The principal features of the CIRIA piezometer are illustrated in Figure 2.

Twin hydraulic leads allow the piezometer tip to be flushed through with de-aired water to remove air bubbles which are liable to accumulate due to water displacement of air in the pores of the concrete as gradual saturation of the concrete takes place, or due to air coming out of solution from the seepage water under certain conditions.

The piezometer tip is as compact as is compatible with satisfactory operation, being designed for installation in a 30 mm dia borehole. Problems associated with differing pore air and pore water pressures are obviated by the use of a 'high air-entry value' porous ceramic element. Pore size of the ceramic is approximately 0.5 micron, resulting in an air-entry value of some 0.2 MN/m^2. Use of this type of element also reduces the need for frequent flushing with de-aired water. The body of the experimental piezometers and of the prototypes installed in Bradan Dam was manufactured in stainless steel to ensure long-term durability on exposure to acidic moorland waters.

On the assumption that the permeability of mature intact concrete is some 10^{-8} to 10^{-9} mm/s the 90% response time is approximately 24 hours. In mass concrete, however, response time is more of academic interest, and is in any event likely to be reduced by the influence of microfissures in the concrete.

The composite nylon 11/polythene hydraulic leads are led to a suitable location in an inspection gallery, the instrument being monitored either by a conventional mercury manometer system or by a portable pressure transducer, as was the case at Bradan Dam. The advantages of the latter technique, where the leads terminate in snap-couplings housed in one recess in the inspection gallery, include a more compact layout and reduced cost if more than a few piezometers are grouped in any one instrument cluster.

Experience at Bradan Dam and in the laboratory indicates that the most reliable method of installation, in existing dams or dams under construction, is to insert the piezometers into specially drilled boreholes. For the Bradan installation the necessary boreholes were drilled to depth into or through Lift 15 in the interval prior to pouring the next lift. The technique of encapsulating piezometers in concrete cylinders in the laboratory and subsequent embedment of those cylinders during concreting is not to be recommended.

Proper sealing of each piezometer element to isolate the point of measurement from the influence of seepage along the outside of the leads and/or down the borehole is absolutely critical. At Bradan Dam a proprietory sealing compound, 'Situseal', was used to form the primary seal in the borehole above the piezometer tip and proved successful, but extreme care is required to ensure total reliability of the seal.

PORE PRESSURES - LOCH DUBH DAM, ROSS-SHIRE

Loch Dubh Dam is a gravity structure some 20 m high located in exposed conditions at an altitude of some 180 m.a.o.d. The dam was constructed partly in conventional mass concrete and partly in colloidal concrete, and was completed in 1955. By 1968 surface deterioration was evident, particularly on the colloidal concrete monoliths, and was attributed in part to saturation from seepage. The latter condition was worsened by leaching action blocking an inadequate and inaccessable relief drain system.

A total of 16 Casagrande type piezometers were installed in boreholes in 1968, the installation being planned to provide two piezometers across each of seven sections, plus single piezometers installed in rock under the toe on each abutment. The piezometers were located at depths of up to 12 m below spill level and a proportion were located, insofar as could be ascertained, on the interface between two successive lifts. The inherent limitations of the Casagrande type unit were accepted in view of the *ad hoc* nature of the investigation and other relevant factors. The piezometers were monitored regularly by means of a dip-meter. Of the total of 18 piezometers, 14 functioned satisfactorily and representative uplift profiles are plotted on Figure 3.

The dry summer of 1969 and consequent low water levels permitted further research, in that a 'packer' rig was developed and employed to determine the macro-permeability of the mass concrete and the colloidal concrete. Representative values of coefficient of macropermeability of the concretes are tabulated on Figure 3.

In addition to yielding useful quantitative data on pore pressure and permeability the results of the Loch Dubh experiments proved to be of value in checking the stability of the dam, and also in designing and subsequently proving the efficacy of additional relief drains drilled into the structure at strategic locations.

PORE PRESSURES - BRADAN DAM, AYRSHIRE

As part of the development programme for the CIRIA piezometer it was planned that proving trials should be conducted in the field if the opportunity arose. Such opportunity was afforded in early 1972, when a total of 16 piezometers were installed in Block 23 of Bradan Dam during construction. Bradan Dam is a gravity structure some 30 m in height, and has been described by Johnston (7). Eight piezometers were installed in boreholes drilled to the Lift 15/Lift 16 interface, the remainder being located at mid-height of Lift 15. Layout of the piezometers, shown on Figure 4, was planned to yield as much information as possible by covering one area of the dam thoroughly rather than distribute available resources over a number of locations to less effect.

Hydraulic leads from the piezometers were laid in rebates sunk in the upper surface of Lift 15 and were brought to one vertical duct leading to a special monitoring chamber formed in the inspection gallery. Installation of the 16 piezometers complete, including drilling, occupied seven days and cost some £750.

As indicated earlier, the pressures were monitored on a portable pressure transducer connected to each lead in turn and calibrated directly in metres head of water. The twin leads were advantageous in providing a check reading for each instrument and thus indicating any need to de-air the piezometers. De-airing and flushing of the piezometers, when necessary, was carried out by a portable bladder-type de-airing unit.

The Bradan piezometers have been monitored regularly for some two years since first impounding, representative envelopes of maximum pore pressure being plotted on Figure 5. It was concluded, after some initial doubts, that all 16 piezometers are functioning, and long-term monitoring by Strathclyde Regional Council personnel is planned.

The value of results recorded to date has been somewhat lessened by virtue of the retained water level being almost constantly at, or within 1 m of, spillway level, some 17 m above the piezometer levels. It has therefore not been possible as yet to determine response to changing reservoir level.

Study of the results available to date is necessarily inconclusive. It is considered that readings over a period of five to seven years and including a complete drawdown cycle will be required before definitive conclusions can be drawn. In general terms, however, the following interim conclusions can be formulated on the data obtained to date :

1 No pore pressures of any magnitude have yet developed, and values lie well within normal design margins.

2 No significant difference is apparent in pore pressures recorded at a point on a lift interface and at the corresponding point in 'intact' concrete at mid-lift.

3 Initial pressure drop is high, but is then surprisingly constant across and through the section. The rapid initial drop is not considered to be attributable to the use of a richer concrete on the upstream face.

4 It is concluded that fissure pattern is influencing the results for certain piezometer locations.

5 The local influence of individual relief drains, contraction joints or other construction features is not yet apparent.

It is considered that the CIRIA piezometer has functioned in a satisfactory manner and that the instrument is suitable for installation in new or existing massive dams.

CONCLUDING REMARKS

The assignation of realistic values to pore pressure when evaluating uplift for analytical purposes is clearly desirable. When retrospectively checking the adequacy of an older dam, which may have been designed with no allowance for uplift or, conversely, with no allowance for the beneficial effects of a well-designed pressure relief system, realistic and reliable data is essential.

The long-term adequacy or otherwise of relief drains is of the greatest significance. Technical Note 63(1) contains suggestions for the classification of relief drain systems in terms of their likely efficiency.

Understanding of the nature and significance of pore pressure and internal uplift is at present limited by the lack of reliable quantitative data. It is considered that the Bradan installation may in the long term contribute some data of the sort required. It is submitted, however, that further field investigations, preferably involving some much older concrete or masonry dams, are a priority requirement. Should such a research programme be initiated the CIRIA piezometer is a suitable instrument.

ACKNOWLEDGEMENTS

The Author wishes to acknowledge the support and cooperation of the Construction Industry Research and Information Association, the North of Scotland Hydro-Electric Board, and the former Ayrshire and Bute Water Board.

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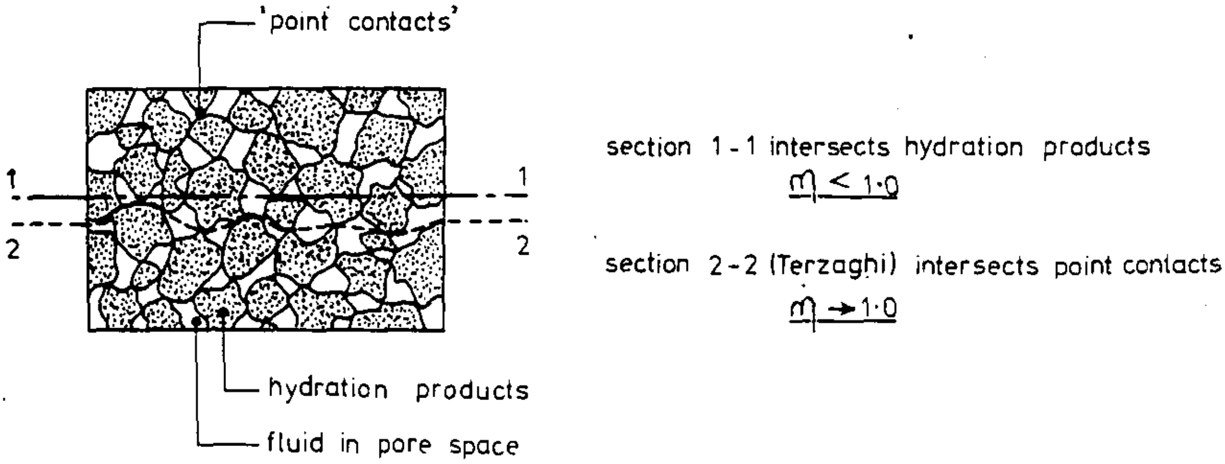


FIGURE 1 : Idealised Microstructure for Concrete Illustrating Area Coefficient $\eta \rightarrow 1{\cdot}0$ (after Terzaghi)

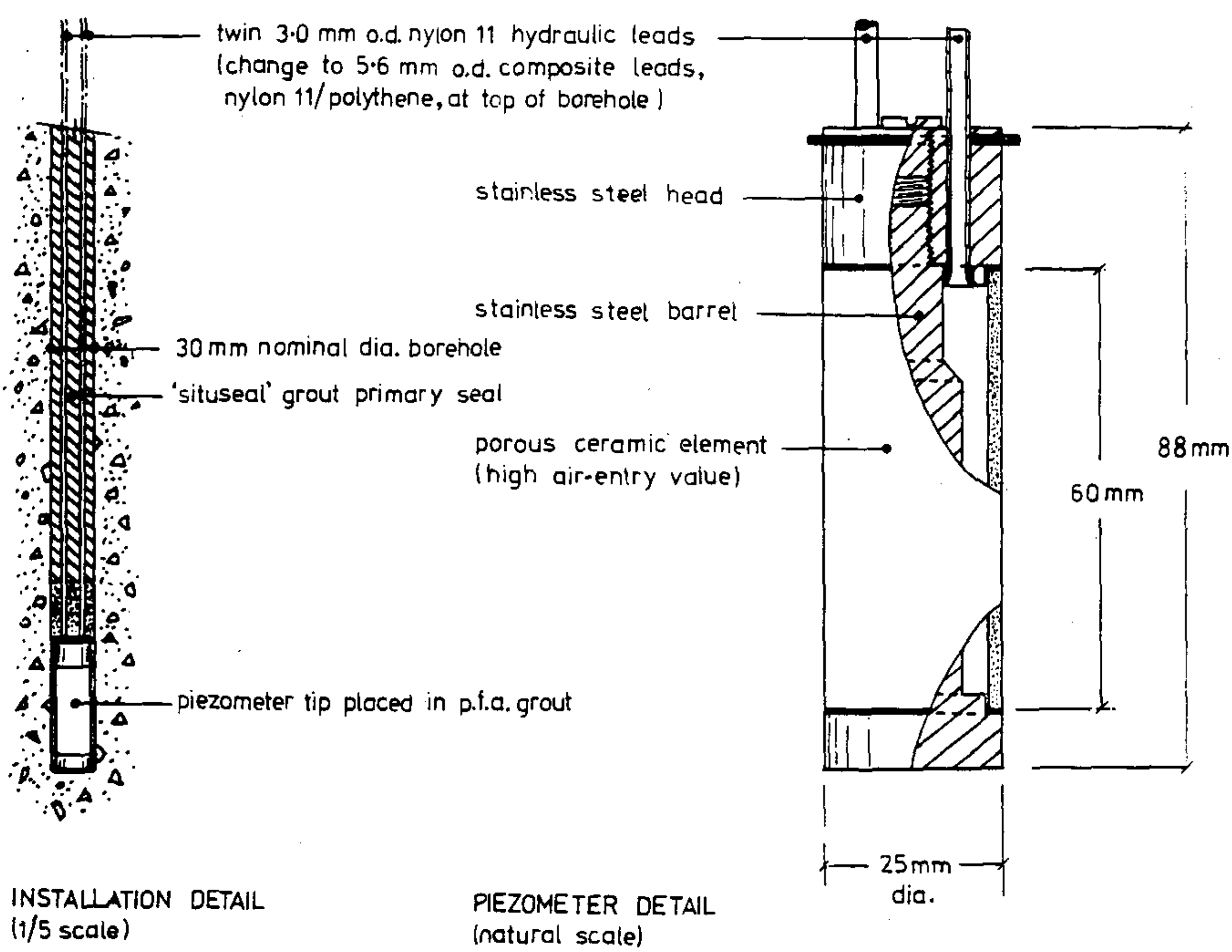
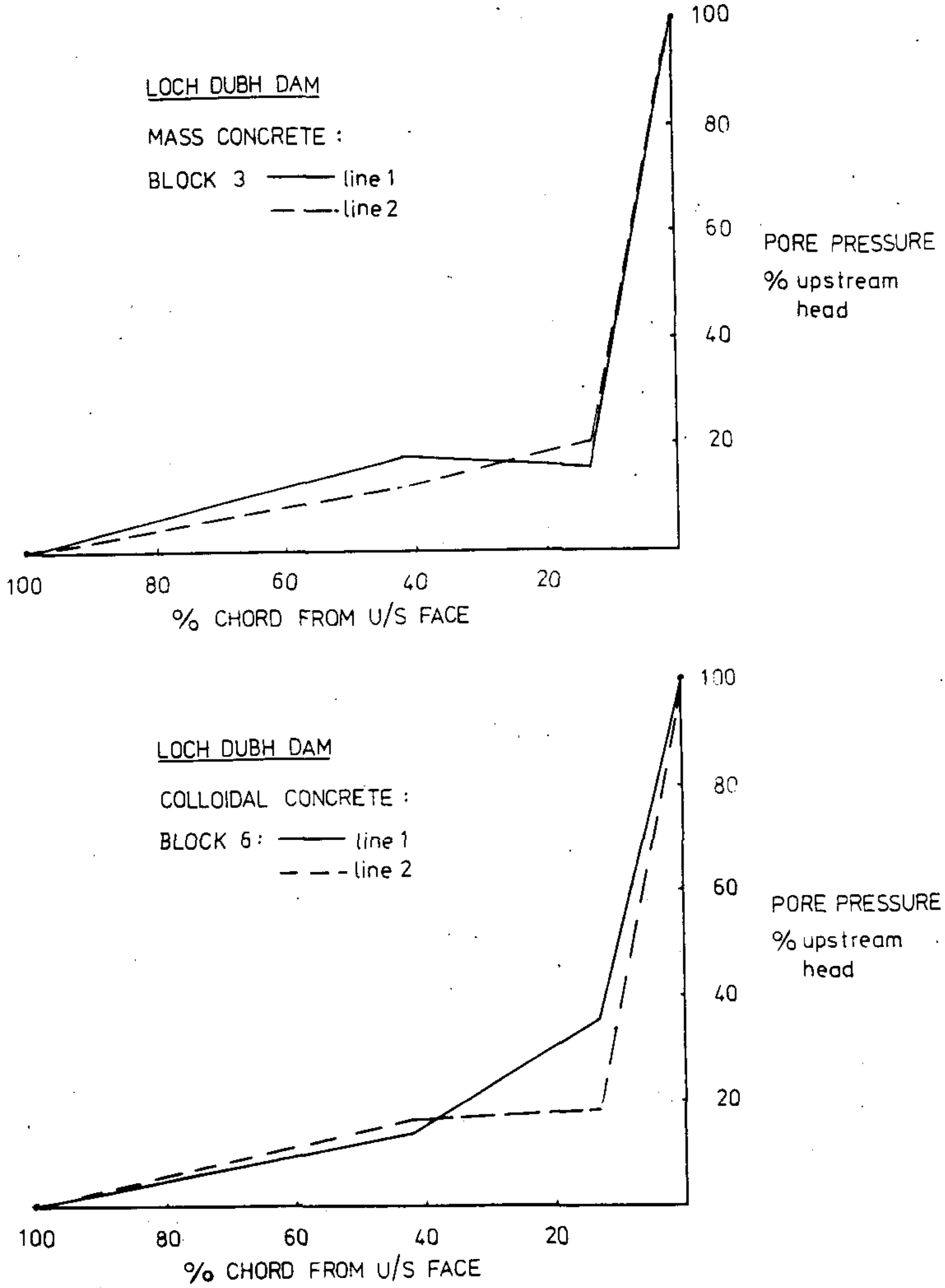
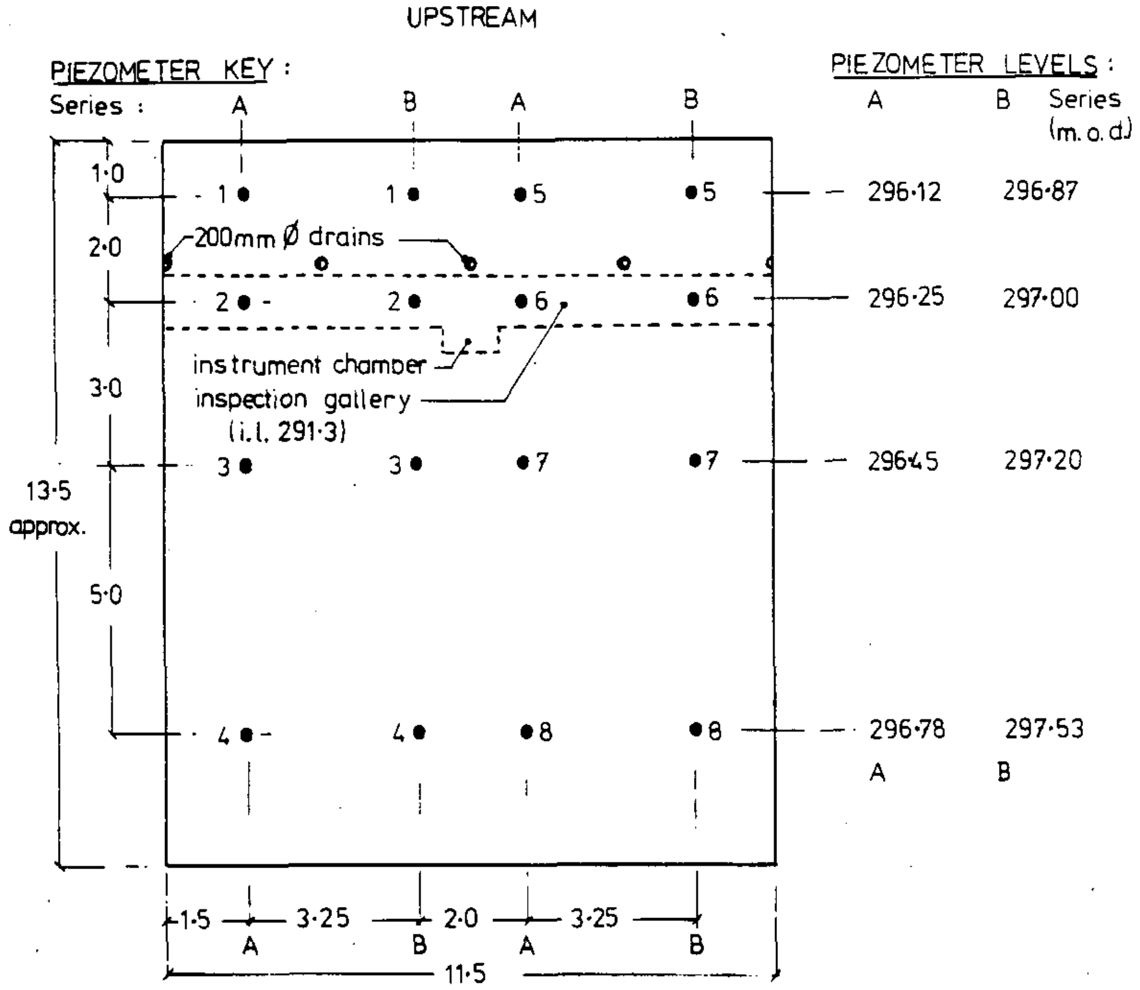


FIGURE 2 : Details of C I R I A Piezometer as Installed in Bradan Dam



COEFFICIENT OF MACRO-PERMEABILITY :		
Mass concrete	:	$k_m = 5 \rightarrow 700 \times 10^{-10}$ m/s
Colloidal concrete	:	$k_m = 0{\cdot}1 \rightarrow 50 \times 10^{-10}$ m/s

FIGURE 3 : Loch Dubh Dam : Representative Pore Pressure Envelopes and Permeabilities 1969 - 1971



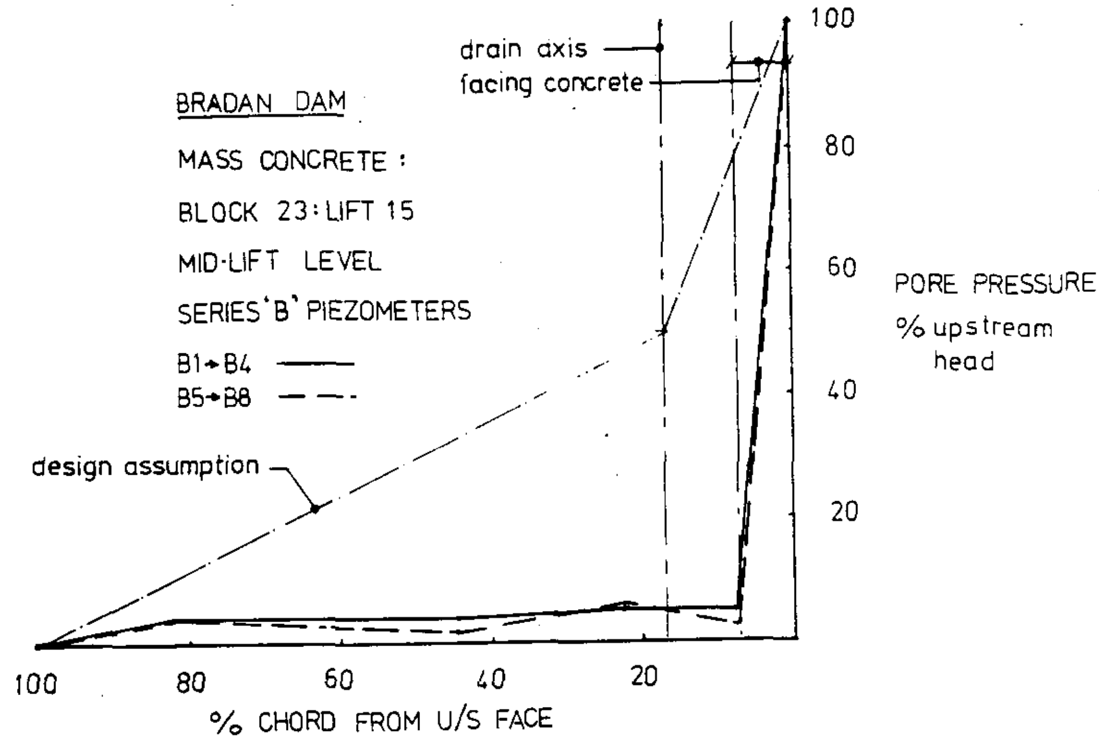
NOTES:

Series A piezometers on Lift 15/Lift16 interface

Series B piezometers at mid-lift Lift 15

Spillway level 313·08 m.o.d.

FIGURE 4: Bradan Dam: Plan of Piezometer Installation, Block 23



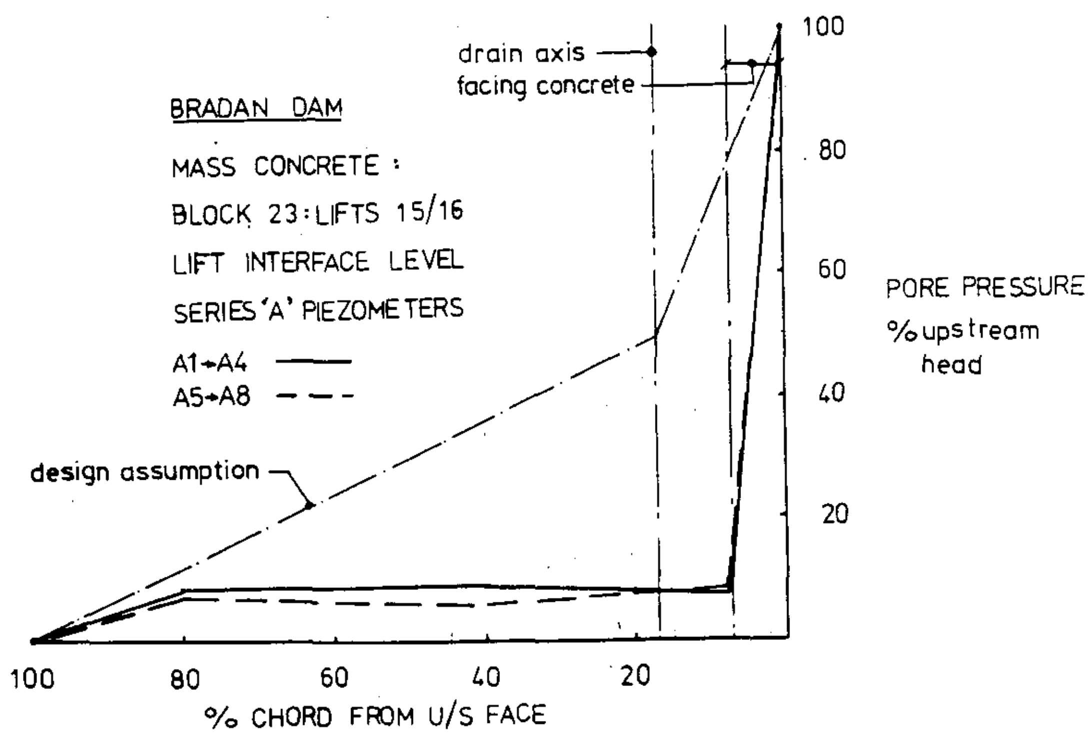


FIGURE 5: Bradan Dam: Envelopes of Maximum Pore Pressure 1973-1975

NON-DESTRUCTIVE TESTING OF CONCRETE DAMS BY SONIC SPEED MEASUREMENT

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SYNOPSIS

Sonic speed through concrete is closely related to Young's Modulus, while a number of factors influence any relationship between sonic speed and concrete strength. The paper discusses these relationships and describes a sonic testing apparatus developed primarily for gravity dams. A case history is presented in which a sonic speed survey readily and economically identified areas of poor concrete quality due to alkali aggregate deterioration. A survey of another gravity dam is also described.

INTRODUCTION

A concrete dam consists of a large number of individual lifts which have been cast over a substantial period of time. Should deterioration occur, it will probably affect only part of the dam. It is, however, generally impractical and uneconomic to core each lift in an effort to assess its quality and the extent of the deterioration problem throughout the structure. Non-destructive testing by sonic speed measurement allows the condition of concrete within individual sections of the dam to be assessed rapidly and cheaply.

The dynamic Young's Modulus of concrete can be calculated from sonic speed and laboratory research has given guidance on the relationship between dynamic and static moduli. Several attempts have been made to correlate sonic speed with concrete strength and these have identified a number of factors which influence any such relationship. It is possible, however, in most cases to determine a usable correlation for a concrete of particular mix proportions and aggregate type by a laboratory test programme.

A variety of sonic test methods and equipment (usually termed 'soniscopes') is reported in the literature but most of these have been used on small specimens or over relatively short distances on arch dams. The present Authors have developed a soniscope similar in principle to the 'hammer blow' seismograph which can be used on test paths ranging from 3 m to 30 m. The equipment can be used without lowering the reservoir level or disrupting the operation of the dam to any significant extent.

RELATIONSHIPS BETWEEN SONIC SPEED AND OTHER PARAMETERS

The speed at which the pressure waves of a sonic pulse travel through an isotropic homogeneous elastic medium of infinite dimensions is related to the elastic constants by:

$$S = \left[\frac{E' g (1 - \nu)}{P(1 + \nu)\ (1 - 2\nu)}\right]^{1/2} \qquad \dots\dots \quad (1)$$

where S = speed of pressure waves

E' = dynamic Young's Modulus

g = acceleration due to gravity

P = density

ν = Poisson's ratio

In practice, it is found that the dynamic Young's Modulus is generally greater than the statically determined value (E''), probably as a result of the quite different stress levels and rates of stressing involved in the two methods. Jones (1) has reviewed the data from comparative studies in normal concrete and suggests the following relationship between the two values:

$$E' = E'' + 6.8 \qquad (\text{kN/mm}^2 \text{ units}) \qquad \dots\dots \quad (2)$$

In hardened concrete, Poisson's ratio usually ranges from 0.25 to 0.30 and it is found to be largely dependent on aggregate type rather than other characteristics of a concrete [2]. Density is related to aggregate type, mix design and method of placement. However, equation 1 is relatively insensitive to changes in these parameters,and tests on a variety of concrete types suggest the empirical correlations between the static and dynamic values of Young's Modulus and sonic speed shown in Figure 1. The static data have been taken from Evans [3] and Baron-Hay [4].

Substantial research has been carried out during the last thirty-five years into relationships between sonic speed, or dynamic Young's Modulus, and concrete strength. It has been found that any correlation is affected by:

a) Aggregate type and grading
b) Mix design
c) Age
d) Moisture content
e) Curing history.

However, a usable correlation can be obtained for a particular concrete in which the above factors are relatively constant, and Jones [2] has explored the effect of varying each of these factors in turn.

The relationships and correlations discussed so far in this section have been prepared entirely from laboratory tests on small samples. When sonic speed is measured within a mass of concrete, such as a dam, three further factors must be considered.

1) EFFECT OF STATE OF STRESS : When the mean compressive stress within concrete increases, the sonic speed also increases. Deterioration may in some cases superimpose substantial increase (or decrease) in stress level upon the design state of stress, which itself varies as a result of the normal operation of a structure. Figure 2 shows a correlation between sonic speed and stress level for a typical concrete in laboratory tests. The corresponding increase in Young's Modulus with sonic speed and stress level is such that the relationships shown in Figure 1 are still appropriate.

2) EFFECT OF TEMPERATURE : Variations in temperature normally have only second order direct effects in sonic testing of structures. However, under some circumstances they can cause significant changes in stress level (increase of 10^{o}C can cause an increase of up to 5 MN/m^2) which will affect results as discussed in a) above.

3) COMPARISON OF FIELD AND LABORATORY SONIC SPEED DETERMINATIONS : In sound concrete the field and laboratory sonic speeds are in good agreement. As sonic speed drops due to deterioration or micro-cracking resulting from poor curing, the ratio of laboratory to field value increases. Figure 3 gives a relationship for a typical concrete.

There is as yet no data on whether equations 1 and 2 or Figure 1 are appropriate for relating field sonic speed measured over long path lengths to the mass Young's Modulus. Nevertheless, significant error is not likely to result from the use of such relationships provided that any deterioration or micro-cracking occurs throughout the tested path length. However, should cracks persist over lengths greater than, say, 0.25% of the tested path length, then sonic speed may under or over-estimate the quality of the concrete depending on whether such cracking is preferentially orientated perpendicular or parallel to the tested path.

To summarise, sonic speed is closely related to Young's Modulus for concrete generally. Although a relationship between sonic speed and strength exists for a particular concrete, it is complicated by a number of factors. However, as the majority (if not all) of these factors are relatively constant throughout a particular structure, a laboratory test programme should generally identify a usable relationship. Thus, field sonic speed measurements can be employed as an index of concrete elasticity and strength throughout a dam, provided that adequate precision and repeatability of the field measurements can be achieved.

DEVELOPMENT OF SONISCOPES

Non-destructive sonic testing of concrete dams has been employed since 1945. Most of the literature refers to work on arch dams where test path lengths range from 2 m to 10 m. The more successful instruments have been based on direct pulse transmission from downstream to upstream face or vice versa because of the high back-scatter which the heterogeneity of concrete introduces. Surveys have generally been limited to concrete above water reservoir level. Leslie and Cheesman (5) describe a soniscope which employed both a piezoelectric transducer source and receiver of 20 kHz frequency together with an electric timing circuit, and this seems to have been the forerunner of a subsequent generation of devices.

The present Authors have worked on gravity dams and have developed a direct pulse transmission soniscope for path lengths in the 3 m to 30 m range. This consists of four major component parts; a seismic source, pulse receiver, signal conditioning module and timing unit, as shown diagrammatically in Figure 4. The latter two components are shown in operation in Figure 5. Because of the necessity to survey operational structures, the seismic source is located on the downstream face while the signal receiver is positioned on the upstream face, water coupling being used below water level and physical coupling above.

CHOICE OF SEISMIC SOURCE

Because of the heterogeneity of concrete, sonic pulses passing through it are severely attenuated. As frequency increases, this attenuation becomes more severe. On the other hand, the lower the frequency the longer the rise time of the initial pulse becomes and this makes the timing procedure less accurate. Again, when using a water-coupled receiver, only about 15% of the pulse reaching the upstream face of the dam is transmitted to the receiver because of the high reflection coefficient of the concrete/water interface.

A variety of source types was investigated including piezoelectric and magnetostrictive transducers, spring loaded impact plungers, and also sparkers located at the upstream face. Generally units of convenient size were not sufficiently powerful or gave results not adequately reproducible, and finally a 1 kgf hammer was chosen as the most promising source.

A series of tests revealed that a single tap of the hammer produced both an adequate and reproducible pulse. A series of clean blows produced signals of virtually identical frequency, usually in the 2 to 3 kHz range, and with a little practice signals of near constant amplitude could be achieved.

Having opted to use a mechanical source, various methods of providing an electrical signal related to the instant of impact were considered. The usual method of attaching an inertia switch to the hammer proved to be too slow in response and lacked reproducibility. A striker plate between the hammer and the dam (which forms an electrical circuit at the impact of the hammer) was equally unsatisfactory. The most reliable and reproducible method found was to mount a horizontal sensing 4.5 Hz geophone on a small purpose-made bracket cemented onto the dam face 150 mm from the centre of hammer impact. To correct observed travel time through the dam, the time required for the pulse to travel from the hammer impact point to the adjacent geophone must be added to the measured times. This correction is determined by mounting two similar geophones on the face of the dam 150 mm apart and measuring the travel time between them while striking the face on the same line as the geophones.

THE SIGNAL RECEIVER

A geophone similar to that used near the hammer impact is employed above water level. The upstream geophone is critically damped, while the downstream one is heavily overdamped because of the proximity of the source. The geophone is fixed to the dam from a bosun's chair or boat as appropriate. A single element piezoelectric hydrophone is used below water level. This is mounted in a specially built carriage, as shown diagrammatically in Figure 6, which facilitates vertical movement up and down the upstream face. The hydrophone element is located some 30 mm from the upstream face and a correction has to be applied for the travel time in this water gap. An assumed value of 1.5 km/s for sonic speed in water is adequate because of the short path length involved.

SIGNAL CONDITIONING AND TIMING

As a pulse passes through concrete its initial rise time is increased, and attenuation (discussed earlier) drastically reduces the amplitude of the pulse. The purpose of signal conditioning is to minimise errors due to these factors.

Both the transmitted and received pulses are fed to individual variable gain amplifiers and the slopes of the initial rise of both pulses are accurately matched using a multi-channel oscilloscope display. A high precision counter timer is used with a gate signal level of about 5% to 10% of the peak pulse level to record the travel time through the dam. The conditioning and timing process is shown diagrammatically in Figure 7. Care must be taken in connecting up the source and receiver detectors to ensure that a positive voltage is generated by the leading edge of the pulse, as the oscilloscope and counter timer are set to trigger on a positive slope only.

SONIC SURVEY METHODS

Because of variability between individual pours of concrete, particularly where deterioration takes place, tested paths through the dam for a particular sonic speed measurement should as far as possible remain within one lift. The average of at least four individual paths should be taken as the result for a particular lift.

After locating the detectors on a particular path, the dam is struck several times with the hammer at the marked position, which allows the operator to adjust the gains of the signal amplifiers as set out above. The times for at least fifteen individual taps are then recorded and averaged, and after correction as discussed earlier, the sonic speed is calculated using a path length usually calculated from 'as constructed' drawings. As each tap constitutes a separate measurement, the quality of which can be assessed on the oscilloscope display, the operator can discard any time reading which appears irregular or suspect. In practice this is found on average to be about five percent of the total number of readings. The sonic speed of the lift is then calculated as the average of the speeds for the individual paths surveyed.

Survey paths should be located sufficiently far from the boundaries of the lift to avoid interference. A combination of 'straight through' and 'diagonal' paths is also to be recommended, as a check can then be kept on any error which may arise related to path length or structure geometry.

ERRORS AND REPEATABILITY

Errors in measurement of sonic speed arise from three major sources.

a) Inaccuracy in pulse timing

b) Path length errors due to mislocation of sensors

c) Path length errors due to inaccuracy in 'as constructed' dam geometry.

Field studies have shown timing repeatability to be within $\pm$ 20 microseconds.

As sensors are mislocated in a plane normal or subnormal to the test path the resulting errors are very small, particularly near the top of the dam. Where a hydrophone is used below water level, the error increases due to deviation of the carriage from the required line. Tests have been carried out on gravity structures of different geometry in which the soniscope sensors have been removed and replaced on the same path length a number of times. These suggest that errors due to a) and b) above range from 3% to 1% as path length increases from 3 m to 30 m.

Errors related to dam geometry will vary from structure to structure. If 'as constructed' drawings do not record geometry, some general checks should be carried out. The order of the likely error revealed will determine whether a detailed geometry survey is necessary. These errors can be quite significant as they constitute a direct error in path length. Nevertheless, for most structures the overall accuracy of a sonic speed measurement should range from $\pm$ 5% to $\pm$1.5% as path length increases from 3 m to 30 m.

CASE HISTORIES

VAL-DE-LA-MARE DAM

This is a gravity structure 183 m long and 30 m high, with a maximum 23 m standing above present ground level. The upstream face is vertical while the downstream face slopes at 1 to 0.7. The dam is straight in profile and is constructed in twenty-six 6.7 m wide blocks built of 1.22 m high lifts.

The concrete of the dam was deteriorating as a result of alkali-aggregate reactivity and sonic testing was carried out primarily to give an assessment of concrete quality and to provide a basis for monitoring further deterioration of the structure. The survey data were also used to locate exploratory drilling and piezometers in the most critical areas in the dam. Another paper to this Symposium by Coombes et al [6] describes the deterioration and its investigation in more detail.

In November 1972 the two hundred and ninety-four accessible lifts were surveyed. Four individual paths were tested in each lift as set out in Figure 8 and these path lengths ranged from 2.7 m at the top to 13.3 m at the base of the dam. Sonic speeds ranged from 3.72 km/s to 4.79 km/s with an average of 4.56 km/s. Overall accuracy for this particular structure was considered to be within $\pm$0.15 km/s for the top three lifts and $\pm$0.06 km/s for the remainder of the dam.

One of the most successful aspects of this initial survey was the correlation established between concrete of low sonic speed and casting date (Figure 9), and this provided quite encouraging indications as to the size of the problem.

Eight months later in July 1973 one hundred and twenty-four lifts were re-logged in an effort to assess the rate of deterioration. These lifts were chosen from the 1972 results on the basis of all lifts below 4.7 km/s and 10% of the balance chosen at random. The 1972 average for these lifts was 4.47 km/s while the 1973 average was 4.46 km/s. A histogram of each set of results is shown in Figure 10.

The sonic speed of forty-nine of the re-logged lifts increased marginally, including nine which increased more than 0.06 km/s. These occurred in the top 7.5 m of the dam and it is worth noting that reservoir water levels were significantly lower during the repeat survey.

It is considered that, with a few exceptions, the re-surveyed lifts had not undergone significant deterioration between the two surveys. Subsequent visual examination of the dam bears this out. In retrospect, the repeat survey may have provided more definite results if it had been carried out at the same time of year and with the same reservoir levels. These factors may have obscured a minor trend of slight further deterioration. In the light of experience to date a repeat survey after three to four years would be more appropriate for this structure.

GRAND VAUX DAM

This is a much smaller dam, 125 m long and 9 m high at its centre. The downstream face is concave varying in slope from 50^{o} to 60^{o} to the horizontal while the upstream face slopes upstream at 1 in 48. A similar configuration of survey paths to that shown in Figure 8 was chosen. Sonic speeds varying from 4.42 km/s to 4.63 km/s were recorded for the central lifts of the dam with the average being 4.54 km/s. These indicated satisfactory concrete strength and quality within the structure.

ACKNOWLEDGEMENTS

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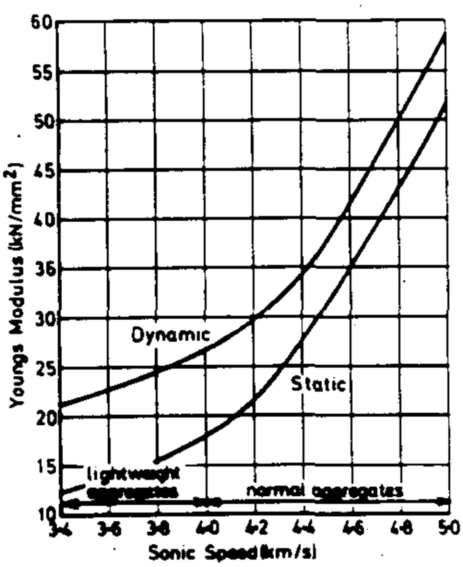

Fig. 1 Empirical Correlations betbeen between Sonic Speed and Young's Modulus from Laboratory Tests.

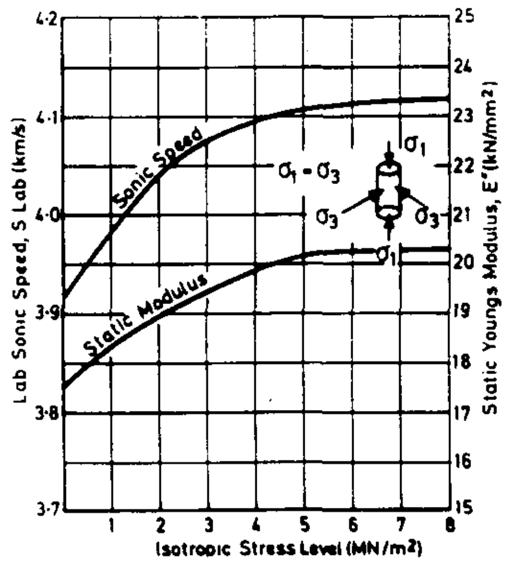

Fig. 2 The Effects of Stress Level on Sonic Speed and Young's Modulus from Laboratory Tests.

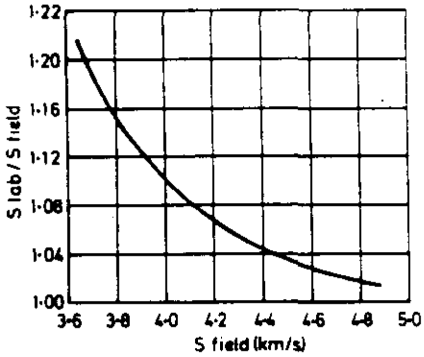

Fig. 3 The Relationship between Field and Laboratory Sonic Speeds for a Typical Concrete

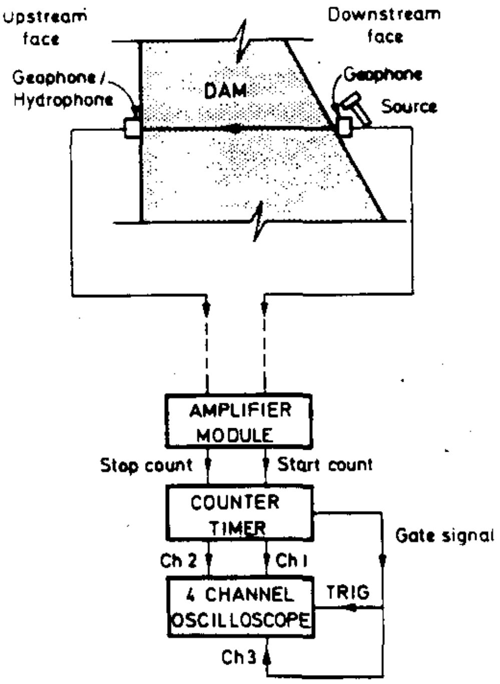

Fig. 4 Block Diagram of Soniscope

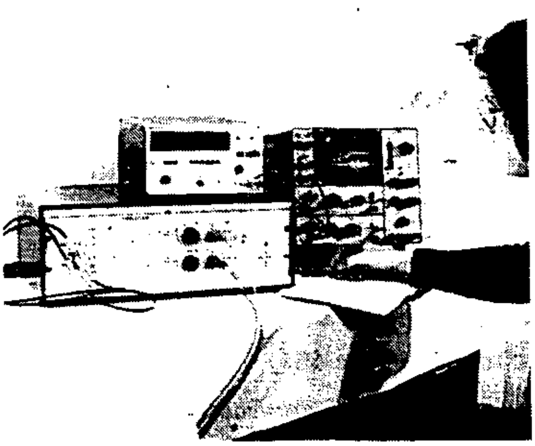

Fig. 5 Signal Conditioning and Timing Modules

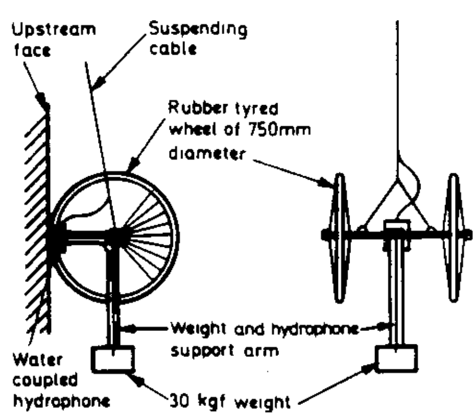

Fig. 6 Hydrophone Carriage for use below Water Level

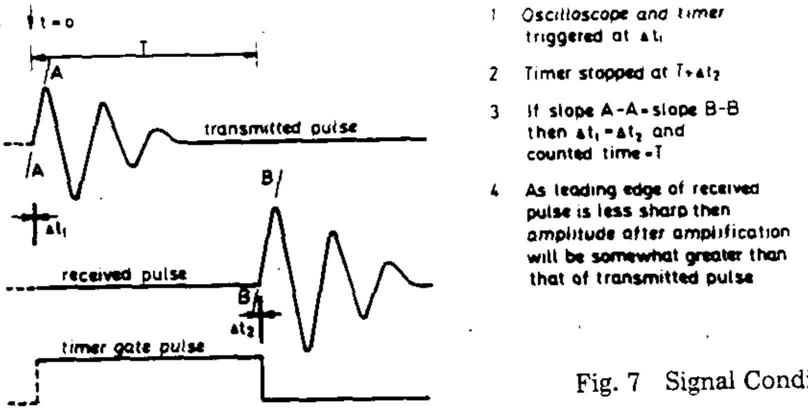

Fig. 7 Signal Conditioning and Timing Process

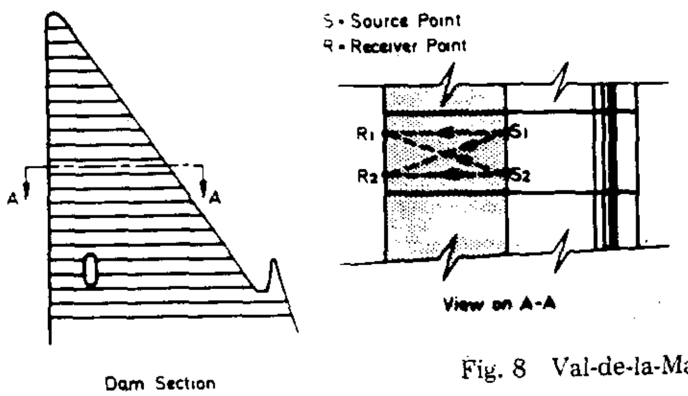

Fig. 8 Val-de-la-Mare Dam Test Path Arrangement

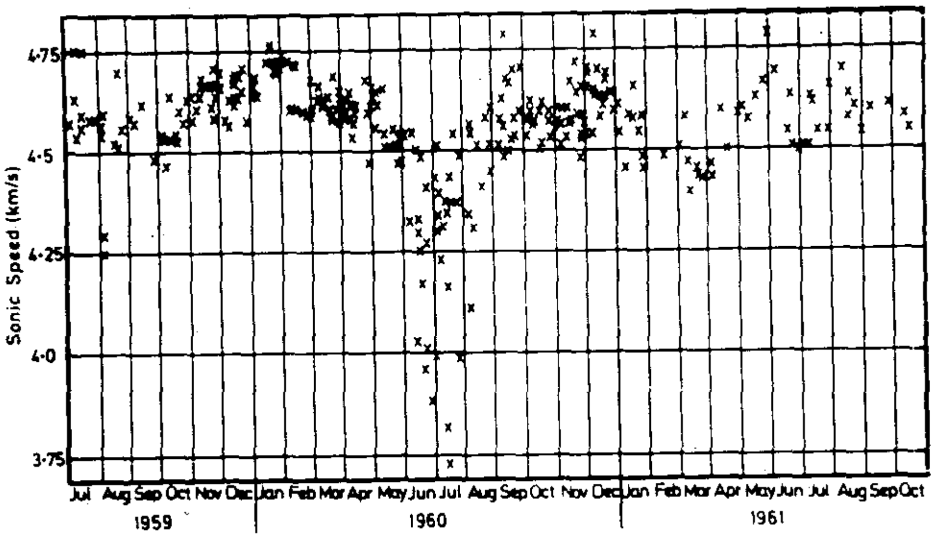

Fig. 9 Val-de-la-Mare Dam Sonic Speed v. Castin Date

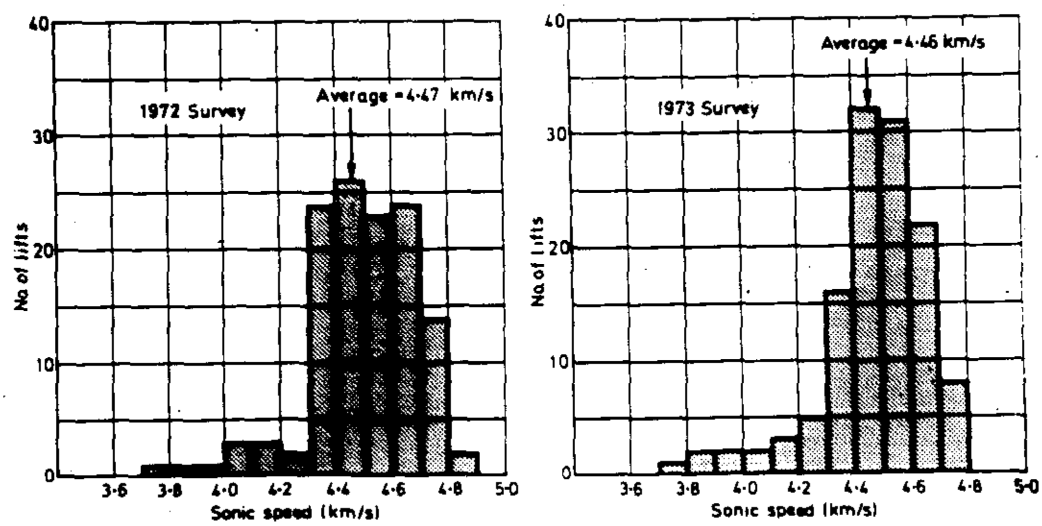

Fig.10 Val-de-la-Mare Dam: Histograms of Sonic Speeds in Re-surveyed Lifts

REMEDIAL MEASURES TO VAL-DE-LA-MARE DAM, JERSEY, CHANNEL ISLANDS, FOLLOWING ALKALI-AGGREGATE REACTIVITY

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SYNOPSIS

Val de la Mare Dam is the principle storage reservoir for the Island of Jersey, with a capacity of 258 t.c.m. The dam has been regularly inspected since completion in 1962.

In January 1971 small upstream movements of the handrail of the crest walkway bridge were noticed. At the same time, darkening and damp patches were observed on the downstream face of the dam and parts of the surface showed random cracking of the concrete.

A programme of investigations was initiated, and alkali-aggregate reactivity was eventually diagnosed as the cause of the defects.

Proposals were developed, and subsequently implemented, for remedial works which included the provision of drainage into the gallery, the grouting of and the installation of anchors in a section of the dam most adversely affected by the reaction, and the installation of appropriate instrumentation to monitor the loads on the anchors and future movements of the particular section.

INTRODUCTION

The dam is located on the western side of Jersey. It has a maximum height of 23 m above the valley floor, a crest length of 168 m, and is constructed of mass concrete in 6.7 m wide blocks. The foundation is of Precambrian sedimentary shales, and in places the formation level extends to 9 m below ground level.

The dam was designed using the middle third rule allowing for an internal uplift pressure of 50% of reservoir head at the upstream face decreasing linearly to zero at the downstream face.

Construction took place over the period 1957 to 1962, with the coarse aggregate being obtained from a local quarry and from oversize material from a beach, the fine aggregate from a beach, and the cement being imported from the U.K. A photograph of the dam is shown in Figure 1. The first signs of trouble appeared in January 1971, following a period of colder than usual weather. Small upstream movements in the order of 6 mm to 13 mm were noticed on certain sections of the concrete handrail of the crest walkway bridge, and darkening and damp patches were observed at the same time on the downstream face of the dam on the same blocks on which the movements had occurred, with the surface showing random hairline cracking. At first alkali-aggregate reaction was not considered to be the cause of the problem as the aggregate quarry had been in operation for several years, and as there has been no published record of an occurrence on Jersey. Several theories, including frost action, sulphate attack and earthquake action, were considered and investigated but eventually discounted. Subsequently however, alkali-aggregate reaction was considered a possibility and further investigations to prove this and to obtain information on the likely effects were initiated.

INVESTIGATIONS INTO ALKALI AGGREGATE REACTIVITY

DEFINITION

Alkali-aggregate reaction in concrete can be briefly defined as being the formation of an alkali silicate gel within the concrete mass caused by chemical action between alkali reactive materials in the aggregates and free alkali from the cement. The formation of the gel leads to the setting up of expansive forces in the concrete, and this is usually followed by random pattern cracking with the soft viscous gel exuding through the cracks and through pores of the concrete.

INVESTIGATION PROGRAMME

The investigation programme included the following :-

1 Consultations with the Cement and Concrete Association, the United States Army Corps of Engineers, the Concrete Research Laboratory in Karlstrup, and the British Museum (Natural History), Department of Mineralogy. The consultations were linked with a review of the existing literature on the subject which revealed that experience of the phenomenon in the United Kingdom was limited. Overseas case histories were therefore examined and discussed, and visits were paid to Jersey by specialists in the field of testing and deterioration of concrete. Reactive materials were identified in some of the materials obtained both from the local quarry and from the beach, and a materials testing programme was initiated.

2 Review of the method and sequence of construction of the dam and of the source of supply of the relevant construction materials - investigations revealed that affected lifts were randomly sited in the dam, but were associated in time.

3 Discussions with the Cement Marketing Company Ltd on the cement shipped to Jersey during the appropriate period of construction. These discussions revealed that cement shipments to Jersey in 1960 during periods significantly related to the time of construction of affected block lifts contained cement with alkali contents of values up to 0.95 per cent, measured on a monthly average basis and expressed as a percentage of Na_2O. (It is interesting to note that none of the British Standards relating to cement, namely, B.S 12 : Part 1 : 1958, B.S. 12 : Part 2 : 1971, B.S 4027: Part 1 1966 and B.S. 4027 : Part 2 : 1972 make any reference to alkali content of cement. The only Standard recommending limiting percentage for alkali content of cement is ASTM C150 - 69. Note 2 of this American Specification states that '*cement containing not more than 0.60 percent alkalies calculated as the percentage of Na_2O plus 0.658 times the percentage of K_2O may be specified when the cement is to be used in concrete with aggregate that may be deleteriously reactive*').

4 Geological survey of, and testing programme on aggregate samples from, the local quarry - carried out by the quarry owners.

5 Examination of the sources and testing of samples of the local beach sand.

6 Expansion testing on cores taken from the dam and mortar bars made from local quarry rock and beach sand - carried out by the Building Research Establishment.

7 In-situ sonic measurements in each accessible lift of each block of the dam - the technique is described in paper 3.2 (1)

8 Core drilling in specific sections of the dam, followed by laboratory description and testing and petrographic examinations of thin sections.

9 Installation of electrically operated piezometers in certain sections of the dam.

ALTERNATIVE REMEDIAL MEASURES CONSIDERED

Following the investigation programme and instrument readings taken over a period of three years, the decision then had to be made on :

(a) Whether or not the dam would eventually collapse due to concrete deterioration.

(b) If so, how long could it remain in service before an alternative water supply source had to be found.

(c) If not, what remedial works were required to keep the dam in a safe stability condition.

It would have been desirable if a completely quantitative decision could have been taken on this matter, but this was impossible. Subjective opinions, based on the whole range of accumulated information, had to enter into the decision.

However, several general factors were of overriding significance and influenced the ultimate decision :

1 Although the reactive material had to be assumed to be present throughout the dam, the degree of attack and concrete deterioration varied. This was borne out by the sonic test results, the crest movements and visual inspection of the downstream face of the dam. The known variation of alkali content of the cement delivered and the fact that most of the reactive material was thought to be derived from a number of veins in the quarry and from beach pebbles containing either chalcedony or opal, also supported this assessment.

2 The expansion test results carried out by the Building Research Establishment on concrete cores from the dam containing suspected reactive material and on mortar bars manufactured from coarse aggregate from the Ronez Quarry and beach pebbles, All containing reactive silica, indicated that the degree of expansion was not as high as the values considered unacceptable in the USA and Canada, based on ASTM C. 33-67 and C.227-69.

3 Case studies of other dams indicated that the initial large rate of expansion caused by the reaction was not maintained with time.

4 There was no proven method of chemical treatment that would either stop or reduce the reaction and expansion.

5 The readings of internal uplift pressures from the installed piezometers were acceptable, except in one case.

Based on the evidence accumulated on this particular dam and on the results of other cases, a conclusion was reached that the concrete would not deteriorate to the extent that it would be incapable of taking the required compressive loads. The danger was that expansive cracking could lead to higher internal uplift pressures than had been allowed for in the design, resulting in instability.

Hence the policy was adopted of proceeding with remedial works to ensure that the stability of the dam was maintained against increased internal uplift, assuming that the concrete would be capable of taking all the stresses applied to it in the future. Also, as the alkali-aggregate reaction varied over the dam, only isolated bad sections would be dealt with, and the remedial method adopted should be capable of being extended in stages to the complete dam at a later date should further deterioration occur. Any remedial works adopted had to be such that the reservoir could be kept in operation at all times, and a minimum of restriction placed on the water level during the work.

Piezometers had been installed in certain sections of the dam, the locations having been chosen on the basis of the sonic test results. The limited number of piezometers installed indicated that the original design uplift pressures were not being exceeded, except in one case in Block 6 where the higher pressures had resulted in a reduction in the design safety factor against overturning. As Block 6 was one of the worst blocks, both visually and from the sonic test results, it was decided to undertake remedial works on this block.

The remedial works considered for Block 6 were :

1 Breaking out the block completely and replacing it with new concrete.

2 Breaking out the block down to the bottom of Lift 8 and replacing it with new concrete.

3 Installing anchor bars through the block near the upstream face.

4 Forming an impermeable membrane on the upstream face.

5 Forming a grout curtain near the upstream face.

It was estimated that the cost of the complete breaking out and replacement of the whole of Block 6, which would involve the installation of a temporary coffer dam around the upstream face of the block, would be approximately five times that of the anchor scheme for the equivalent result.

The alternative of replacing only part of Block 6 was manifestly cheaper than complete replacement, and, in addition, would not require a temporary coffer dam if the breaking out of the concrete immediately followed the seasonal drop in water level. However, this alternative was rejected on the grounds that only concrete above the lowest downstream ground level would be replaced and that doubt would still exist about the concrete below this level. Also, should unexpected difficulties or delays occur in the cutting out or replacement of the concrete, then restrictions might have to be placed on the rising water level. Lastly, the remedial measures adopted for Block 6 were intended to be applied to further sections of the dam at a later date should further deterioration occur - this particular approach was not practicable for the deeper sections of the dam.

The formation of an impermeable membrane on the upstream face of the dam was rejected on the grounds that the necessary works would involve the complete emptying of the reservoir. The formation of a grout curtain near the upstream face of the dam was considered to be the most uncertain with regard to successful implementation, due to lack of knowledge regarding the possible penetration of the cracks by the grout.

It was decided, therefore, that the anchor scheme would be adopted for restoring the stability of Block 6. At the same time, a trial on forming a grout curtain would be undertaken whilst installing the anchors in case this method proved to be viable and cheaper for any future work. Piezometers would also be installed to check the effect of the curtain.

On the remainder of the dam it was not considered essential to undertake any remedial works. However, since drilling equipment would be mobilised for work on Block 6, it was decided to take advantage of this by drilling drainage holes approximately 1.2 m from the upstream face of the dam in the blocks where access was available from the inspection gallery. It was hoped that if cracking became serious, the holes would intercept seepage flow and reduce the internal uplift pressure downstream of the holes.

REMEDIAL WORKS

GENERAL

Remedial works were carried out on the dam during the period June to December, 1974. The remedial works consisted of the following :-

(a) An initial site investigation of the foundation rock beneath Block 6.

(b) The installation of a high-strength reinforced concrete spreader beam along the top of Block 6.

(c) Grouting of Block 6 using an inert polythixon grout.

(d) The anchoring of Block 6 to the foundation rock beneath the dam.

(e) The installation of instrumentation to monitor the behaviour of Block 6 during the stressing of the anchors and in the future.

(f) The drilling of relief drainage holes in Blocks 10 to 20 inclusive.

Block 6 is situated near the right abutment, and is clearly indicated on Figure 1 by the presence of the Contractor's scaffolding and equipment. Blocks 10 to 20 are in the centre section of the dam between the two gallery access openings sited against the downstream toe.

SITE INVESTIGATION

As a necessary preliminary to the design of the anchors for Block 6, an investigation of the foundation rock beneath Block 6 was carried out. Two vertical boreholes and one inclined borehole were drilled from the downstream toe, the latter hole angled to penetrate through the middle of the proposed anchorage zone.

In addition, one vertical borehole was drilled from the crest of Block 6 down through the concrete of the dam into the proposed anchorage zone. The position in plan of this hole coincided with one of the proposed grout holes in the centre of the block (refer to Fig.2). Laboratory tests were carried out on cores recovered from the boreholes to produce data on the Elastic Modulus and Poissons Ratio of the rock, and on the shear strength of the joints. All boreholes encountered very jointed and fissured rock.

In addition to the boring, rock outcrops in the vicinity of the dam were mapped geologically to supplement the borehole information.

INSTALLATION

In order effectively to distribute the loads from the anchors, a high-strength reinforced concrete spreader beam was introduced as part of the crest lift of the downstream face extending for the full block width between the walkway supports. Basic details of the arrangement are shown in Figure 3.

During construction, water level in the reservoir was maintained at elevation 44.5 m or below, and this satisfied the operational requirements of the Jersey New Waterworks Company. To ensure stability of the crest lift after cutting out for the spreader beam, short resin-bonded rock anchors were installed down through the uncut concrete of the crest lift. By the use of this light post-tensioned bolt system, the reduced crest thickness was designed to be stable for the maximum water elevation of 46.5 m. Thus, under flood conditions, storage to 46.5 m would be available and design flood water spilling over Block 6 during construction would cause only minor damage to shuttering, scaffolding and equipment.

The spreader beam was installed before any drilling work was carried out for the grouting, anchor tendons and instrumentation, and to avoid drilling through the freshly placed concrete of the spreader beam mild steel tubes were fixed in appropriate positions through the beam.

GROUTING OF BLOCK 6

Grouting was carried out in Block 6 as a trial measure to ascertain whether a curtain could be introduced near the upstream face of the block to reduce seepage uplift pressures and to reduce future alkali aggregate reaction.

Six holes were drilled vertically from the bottom of the spreader beam down through the concrete of the dam at a distance of 1.2 m from the upstream face and between 0.9 m and 1.3 m apart. Five of the holes were drilled in 82 mm dia and the sixth hole, chosen as the position of the site investigation borehole, was cored in 102 mm through the concrete and foundation rock for a depth of 33 m. The depth of the five holes varied from 13.4 m to 17 m such that the bottoms of the holes were approximately 0.6 m into the concrete of the cut-off trench. The positions of the grout holes are shown on Figures 2 and 4.

A water test was carried out in each hole to test the permeability of the surrounding concrete and showed the permeability to be very low. The material chosen for grouting was Polythixon 60/40 DR grout. This is an oil-based chemical grout supplied in the form of two liquid phases which, when mixed, form a cross-linked polymer that sets to form a rubber-like substance. At the time of mixing, the grout has a very low viscosity and is therefore suitable for penetrating fine cracks. The quantity of Polythixon used in the grout holes was only slightly greater than the quantity of water used in the water tests, so it is probable that very little penetration was achieved.

ANCHORING OF BLOCK 6 - BASIC DESIGN

The anchoring of Block 6 consists of 3 No. 40 mm diameter 'Macalloy' high tensile anchors fixed in 114 mm dia holes positioned symmetrically across the width of the Block at 2.23 m centres and drilled through the concrete of the dam into the foundation rock - refer to Figs 2 and 3. The required anchor force of 85 tonnes per tendon was determined from static considerations of stability to provide for a minimum factor of safety against overturning of 1.7 under full flood condition loading and with hydrostatic uplift assumed to act over 100% of the plan area of the block, with full hydrostatic head at the upstream face decreasing linearly to zero at the downstream face. The restraining and load spreading effects of the adjacent blocks were ignored in the design.

Calculations for the proposed depths and lengths of the anchorage zones were carried out using data from the laboratory testing of rock cores. Using the depths and anchorage lengths resulting from this analysis, a computer analysis of stresses and displacements in the dam and the foundation was carried out using a finite element technique based on the linear elastic plane strain model.

The computer analysis was carried out for four loading conditions, namely, dam alone, dam plus water, dam plus anchors, and dam plus water plus anchors. In each stage, gravity loading was used for the foundation, thus enabling the change in displacements and stresses between each loading stage to be easily determined in addition to the overall picture. The results of the computer analysis showed that there would be no areas of high stress concentration after the installation of the anchors and no excessive movements of the structure.

The final decision regarding the minimum depth of the anchorages was eventually largely influenced by the rock conditions encountered during drilling for the anchors. A major shear zone was detected below Block 6, lying parallel to the bedding and therefore dipping steeply upstream and towards the centre of the valley. Numerous fissures and areas of highly fractured brecciated rock associated with this shear zone were observed.

The initial design provided for the bonding of the anchors to the rock with epoxy resin grout. This type of grout was chosen to eliminate any risk of any reaction occurring in the future in the foundation rock beneath the dam, since it was possible that small traces of reactive minerals might be present in the foundation rock. However, detailed enquiries during the final design stage into the use of epoxy resin grout for this type of rock anchoring revealed that a rather higher than previously anticipated shrinkage could take place during the setting of the grout. Since the drilled anchor holes were relatively so much larger than the smooth anchor tendons, this particular property of the epoxy resin grout was considered to be unacceptable, and the anchorage zones were redesigned using a low alkali cement grout.

In view of the fact that it was not certain that there were reactive minerals present in the foundation rock and that only traces were likely to be there if present at all, the risk of any reaction taking place in the anchor zones due to the use of low alkali cement grout was considered to be negligible.

ANCHORING OF BLOCK 6 - DRILLING FOR ANCHORS

No problems were encountered drilling through the concrete of the dam - drilling was carried out using a 114 mm dia down-the-hole hammer. Drilling through rock was initially good, until highly fractured zones were encountered where the drill repeatedly jammed. When the hammer was freed gravel size fragments of mudstone and some clay were flushed up the borehole. Collapsing of the holes frequently occurred, giving concern over the possibility of cavities being created in the fault zone.

In order to prevent further collapse in the holes the method of drilling was changed from rotary percussive to coring using water flush, and a drilling/pressure grouting technique was employed. Repeated pressure grouting and redrilling resulted in a build-up of grout in the fault zone, and the area was eventually stabilised.

One of the advantages of the change in drilling method was that the rock cores could be examined and a better appreciation of the geology of the zone could be obtained. The depths of the anchor holes were extended beyond the depths confirmed by the finite element analysis as being acceptable until the cores showed the rock to be sufficiently competent to carry the anchorage loads - the final depths are shown on Figure 2.

ANCHORING OF BLOCK 6 — FISSURE GROUTING

Although the cores showed the rock in the chosen anchorage zones to be suitable, fissures and joints were noted and water tests showed a sufficiently large take from the anchor holes for fissure grouting to be considered necessary.

Grouting was carried out in stages up each hole using hydraulic packers — the quantities of grout pumped were not high, indicating that no large voids were present. When set, the grout was redrilled and the grouting proceeded rapidly until water tests showed the anchorage zones to be tight.

ANCHORING OF BLOCK 6 - INSTALLATION

The anchors consisted of 3 No. 40mm 'Macalloy' bars. A 225 mm long serrated cast iron 'Macalloy' sleeve was fixed on the bottom of each anchor. As the anchors were being installed, 'Denso' tape was wrapped around the bars for approximately 3 m above the bonded zone and above this silicon grease was sprayed onto the bars. The presence of the 'Denso' tape immediately above the bonded zone ensured that there would be no transfer of bond stress between the anchor tendon and the rock at a higher level than was desirable.

The anchors were bonded to the rock over a 9.7 m anchorage length using a 0.45 w/c low alkali cement grout with a 14-day cube strength of 35 MN/m^2. The grout was pumped through a 13 mm dia pipe that was temporarily fixed to the lower end of the anchor sleeve and raised as grouting proceeded.

As a test of the ability of the foundation rock to carry successfully the anchor loads at the chosen depths the centre anchor was stressed before drilling of the two outer anchors had been completed.

Stressing on all three anchors took place in 10 t increments up to 90 t with readings being taken of the anchor bar extension, the load cells, the electrolevel and the inverted pendulum after each increment. Readings were plotted so that anything unusual would have been detected. After a waiting period of approximately 15 minutes, the load was increased by a further increment. At 90 t the load was held for 30 minutes before being relaxed to the design figure of 85 t and locked off. Readings of extension were taken before and after each stressing operation.

When the two anchors had been stressed the annular space above the bonded length of each anchor tendon was filled with Polythixon FR special grout to prevent corrosion.

INSTRUMENTATION INSTALLATION

Instruments were installed in and adjacent to Block 6 to monitor behaviour during the stressing of the anchors and at regular intervals after the stressing. The individual sitings of the instrumentation are indicated diagrammatically on Figures 2 and 4.

In Block 6 itself, the following instruments were installed:-

(i) Vibrating wire load cells at the head of each anchor.

(ii) A vertical extensometer down from the walkway bridge to a level 1.5 m below the deepest anchor.

(iii) An inclined extensometer from a point two-thirds down the exposed downstream face of the dam through the cut-off trench into the foundation zone immediately upstream of the dam.

(iv) An inverted pendulum vertically down from the walkway bridge to the middle of Lift No. 7.

(v) Two electrically operated piezometers inclined down from the downstream face of the dam into the middle of Lift No. 5, the individual piezometers being positioned 1.5 m and 3.4 m from the upstream face of the dam.

(vi) Twenty four mechanical strain gauge positions (three points per position) on the downstream face of the dam distributed across the lift joints and the vertical joints at each side of the block.

In addition to the above, four electrolevels were positioned on the walkway piers, each electrolevel spanning the joint between adjacent blocks for Block Nos. 4 to 8.

RELIEF DRAINAGE HOLES

The relief drainage works consist of 76 mm diameter holes drilled from the roof of the drainage gallery up to near the crest of the dam, and from the bottom of the gallery down to elevation 3.4 m as shown on Figure 5.

The upward holes are vertical with respect to the dam elevation in Blocks 10 to 18, and were drilled generally at 3.4 m centres. Holes inclined with respect to the dam elevation were provided for Blocks 19 and 20. The downward holes are vertical with respect to the dam elevation, again at 3.4 m centres, and were extended into the foundation rock to supplement the existing foundation relief drainage system.

The upward holes were capped with removable caps at their exit points near the crest. Both the upward and downward holes were provided with suitable capped ends in the gallery to facilitate measurement of flow.

FUTURE MONITORING

Following the remedial works it is intended to continue surveillance of the dam, principally by visual observation of the downstream face and of any movement at the crest of the dam and by sonic testing at three year intervals. This will be supported by records of the flow from the drainage holes and by readings on the piezometers and other instruments.

Following the sonic testing of the dam, electrically operated piezometers were installed in certain sections of the dam - one was installed in Block 6, four in Block 11 and two in Block 17. Unfortunately, due to two strikes of lightning which occurred during the period of the remedial works all the piezometers in the dam were damaged. Prior to this, it had been intended to have additional piezometers installed in Block 6 and Block 11 so that more complete information was available on internal uplift pressures. Due to the expense involved, however, it was decided not to replace the damaged piezometers but to install two additional ones on Block 6, so that the effect of the grout curtain can be monitored.

If the sonic tests and visual inspection indicate deterioration in any block in the future, piezometers would probably be installed and observed for a period before deciding on the remedial works. Due to the success of the anchor installation, it is likely that this method would be adopted in the future.

ACKNOWLEDGEMENTS

The Authors are grateful to the Directors of the Jersey New Waterworks Company for permission to present this paper.

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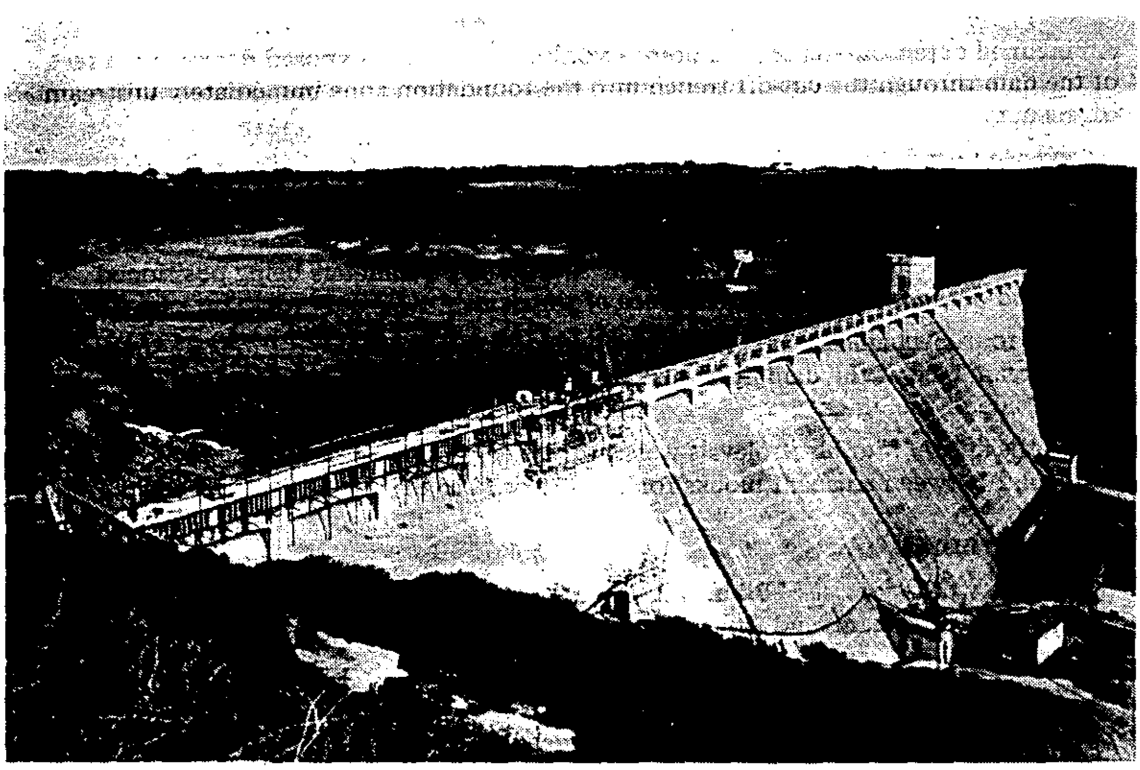

Fig. 1 Val de la Mare Dam

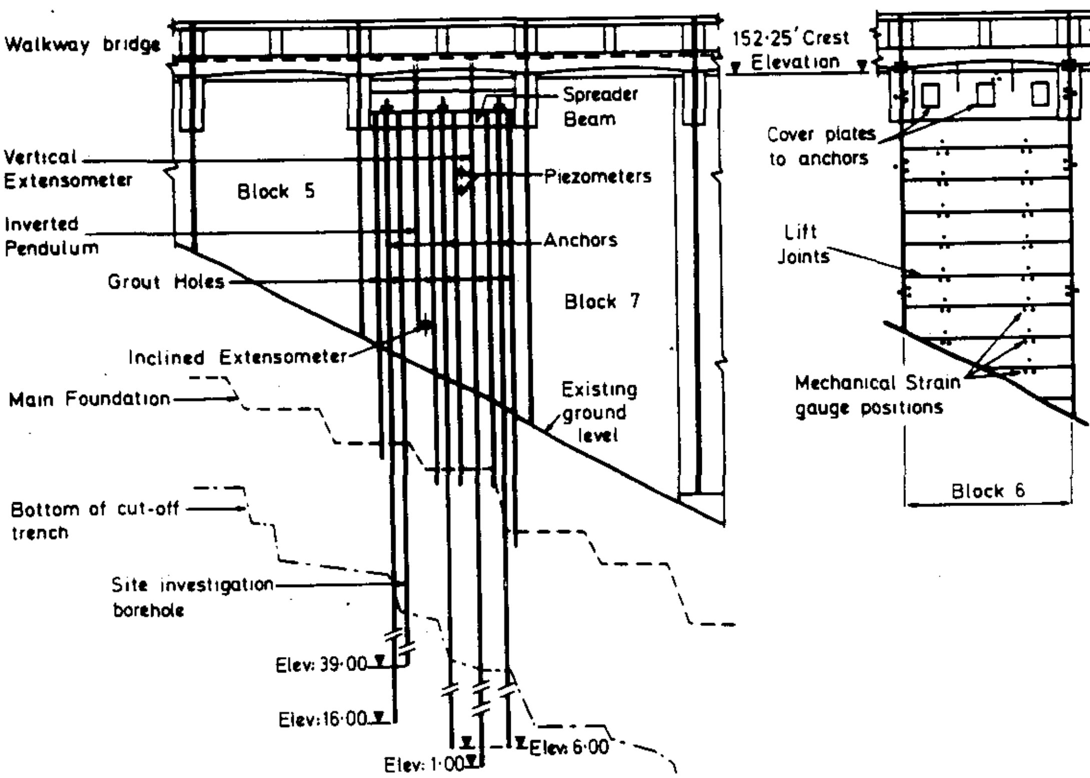

Fig. 2 Locations of Anchors, Grout Holes and Instrumentation in Block 6

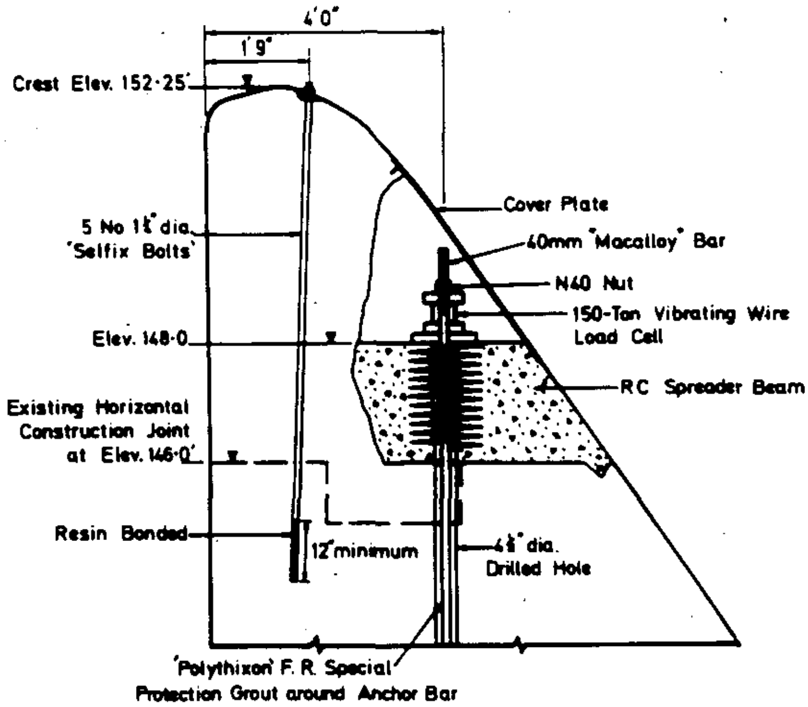

Fig. 3 Basic Detail of Spreader Beam and Tendon Anchorage at Crest of Block 6.

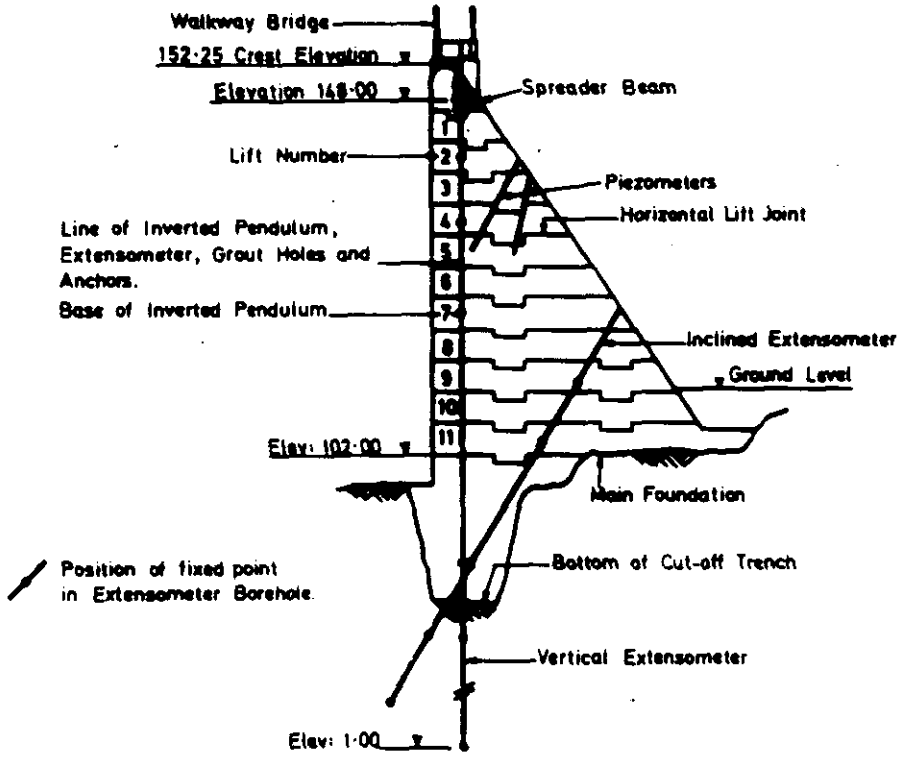

Fig. 4 Section through Block 6 showing Arrangement of Anchors, Grout Holes and Instrumentation

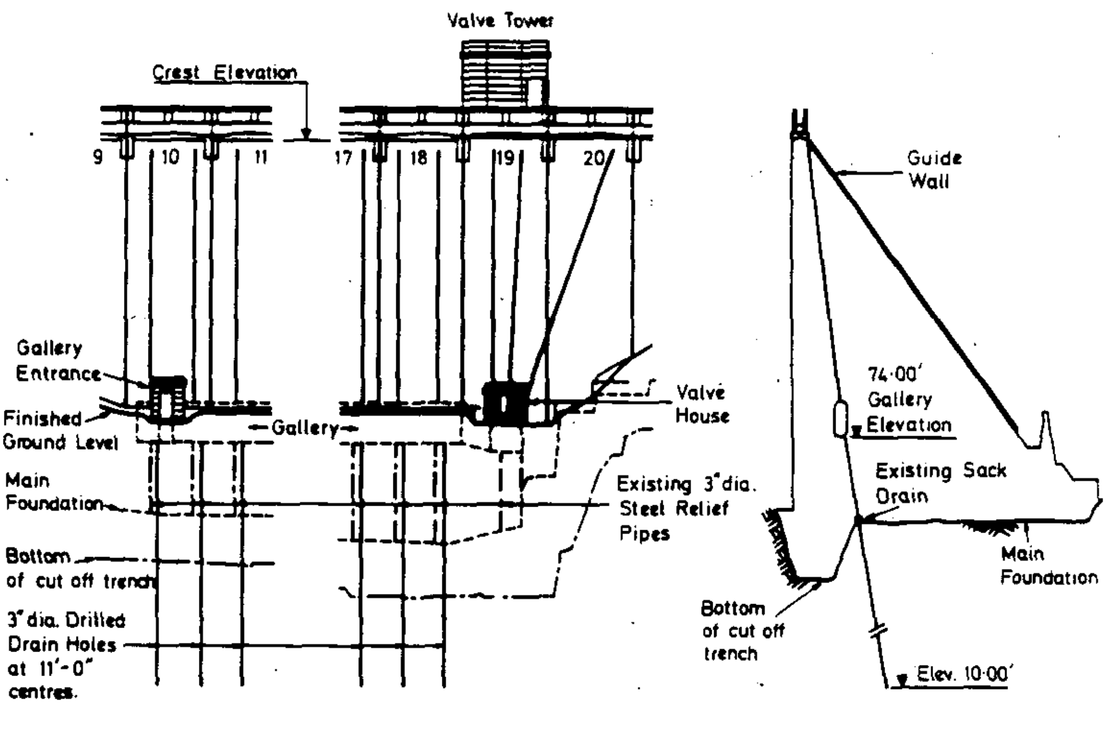

Fig. 5 Relief Drainage Holes

ALTNAHEGLISH DAM, CO. LONDONDERRY, NORTHERN IRELAND

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PARTNER,
WATERHOUSE AND PARTNERS

SYNOPSIS

Altnaheglish Dam is a mass concrete gravity structure 36 metres high and 110 metres long at the crest. The dam was constructed in the period 1930 to 1934 and designed before the introduction of the Reservoirs (Safety Provisions) Act,1930.

Spalling of the face which had been reported in 1952 had increased on an inspection in 1959.

Investigations into the condition of the dam were carried out between 1960 and 1964. Concrete samples were tested for strength, chemical deterioration etc, as was the foundation rock and aggregates.

A new underdrain system was drilled and piezometers installed in the body of the dam observed from 1964 to 1968 to check uplift conditions.

INTRODUCTION

HISTORICAL

Altnaheglish Dam formed part of the Banagher Water Scheme,for which the authorising Act was passed in 1918.

Mr Criswell, MICE, who had been Resident Engineer on earlier works became responsible for the design and construction of the dam in early 1929, with Mr W S Osman, MICE, as his chief assistant who also acted as Resident Engineer. Mr E Sandeman, MICE, was consulted on the design of the dam and inspected the rock foundation.

The dam was constructed by direct labour under the direction of Mr W Criswell and was finally designed as a mass concrete structure with displacers, and although designed to act as a gravity structure was constructed to a radius of 153 metres on plan.

The concrete was generally a 6 : 1 mix except for the facing to the steps and the top portion of the dam which was 5 : 1. The cement was a slow setting Portland cement (initial set 2¼ hours) and the aggregate a quartz mica gneiss was obtained from a quarry adjacent to the site and crushed on site (1). The coarse aggregate was graded from 37 mm down and the fine aggregate produced from the same source via granulators, 'meal' and dust being excluded. No water content was given in the original specification other than the requirement that 'Only sufficient water is to be used to make the mixture plastic but not sloppy'. The concrete was placed in lifts approximately 2 m high and averaging 90 m^3 per pour.

Strengths of cubes taken on site averaged 13.5 MN/m^2 for the 5 : 1 mix and 11.5MN/m^2 for the 6: 1 mix. These strengths were low for the mixes used and indicate a very wet mix. This was commented on during the discussion on reference (1).

The underdrainage system comprised several independent systems of drains 100 mm square in section, formed of concrete or concrete bricks with precast covers and with at least two rising pipes from each system terminating in the gallery or steps to allow for observation.

The general plan of the dam is shown in Figure 1.

INVESTIGATION

PRELIMINARY

A general report (2) on the water supply for Londonderry City was prepared in 1952,in which it was noted that 'some disintegration of the downstream face was observed, and it was recommended that minor repair measures should be considered.

Extensive spalling of the face of the dam was reported by the then City Engineer and Surveyor, Mr J C Mackinder, OBE ERD MIMunE, in late 1959, and the Author's firm were instructed to investigate and report on the position. An initial inspection of the more readily accessible areas was carried out in early 1960 and showed that spalling had taken place generally over the whole downstream face of the dam and varied from 15 to 80 mm in depth. In the first stages of the investigation the chemical and mechanical properties of the concrete were examined. A complete record of the lifts of concrete placed during construction was available giving extent, position and date. From these it was possible to identify with a fair degree of accuracy the lifts from which the original test cubes had been taken. A series of cores were obtained by diamond drilling including a number from the same lifts as the original test cubes. These were sent to the testing laboratory responsible for the original tests.

No chemical deterioration of the concrete was found, but the tests indicated that for the samples taken the mix varied from 5 : 1 to 10 : 1 and confirmed the visual observations of poor grading, i.e. too much fine material.

The results of the tests of the core samples after a period of 27 to 30 years gave average strengths of $29.3 MN/m^2$ for the 5 : 1 mix and $29.7 MN/m^2$ for the 6 : 1 mix, giving an average increase in strength over the period of some 100%. The tensile strength of the samples tested averaged $1.2 MN/m^2$.

Samples of the foundation rock, particularly at the abutments, were tested and gave strengths between 17 and $94 MN/m^2$ as compared to 90 to $101 MN/m^2$ on samples taken during construction.

Samples of the aggregate were tested for moisture movement and found to be in the range 0.06 to 0.07% for movement perpendicular to the rock strata. This is a high result, as some South African sandstones with moisture movements from 0.04 to 0.08% are reported to have caused rapid and severe deterioration of concrete [3]. The deterioration caused by moisture movement is mainly dependent on alternate wetting and drying cycles. It was noted that over a period of time the downstream face of the dam never really dried out.

A more detailed examination of the whole downstream face of the dam from the scaffolding erected for the diamond drilling showed that the average loss by spalling over the whole face was between 12 and 25 mm over a period of some 27 years. It was felt that unless a marked increase in the rate of loss took place no remedial action was necessary, and this has been borne out by the experience of the last 14 years.

GENERAL

At this stage it was apparent that the continuous damp on the downstream face was not an effect of the notorious Irish weather but probably due to seepage through the body of the dam, and that the problem could be much more serious than surface damage to the downstream face. Tests were carried out with various colours of dyes injected at carefully controlled pressures via the holes drilled in the dam, with results varying from little apparent seepage to a case where 3,400 l were used in one hole in 22½ hours, and on one hole on the spillway no pressure could be applied as the hole could not be filled. Colour appeared on joint planes up to 6 m from the point of application, and it was apparent that there were many leakage paths in the dam.

A series of calculations for the stability of the dam was carried out using the actual concrete densities obtained from the cores. They took into account the report of the Interim Committee on Floods [4] and used different assessments of uplift factors.

With minimum uplift taken at 50% on the upstream face reducing to zero at the downstream face with acute' catastrophic flood' conditions the resultant fell outside the middle third by 0.8 m. With 100% uplift reducing to zero in the last 1.2 m from the downstream face the resultant fell 4 m outside the base of the dam. This obviously revealed a serious condition and steps were immediately taken to reduce the Top Water Level by a minimum of 3 m and to modify the scour arrangements so that the scour valve could be opened fully. A general warning system was set up with the co-operation of the local meteorological office and the police.

An examination of all the run off and level records etc of the dam since 1934 showed that in the last 27 years the maximum recorded flood had not exceeded that taken in the original design, i.e. 25.5 m^3/s, although it had approached this figure on about three occasions.

REMEDIAL WORKS STAGE 1

In order to try to increase the stability of the dam by reducing uplift it was decided to carry out grouting, and to install a system of piezometers to measure the uplift before and after grouting.

The following works were carried out in 1964 :

1 Holes were drilled at 1.5 m centres along the crest of the dam, terminating some 500 mm above the junction of the dam and rock foundation. These were grouted under controlled pressure conditions (see Figure 2).

2 On the completion of step 1, alternate holes were redrilled and carried down into the foundation rock in stages, being grouted and tested at each stage to provide a grout curtain of the same height as the dam. In carrying out this work tests were carried out on the underdrainage system which involved emptying the stilling pond at the foot of the dam, opening up an original sump in the pond, and uncapping the ends of certain underdrains.

It was found that the underdrains covering some two thirds of the base of the dam at its deepest section were blocked and inoperable, those remaining open being in the area of the toe of the dam. During grouting these particular underdrains were flushed out continuously.

3 During the carrying out of the grouting programme a series of new underdrains were drilled into the base of the dam. As each section was grouted these holes were kept clear by a continuous circulation process during the remainder of the grouting and in the end covered rather more area than the original system.

4 A series of 12 piezometers were inserted in holes drilled in the dam to provide coverage across the base which, with vertical observation holes in the crest of the dam, would provide a measure of the uplift, particularly in the highest blocks of the structure.

5 A number of exploratory holes were drilled into the rock abutments so that an assessment could be made of any possible arch action. Core recovery was good in the holes drilled in the South abutment, but the North abutment did not prove so good and further holes were drilled to obtain more information.

RESULTS STAGE 1

The amount of cement used in grouting of the dam proper was minimal and could be measured in grammes per square metre, average 200 gms/m^2.

In the rock section there was a three to fourfold increase in the amount of cement used, but even this amount, 600 to 800 gms per m^2, was low.

During the grouting and installation of the piezometers the water level in the dam had been lowered 9 m below Top Water Level. Once the piezometers had settled down there was no appreciable variation in the readings before and after grouting. After completion of the grouting the water level was allowed to rise by some 4.5 m over a period of 20 days, and again no appreciable variation in the piezometer readings occurred. The water level in the vertical observation holes in the crest of the dam, which were situated 1.4 m from the upstream face, followed the variation of water level in the reservoir very closely and quickly.

The exploratory holes in the abutments indicated that the South abutment was founded on solid rock to a reasonable depth, but those on the North abutment indicated a shattered rock to some depth, with seams of clay infilling the cracks. Drilling in this material was difficult and core recovery varied from 10% to 90% (see Figure 3).

The amount of grout taken in these particular holes was small and it was felt that, whilst the abutment was dubious from a strength point of view as regards arch action, there was no evidence of any noticeable degree of leakage around this end of the dam.

REMEDIAL WORKS STAGE 2

The piezometer readings and water levels in the observation holes on the dam crest were observed daily for a period of 12 months, and whilst the level in the observation holes continued to follow the water level of the reservoir only minor variations were noted in the piezometer readings. During this period an induced head was applied to all the piezometers to ensure their correct operation and in all cases the readings reverted to their original levels within a short time.

During this period flow nets had been produced for each block and when compared with the piezometer readings gave a reasonably close correlation (see Figure 4), the latter giving an uplift figure at the deepest sections of the dam of approximately 75% at the water face to zero at the downstream face.

At this stage three methods were considered for improving the stability of the dam :

1 Alterations to the spillway to reduce the flood level by some 2 m.

2 Insertion of post-stressed anchor cables into the rock foundation.

3 Provision of siphons to limit the reduction in Top Water Level and thus in storage capacity.

In the end the decision was taken to keep the spillway at its present width, which involved cutting it down by 1.9 m. Ancillary works included a reinforced concrete bridge across the spillway and a remodelling of the measuring weir downstream of the dam to give more accurate assessment of flood flows.

ACKNOWLEDGEMENTS

The Author wishes to acknowledge the assistance rendered by the then City Surveyor and Engineer, Mr J C Mackinder, OBE ERD MIMunE and his staff, on whom the main burden of taking readings and, at one stage, of keeping a 24 hour watch on the dam, devolved, and also to the former City of Londonderry Corporation, as the then responsible Authority.

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3 Stutterheim N (1955) *Deterioration of Concrete* South Afr Arch Jnl 40 · No 1

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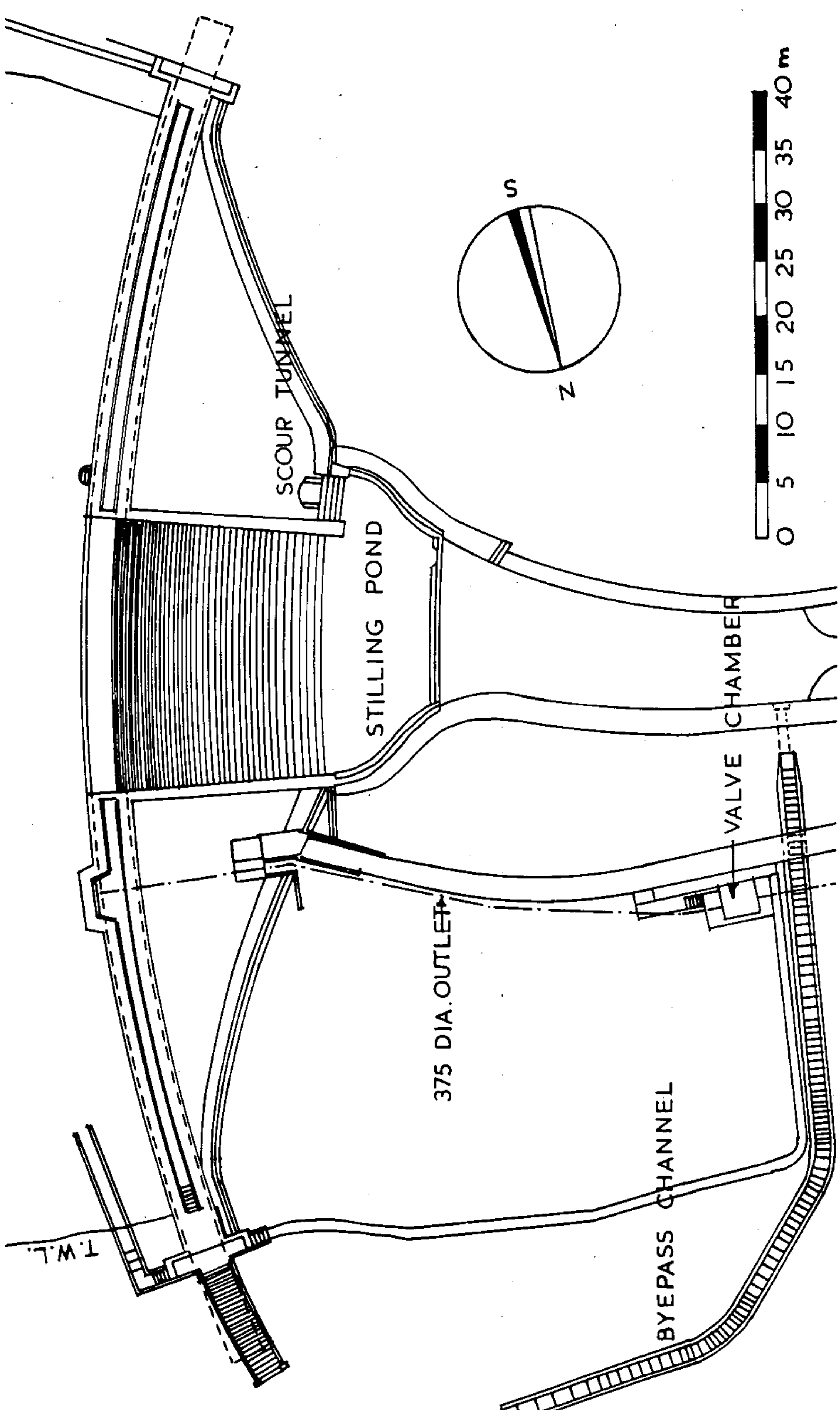

Fig. 1 Plan of Altnaheglish Dam

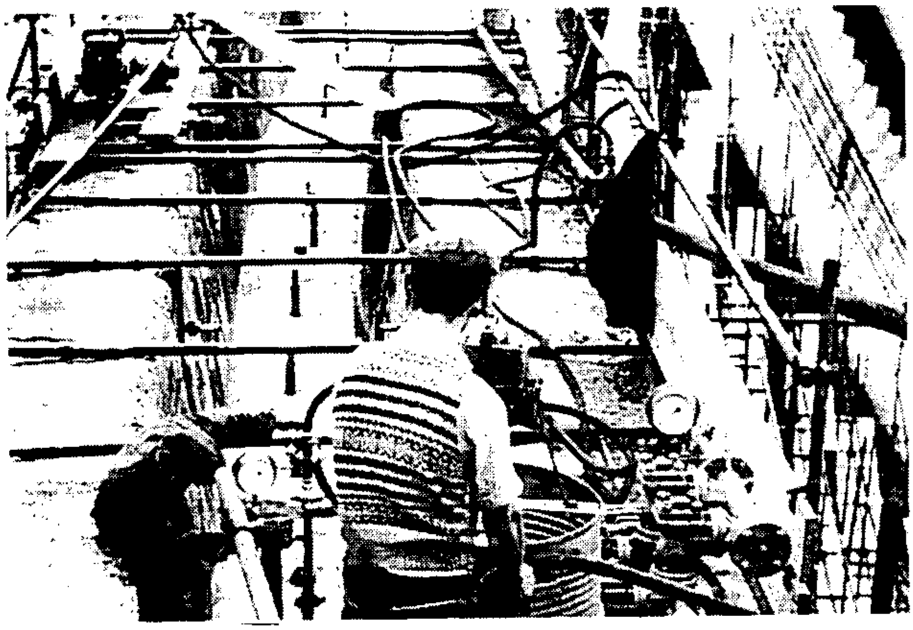

Fig. 2 Grouting in progress

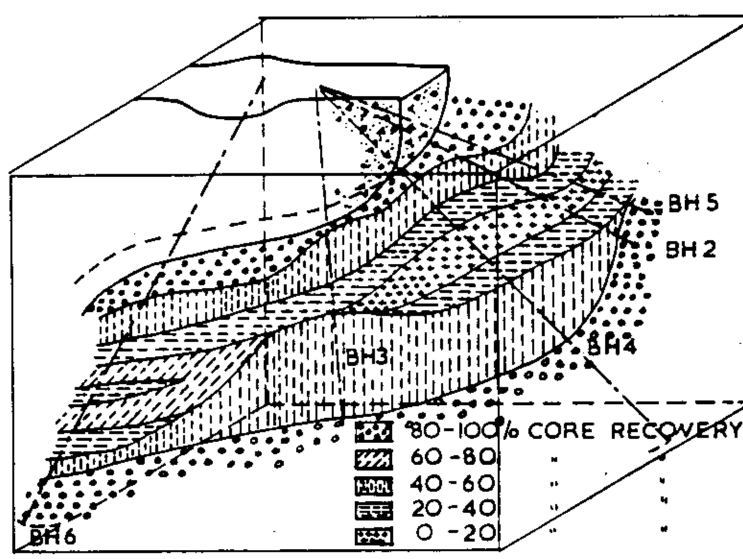

Fig. 3 Block Diagram North Abutment

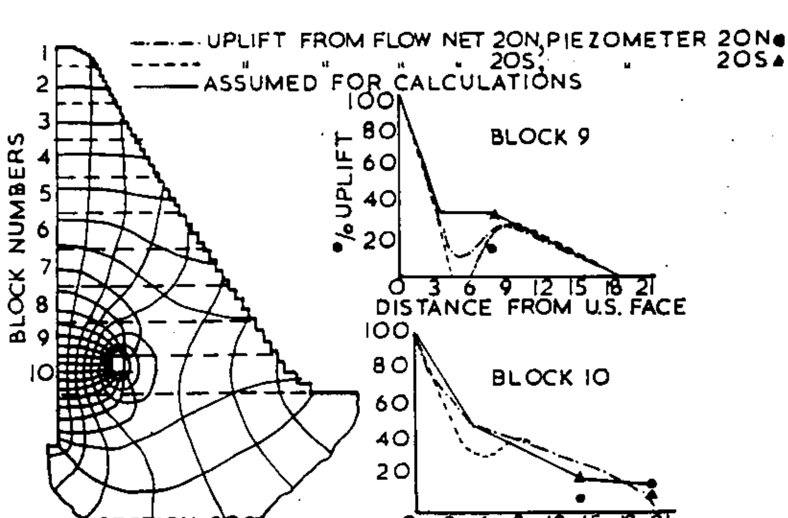

Fig. 4 Flow Net and Uplift Diagram

SEISMIC VELOCITY INVESTIGATIONS OF CONCRETE DAMS IN GREAT BRITAIN

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SYNOPSIS

Longitudinal wave velocity measurements made inside concrete dams using hammer seismic equipment provide a rapid and inexpensive means of verifying the in situ mass properties of concrete. Observations at seven dams demonstrate the anisotropic influence of construction joints and variations in concrete quality on the measured in situ velocity.

INTRODUCTION

Ultrasonic methods of non-destructive testing now form a standard technique for verifying concrete quality, particularly when precast components are being manufactured on a production line basis. Such techniques can also be readily applied to cast in situ concrete and have been used successfully in mass concrete dams (1). One of the practical problems arising from the application of such methods is that the relatively high frequency (50 to 200 kHz) waves tend to be damped so the method cannot be readily applied over long path lengths. An alternative method which overcomes this difficulty involves the application of hammer seismic equipment originally developed for shallow geophysical investigations. Such equipment, in relying on a hammer blow at a low frequency (5 to 30 Hz) energy source, remains a non-destructive technique and can be used over maximum path lengths of 50 m to 100 m but, in good quality concrete, cannot be used over concrete thicknesses of less than about 10 m. As part of a study of the seismic wave velocity in rock foundations of dams in Great Britain (2), data was also collected systematically from the concrete of seven dams; the paper reports the results of these investigations.

PROCEDURE

The velocity measurements were made using an MD-1 Refraction Seismograph, which is a commercially available single-channel seismic instrument. The energy source used was a hammer blow directly onto a concrete surface. Attached to the hammer is a spring switch connected to the seismograph by means of a cable which closes a circuit at the time of hammer impact. The timing mechanism in the seismograph operates on a binary system with a minimum time readout of 0.25 ms. The first arrival of energy at the geophone, which rests on the concrete adjacent to the seismograph, closes the timing circuit enabling the time lapse between hammer blow and arrival to be recorded. The seismograph has a gain control which must be adjusted to ensure that the 'first' arrival, associated with the longitudinal wave arrival, is correctly identified and interference of background noise suppressed. By moving the hammer point progressively away from the geophone, a time-distance graph can be constructed from which the mean longitudinal wave velocity can be calculated.

Measurements were made at the seven concrete dams listed in Table 1.

Name	Year of Completion	Height (m)	Type
Altnaheglish	1934	42.1	gravity
Blackwater	1909	26.5	gravity
Glen Finglas	1965	39.6	gravity
Laggan	1934	54.4	gravity
Monar	1963	39.0	cupola
Stithians	1965	38.4	arch & gravity
Wet Sleddale	1967	29.3	gravity

Table 1 Information on Dams Investigated.

At three dams (Glen Finglas, Monar and Wet Sleddale) measurements were made during construction, at one dam (Stithians) they were made both during and after construction and, in the remaining three cases, the measurements were made thirty years or more after completion.

The procedure at each site was arranged so that longitudinal wave velocity measurements could be made both horizontally, parallel to the construction joints between pours but across the vertical contraction joints separating individual blocks, and vertically where possible. The vertical measurements were made between an inspection gallery and the crest of the dam (Altnaheglish, Laggan and Stithians) but in other cases, particularly during construction, measurements were made up the vertical upstream face of the dam. At Blackwater, where the reservoir was somewhat drawndown at the time, the hammer blows were made from a boat. The horizontal measurements were made along galleries, across the dam crest, along the upstream face during construction,or across the top of uncompleted blocks.

At two of the dams, Altnaheglish and Blackwater, deterioration of the concrete has been identified and remedial works carried out. At Altnaheglish the concrete was made from a local aggregate source including a weathered biotite schist which has contributed to deterioration. At the very much older dam at Blackwater, on the other hand, the deterioration may be the consequence of unsophisticated concrete control at the time of construction. It is of interest in this latter case to note that the effect of corrosive action on the concrete has been known for a considerable period (3). The concrete at the other sites, as far as could be established, was in good, if not excellent, condition.

At a number of sites small samples of concrete were collected for laboratory-scale velocity measurements which were made with an ultrasonic tester; the observed velocities ranged between 4000 and 5200 m/s as might be anticipated from general considerations.

DISCUSSION OF RESULTS

The original objective of measuring velocities in the dam concrete was to ensure that a record was kept of the mass properties of the dam in contrast to the rock of the foundation. Frequent observations were made of the in situ velocity along dam toes and these consistently indicated in situ velocities in the range 3 800 to 4 700 m/s. At Altnaheglish Dam, however, the condition of the concrete prompted a more comprehensive study based on horizontal, vertical and inclined measurements (made down the downstream face of the dam); these data are summarised in Table 2.

Orientation of Measurement	Velocity m/s (No of Measurements)	
	Altnaheglish	Blackwater
Vertical	2500 (3)	2070 (12)
Horizontal	3350 (3)	3410 (3)
Inclined	4200 (2)	5090 (2)

Table 2 : Summary of Longitudinal Wave Velocity Measurements made at Altnaheglish and Blackwater Dams.

These observations indicated very clearly the anisotropy of the in situ concrete, which was presumably a response to the damping influence on the sonic waves of the horizontal construction joints separating individual pours. At that time it was suspected that the anisotropic velocity distribution was a response to accentuated deterioration of the concrete along individual construction joints. However, in the event, all the dams investigated have demonstrated this anisotropy extremely clearly; the essential results are summarized in Figure 1.

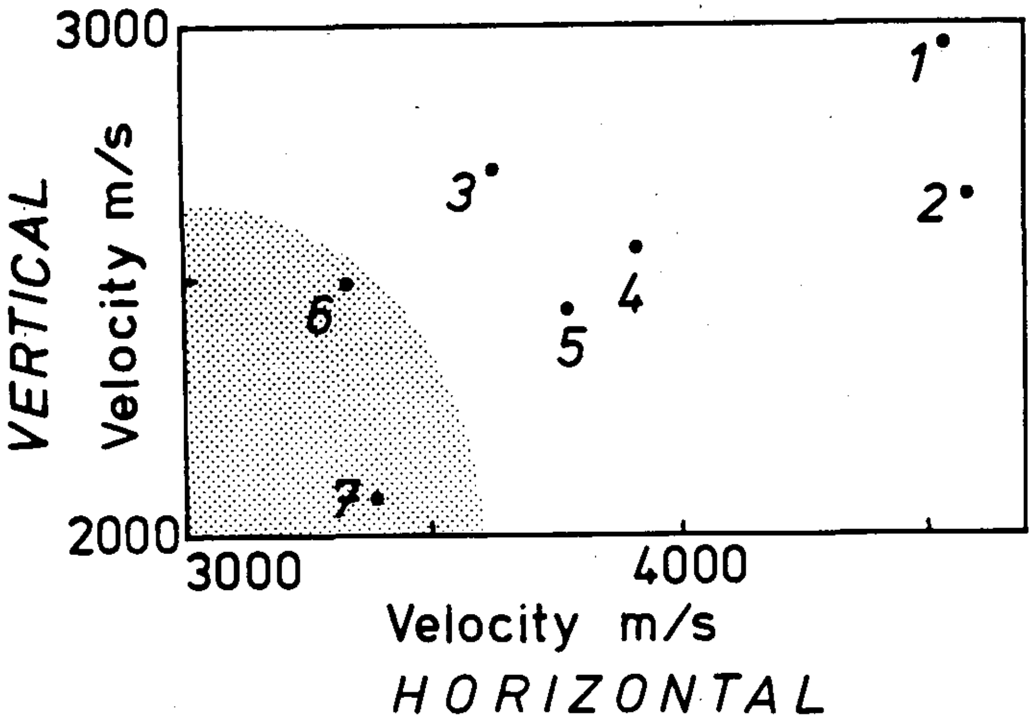

Fig 1: Relationship between Horizontal and Vertical Velocities.

[1. Laggan, 2. Glen Finglas, 3. Monar, 4. Stithians, 5. Wet Sleddale, 6. Altnaheglish, 7. Blackwater. (Area stippled corresponds to probable field of suspect concrete)]

The ratio between the vertical and horizontal velocity has a mean value of about 1.5:1 and this does not appear to be sensitive to the relative quality of the concrete.

Recognising this influence of the horizontal joints, measurements were made across vertical contraction joints which demonstrated that the maximum delay likely to be imparted by a single such joint is about 0.25 ms. The maximum influence of such widelv separated joints, therefore, in the best quality concrete would be rather less than 10% on the measured velocity. In concrete of average or mediocre quality the influence on the measured velocity would be significantly less. Calculations would suggest that the delay imparted by individual horizontal construction joint to the longitudinal waves is almost identical to that imparted by the vertical joints.

A further point arose at Monar and Stithians dams where it was established that the velocities measured in thin arch sections were noticeably less than those determined in mass concrete dams.

The major objective of ultrasonic testing of concrete is to establish a basis for quality control ; Breuning and Roggeveen (1) have, for example, suggested the following scale which has been adapted to metric units:-

Velocity (m/s)	Concrete quality
> 4570	Excellent
3660-4570	Generally good
3050-3660	Questionable
2130-3050	Generally poor
< 2130	Very poor

On the basis of the horizontal velocity measurements (Fig 1) above, the average velocities measured at Altnaheglish and Blackwater dams fall well within the 'questionable' class, whereas the concrete of the other five is 'generally good'. However, a very different conclusion can be derived if the vertical measurements are considered in isolation. For this reason a field of 'suspect concrete' is stippled in Fig 1 which relates to both horizontal and vertical velocity. It may be concluded that, if in situ seismic or ultrasonic measurements are so orientated or are over a scale which incorporates construction joints, then directional measurements should be carried out if the data is to be used for the purposes of quality assessment.

There is a further point of interest which can be derived from the observations at Altnaheglish and Blackwater. Measurements made parallel to the downstream dam face at both sites indicated exceptionally high velocities compared to other observations. These results presumably indicate the presence of a high velocity layer resulting from the precipitation of calcium carbonate (leached from the cement of the dam) in and on the concrete immediately adjacent to the downstream face. At both dams the downstream faces were streaked with such precipitates which took the form of thin, brittle and platy deposits.

CONCLUSIONS

Hammer seismic equipment provides a rapid and economical method of in situ assessment of concrete quality in dams. Horizontal construction joints have a significant influence on the measured longitudinal wave velocity and this situation contributes to significant velocity anisotropy. Despite some limitations, the technique has considerable potential as a means of assessing the concrete in old dams and in the regular inspection of concrete structures.

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LEAKAGE AND STABILITY OF THE UPPER GLENDEVON DAM OF THE FIFE REGIONAL AUTHORITY, SCOTLAND

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SYNOPSIS

The Upper Glendevon concrete gravity dam, 45 m in height, was completed in 1955. Following impounding, leakage was about 25 l/s and measurements indicated unusually high uplift pressures. In 1959/60 the reservoir was emptied, an upstream grout curtain was injected, bitumen seals were placed at the vertical joints and other grouting took place. In 1968, again leakage was substantial with indications of high uplift pressures. Cores showed little evidence of grout and disclosed several lava flows, faults, other discontinuities and some calcite in the underlying rocks. Leakage approached 10 l/s . Observations of uplift pressures have continued revealing some slight progressive changes. Pressure relief measures are being installed. The Paper relates the circumstances to stability of the dam at stages in its history.

BRIEF HISTORY

TO OPENING OF THE RESERVOIR, JUNE 1955

The 390 m long dam is of traditional cross-section for a concrete mass gravity dam. It has a 0.65:1 downstream batter and a small, verying, upstream batter, giving a base to height ratio of 0.76 at its maximum height of 45m. For a width of 30 m where it is of this height there is an overflow section and a stilling basin. Construction commenced in 1950.

It appears that little investigation was undertaken of the rocks on which it is founded, and that they were excavated to shallow depths only except in a narrow cut-off trench which varies between about 2 m and 12 m deep. No grout curtain was provided. The waterstop between the monoliths of the dam was a single copper strip. Generally, concrete lifts were 0.6 m deep. The concrete was harsh and segregated badly and an air entraining agent was introduced into the concrete in 1953.

It is recorded that in 1951, when cleaning up of foundations exposed large boulder masses with clay joints, this was accepted as perfectly suitable for the dam.

Generally, photographs give no obvious indication that the concrete was vibrated or cured, or that the surfaces of lifts were treated, other than by brushing away loose stones before applying successive lifts. The concrete is shown generally within lifts to have been stopped at incompletely compacted faces approximately parallel to the upstream face and lying at the concrete's natural angle of repose, at the tops of the lifts to have sunken keys roughly parallel to the upstream face, and to have demonstrated frequently the seepage of water through it during construction.

Water was impounded to heights of 4 m by June and 13.5 m by October 1954. By July 1954, water had appeared in such quantity during the construction of the stilling basin that sub-drainage was provided. Water was showing on the downstream face of the dam at nearly all those vertical joints between monoliths which extended below the water level when a height of 25 m was impounded in April 1955. At about that time instructions were issued for drill holes to be put in at ground level at the joints, and a 150 mm tile drain laid along the toe of the dam collected the water coming through the drill holes and took it to the stilling basin. Some joints were caulked progressively upwards at the downstream face to contain leakage within the dam.

A second Preliminary Certificate issued on 3rd May 1955 under the Reservoirs (Safety Provisions) Act 1930 permitted the reservoir to be filled to overflow level. The Official Opening took place on 8th June 1955.

OPENING TO FIRST STATUTORY REPORT, ISSUED DECEMBER 1968

Correspondence about leakage proceeded between the parties concerned. The Final Certificate under the 1930 Act was issued on 24th March 1956. Leakage was measured as nearly 25 l/s. An independant engineer, recommended by the President of the Institution of Civil Engineers at the request of the owners, reported on the dam in 1957, including conclusions that the leakage was most certainly excessive, that records should be obtained, and that there was no cause for immediate alarm that the dam would not remain stable for a ten-year period. He recommended that the leakage be reduced in the joints and also, below the dam, by grouting.

Fluorescein tests from April 1956 demonstrated numerous and varied water paths through the concrete. Boreholes were drilled into the concrete and rock, substantial water flows and pressures were obtained from the holes and interconnections were revealed. At the change from concrete to rock the drilling rods tended to drop suddenly for distances as much as 150 mm and on occasion muddy water was discharged from the holes. Early in 1958 uplift measurements commenced and an indication of results obtained at Monolith 15 is given in Figure 1.

Analysis of water samples showed much lower amounts of organic matter and less ammonias in the waters that had passed through the rock, which suggested filtration by the rock.

Following a report in 1958 by another independent engineer (which recommended keeping the reservoir normally 4 m below overflow level) remedial measures, designed and supervised by that engineer, commenced in 1959 after the reservoir had been emptied. They included (i) curtain grouting, blanket grouting to a depth of 4.5 m downstream from the curtain to seal any fissures there, stitch-grouting near vertical joint seals and additional grouting of porous concrete and, (ii), drilling for, and pouring, bitumen seals at all vertical joints and grouting beneath the seals.

Following restricted impounding some minor leakages were troublesome and additional grouting was undertaken, some within the downstream face of the dam, and included the use of sodium silicate. Some vertical joints at the face were grooved to confine leakage. In April 1960 it was reported that the leakage had been reduced to a fraction of 1% of what it had been and the remedial works were certified as having been substantially completed by 1st May 1960. The quantities of cement injected were shown to have been as follows:-

Curtain grouting	520 t
Blanket grouting	200 t
Stitch grouting	10 t
Bitumen seal plugs	40 t
Porous concrete	300 t

In April 1961 it was reported that quite a lot of the joints were at that time showing dampness over a considerable length. Further Certificates expected were not issued, and the filling of the reservoir was left 'to some extent experimental'.

Apart from final payment being made in April 1962, after protracted negotiations, and the statutory entries in the Record required to be kept under the 1930 Act, no subsequent developments are recorded.

The Author was appointed in March 1968 to make an inspection of the reservoir and then became associated with it for the first time. It was observed that leakage was substantial and that there were indications of high uplift pressures. In relation to assessing the stability of the dam, recommendations were made to investigate the rock and the uplift pressures beneath the dam and to record leakage continuously. At that time, available records of the dam were few, and one recommendation was that enquiries should be made to procure details of the original and remedial works and any calculations that had been made.

DECEMBER 1968 TO JULY 1975

Records produced in consequence of the 1968 Statutory Report enabled a history of the dam to be compiled and it has been outlined above. An investigation of uplift pressures at the dam and of the underlying rock was, however, necessary. To determine uplift pressures piezometers were installed (i) at the base of Monolith 17 at locations 1 to 6 as shown in Figure 2, (ii), at Monolith 18 as piezometer locations 1, 3, 5 and 6 only, and (iii), at Monolith 19 as piezometer locations 1 to 6 inclusive.

Monolith 18 is central to the stilling basin and Monoliths 17 and 19 lie at either side. In drilling the required holes cores were obtained, and tests were made throughout to determine where water was seeping into the drill holes. The concrete was found to contain occasional areas of voids (some of which were partially filled with grout) and, in sundry places, pieces of wood. The cores when handled yielded sand grains. Construction joints were generally sound, although occasionally they were dirty and showed sand particles. The contact with the rock was similarly variable showing, in different places, both good and poor bond, instances of the latter containing some dirty fines. A porous tip protected by a sand filter was installed at the base of each hole. Each tip was connected to a 20 mm pipe grouted into the hole and leading to the face of the dam through which to 'dip' the standing water level.

Tests on samples of concrete cores gave the following results:

Tensile strength	:	0.3 — 1.9	MN/m^2
Compressive strength	:	14.1 — 48.2	MN/m^2
Density (dry)	:	2.10 — 2.34 (avg 2.21)	t/m^3
Density (saturated)	:	2.22 — 2.42 (avg 2.33)	t/m^3

Of five tensile test samples containing a construction joint (inclined to direction of stress) failure occurred at the joint in only one instance.

The main investigation of the underlying rock was undertaken by boreholes drilled from the part of the stilling basin close to the dam. All of the holes produced water under artesian pressure and showed inflow of water throughout their entire lengths in the stages tested. The largest inflow was from borehole 17, which passed from the stilling basin into the area downstream from Monolith 17 and penetrated porous and jointed rock between 11.3 and 14.3 m along its length; the flow amounted to 1.4 l/s and when closed off developed a pressure of 7.9 m head of water at the level of the stilling basin floor. These figures reduced as other holes were drilled. The investigation took place when the depth of water impounded varied from 38 to 44m.

The Institute of Geological Sciences reported on the 'geological' holes. The report comments that the rocks at the site are entirely volcanic lavas and that three successive lava flows are evident in the vertical depth of about 13.7 m investigated. The horizontal continuity of the flows is interrupted by a near vertical system of several east-west faults, into some of which there have been later intrusions creating dykes of igneous rock. The throw at some of the faults is of the order of a metre or so downwards at the north side. The appearance of crushing is considered to be the effect of a number of small faults on a type of volcanic rock which is formed from pieces of rock which were surrounded by molten lava. Throughout the lavas, there are calcite and other minerals occupying veins, steam holes and other irregular spaces; open voids were found in borehole 17 at about 13.7 m in, where it is pervious between 11.3 to 14.3m. This porosity was found to be connected to boreholes 2 and 4 at Monolith 17. Between 0.6 and 2.1 m in borehole 18 there was sand, probably from the old river channel, and it is considered that river gravel may have been penetrated by borehole 19, at which there was also some pervious rock. With regard to seepage, the view is expressed that, although over much of the investigation the rocks themselves appear relatively impervious, leakage paths are associated firstly with open voids and secondly with faults, joints and dyke margins. The report does not anticipate that leakage will increase materially in the course of time because, in the pervious areas of rock, no cavities occupied by calcite or siltstone were observed (implying that such materials were never present or have been completely removed by percolating water) and the calcite occurring is in generally very narrow fractures which it completely seals, so that the development of significant leakage paths along these lines is likely to be slow.

No evidence was found of instability in the foundation rocks of the dam. It is commented that there was no grout found in the cores from boreholes across the stilling basin, that a few narrow joints filled with cement were noticed in cores from beneath the dam, and that it is somewhat surprising that the large quantity of grout which is reported to have been injected is not more obvious in the boreholes.

During execution of the investigation work at the dam, in October 1969, the Author noticed for the first time a leakage of water coming up through the bed of the river at about 30 m downstream from the dam.

In 1970 the reservoir level fell to an impounded depth of about 20 m. Photographs taken of the upstream face show many instances of poor workmanship and water continuing to seep out of the dam at joints several days after the reservoir level fell below joint level. At some seepages there was a build-up of lime efflorescence.

In 1971 the observation of movement of the dam was recommended and the systems described by Kitching in Paper 2.7 were subsequently installed. As a consequence of these observations the reservoir was permitted to rise under control up to overflow level and was kept at about that level for several weeks as intermittent observations continued. Arising from the further results unrestricted use of the reservoir has been permitted as movements of the dam have continued to be monitored.

In October 1973 it was recommended to the owners that the high uplift pressures under the downstream toe of the dam and the stilling basin should be slowly relieved by undertaking drilling into the rock beneath the dam in stages. Such pressure relief holes were due to have been provided in the summer of 1974, but the work was postponed on account of the need for the reservoir to be lowered during execution and the possibility of this exacerbating the water shortage that existed at that time. A contract for the work was let in July 1975 for commencement the following month.

LEAKAGE

The substantial leakage which occurred upon impounding was widespread through the dam and the rock beneath. Fluorescein tests across the upstream face indicated major points of entry into the dam. Within the dam, sunken keys and poor bond at the tops of lifts and segregated concrete, particularly along construction joint faces, would contribute significantly to seepage paths through the dam. Similar considerations would have applied at the concrete/rock contact. In the rocks beneath the dam open voids, faults, joints and dyke margins would have permitted the leakage. The continuation of these rock features downstream will have assisted leakage to travel further through the rocks. Similarly, the sub-drainage to the stilling basin will have assisted downstream travel of leakage there. These movements are substantiated by the relatively high uplift pressures obtained at the downstream toe of the dam and the appearances of leakage in the river and in a side stream away from the dam.

The 1959/60 remedial measures appeared to have been very successful in reducing leakage, although in some places to do so proved troublesome and leakage was not eliminated. They will not have affected leakage paths downstream from the dam to any significant extent, but the subsequent uplift measurements suggest the forcing of leakage to below the grout 'blanket' under the dam. After 12 months there appeared to have been a noticeable deterioration and after eight years leakage had increased to about 40% of its early values, i.e. to 10 l/s with the reservoir at overflow level. In the subsequent seven years continuous recording has shown that there have not been substantial changes in leakage, suggesting that some failure of the grouting occurred mostly in the first years after it was completed. It is considered that this would have occurred from organic and other deposits, placed in seepage paths during five years of substantial leakage, having prevented the satisfactory hardening of the grout and having assisted its gradual removal by continuing percolation. It is expected that this effect will have been less significant in the dam than in the rock beneath, where dissolving of some calcite will have been a slight and continuing effect. Protracted seepages from the dam into the reservoir after lowering of its level are indications of substantial leakage paths and voids remaining in the dam.

Leakage is of itself unlikely to be detrimental to stability, but will be of disadvantage indirectly on account of its further opening of the seepage paths and its increasing of uplift pressures downstream from water barriers. Some sand has been deposited from leaks into the stilling basin during raising of the reservoir level. The rate of dissolving of calcite by reservoir water has been assessed from trials and calculation.

Continuous recording of leakage is considered of importance so that any tendency to long-term change and any sudden change that might be indicative of movement relevant to stability of the dam may be detected, since the uplift records suggest that in addition to increasing with rise in reservoir level leakage may from time to time vary slightly in its distribution.

UPLIFT

Higher uplift pressures than usually adopted in design, suspected earlier from leakages occurring at high levels at the joints between monoliths and at locations in the stilling basin and river downstream from the dam, have been a known feature of the dam since measurements were commenced. They were confirmed by pressures recorded during exploratory drilling in 1969 and are shown to be sensibly the same from the toe of the dam to about three-quarters of the way to the cut-off trench, indicating little resistance to flow. The blanket grouting beneath the dam, the downstream grouting carried out within the dam in

1959/60, and earlier caulking at the downstream face of vertical joints between monoliths in order to conceal leakage, will all have increased uplift pressures, but the 1959/60 curtain grouting, however, will have reduced them. The downstream pressures are increased by lack of relief at the downstream toe and long seepage paths beyond. Typical recorded uplift pressures are shown in Figure 1. It is expected that uplift pressures will generally tend to be lower at the sides of monoliths than beneath them except where the joints have been caulked downstream, where the pressures at joints may be higher.

In 1973, a slight trend for pressures to have increased towards downstream and to have decreased upstream was noted and reported on and there have been further slight changes since.

STABILITY

It is considered that the dam will have had least stability in its first few years prior to execution of the 1959/60 remedial measures. In those years impounding was unrestricted, so that the reservoir often overflowed and uplift pressures were at their highest. Adopting, across the complete area, values of uplift subsequently found for these conditions the traditionally-calculated factors of safety against sliding and overturning are found to be less than customarily recommended, and tension is shown to exist in the upstream face. The extent of the calculated tensions is dependent upon the method of applying the uplift pressures, i.e. whether they are incorporated across any section by deducting them directly from the other stresses, or whether they are compounded into a single force applied across the section, which is assumed to remain plane and behave elastically thus maintaining linear distribution of stress across the section. The Author believes the former method of applying uplift pressures to be the more satisfactory.

Earlier calculations had apparently assumed the pressures to reduce to zero at the downstream toe. This was shown not to be the case during the 1969 investigations. The uplift conditions in this location and under the stilling basin, combined with the general level of the base of the dam relative to the stilling basin floor, indicate that some instability of the dam could result from lifting of part of the floor and the rock immediately beneath it close to the dam.

Although tension in the upstream face is theoretically indicated and is accepted as existing, unrestricted impounding is at present permitted because : (i) the dam has remained stable during apparently worse conditions of loading, and (ii), all features of subsequent instrumentation have given no cause to suspect near-instability. The 'factor of safety' is, however, unknown and pressure relief holes are to be installed to increase it. During drilling of these holes and thereafter they will be provided individually with means of quickly regulating any flows. As each hole is completed the valve at its upper end will be closed. Opening of valves will be undertaken systematically in controlled stages in association with continued measurements of leakage, pressures and movement. As leakage should increase with opening of the valves careful visual examination will be maintained downstream of the dam during this operation.

CONCLUSIONS

Uplift pressures do not follow uniform patterns of distribution in and under the dam on account of the presence of frequently differing permeabilities in and between the rock and concrete masses, combined with inter-connections and irregular leakage paths throughout. Assessment of the safety of the dam by calculation is inappropriate and renders assessment by performance necessary because: (i) these pressures are proved to continue (often in excess of normal design assumptions) and to be changing, and (ii) in view of the circumstances of rock excavation (particularly the depth removed, the apparently incomplete removal of river bed material, the lack of appropriate benching and other shaping, and the high level of the bottom of the cut-off trench and excavation generally, in the centre, relative to the stilling basin) and of concreting throughout the dam.

With the installation and use of pressure relief holes at the dam and continued monitoring of pressures, leakage and movement, some possibly at longer intervals than heretofore, the continued utilisation of the reservoir without further major remedial works should be assured for a very long time. The adoption for concrete dams of traditional design criteria of factors of safety against overturning and sliding and of avoiding upstream tension, which were produced for stone masonry dams, although generally providing adequate safety are considered often to result in extravagant design. A dam is unlikely to fail by overturning, as there would earlier be crushing at the downstream toe leading to failure by sliding. Calculations proving no upstream tension which are based on simple elastic theory are unlikely to be realised in practice, and other methods of analysis prove the existence of tensions. Some tension and horizontal upstream cracking may safely be accepted, and it is the Author's view that experience of dams such as Upper Glendevon and other gravity dams in unusual situations calls for new thinking to achieve realistic designs for such structures.

ACKNOWLEDGEMENT

The Author gratefully acknowledges the permission granted to him by the Owners of the dam for this Paper to be submitted. Matters of opinion are expressed as being personal views.

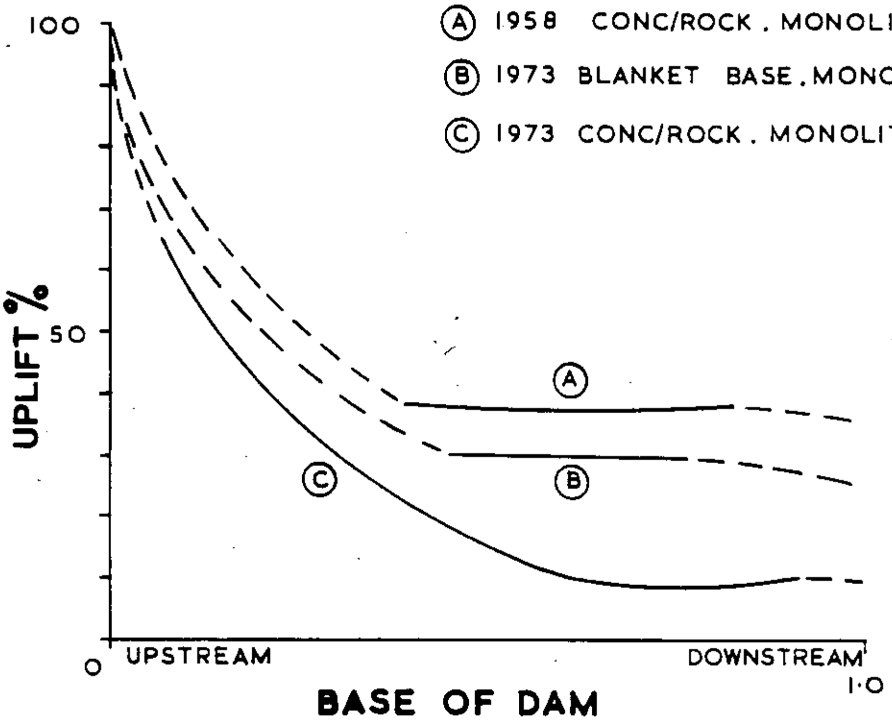

Fig. 1 : Typical Recorded Uplift Pressures

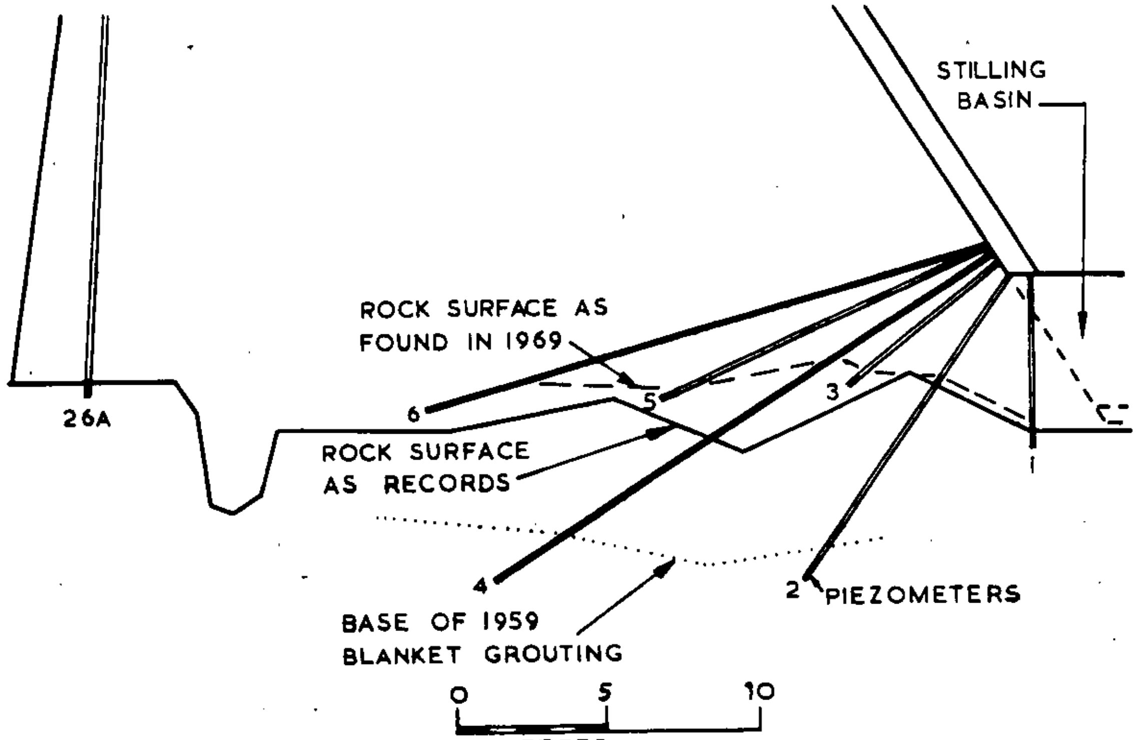

Fig. 2 : Cross-Section at Monolith 17

DISCUSSION : TECHNICAL SESSION 3

PROBLEMS OF MASSIVE DAMS AND REMEDIAL MEASURES

Session Chairman : E J K CHAPMAN BSc CEng FICE FASCE
Consultant
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General Reporter : A C ALLEN BSc CEng FICE FIStructE FIWES FGS
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CHAIRMAN : E J K CHAPMAN

It is my privilege to open Technical Session 3, dealing with 'Problems of Massive Dams and Remedial Measures'. My name is John Chapman. I have recently retired from my firm, James Williamson and Partners, and also from the Chairmanship of BNCOLD. My main interest in dams has been in the hydro-electric field and has been largely concerned with concrete dams and I am a member of Panel 1.

To introduce your General Reporter, Arthur Allen is a very experienced consulting engineer with his own firm, Allen, Gordon and Company. His practice is concerned mainly with dams, particularly concrete dams, and he is also a member of Panel 1.

REPORTER : A C ALLEN :

Our Session this afternoon is concerned with problems of massive dams and remedial measures associated with reservoir safety. This topic obviously overlaps to some extent, especially as regards instrumentation, with Sessions 1 and 2. The papers presented deal entirely with concrete gravity dams - none with masonry dams.

When faced with a massive dam and having to decide upon its stability there are four features which in my view are of paramount importance, excluding in this instance overflow and outlet arrangements. Those features are any appearances of movement, of leakage - from the nature of which some indication of uplift may be gained, of defects in the concrete and, fourthly, a knowledge of the design and history of the dam.

The six papers in this Session usefully cover all these features by giving case histories of three dams and expositions on the nature of uplift and on the testing of concrete quality by velocity measurement techniques.

As Mr Moffat explains in Paper 3.1, uplift is the least determinate of the primary loads for design, and he states that criteria applied in practice are notable only for their variability and, on some cases, questionable underlying philosphy. He makes it clear that pore pressure and uplift are now better understood but that much still remains to be explored. He rightly emphasises that uplift is not necessarily greater where concrete is of poor quality or leakage is apparent. At each of three dams where there has been doubt about safety piezometers have been installed to assess uplift, which is proof of the need for better information. The types of piezometer varied from the simple Casagrande dip-tube type to the newer and more sophisticated CIRIA twin-tube hydraulic type. The only electrical ones referred to were subject to an unfortunate mishap. Comments on experience of types of piezometer would be valuable contributions to this discussion, as I am sure that their use and installation in existing and new dams will develop.

The results obtained from piezometers have varied. Compared with design assumptions, pressures have generally been found to be higher upstream and lower towards downstream. Only at the Upper Glendevon Dam have higher downstream pressures been recorded. It will be interesting to see if the pressures recorded in the concrete of Bradan Dam increase with time and, if not, to consider why not.

Mr Coombes and his co-authors of Paper 3.3 indicate that, for the Val de la Mare Dam, quantitative assessment of stability was abandoned and that the anchoring scheme to restore stability was based on the belief that dangerous expansive cracking of the concrete could lead to higher internal uplift pressures than had been allowed for in the design. It would be helpful to understanding of this reasoning if the design and actual uplift figures could be given.

The general effectiveness of upstream grout curtains beneath dams is proved, as would be expected. Contributory to reducing uplift within a dam is creation of an upstream barrier to seepage and having general permeability or providing free drainage elsewhere. Upstream cement and chemical grouting was attempted in the concrete of the Altnaheglish Dam (Paper 3.4) and the Val de la Mare Dam. Some of the concrete in the latter was of low permeability, but it seems that the authors of the relevant papers may have doubts about the effectiveness of such grouting and they may wish to express their views further.

Turning to concrete quality, Professor Knill in Paper 3.5 gives a convincing description of seismic velocity investigations at several dams, his clear conclusion being that the method is a valuable technique but that it must be appreciated that construction joints - and I presume shrinkage and other cracks - affect the results. Figures he quotes suggest that the concrete of Val de la Mare Dam, similarly tested and reported on by Sherwood and others in Paper 3.2, was generally 'good' to 'excellent', and that at Altnaheglish Dam was 'questionable'. The latter suggestion appears to agree with the other reports given of the dam, but it would be interesting to have further debate on the suggestion of 'good' which appears inconsistent with the report of Val de la Mare Dam in Paper 3.3. It would be valuable if participants would discuss whether velocity methods can reasonably be related quantitatively to concrete strength.

Sherwood and his co-authors state that one of the most successful aspects of the initial survey at Val de la Mare Dam was the correlation established between low sonic velocity and concrete casting date (Paper 3.3, Fig 9), and that this provided encouraging indications as to the size of the problem, but they say no more about it. I was left wondering and ask them how do they define the correlation established, what do they suppose were the reasons for it, and, to the size of what problem were the indications an encouragement?

Even so, I question the extent of the value of such interesting methods in relation to reservoir safety, where there are large exposed surfaces to give indications of concrete quality, surfaces at which deterioration will probably be most rapid, and from which to make a visual assessment in association with selective coring and tests. It is important to realise that identical methods of dam design would probably have applied to masonry dams built in lime mortar.

The rate of surface deterioration of poor concrete to a maximum depth of 80 mm is clearly shown in Fitzgerald's paper (3.4) on the Altnaheglish Dam and, with the features of the concrete mix which he gives, it is further evidence of high water/cement ratios leading to lack of durability. It is, however, of utmost relevance to the care of old dams that in this case it was felt that unless a marked increase in the rate of loss took place no remedial action was necessary, and that the validity of that decision was borne out by the experience of 14 years.

Having to decide on the safety of a doubtful dam, striking the balance between taking adequate precautions on the one hand and avoiding undue alarm and the serious consequences of loss of water on the other, is a great mental strain. Considerations of the history of the dam and the nature and consequences of any likely failure are vital to any such decision. I think that in most cases it is reasonable to adopt some measure for avoiding maximum loading of the dam, at the same time introducing monitoring of the essentials I referred to earlier, i.e. movement, leakage and uplift, on which to base a firm decision on the dam's future. This has been done, as I have described in Paper 3.6 for Upper Glendevon Dam, and the views of participants on such a philosophy are invited.

In the dams described in the papers the amount of instrumentation of all kinds has varied greatly, and so no doubt have the costs. As referred to in Session 2, instrumentation can only be justified by need. The cost of it is not only installation but maintenance, monitoring, data-processing, comparison and evaluation, and unless instrumentation results are evaluated promptly they lose their value in the context of reservoir safety. It is also significant that some forms of instrumentation require replacement before others and some, if they become unserviceable, may be incapable of replacement.

The two papers on the Val de la Mare Dam are intensely interesting in the variety of investigations and measures to which they refer, and it is disappointing that because of the limit necessarily imposed on papers much has obviously had to be left out. I would list six items where I think that if the authors could expand on their papers in the discussion it would be of great value:

1 The original concrete mix and workmanship, the nature of the alkali-aggregate reaction, its relationship with time and its occurrence at depth in the dam and, incidentally, elsewhere in Jersey.

2 The reasons for the handrail movements appearing after nine years, what the movements were between blocks of the dam, particularly the highest ones, and what have they been, and how measured, since 1971.

3 How leakage and uplift have changed.

4 The average of the upward and downward percentage variations of the changes in sonic speed between 1972 and 1973.

5 The changes in downstream face stresses, with the reservoir empty, induced by the upstream strengthening of the dam.

6 How the computer analysis showed that movements would not be excessive.

I wish to draw attention to one very important feature which arises from experience with existing massive dams. It is that the construction workmanship, in which the contractor and those responsible for supervision alike have their part to play, is vital to fulfilment of the design function and to safety. Sound concrete surfaces, true to line and free from voids and blemishes can generally be taken as an indication that there is similar quality within, and that is important. Any uplift theory is upset if the horizontal construction joint is badly honeycombed for two-thirds of the way from the upstream face and very tight thereafter.

In conclusion, in this Report I have posed questions for the authors - except myself - but I hope to supplement my own paper with some slides. The ultimate purpose of this Symposium is to improve safety of operation of reservoirs, and towards that admirable objective it would be valuable if other participants would give their experience of problems of massive dams and of the remedial measures adopted. I do not believe that such problems exist only in the Channel Islands, Northern Ireland and Scotland. Massive dams became more popular with the publishing of Rankine's Theory in 1872, and many of the early masonry dams still exist. If there are no problems with them perhaps there is a fundamental lesson to be learnt.

A I B MOFFAT (University of Newcastle upon Tyne) :

S A misapprehension common among civil engineers is the notion that mass concrete is, in general, an inert, durable and well-behaved construction material which will fulfil its structural function for an almost indefinite period of time. That this is not the case is clear at least to engineers involved with concrete dams. It has been clearly demonstrated from laboratory work done by the Corps of Engineers in the United States that unstressed samples of concrete which are kept under moist conditions will gain strength asymptotically over very long periods. If exposure to moisture is arrested, performance of the concrete will still remain satisfactory. Given sensibly constant loading conditions, however, with the added possibility of leaching action from seepage within the body of a dam, the possibility of a significant regression of strength occurring even within the space of a few decades becomes very real. We therefore have to constantly bear in mind the fact that, its other deficiencies such as cracking and dimensional instability apart, concrete is far from an 'ideal' fill material for dams.

Alongside this we must appreciate the limitations of our understanding of the behaviour of the dam itself. I believe it was an eminent Spanish engineer who once stated that 'gravity dams are not subject to rational analysis'. He was not terribly far from the mark, and such truisms and the limitations of present materials aside it is essential that attempts are made to clarify certain fundamental issues. Paper 3.1 outlined research into just one such issue, internal uplift, with reference to the CIRIA sponsored work carried out at this University. In experimental terms the primary objective of the programme was to develop and prove a piezometer for mass concrete capable of installation within existing or new construction dams.

Laboratory development and proving of the piezometer and of the best sealing technique occupied some six months. A total of some 24 piezometers was manufactured to cover trials at the Bradan installation.

In planning the Bradan trial installation it was decided that more would be gained by thorough coverage of one location within the dam rather than by dissipating the limited resources available in attempting to cover different areas. For this reason attention was confined to Lift 15 of Block 23, and the simple but effective instrument layout illustrated in Fig. 4 of Paper 3.1 adopted.

At Bradan the piezometers were saturated and then dropped into a PFA grout 'cushion' at the bottom of each 30 mm dia. borehole. 'Situseal' grout was then tremied into the boreholes, which were filled to lift-surface level. The initial vertical length of hydraulic lead was run in small diameter tubing which expanded to standard piezometer tubing for the horizontal runs.

Following installation the Bradan piezometers were left until impounding was in progress, at which stage they were de-aired using portable equipment. The decision to use a portable pressure transducer to monitor pressures resulted in a very compact layout inside the inspection gallery and, once minor problems over making connection via the snap-couplings were overcome, proved to be straightforward and entirely satisfactory.

The cost of manufacture and installation, £750 at 1972 prices exclusive of portable equipment, could be somewhat reduced in real terms in 1975.

In conclusion, returning to the question of uplift design criteria, the present use of a convenient emperical pressure intensity of, say, 50% or 67% of upstream head is to be deplored. Further quantitative data gained from field measurements in a representative selection of concrete dams of different types and ages is required. The low intensities indicated at Bradan Dam and summarised in Paper 3.1 probably reflect the influence of efficient and clear relief drains in a new dam. This is unlikely to be the case with dams which have been in service for some time, e.g. Loch Dubh, and lends emphasis to the case for well-designed relief drains which can be readily inspected and reamed out.

R M CLARKE (Jersey New Waterworks Company) :

S Before I had the task of managing the Val de la Mare Dam I had not heard of the alkali-aggregate reaction (AAR) disease - I say that to my shame. For those of you who have not seen AAR it looks awful, as will be apparent from my slides. AAR at Val de la Mare is our nightmare, and I should start by telling you that in 1963 we had the Summer Meeting of the British Water Works Association in Jersey and I was then proud to take that Meeting to Val de la Mare.

In Paper 3.3 we referred to the remedial work we did on Block 6. Progress of the deterioration even between June 1972 and December 1974, revealed by a light discolouration, was apparent on observing the face of the dam. In close-up, the appearance of AAR deterioration is fairly distinctive, and does not present a pretty picture. I should say that it is now possible, with the knowledge we have, to avoid problems like that at Val de la Mare. I think that as engineers we owe it to our employers.

Referring back to one or two things that have come out of discussion at this Symposium, I think it was Mr Kitching who spoke about the movement of defective dams. With respect to Val de la Mare, one of the first things we did in 1972 was to fix welding rod onto the bridge abutments at the top of each block, spanning the construction joint, and then cut the rod. Those welding rods are still there and it is comforting, in my position, to see just how close the two ends of each rod still are. It gives a little more assurance, I think, than looking at some of the print-outs and diagrams of more sophisticated instruments.

The entire bridgeway surmounting the monoliths, including the handrail, is precast, and any movement which is measured at the handrail is obviously magnified from the actual monolith below. The differential movement is a measurement of how one precast handrail is buckling against the other, and for Block 6 the maximum movement we got between any two handrails is 28 mm. That does not mean that the monolith has moved 28 mm, it is something considerably less, but the handrail movement is a good indication of what movement is still continuing. It is interesting to note that by January 1974 we were beginning to get a little measurable movement in the handrails further along, towards the left abutment, which confirms our feeling that the reaction is continuing.

The Reporter asked the reason for the movement. I think that the reason for the movement, as Mr Coombes will explain, is AAR and the stresses it sets up. The reason that we saw it when we did - it almost happened overnight, as far as we are aware - is probably related to the period of the lowest temperatures we have had since the dam was built. We had 11 days where the temperature remained below or at freezing point and the dam, we think, took up its smallest dimension, the adhesion between monoliths eased off, and the affected blocks were thus allowed to take up a new equilibrium and we saw the movement at once. The movement was upstream, and at that time it was four blocks which had moved over a period of a few days.

CHAIRMAN : E J K CHAPMAN

Mr Clarke, we know that not all the laws of England and Wales apply to the Channel Islands, can you tell us whether the Reservoirs Act applies to the Channel Islands?

R M CLARKE :

No, but my Company operates on the sensible assumption that the Act does apply, and our structures are inspected by a 'qualified engineer'.

I do not wish my remarks from the rostrum to give the impression that I consider the engineering at the time of building of Val de la Mare dam was deficient. This is not my intention nor my belief. I think the information available in this country in 1959 on alkali-aggregate reactivity was almost non-existent. My object is to make sure that all engineers using concrete are now fully aware of the possible effect of AAR and that they take what precautions may be necessary.

L H COOMBES (Engineering and Resources Consultants Ltd.) :

It is, I feel, appropriate to supplement our Paper 3.3 and give further explanation of the relatively rare phenomenon of alkali-aggregate reaction, or AAR.

During the early stages of the investigation into the condition of the dam, an exhaustive literature review was carried out and a bibliography prepared. The review covered problems with concrete, with particular reference to hydraulic structures, deleterious materials in concrete, deleterious action on concrete, relevant investigation and testing procedures, instrumentation and monitoring.

The bibliography contained a list of from 400 to 450 publications, a good proportion of which mentioned AAR in their titles. A few of the publications originated in Australia, Denmark and the United

Kingdom, but by far the majority originated in the United States. The bibliography is mentioned to emphasise the fact that although the incidence of AAR in the United Kingdom up to the present time has been rare, occurrences outside this country have been numerous and much has been written on the subject. The Cement and Concrete Association are at the moment preparing their own bibliography on AAR, and this should be available shortly.

The alkali reactive materials are all silicas, which are the basic constituent of quartz, chert and flint. In the amorphous or cryptocrystalline form this material can be reactive with alkali, giving rise to the sodium silicate or potassium silicate gels that are the expansive agents causing disruption in concrete, sodium salt being the most disruptive.

The reactive materials which are most likely to react are typically chalcedonic, namely beekite and jasper, with associated opal. All of these were present in varying proportions in the quarry supplying the coarse aggregate for Val de la Mare Dam, as veins secondary to the host rocks.

It is generally accepted that a cement alkali content of under 0.6% will not give trouble, although in the United States there has been at least one case where reactivity has occurred with a lower alkali percentage.

Whether or not a material reacts disruptively in concrete is dependent on many parameters, one of which is the quantity of reactive material present in the concrete, and its form. The 'pessimum' quantity is that proportion which gives the greatest disruption or expansion and will be dependent on the mix and cement used. With a very small amount of reactive material the reaction will be minimal - a large amount, however, will swamp the reaction and form a pozzolan.

The form in which the reactive material is present is also decisive, as is its manner of inclusion, i.e. single uniform stones or within veins largely surrounded by parent rock. The first form gives rise to 'pop outs' whereas the latter is less likely to exhibit this effect on the face of the concrete. The presence of reactive material in fine aggregate will tend to lead to the formation of pozzolans since the large surface area induces rapid reaction.

The hypothesis that the reaction requires lime to allow a regenerative process is now discounted in the United States, though this lack of calcium hydroxide in the cement paste is looked upon as being indicative that reaction is taking place or has taken place.

Much work has been carried out in many parts of the world on this phenomenon, but the precise nature of the reaction is still in some doubt. American experience indicates that the initial movement is usually the greatest and movement generally does not recur on the same scale. Also, the inside of a reacting body is restrained by body forces and will not expand so much as the outer parts. Additionally, if cracks remain open, gel can exude into them and the remaining movements may not be so great.

Tests for potential AAR fall into two main types, chemical tests and mortar bar tests. The methods are specified in ASTM 289. The difficulties are that the chemical tests are not definitive, and the physical tests depend on long-term programme results - preferably over about two years' minimum.

In the CIRIA Technical Note 63 — 'A Review of Pore Pressure and Internal Uplift in Massive Concrete Dams' by Mr Moffat, comparisons are made between electrical and hydraulic piezometers for the measurement of pore pressures. It is pertinent in relation to the paper on the Val de la Mare Dam to mention the reasons why, at the time of installation in 1973, electrical piezometers were chosen.

The main reasons for the choice were that the electric piezometer has a rapid response time, is easy to read even by unskilled operators, and the piezometer and readout system is compact and was able to be readily fitted within the existing gauge house. Also, with hydraulic piezometers, limitations are put on the relative levels of piezometer, highest point of connecting lines and readout position to prevent formation of air pockets. The cost of electric equipment was comparable with hydraulic equipment when all the ancillary hydraulic and de-airing units were included.

Referring to the suggested disadvantages of electric piezometers listed by Mr Moffat in Appendix I of Technical Note 63, the following observations are made :

1 It is agreed that the inability of an electric piezometer to differentiate between pore air and pore water pressure could be a problem.

2 The zero drift of a piezometer tip may be specified but is generally 0.05% maximum of full scale. The zero drift of the electronics may be checked.

3 It is not generally necessary to recalibrate in-situ.

4 Extra lines can be put in the pocket at the time of installation should it be required to use an electric piezometer in reverse to measure permeability.

5 All piezometers are subject to hidden damage during installation - the only precaution is to carry out the installation using experienced staff.

6 The long-term durability of an electric piezometer is questionable, but this has not caused any difficulties in our experience excepting the lightning strikes referred to in our paper.

In answer to the Reporter's question regarding concrete mix used in the dam contract, the mix by weight was 4.1 : 2.9 : 1 and water/cement ratios ranged from 0.61 - 0.72. From experience of previous dam construction on Jersey emphasis was placed on density, and figures ranged from 2403 kg/m^3 to 2443 kg/m^3. Due to the restriction imposed by the pumping equipment, the nominal size of the maximum aggregate was limited to 25 mm. A dispersing or wetting agent, 'Lissapol', was added to the mix to aid pumping.

Regarding the question on how the finite element analysis showed that movement would not be excessive, the standard output of the finite element analysis programme recorded the magnitude and direction of the three principal stresses at the centre of each element, and also the displacement in two directions at each node point of each element.

A C ALLEN (Allen, Gordon and Company) :

(S) On photographs taken of Upper Glendevon Dam in 1955, shortly after the dam was completed and impounding had taken place, some leakage is evident at horizontal joints. Two primary points of leakage at that time were the vertical joints between blocks in the spillway section, and although general leakage had been collected at that time into a pipe running underground there was still much coming overground and falling down into the stilling basin. Remedial measures were taken in 1959/60, and I imagine that the dam's appearance in 1968 photographs is quite similar to six or eight years previously.

As outlined in Paper 3.6 I was appointed Panel Engineer for the dam in 1968, and further investigation into continued leakage commenced in 1969, the situation then being that artesian pressure was being relieved within the area below the dam immediately downstream of the original V-notch weir, in the masonry of the tailrace area and also in the river channel. At these points artesian water is visibly emerging. A further V-notch and recorder was installed to give a continuous record of leakage.

Examination of these seepage exits in more detail indicates, for example, emergence just below the original V-notch and in the floor of the stilling basin floor. With the latter, water is clearly emerging and grey streaks of sand brought up by that water are visible on the floor. Moving down into the river channel, one can see in the river bed, which has normal detritus over the gravel, an area which is clean, indicating that emerging artesian water is keeping the dirt off the stones. A little surface ripple is also apparent in the river at this point, some 30 m downstream from the dam itself.

Turning to the upstream face of the dam, on photographs taken in the early 1970's at a time when the reservoir was very low and long after the 1959/60 remedial work leakage from horizontal construction joints and the vertical joints was apparent at a number of locations along the dam a considerable time after the reservoir level had fallen below these joints. At points on some construction joints lime was still leaching out. These are some indicators as to indifferent workmanship, a point mentioned in my Report. I think this kind of thing is indicative of what we also may reasonably expect inside such a dam.

Some vertical joints in the spillway which leak from a very high level were caulked up in the early days to try and hide the leakage, probably bottling up water to a considerable height and increasing internal pressures. Landslips associated with high water tables in the area of the left abutment were also apparent. As regards the piezometers installed at Upper Glendevon, and referred to in Paper 3.6, essentially they consisted of a porous candle located in a perforated PVC tube which continued up in normal PVC tube to the face of the dam to form a dip-tube. The perforated tube had sand around it, enclosed in hessian and protected with gauze for installation purposes. A sealing washer was located above the porous element, the annulus round the dip-tube being filled with neat cement grout. This technique was entirely successful

It is also noteworthy that artesian pressure and flow issued from one of the geological holes after it had been completed.

J L ADALID (Ministry of Public Works, Spain):

(S) I would refer to a major incident which occurred in Spain, affecting a large dam on the River Ebro and located some 150 km inland.

Mequinenza Dam is a concrete gravity structure some 85 m high and 460 m in length, impounding some 1530 x 10^6 x m^3. The foundation conditions consisted of alternating strata of limestone, marl, gypsum and coal measures.

On March 27 1967, with reservoir level some nine metres below TWL elevation of 121 m, leakage appeared on the right abutment at elevation 87 m. Leakage flow increased progressively from 8 l/sec that morning to 500 l/sec the following day and to 17 000 l/sec that afternoon. The leakage path was traced using aniline dye. Abandoned coal workings in the abutment were initially suspected, the tracer dye emerging downstream in some 10 seconds.

Remedial action consisted of lowering the retained water level until leakage ceased, and exploring and excavating galleries in the abutment. A concrete 'dam' or wall up to 10 m high was cast inside the excavations in the abutment. The source of the trouble was, in fact, traced to weak and comparatively easily eroded strata between the limestones, and similar treatment was therefore extended to the other abutment.

Since completion of the remedial work the reservoir has performed satisfactorily.

R D FITZGERALD (Waterhouse and Partners) :

The *ad hoc* approach to the subject of deterioration at Altnaheglish Dam (Paper 3.4) was conditioned by the fact that we had first to find out what questions to ask before trying to find the answers. It has been suggested that perhaps responsible engineers should be made to have their site offices etc. below the dam. We started like this at Altnaheglish, but shortly after our investigations were under way moved them to above TWL !

Piezometers, which were of the Bishop type, and also the simple vertical dip-holes in the crest, were read and measured daily for some 12 months, and weekly thereafter. Readings were discontinued after four years as they showed no general change and had remained steady for the last three years. The instruments and hut were removed from the site, the tubes were cut back, plugged and pushed back into the holes drilled for the piezometers which had been filled up to 300 mm below the dam face, and the holes sealed.

In answer to the Reporter's query it was not thought that the grouting had any effect, at least no appreciable variation in the piezometer readings before and after grouting was noted. The only holes to take any large amount of grout were adjacent to the scour pipe which was then opened, disgorging a large amount of grout. The boreholes were checked during drilling to ensure that the scour and eduction pipes were not penetrated, and it was subsequently assumed that the scour pipe had sheared or cracked in the plug in the body of the dam during or after construction.

I sympathise with Mr Clarke after his comments on the state of the face of Val de la Mare Dam, but if he looked at the condition of the downstream face of Altnaheglish he would not be so depressed!

E B WILSON (Sir William Halcrow and Partners) :

I was very interested in Mr Allen's Paper (3.6) on the Upper Glendevon Dam.

I think we engineers are now facing a considerable problem which has been brought to a head by the Flood Studies Report. Are we to reassure ourselves about the ability of many dams to withstand the much higher design floods which are recommended in that Report? Obviously, before we recommend any major reconstruction we must know much more about the real factors of safety within the existing structure. I do not think that we are very well placed to form reliable opinions about limit state design of gravity dams. We have not yet had sufficient failures. We are observing very carefully, and some people suggest too carefully, the behaviour of our successes in the hope that by doing so we can estimate just how near to the bone they come.

The example of Upper Glendevon Dam does help us a little. There I counted about a dozen major departures from accepted practice of design and construction. All these would now be considered serious faults. For instance, no internal drainage, no grout curtain and rather low density concrete. This dam did not fail, however, even before the remedial measures were carried out. That seems to me to indicate that there are hidden factors of which current theory takes no account. I have estimated, rather roughly, that the original construction cost of Upper Glendevon Dam would have been increased by at least 50% if all the alleged faults had been avoided at the start. I wonder if the Author could tell us the cost of the remedial work since the dam was constructed? I doubt very much whether the amount was as much as half the original cost.

I would therefore suggest that it could be worthwhile to spend large sums on research with models to determine exactly where the borderline of safety lies with gravity dams. Such a study ought to be Government sponsored, of course. There is the fairly certain hope of quite majestic savings in the capital cost of any unnecessary reconstructions.

The Reporter asked for comment on the clogging of internal drainage systems. It is possible, is it not, that there may be a correlation here with the amount of grout accepted by the foundation during construction? If that is so, perhaps we should look for better cement than OPC for grouting.

D W BERRY (Howard Humphreys and Sons) :

S In reply to the Reporter's request for information on existing dams, I would like to give you a few facts about a concrete dam in Jamaica about 48 years old and where we are currently carrying out site investigations and remedial works. The maximum height of the dam is about 46 m and length is about 230 m. The foundation is a conglomerate, the series repeating about every 2.5 m from fine to coarse, and dip upstream is about 80^0.

The interesting thing about this particular dam is that it has no drainage system whatsoever, and I would like to compare the uplift which was measured there with the uplift given in Mr Moffat's Paper 3.1. We put in piezometers (Fig A), very crude compared with some of the ones we have seen — and measured uplift on three sections. We measured over a period of a few months when the water level varied from full height to about 50% height, and the interesting thing is that we got a very close correlation between the uplift expressed as a percentage of water depth. Mr Moffat's and other papers I know talk about uplift varying from 0% to 100%, but what in fact we recorded was as shown in Fig B. The picture was pretty frightening compared with some of the things that we are told we should assume.

This brings me to the point of what happens if the drains clog up. We are going to get the sort of uplift profile shown in Fig. B rather than the simple triangle which some of the textbooks give.

A point Mr Moffat also mentioned was the question of whether uplift applies over 100% of the base area or some lesser value. As a matter of interest I worked out for this particular dam that if we had that configuration of uplift over 100% area the resultant, on a two-dimensional analysis, shot off the page! As the dam had not failed we must assume it had not acted over 100% of the base area. Our geologists had recommended a safe load under the toe, and we found that if you assumed the uplift to act over 70% of the base area we could just about stay within the geologists' recommendation for maximum load on the rock.

Finally, also on this dam, I would like to say something about the concrete mix, based on very old records. It appears that a 1:12 mix plus displacers was used, so it is likely to be a pretty weak mix. Nevertheless we took cores at several points when we were putting in our piezometers and we got core strengths of 17 MN/m^2 to 35 MN/m^2, which is, of course, very much more than the stresses one would expect to get in the body of that dam.

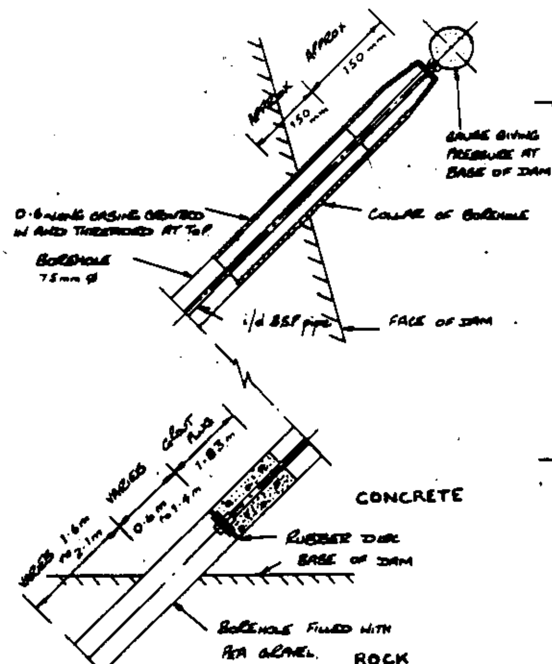


FIGURE A

TYPICAL DETAIL OF PIEZOMETER.

(NOT TO SCALE)

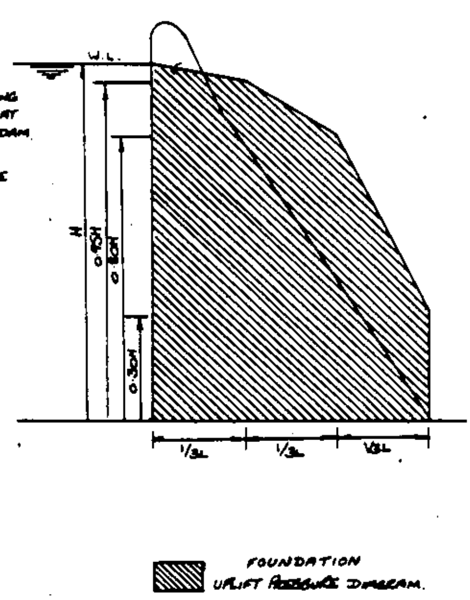


FIGURE B

UPLIFT - AS MEASURED ON SITE

BERRY. D W - HERMITAGE DAM : JAMAICA

C GROBELAAR (Department of Water Affairs, South Africa) :

Ⓢ I have come to a couple of conclusions, and the first is that it is very easy to abuse any type of instrument. Secondly, any type of instruments correctly used are usually satisfactory, there are very few exceptions. There are, however, limitations in practice. Considering geodetic surveying to record deflection, if, for example, a 50 m high arch dam deflects under filling and an upstream heel crack then develops, we can do theoretical calculations of the further deflections. The amount of extra deflection to be expected is of the order of about 20% of initial deflection, and for most dams this is within the error of survey so that you cannot normally detect the effect of a crack at the upstream heel by geodetic survey.

A second point I want to make is that there are more dams subject to alkali aggregate reaction (AAR)) than is normally realised, and the amount of reaction required to invalidate strain measurements is very small. We have several dams where for all practical purposes reaction is minimal, but if we take strain measurement we take the additional precaution of putting in 'corrector' gauges which are made as closely as possible of the same mix and placed in the same conditions in the dam except that they are isolated from stress. There are several instances where we have installed such gauges, and one can monitor 'corrector' strains of 200 microstrain even before filling.

At a particular dam that we are investigating at the moment with that type of performance before filling, we have taken the additional step of putting strain gauges in groups of four instead of the normal three in order to get a measure of the accuracy of the strain gauge. We all know from theory of elasticity that the sum of readings on gauges 1 and 3 must be equal to the sum of readings on gauges 2 and 4. If one gets anything in the nature of AAR, however, the resulting strains do not conform to elastic principles. I just want to point out that even if you have got so little AAR that it does not matter whatsoever in regard to durability or safety of the dam one should not try to install strain gauges and calculate stress from strain measurements.

C F GRØNER (President ICOLD; Chr F Grøner A/S, Norway) :

Ⓢ The General Reporter asked if it was only in Northern Ireland, Scotland or Wales that troubles arose with massive dams.

We have seen Spain and I can tell you we have also a lot of difficulties in Norway. We built a lot of massive dams from 1905 to 1920, and most of those dams deteriorated in 10 to 30 years. There are a lot of reasons for this, of course. Two of the reasons are that we have a heavy rainfall with freezing and thawing, but we also have acid waters. There are of course other factors such as bad concreting and so on, but in any event we stopped building massive dams about that time. We are now starting again and I hope we will be more successful.

I would give two practical examples of what might happen when massive dams deteriorate. We have a dam built in 1915, and in the 1950's it had to be reconstructed because it had leakage all over. It was grouted - which we do not like - and the porosity was so high and the density of the concrete was so low that if you grouted up you got high uplift! We had one other case at about the same time, also with a massive dam which had heavy leakage. On grouting up near the crest it was decided to tighten the downstream side first. Never do that! As a result of this the horizontal construction joint some 3 m down was subject to uplift and disruption.

M F KENNARD (Rofe, Kennard and Lapworth) :

One would have thought that by now with all our experience of concrete dams we would no longer have any problems. Unfortunately this is not quite so, and one of the reasons for this is the change in cement over the years. When the Hydro-Electric Board was building a large number of dams in the 1950's they changed from Low Heat cement to Ordinary Portland Cement (OPC) to BS 12 with quite satisfactory results. In recent years, however, it has been found that the practice then adopted has not been quite so satisfactory.

At Clywedog Dam in the mid 1960's I believe they had problems that were traced to the very fine nature of the OPC then being produced. This was because BS 12 had no upper limit for fineness, and it was in the cement company's interests to produce a finer cement with rapid-hardening properties – really to help the precast concrete industry – but Engineers have used OPC thinking it is still OPC as of old. Subsequently to this we had the same experience at Cow Green Dam. It led to excessive shrinkage in the lifts, which curled up slightly, opening up along the horizontal joints and around the waterstops near the vertical joints. Subsequently this has also been found at the Meldon Dam and the Tamar Dam. In all these dams PFA was used as a partial replacement for cement to the extent of about 25%, but even this was not sufficient to counteract the excessive heat of hydration of so-called OPC. The recent energy crisis may have affected this, as I believe that the cement is now coarser than it then was in order to save on the cost of grinding, but I just add a note of warning that whereas one found in the past that

OPC was satisfactory it is not necessarily so today. In Meldon and Tamer a certain amount of remedial grouting using a resin grout was required in order to seal the very finely opened joints that resulted from the excessive heat of hydration.

T A JOHNSTON (Babtie, Shaw and Morton) :

I have been trying to decide if Mr Moffat had his tongue in his cheek when he wrote up certain of the interim conclusions of his investigations at Loch Dubh Dam and Bradan Dam in Paper 3.1. I have been looking at his Fig.5, Bradan Dam, which shows a very significant drop in the uplift pressure taking place nearly coincident with the inside of the upstream facing concrete. Facing is provided to the upstream face to be more durable and less permeable, and that was done at Bradan where the facing was 5.7 : 1 and the hearting 10.7 : 1 by weight. Fig.5 thus appears to confirm theory, but in Mr Moffat's third conclusion I find that he says that the rapid initial drop in uplift was not considered to be attributable to the use of rich facing concrete. Does that mean that hearting concrete throughout would result in the same diagram as he shows and, indeed, could we abandon facing concrete and save some money?

A I B MOFFAT (University of Newcastle upon Tyne) :

I think the answer to Mr Johnston's question about the effectiveness or otherwise of a rich upstream facing lies in the fact that with two different mixtures of concrete, one of which is facing of the order 0.5 m to 1.0 m thick which we assume is intimately bonded to a rather leaner concrete internally, there is nevertheless bound to be some differential movement and micro-cracking even within the depth of one lift. I think the answer also lies in the fact that with the richer facing concrete the risk of shrinkage, which Mr Kennard referred to, is also much increased, again giving rise to fine cracks.

For both those reasons I feel that we cannot say that the rapid fall in the intensity of the uplift pressure is attributable to relative impermeability of the upstream facing concrete, since the influence of micro-cracks would be predominant. As regards the final point Mr Johnston made, I am not convinced that a richer upstream facing, except possibly as a defence against acid water, serves any real purpose. There may be advantages in a rich downstream facing from the point of view of frost protection and durability.

Mr Berry referred to uplift figures for a dam in Jamaica where he recorded very high uplift pressures throughout the width of the dam. I believe he quoted it almost in the form of a question as to whether uplift acted over 100% of the area since, on the basis of his recorded measurements, that dam would be unstable. I do not think the dam is unique in that respect; there are many old dams built without any allowance for uplift, and thus of slender section, still standing. I think that for the answer we must look to various other influences, notably the fact that concrete or masonry does have a finite tensile strength, or to three-dimensional effects or the constraints of local topography, to give a few examples. There is also the unknown effect of certain secondary stresses which may partially offset some of the effects of uplift.

As a final point, in 1972 the CIRIA prototype piezometer cost £15 to make in the University workshop. A commercial electrical unit then cost £65, and I think that saving is significant, particularly in view of the relative merits of the different types even when balanced against total *in-situ* costs.

REPORTER : A C ALLEN (Concluding Summary) :

I shall go through my notes as I have made them and comment on the various contributions as they have occurred.

May I firstly say how useful I think this discussion has been - I think it has brought us out of our shells, and we are not now quite so afraid to say that we do not know all the answers. I think it has come out very clearly, especially in relation to massive concrete dams, that we are beginning to learn that we still have quite a lot to learn!

If I deal with the contributions one by one, I do not think I have any comment on Mr Moffat's first contribution which was by way of further explanation of Paper 3.1. As to his later contribution, I will simply say that I associate myself entirely with the remarks he has made.

Turning to Mr Clarke and the Val de la Mare Dam, it was very interesting to hear his comment that the effect of the AAR became evident more or less overnight. The handrail movements would appear to indicate something collective, something progressive, but on the other hand his welding rod indicators were still reassuring. I am a little bit confused on that one, and if we can get more information about Val de la Mare that will prove most useful in helping us all to assess the kind of problems which have occurred there. Similarly with Mr Coombes' comments on that dam and his helpful explanations of AAR, his electric piezometers and on the concrete mix, where he brought out certain facts and figures, some of which may be relevant to the problems that have developed. I noted the high water/cement ratio of up to 0.72 occurring here and the density figures he quoted. He also mentioned 25 mm aggregate and use of 'Lissapol' additive, both of which may have been relevant factors.

Again on concrete mixes Mr Fitzgerald, in Paper 3.4 and subsequently, quoted very interesting facts about the specification for Altnaheglish Dam. It perhaps caused a smile to think that engineers used to specify concrete like that. We have come a long way. The specification for Upper Glendevon included : 'Only as much water is to be used in mixing concrete as is required to allow it to fall out of the mixing machine... Concrete is to be laid down in horizontal layers not exceeding 600 mm thick, evenly spread out and beaten or trodden down, and must not be allowed to fall from a height.'

Mr Adalid had a very interesting example of abutment foundation failure at Mequinenza. It reminded me of the Baldwin Hills failure in Los Angeles in the rapidity with which it developed. One wonders whether grouting might have avoided that kind of effect. It is not unique to Spain or the United States – I did have association with a dam in Scotland where some 23000 tcm of water went down one of our rivers in about a fortnight after a low dam with a very large surface area of water behind it failed by precisely that same kind of thing. No one heard anything about that one! Regarding Mr Fitzgerald's mention of grouting of the scour pipe, again there have been similar experiences elsewhere - the difficulty is when they grout up the valve and you cannot do anything about it!

Mr Wilson made a very thoughtful contribution. We are not at all well placed to assess gravity dams, I would agree. We do not really know how our gravity dams act - there are some standing that should not be and there may be others that appear secure but may well not be - we have got to be most careful. Inadvertent grouting of internal drainage systems has certainly happened. In answer to Mr Wilson, the cost of remedial works at Upper Glendevon was perhaps £60000 or £70000 in 1959/1960, nowhere near half the cost of the dam itself.

Mr Berry gave another interesting example of high uplift pressures and queried why the dam was still standing. A point for consideration, very definitely, on a dam with a poor concrete mix of about 1:12 plus 'plums' and a high water/cement ratio.

Mr Grobbelaar's comments on instruments and deflections were very helpful. It may not be generally realised that an upstream crack in a massive gravity dam can extend for quite a distance into the upstream face with the dam still remaining stable. AAR is obviously well known in South Africa, and his discussion on strain measurements was most valuable.

I would thank Mr Grøner for his contribution on two examples of dams and his general remarks about the life of dams in Norway. It really is incredible, as he said, that someone grouts up the downstream face first - it is asking for trouble in a doubtful dam.

Mr Kennard ventilated the problems with fine cement leading to shrinkage cracking and to the need for proper and prolonged curing. If he could perhaps say a little bit more about the size and height of lifts at Meldon and Tamar it would be useful. I am certain we shall hear much more of resin grouts for sealing. An interesting thing about Mr Kennard's contribution was also that it was the only example given of trouble in England.

WRITTEN CONTRIBUTIONS

D E SHERWOOD (*Soil Mechanics Ltd*):

PIEZOMETERS

It may be helpful to summarise our experience in the use of, and present philosophy in the choice of, piezometers for instrumentation of existing dams. Three main types of piezometer are currently available and are discussed separately below.

Standpipe type piezometer : The standpipe piezometer is open to atmosphere and is a slow response instrument, particularly if installed within sound concrete. It is necessary to have access to the top of the standpipe to record water levels and this can increase cost in the case of a concrete dam. The standpipe can be converted to remote reading by introducing an air-bubble system, but the instrument remains slow in response and the standpipe is usually only installed in embankments where the fill is relatively pervious.

Hydraulic piezometer : The twin line hydraulic piezometer of the Bishop or CIRIA type is a closed system which uses manometers or Bourdon gauges to measure water pressures. It is a fast response instrument and is technically attractive where it can be used, as de-airing ensures and demonstrates continued satisfactory operation. There are, however, two factors which often preclude use in existing structures :

1 Where significant periods of sub-zero temperatures occur, freezing and consequent damage to lines and monitoring equipment result. Anti-freeze has been used but in many cases caused problems.

2 Hydraulic systems will not tolerate high negative pressures within the tip or connecting lines, as the water column breaks down. This provides severe restraints both on piezometer and connecting line locations.

In our experience data quality is usually poor unless skilled staff are employed, as de-airing is neglected or carried out without sufficient care.

Pneumatic or electric transducers : Transducers can be installed at any elevation within a structure and their connecting cables can be run in any convenient manner provided they are suitably protected. The electric transducer can be used with any cable length, while the pneumatic unit becomes progressively more time consuming to read as line length increases. We prefer not to use the latter with line lengths greater than 150 m. In commercially available form neither transducer incorporates de-airing facilities, which is a severe draw-back. We have, however, on occasions modified units to incorporate de-airing lines to the diaphragm face, but when these are located so that high negative pressures would develop in the de-airing lines it is necessary to maintain the lines open to atmosphere which, in turn, significantly slows the response of the system.

The electric piezometer can be read by unskilled personnel and it can be incorporated cheaply in an automatic monitoring system. It can be subject to damage by lighting in certain circumstances, but measures can be taken to reduce this risk to acceptable levels. The pneumatic piezometer should be read by skilled staff if errors are to be avoided, and automatic monitoring is much more expensive.

NON-DESTRUCTIVE TESTING OF CONCRETE DAMS

The table relating sonic speed to apparent concrete quality included by Professor Knill in Paper 3.5 can be traced back to 1949, minor modifications having been made at the low end along the way. All definitive classifications we have found in the literature can be traced back to the same source.

Leslie and Cheesman (1949) report having measured concrete speeds as low as 300 m/s on Ambursen dams, where they formulated their classification. A dry compact sand and gravel would typically have a sonic speed of 1600 m/s while the same material saturated should have a speed of about 2200 m/s. Weak rocks such as chalk or Tertiary sandstones have speeds ranging from 2100 m/s to 4200 m/s. The speed of sound in air is 330 m/s. It is pertinent to ask, then, whether the basic data on which the classification was based is reliable. It should be noted that the paths measured do not appear to have crossed construction or lift joints and that their measurements were made *through* the structure.

Sonic testing of various other structures is reported in the literature using similar equipment and the results have tended to confirm the original classification. Several of these surveys have included paths which intersect construction joints.

We consider that the important factor in all of this earlier work was that frequencies of the order 20 kHz to 50 kHz were used. Higher frequencies are more severely attenuated at cracks and construction joints, and direct transmission between transmitter and receiver may have been almost completely attenuated. In such a case the actual path along which the received sound travelled would have been made up of a series of straight paths between the extremities of individual cracks, which would be substantially longer than the direct distance between the transmitter and receiver. This would cause measured sonic speeds to be low.

In developing the method used at Val de la Mare we purposely used a lower frequency to ensure that direct transmission took place. That this has generally occurred is borne out by Fig.3 of our Paper 3.2. We considered it important that paths should not cross construction joints and that the average speed along several paths within any lift of concrete should be used as an index of its quality (Paper 3.2, Fig.8). The agreement between the speed calculated for the indivisual paths was generally excellent, so we feel happy that the sonic speed calculated was for direct transmission.

Although Professor Knill has used a low frequency source his measuring equipment was somewhat crude, and he has chosen to make measurements along the downstream face of his structures ignoring the existence of construction joints. It may be that his difference between vertical and horizontal sonic speed reflects the large number of horizontal to vertical lift joints in the typical structure. His data will also be influenced to a large extent by the quality of the skin of the dam, which is invariably poorer as cracks will open etc. It is considered then that Professor Knill's agreement with the Leslie and Cheesman classification may to a large extent be fortuitous, and that the sonic speed range corresponding to 'very poor' to 'excellent' quality should be 3500 m/s to 4600 m/s.

One of the problems facing an engineer studying a deteriorating dam is to assess how widespread within the structure the deterioration will eventually be. This may have a profound influence on whether the dam is repaired or replaced. Fig 9 of Paper 3.2 shows a plot of sonic speed versus casting date for Val de la Mare, and this clearly tied the problem to a period in June and July 1960. A vein of particularly poor aggregate was subsequently shown to have been worked in the quarry supplying aggregate about this period. This established that severe deterioration was unlikely to take place throughout the structure, but would probably be limited to lifts cast around the middle of 1960.

The need for sonic surveys has been questioned by the Reporter. At Val de la Mare, deterioration became apparent on the surface at about 4250 m/s. Thus only twelve or thirteen lifts could be suspected from superficial appearance at the time of the 1972 survey, although block movements suggested the problem was more serious. The survey revealed a significantly larger number of lifts to be suspect and many of these have subsequently shown deterioration on their downstream surface. We hold the view that in cases of selective deterioration the sonic survey provides a sound basis for locating coring and testing sites and can give prior warning of the likely extent of deterioration.

J D EVANS (South West Water Authority) :

Following Mr Kennard's comments concerning cement, I would draw your attention to Crowdy Dam, recently constructed in Cornwall. Here Sulphate Resisting Cement was used, not of necessity but because of its considerable low heat properties. It had the added advantages that it was much easier to obtain and somewhat cheaper than Low Heat Cement.

The temperature measurements were made during construction in a 19 mm dia. water-filled PVC tube cast into the concrete, using a dip thermometer. Although there were some problems due to displacement of water while lowering the probe into the tube, the method provided a simple and cheap method of monitoring the heat generated within the lifts.

Crowdy was also interesting in that the aggregates were both waste materials obtained locally. The coarse aggregate was slate quarry waste which was passed through a jaw crusher to produce an 'all in' aggregate graded from 75 mm to dust. By normal standards the aggregate produced is rather flakey and elongated and, with the dust content, looks extremely 'dirty'. However, the broken stone has a rough surface which ensures a good bond and the dust is an important factor in producing a concrete with a density of about 2500 kg/m^3. The sand was waste from a china clay pit and used only to make up a slight deficiency in the grading of the fines in the slate aggregate.

Dr. J E BUTLER (Portsmouth Polytechnic):

During a research programme conducted at Portsmouth and investigating the effects of water pore pressure on concrete tests were carried out which simulated uplift in concrete. One series of tests involved continually replacing the uplift force, created by a water pressure gradient, by an external mechanical tensile force. From the observed changes in strain, the area over which the uplift force existed, (η), was calculated. The calculated values for three concrete mixes are shown in Table A.

The principal conclusion drawn from these tests was that for mass concrete with no major cracks or construction joints the effective area (η) for concrete was initially a quantity substantially less than unity, but which changed with concrete micro-cracking to unity at failure. Uplift in inflawed concrete is a dam may therefore act on an area substantially less than unity. However, since the material in a concrete dam will have major construction joints and cracks the real effective area may be substantially larger.

Mix	w/c ratio	a/c ratio	η*
1	0.47	3.6	0.46
2	0.59	4.2	0.55
3	0.71	7.6	0.57

* concrete loaded to 50% of its ultimate tensile strength

Table A : Effective Area Coefficient: Portsmouth Tests

W J H RENNIE (Bush and Rennie Associates Ltd.) :

Mr Berry made reference to the Hermitage Dam in Jamaica. I was engaged in the Contractor's Office on the planning and the preparation of the tender and, later, was resident on the site during execution of the work. It may therefore be of interest if I fill in some of the details, as I understand proposals to heighten the dam are actively under consideration.

The gravity dam, some 230 m long and 38 m high above river bed, with a 6 m wide crest and 27 m thick at the base, was specified 'to be constructed of cyclopean masonry consisting of cement concrete in which displacers are embedded'. The corresponding item in the Bill of Quantities was simply 'Cyclopean Masonry in Dam'. The Contractors, Sir W G Armstrong Whitworth and Co Ltd, were well known in Newcastle upon Tyne as a heavy armament firm and had formed a new Civil Engineering and Contracting Department. In submitting their tender for the dam they intimated that their prices were based on the concreting being carried out with the Insley Concrete Chuting Equipment. In essence, this involved mixing the specification nominal 1 : 3 : 6 mix concrete with an additional 20% of coarse aggregate of 150 mm size - plums - through the mixer, making the combined mix a nominal 1:3:6.3. Mr Berry's figure for the resultant mix was, I think, given as 12:1. In my view, however, the figure for the mixed concrete is nearer to 8:1 and the crushing strengths which he obtained seem to confirm the latter figure.

The Hermitage Dam is sited at the confluence of two small rivers in the Blue Mountains, in an area subject to heavy precipitation - 240 mm had been recorded in 12 hours, representing a flow of about 85 m^3/s, and I have a note that in the year 1909 over eight consecutive days the rainfall was 3.43 m. During the period of construction the heaviest we experienced was 200 mm in 10 hours.

It was for this flood problem, as well as unexpected rock conditions met with in the foundations, that a number of modifications were introduced:

1 The dam was redesigned to a spill profile and the spillway length reduced to 142 m.

2 The dam section on the left bank, for a length of 52 m, was redesigned and constructed as a retaining wall 27 m high, 8 m thick at 15 m and below and with an embankment of selected materials from the excavation and the quarries.

3 The dam was increased to 230 m in length and the height of the dam at the deepest point was 43 m.

4 A concrete retaining wing-wall at the head of, and the spillway channel along, the downstream face of the left bank were additional works.

It should be noted that the original dam had no allowances for uplift and in consequence the redesign *also had no provision* for uplift.

The cement was shipped in barrels from the United Kingdom and delivered at £4.50/tonne. The coarse aggregate was a sound local limestone. A quarry was opened up nearby and it was crushed and screened to a maximum size of 38 mm at the dam.

The excavation was carried, often through densely packed granite boulders, down to the sound conglomerate, which was cleaned and washed by air and water under pressure. The first 300 mm of concrete was a 1:3:6 mix without plums, and on this bed the cyclopean masonry commenced. In addition to the 150 mm aggregate, displacers of selected granite boulders up to five tonnes in weight were handled by derricks and embedded in the concrete, while smaller boulders from the river or stone from the quarries were placed individually in the mixed concrete. The concrete was placed in 0.9 m to 1.2 m lifts across the dam without any contraction joints.

On the upstream face of the dam, below ground level, a concrete mix of 1:2:4 about 750 mm thick - minimum 500 mm - was deposited simultaneously with the adjoining cyclopean masonry, and above ground level to the crest the face was rendered with 13 mm thickness of cement and sand mix 1:2 applied by a cement gun in two thick coats. The specification read 'The dam on completion is to be left thoroughly water-tight to the satisfaction of the City Engineer'.

I understand that the reservoir silted up some years ago as a result of flooding and hurricanes but has been cleaned out by dredging and also that proposals for increasing the storage capacity by heightening and strengthening the dam are now being investigated by the Water Commissioners.

It may be of interest to record that the Contract Documents consisted of 24 pages including :

1 The Indenture — between the Contractor and the Client — 6 clauses.

2 General Conditions — 49 clauses, including four drawings.

3 Specification — 20 clauses.

4 Schedule of Quantities and Prices — 11 items.

5 Tender for Dam and Appurtenant Works — 2 pages.

6 Instructions to persons tendering — 1 page — 7 clauses.

R G COLE (T and C Hawksley) :

On page 2 of Paper 3.3 (Val de la Mare Dam) are listed several items that were investigated over a period of three years. I propose to amplify three of these items, which will also answer some of the points raised in the General Report.

MORTAR BAR EXPANSION TESTING

This test was developed in the United States and the details are given in ASTM C227. In principle it provides accelerated AAR by the use of high alkali cement and high storage temperatures. It is used as a guide to the degree of AAR to be expected from the use of a particular aggregate and to the resulting expansion. The ASTM also suggests limits on test expansion below which the aggregate would be considered suitable for use in concrete, these limits having been derived from experience on structures in the United States.

The tests on the various aggregates used at Val de la Mare and on a control aggregate were undertaken at the Building Research Establishment. (BRE). Summarising the results, Fig. A indicates that the expansions of the dam aggregates were up to twice that of the control, but in all cases were well below the ASTM limits of 0.05% at three months and 0.10% at six months. The quarry aggregate sample was from the area of the quarry used for the dam construction, consisting of granite and diorite and containing chalcedony and opal. Beach gravel 1 originated from the area on St Ouen's Beach used for the dam and consisted of rhyolite, granite, diorite, quartz, flints and acid volcanic glass. Beach gravel 2 was taken from the remains left in the storage bins at the dam site. Tests on the acid volcanic glass pebbles making up to 75% of the beach samples indicate this material to be non-reactive.

BRE has developed its own mortar bar expansion test (Research Paper 25, Part VI). This test is similar to the ASTM method but a higher alkali content is used, and the bars are stored at a lower temperature. The results are also shown on Fig. A, from which it can be seen that the expansions are generally similar to the ASTM results but with slightly higher expansions.

One conclusion that may be drawn from these results is that the ASTM limits are too high. The American reaction to this would probably be that the limits were established for aggregates and cement used under their conditions and were not intended for use in the Channel Islands. This indicates the danger of using standards derived for a particular set of circumstances on a universal basis, as the aggregate samples selected at random for these tests and containing reactive materials on the basis of this test alone would have been considered suitable for the dam.

Greater expansion has taken place in the actual dam, however, and it is clear in my opinion that higher intensities of reactive material, possibly approaching the 'pessimum' quantity, have been present in the aggregates at certain times during construction. Although the alkalis in the cement were known to vary they were never as high as in the cement used in the mortar bar tests, and this, in my opinion, is less likely to be the answer as to why there is such a difference in the expansion results from the tests and the expansion that has occurred in certain parts of the dam structure.

SONIC VELOCITY TESTING

Mr. Allen, in his Report, questioned why the table of sonic velocity to concrete quality quoted by Knill in Paper 3.5 indicated that the concrete at Val de la Mare was classified as 'generally good' or 'excellent' when, based on the results given by Sherwood, Marriott and Smith in Paper 3.2, this conflicted with concrete deterioration from AAR.

In my opinion such a comparison is not valid, as it is essential to appreciate that a correlation between sonic velocity and concrete quality should be developed for a particular dam structure. This is supported by comments in both the above papers that the velocity readings are affected by cracking and joint layouts within the mass concrete.

Several tables have been produced in addition to the one quoted in Paper 3.5. Mr Mather of the US Corps of Engineers developed one from information on Tuscaloosa Lock, which suffered from AAR, and one was also produced for Val de la Mare as below:

Sonic Velocity (km/sec)	Concrete Quality
4.26 - 4.87	Good
3.65 - 4.26	Doubtful
Less than 3.65	Poor

All these tables give a very basic correlation, but it has proved useful at Val de la Mare where readings at frequent intervals indicate the trend in concrete deterioration. Even then, though, it has proved better to undertake the subsequent tests during similar climatic and reservoir conditions to those existing during the original test.

At Val de la Mare we attempted to derive a more quantitative relationship between sonic velocity and concrete strength based on the results from concrete cores taken from the dam following sonic readings. No correlation was established with compressive strength of concrete. As the AAR had resulted in cracking, a correlation between concrete tensile strength and sonic velocity was tried. Fig.B shows the results of indirect tensile strength tests to BS 1881, plotted against laboratory sonic velocity. As can be seen there is a considerable scatter of results. However, if the *average* of the samples for each borehole is used, also shown on Fig. B, a reasonable relationship is obtained.

To complete the picture, the correlation between laboratory sonic velocity and in-situ velocity of the structure is shown in Fig. C, from Val de la Mare results. Again there is considerable scatter, but by taking the average of the laboratory results for each borehole the curve shown is obtained. This is similar to the curve given by Sherwood in Fig. 3 of Paper 3.2

INTERNAL UPLIFT PRESSURES

Piezometers were installed in three different monoliths depending on the sonic velocity concrete quality classification of 'poor, doubtful and good'.

Fig. D shows the results in a 'good' block (Block 14), where it can be seen that the internal uplift pressures are negligible and substantially below the design assumption.

Four piezometers were installed in Block 11, the results from which are shown in Fig. E. The lower two are in 'good' concrete and the upper two in a 'doubtful' concrete lift. In the latter case the pressure exceeds the design assumption near the upstream face but quickly dissipates, with little net change in stability compared to the original design.

Fig. F shows the results in a 'poor' concrete lift (Block 6). The uplift exceeds the original design assumption and this, together with several other factors, resulted in the decision to install anchor bars in this block to cater for the revised design assumption shown on the figure.

The average cost of installing piezometers at Val de la Mare, including mobilisation charges, temporary staging, drilling, supplying and installing the piezometers, grouting, cables and digital read-out equipment was £1200 to £1500 each.

FIG. A - MORTAR BAR EXPANSION TESTS

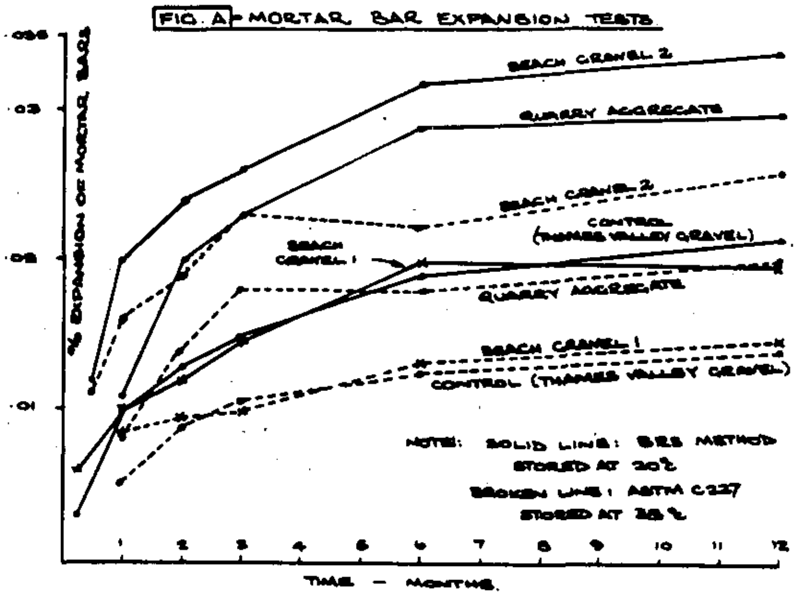


FIG B.

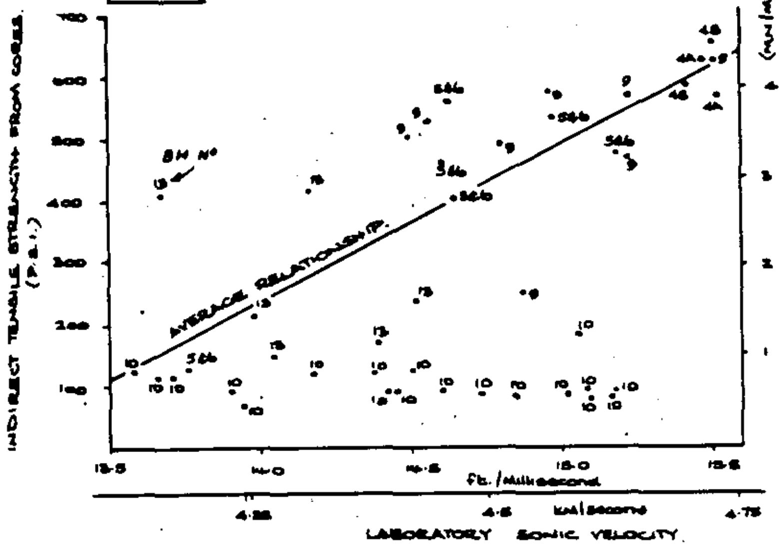
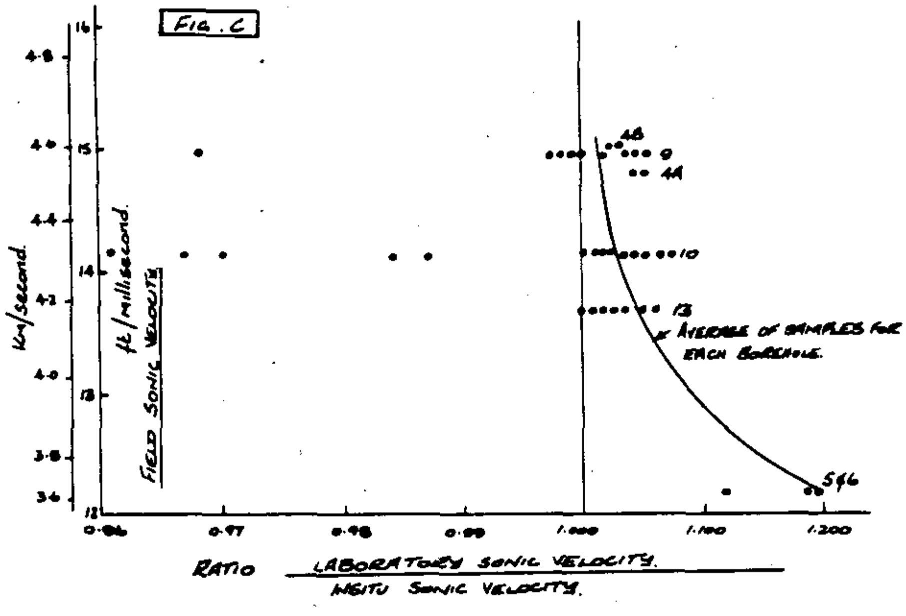
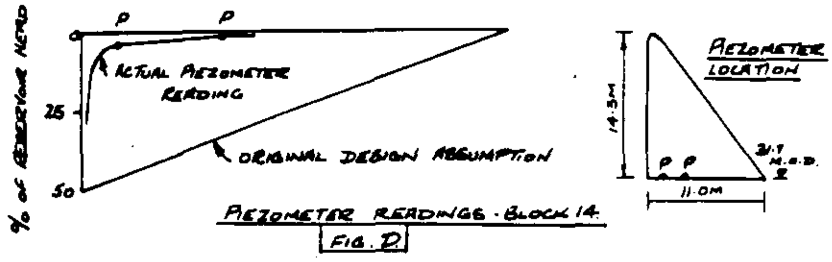

INDIRECT TENSILE STRENGTH FROM CORES (P.S.I.)
(MN/M²)
BH No
AVERAGE RELATIONSHIP.
Ft./Millisecond
KM/Second
LABORATORY SONIC VELOCITY

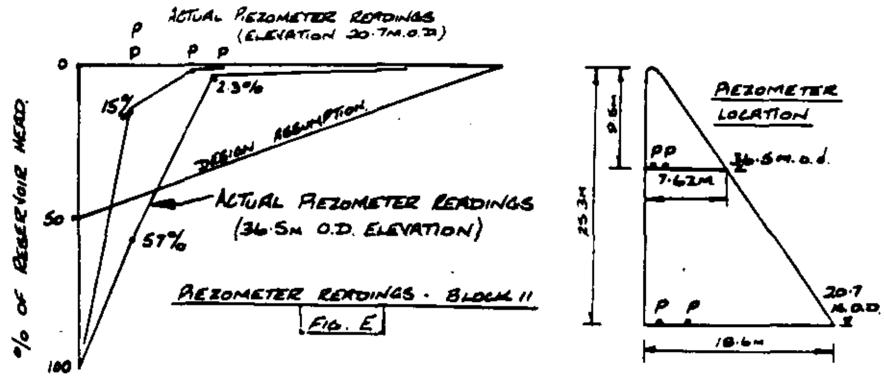
FIG. C


Km/second.
ft/millisecond.
FIELD SONIC VELOCITY
AVERAGE OF SAMPLES FOR EACH BOREHOLE.
RATIO
LABORATORY SONIC VELOCITY.
INSITU SONIC VELOCITY.



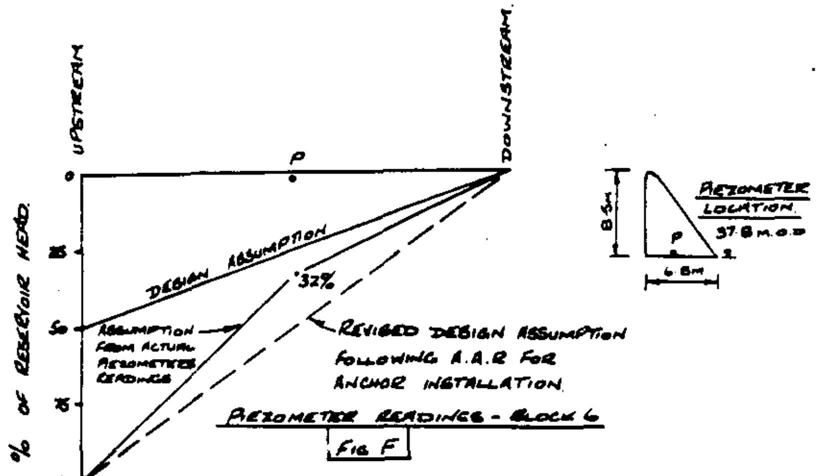
PIEZOMETER READINGS - BLOCK 14

FIG. D



PIEZOMETER READINGS - BLOCK 11

FIG. E



PIEZOMETER READINGS - BLOCK 6

FIG F

COLE, RG - VAL DE LA MARE DAM

R M ARAH (Binnie and Partners) :

In looking for an explanation for the steeper pore-pressure gradients produced by richer facing concrete have the biologists been consulted? Many reservoir micro-organisms are surrounded by gelatinous envelopes which could well clog the lattice of the cement structures. There was a suggestion that any richer facing conrete would be most useful as a protection against weathering at the downstream face, but the most frost-vulnerable zone is surely near the reservoir water-level?

PROCEEDINGS: TECHNICAL SESSION 4

FLOOD ANALYSIS, PREDICTION AND DESIGN IN RELATION TO RESERVOIRS, DAMS AND SPILLWAYS

Cont.

SELECTION OF DESIGN FLOOD – THE ENGINEER'S DILEMMA

K T Bass, BSc CEng FICE FIWES MConsE

PARTNER

ROPE KENNARD AND LAPWORTH

SYNOPSIS

As a result of a study of published data relating to the design of spillways for dams constructed prior to 1960 it appears that an estimate of the maximum flood plus an allowance for the unknown formed an accepted basis which could be adequately substantiated if need be. Following the Report of the Institute of Hydrology, the responsibility of the Engineer has changed in that he has to select a design flood from a wide range with varying return periods,which is no longer an engineering decision. It is suggested that the Engineer should be relieved of this responsibility.

INTRODUCTION

There is but one thing certain about a 'Design Flood' which is that the chance of its occurrence is so remote that it can be ignored. This is so whether the return period of the 'Design Flood' is small or large,for floods by their nature are rare and the storm patterns producing these are unique. Hence flood hyrographs are unique.

When a flood occurs it may be below the 'Design Flood' and the engineering works provided will almost certainly prove adequate and nobody is worried or suffers as a result. On the other hand the flood may be larger than the 'Design Flood' in which case concern will be occasioned as to whether the design was adequate. The extent of criticism of the design standard will probably depend on who suffered rather than on what damage and difficulties resulted from the engineering works being overtopped. Moreover, no account will be taken, or credit given, for any reduction in damage due to the works.

DESIGN FLOOD APPLIED TO SPILLWAYS PRIOR TO FLOOD STUDIES REPORT

The Reservoirs (Safety Provisions) Act 1930 restricted the design of reservoirs to those Engineers appointed to a Panel approved by the Secretary of State. It thereupon became incumbent on these individuals to select 'Design Floods' for impounding reservoirs for which they were responsible. By a study of published data on the design of spillways for reservoirs constructed in the United Kingdom it was hoped to show how engineers arrived at the design flood, and to indicate the various standards adopted in pre-war and the immediate post-war periods. Unfortunately due to lack of details of some types of overflow coupled with varying usages of the terms 'design flood', 'maximum flood', 'catastrophic flood', etc. it has not been possible to produce a meaningful table.

The only clear indications are that where bellmouth overflows are involved a generous figure for the gorging condition has been adopted as compared with the assumed 'catastrophic' flood for other types of spillway, but in these cases some extra freeboard together with wave wall or parapet has been provided which, whether intended or not, caters to some extent for the unknown. One problem is that a similar allowance in the spillway channel gives a smaller increase in capacity than the extra freeboard would give in free discharge over the spill weir. The Author's recent experience suggests that many reservoirs will be found to suffer from the inadequacy of the spillway channel when the new standards are applied.

It would appear that with the state of knowledge in 1930 and for years afterwards spillways were designed to pass a flood which the Engineer, based on his own experience, considered to be a maximum for the catchment. No doubt the Engineer had his own empirical formula or method for assessing design floods. Several such formulae were quoted by D B Richards in 'Flood Estimation and Control', first published in 1944. The methods range from a detailed study of rainfall/run-off, taking into account various factors relating to the catchment, to an adjustment of the ICE 'Normal Maximum Flood' by a factor to allow for either greater or lesser degrees of exposure or of altitude etc. On the whole, as these were used by Engineers with considerable experience, they seem to have worked very well possibly, aided by the additional freeboard referred to above and the fact that reservoirs are usually drawn down in the summer when thunderstorms are most likely.

Fortunately no disasters have occurred in the United Kingdom since 1930 which can be directly attributed to overtopping of a dam as a result of providing an inadequate spillway capacity. However, even if a disaster had occurred, it seems certain that with the state of knowledge available to the Engineer at that time he could have produced evidence of the enveloping curve type to substantiate his estimate of design flood. Having provided additional capacity above the calculated figure he could not reasonably have been accused of negligence in assessing the design flood. The Engineer's responsibility was purely to assess the largest flood which could be expected to enter the reservoir and to make adequate provision for its safe discharge below the dam. This was substantially an engineering exercise well within his experience and the technical knowledge of the time.

DESIGN FLOOD FOR SPILLWAYS IN THE FUTURE

With the Report of the Institute of Hydrology on Flood Studies the situation has changed somewhat dramatically. By following the methods set out in the Report it is now possible to make an estimate of floods having return periods of at least up to 1 in 100,000 years, and furthermore to obtain an estimate for the Probable Maximum Flood (P.M.F.) based on the Probable Maximum Precipitation (P.M.P.). Firstly it is very difficult for an engineer to appreciate why the P.M.F. could not be awarded a return period as other floods and why P.M.F. + 1 is impossible. It becomes even more difficult to appreciate what is intended by this term when one considers the results which can occur. For instance in the case of the floods estimated for Ardingly Reservoir, which were provided in a report on the catchment by the Institute of Hydrology, the following figures were given or can be obtained.

the 150 year flood	22 m^3/s
the 15,000 year flood	61 m^3/s
the 100,000 year flood	90 m^3/s
the Probable Maximum Flood (PMF)	156 m^3/s

There is thus an enormous range between the 150 year flood and the P.M.F., and one wonders how an Engineer can be expected to select a design flood from this range. It is often not too difficult to cope with the problem when a new reservoir is being designed since a high figure for the design flood can be accommodated with little increase in cost, but when the reservoir already exists and may well have been in being for nearly 100 years there is a very real problem in deciding what additional overflow facilities should be provided. The Institution of Civil Engineers Floods Committee appreciated this point and, in their Discussion Paper, (1) suggested that provided the existing spillway would pass a 150 year flood the remainder could be put over a supplementary spillway.

In the case of Ardingly Reservoir this would mean that should the P.M.F. occur the supplementary spillway would have to take something like 6 times the quantity of water passing down the original overflow channel. This proportion seems quite unrealistic. A more acceptable distribution between main and supplementary spillways might be 50 : 50 such that each should be capable of passing 78 m^3/s. This is practically equal to the Upper Limit Flood recommended in the suggested screening process 3.7 of the Discussion Paper, which for this catchment amounts to 77 m^3/s and represents a flood having a return period of about 40,000 years. Is the screening process useless or the P.M.F. too big?

Rare events do occur, but it is only the disasters, rare events usually involving loss of life, that hit the headlines. In such cases the general public are the judges and in respect of the failure of a dam by overtopping an answer to the question posed above based on an engineering assessment is not likely to get universal acceptance.

A look at the statistics relating to deaths in the Registrar General's returns may give some idea of public opinion on such matters and a guide as to the standard for design floods.

The odds of suffering a violent death (taking male and female together) are:-

By homicide, about 1,500 to 1 against
Due to falls, about 110 to 1 against
On the roads, about 85 to 1 against
Any violent death, about 27 to 1 against.

The above are affected by or are a direct result of human behaviour, and the odds are not long. Risks which may be nearer to natural events, albeit still dependent on medical achievement, are the following:-

Stillbirth,about 90 to 1 against

Death before the age of 1 year about 60 to 1 against

The above two together about 35 to 1 against.

These odds are not so different from those of violent death.

The above events in terms of tragedy for the family are more or less equal, yet the reaction of and the cost to the community are greatly different. No attempt has been made to ascribe a cost to a death in each of the various categories, but there can be no doubt that, taking into account the police investigation, public trial and subsequent maintenance of the convicted criminal, a homicide which has the longest odds also involves the greatest cost. On this basis there can be no doubt that the recommended standard for community protection as given in the Discussion Paper, whereby the spillway should be designed to pass the outflow from a P.M.F, is the only acceptable answer. Moreover there can be no relaxation for existing reservoirs but can the country afford to spend much to make a rare event even more rare?

It has been shown by Lowing (2) that the ratio of P M F to the 150 year flood may vary between 2.8 and 8.9, and further examples have been given by Farquharson (3). It appears therefore that the P M F may be only a little more than or possibly twice the 100,000 year flood. With this uncertainty from an engineering point of view it seems that the P M F has received an undue prominence in the recent Flood Studies and has cast a somewhat academic cloud over an already difficult problem. Since the Upper Limit Flood previously referred to was based on the floods at Lynmouth and in the Wray Valley, it seems clear that anything above a 100,000 year flood is bound to constitute a national disaster even without a reservoir being situated in the valley. In these circumstances compensation and repair of damage would be met on a national basis and it is not unreasonable to expect the country as a whole to accept the added risk of a reservoir.

No matter what the Floods Committee may ultimately recommend with regard to reduced standards for reservoirs where loss of life is unlikely the Engineer will still have to carry the full responsibility of selecting anything less than the P M F unless the Committee's recommendations are given some form of Government backing. Table A.1 of the Discussion Paper sets out four categories for reservoir flood protection. Something on these lines, if generally accepted by Panel Engineers, could reasonably form a basis for statutory limitation. In the Author's opinion the table itself should be modified slightly to give higher flood protection to the lower risk reservoirs, and the following is put forward for consideration:-

Category A : All reservoirs posing a threat to life and new reservoirs.

This category includes all new and existing reservoirs other than those in Category C where a breach of the dam may endanger lives or result in extensive damage.

Such reservoirs should generally be capable of passing at least a 100,000 year flood, but would be considered satisfactory if they are able to pass a 10,000 year flood with negligible risk of a breach if the larger flood occurred.

Category B : Existing reservoirs unlikely to cause much damage.

This category includes all existing reservoirs where a breach of the dam would pose a negligible risk to life and would only result in limited and short term damage.

Such reservoirs should generally be capable of passing at least a 10,000 year flood but would be considered satisfactory if they are able to pass a 1,000 year flood with negligible risk of a breach if the larger 10,000 year flood occurred.

Category C : Small reservoirs and other special cases (new and existing).

This category includes reservoirs where no loss of life can be reasonably foreseen as a result of a breach of the dam and such a breach would either;

a) not add significantly to the effect of the flood or

b) cause very limited damage.

Such reservoirs should be capable of passing a 1,000 year flood.

At the end of Table A.1 of the Discussion Paper note (iii) suggests that after some experience 'n x 150 - year' be substituted for the return period quoted in the text. With a choice of growth curves for somewhat arbitrary regions there seems to be a better case for adopting 'n x 150 - year' now and substituting the return periods when more experience of their estimates are available. The relationship between the categories suggested above can be seen from the following table based on the Ardingly figures.

Table giving ratio of:- $\frac{\text{Flood at head of column}}{\text{Flood at beginning of line}}$

Return Period Years	150	1,000	10,000	100,000	1,000,000	P M F
150	1	1.5	2.5	4.2	6.2	7.1
1,000	0.69	1	1.8	2.8	4.3	4.9
10,000	0.39	0.57	1	1.6	2.4	2.8
100.000	0.24	0.36	0.62	1	1.5	1.7
1,000,000	0.16	0.24	0.42	0.67	1	1.1
P M F	0.14	0.21	0.39	0.58	0.9	1

SUMMARY

It would appear that the problem facing the Engineer has changed from a purely technical one of assessing a maximum flood,which he could reasonably be expected to handle, to a sociological one of selecting a degree of protection for the public and property downstream of the dam. It is suggested that the Engineer should be relieved of this responsibility and that the country should carry the risk of floods in excess of 100,000 year return periods for new dams and for a lesser return period for existing reservoirs. The Engineer would then merely be responsible for making a proper estimate of these figures and revert to his original role of making estimates and decisions on purely engineering matters.

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FLOOD ASSESSMENT FOR HYDRO-ELECTRIC PROJECTS

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SYNOPSIS

A brief description of the current practice of the North of Scotland Hydro-Electric Board in relation to floods is given. The Beauly hydro-electric scheme is used to illustrate application of the methods adopted with an indication of how operating procedures have been developed to cope with specific problems.

INTRODUCTION

The North of Scotland Hydro-Electric Board have now operated most of their schemes for 15 or 20 years and it was considered necessary to review the flood capabilities and operating procedures for a number of them, particularly those which have been found to have specific flood problems. Most of the Board's schemes were originally based on the Interim Report on ' Floods in Relation to Resevoir Practice ' published by the Institution of Civil Engineers, although on occasions recourse was made to an analysis of the historic record of floods if this was available. Without going into detail, the Report is now felt to be weak for large catchments where hydrograph shape is undefined and initial conditions unrealistically severe and, in addition, the range of return periods which can be associated with a normal maximum of catastrophic flood is too great to be of use.

The work reported in this paper was carried out prior to the publication of the recent Flood Studies Report of the Institute of Hydrology, and some adjustments will be carried out in future work to comply as far as possible with the recommendations of that report. As the general approach of associating a return period with flood magnitudes is accepted, it was necessary to set down general flood protection standards geared to this approach. These are given in detail in Paper 2.3 (1) from which it will be seen that the 1000 year flood has been adopted as the basis of the standards. It is intended that these standards be revised to comply with those being prepared by the Institution of Civil Engineers, and at present under discussion, when the latter are finalised.

BEAULY SCHEME

At the time of writing, reviews of the Awe, Shin and Beauly schemes have been completed but this paper limits itself to consideration of the Beauly Scheme as being more complex than the other two. Figure 1 shows the layout of the scheme, from which it will be seen that the catchment area can be split up conveniently into six sub-catchments. Although Loch Benevean provides a useful amount of storage for operational purposes, major storage for flood relief is confined to Loch Monar and Loch Mullardoch. During floods the maximum flow in the Rivers Affric and Farrar is fairly critical but concern is mainly directed towards the maximum flow passing Aigas and Kilmorack dams in relation to the capacity of these dams.

HYDROLOGICAL DATA

The objective of this study was to obtain an estimate of the run off which would result from a storm event with a 1000 year return period. The run off depends on a number of parameters of varying importance and these may be listed as follows

a) Rainfall depth
b) Rainfall duration
c) Rainfall profile
d) Antecedent conditions
e) Snowmelt

A storm of a specified duration and return period can result from a rainfall depth of that return period combined with the other parameters set at their most likely (median) condition. It is equally valid to set these parameters at other than their median condition and, by adjusting the return period of the rainfall, derive other storm events with the specified overall return period. It was considered sensible, however, to set the antecedent conditions and snowmelt to their median condition and to consider only combinations of rainfall depth, duration and profile.

RAINFALL DEPTH AND DURATION

This topic is dealt with fully by Reynolds (2) and it is sufficient to say that rainfall depths for storms of 24, 48 and 72 hours duration, and for selected return periods between 50 and 1000 years, were extracted for each sub-catchment, making allowance for areal reduction and snowmelt.

RAINFALL PROFILE

The profiles of rainfall with time were based on information supplied by the Meteorological Office for the median case and for exceedance probabilities of 25%, 10% and 5%.

ANTECEDENT CONDITIONS

Experience indicates that major rainfall events in the Scottish Highlands are often preceded by wet weather for a week or so before the major event itself. It was felt therefore that median conditions immediately prior to such an event cannot be considered to be the same as the long term median conditions. A pilot study of rainfall prior to several major rainfall events was carried out, and it was concluded that the median daily rainfall for the previous week could be considered to be approximately twice the average daily rainfall. Accordingly an antecedent river flow of twice average annual flow was adopted. It was also considered that under these conditions soil moisture deficit on the relatively impermeable catchments in this region was unlikely to be significant and zero deficit was adopted throughout.

BASE FLOW

If all the rainfall during an event is considered to contribute to flood run off then it is logical that the base flow should be allowed to recede as it would without further rainfall. An inspection of river flow records suggested that an initial river flow of twice average annual flow would typically fall to half that flow in 72 hours. The relatively small flows involved did not justify the use of a sophisticated decay curve and linear decay was adopted.

UNIT HYDROGRAPH

The unit hydrograph method was used for the conversion of the rainfall depth and its associated profile into a storm run off hydrograph. Various shapes of hydrograph were tried with a range of rainfall profiles and durations and the shape finally adopted chosen rather subjectively, on the basis of Board experience with typical run off hydrograph shapes. The point should be made here that for this particular study, where the storm duration is fairly long in relation to the time of concentration of the sub-catchments, the shape of the unit hydrograph was not critical.

The Flood Studies Report differentiates between the flood event with a given return period and the flood caused by a storm event of that return period, and indicates the storm duration, storm profile and antecedent conditions which should be used in conjunction with the rainfall depth of the desired return period to yield the run-off with the same return period.

The Author submits that, when dealing with a reservoired catchment, where interest is largely confined to the routed outflow, it is not possible for a general recommendation to be made. For example it is very apparent that the size of a reservoir dictates to a great extent the relative importance of rainfall depth and rainfall profile. It is therefore felt that in such cases, and in particular when dealing with multiple reservoir systems, that combinations of the important parameters require to be studied to establish the most critical combination for the particular investigation. In order to apply this philosophy it becomes necessary to redefine the T-year flood as the flood resulting from a T-year storm event comprising the worst combination of parameters that can be found consistent with maintaining that return period. It is considered that, although involving considerable extra calculation, this approach is necessary and in addition gives more confidence to the designer by providing information as to the relative importance of the parameters involved and giving a better understanding of a problem.

FLOOD ROUTING

A computer programme was developed to route floods through the whole basin with sufficient flexibility to allow the effect of variations in hydrological parameters, reservoir storage, aqueducts and machines to be investigated. Space does not permit a full description of the programme to be presented, but in outline it performs routing calculations for each reservoir based on the run off hydrographs for the appropriate sub-catchment after making adjustments for aqueduct and machine flows and, where applicable, for the discharge from upper reservoirs. Where attenuation in a river reach is significant this is taken into account, but generally it was considered adequate to simply use a delay time based on field measurements.

In addition to the normal numerical checks of the programme, an attempt was made to verify the programme by trying to reproduce the conditions which occurred during a severe flood in December 1966. The rainfall depths of the six sections of catchment were known reasonably accurately and sufficient information was available from recording rain gauges to allow an estimate of rainfall profile to be made. With the initial base flows modified to estimated actual values, the programme gave routed hydrographs throughout the basin which were sufficiently close to the recorded hydrographs to give confidence in the programme.

MONAR GATES

At the design stage of the Monar scheme a restriction was placed on the maximum permissible water level. The Board wanted to make the fullest possible use of the available storage and ten tilting gates were installed in preference to a fixed spillweir in order that the rise in reservoir level to pass a design flood would be kept small. With this arrangement the discharge from the reservoir during floods was similar to that from a fixed spillweir of about 1500m in length and, despite the increased reservoir area following impounding, it was inevitable that discharge from the reservoir would substantially exceed the pre-scheme discharge from the loch if the initial level in the reservoir were high. As a consequence the Board became very conscious of the need to maintain freeboard in the reservoir and the introduction of operating procedures aimed at so doing negated the original intention of fully utilising the available storage. The Board have now locked the tilting gates in fixed partially open positions, thereby effectively producing a fixed stepped spillweir incapable of passing greater flows into the river than occurred prior to the construction of the scheme. This arrangement has the additional advantage of eliminating the need for de-icing heaters and anti-chatter devices, both of which gave rise to some concern. The latter required manual insertion and withdrawal of locking pins on each gate.

This section has been included to illustrate the point that, while there obviously must be sufficient discharge capacity for structural safety in any design, it must be borne in mind that by trying to restrict the reservoir rise by installing gates or a long spillweir, flood problems which did not previously exist may be introduced into the river downstream of a structure.

RESIDUAL FLOOD

The residual catchment area below Benevean and Beannachran dams is 313 km^2 but, bearing in mind that the storage in these two reservoirs is comparatively small the uncontrolled catchment area is effectively 609 km^2. This is a high proportion of the total catchment area of 878 km^2 and there was concern that short duration storms with a very 'peaky' profile might give rise to high uncontrolled inflows to Aigas headpond. This aspect was investigated by considering the 24, 48 and 72 hour storms with Monar and Mullardoch initially sufficiently low to prevent spill occurring. With each storm duration four rainfall profiles were tried each being associated with a rainfall depth of a return period calculated to retain the overall return period of 1,000 years. These twelve storms were routed through the basin using the computer programme, each yielding an inflow hydrograph to Aigas headpond. Figure 2 shows the results obtained and was interpreted as serving to show that for this catchment it is adequate to consider only median profile storms and that 24 hours is the critical rainfall duration although the sensitivity to duration is not great.

Recognising the possibility of partial gate failure or machine outage it was considered prudent that the maximum inflow to Aigas headpond, consistent with not flooding Aigas or Kilmorack power stations, should be restricted to 1130 m^3/s. The flood peaks shown in Figure 2 do not exceed this restriction.

UPPER RESERVOIRS

Median profile storms of 24, 48 and 72 hour duration were again routed through the basin, in this instance with the upper reservoirs initially at spill level. The 48 hour storm was found to yield the highest inflow to Aigas headpond with 1310 m^3/s, which exceeds the design limit imposed. It is interesting to note that the critical duration changes from the 24 hour storm, when dealing with the residual catchment only, to the 48 hour storm when the storage of the upper reservoirs is involved. This study indicated that, despite the improved spill characteristics at Monar Dam, some freeboard at one or both of Monar and Mullardoch reservoirs was still required to restrict the inflow to Aigas headpond to the design limit.

FLOOD RELEASE LEVELS

For all their major reservoirs the Board have assigned a level, referred to as 'maximum output level' (MOL) below which generation is at the discretion of the Board's Central Control and dictated by the requirements of the electrical system. When a reservoir exceeds this level, the importance of flood regulation is recognised and generation decisions are transferred to the discretion of the Generation Engineer for the reservoir in question subject to adherence to any flood procedures in force. In this event, a programme of heavy generation is normally implemented, frequently being the maximum possible of full load for 24 hours per day. These MOLs have been derived purely on the basis of the value of the water liable to be lost in spill and without reference to flood regulation or possible flood damage. It was decided that for Monar and Mullardoch reservoirs additional levels would be assigned at which all possible measures should be taken to restrict further rises in level. These levels are referred to as 'flood release levels' (FRL) and are defined as the levels which, if occurring both at Monar and Mullardoch at the onset of the worst 1000 year rainfall event, would prevent the peak inflow to Aigas headpond exceeding 1130 m^3/s. In practice, when an FRL is reached in a reservoir, contributory diversion aqueducts are shut off and generating plant is utilised to its absolute maximum. If this is insufficient to contain the reservoir rise then the ground sluice is opened, if necessary to its maximum.

DETERMINATION OF FLOOD RELEASE LEVELS

A series of computer runs was carried out for the 48 hour median storm in which the initial level in Monar Reservoir was sufficiently low to prevent spill occurring there while the initial level at Mullardoch Reservoir was progressively reduced from spill level in order to assess the value of it's freeboard storage. The result is shown in Figure 3 from which it can be seen that, from the point of view of peak inflow to Aigas headpond, no further benefit can be derived by increasing the initial freeboard at Mullardoch beyond about 1.8 m. A similar series of computer runs was carried out with Mullardoch Reservoir initially sufficiently low to prevent spill while the initial Monar level was progressively reduced. The conclusion reached in this case was that no further benefit can be derived by increasing the initial freeboard at Monar beyond about 0.9 m. These studies suggest that in practice it is desirable that the freeboard at Mullardoch is twice that at Monar, and accordingly a final series of computer runs was carried out in which the initial levels at both reservoirs were progressively reduced together while preserving the ratio of two between the initial freeboards at the two dams. The result of this study is shown in Figure 4 which enables the initial levels at the two reservoirs to be selected as a pair to meet a chosen criterion. The levels at present adopted are 224.3 m at Monar and 247.8 m at Mullardoch, which should limit the inflow to Aigas to 1110 m^3/s for the worst 1000 year storm event. These levels are the FRLs mentioned in the last section.

STRUCTURAL SAFETY AT AIGAS AND KILMORACK

Since the machines and ground sluices are capable of extracting about eight times average flow from both Monar and Mullardoch reservoirs it is very unlikely, even bearing in mind the possibility of machine outage, that the FRLs will be significantly exceeded at the onset of a severe flood. It was considered that the risk of loss of initial freeboard was acceptable from the point of view of material damage but unacceptable for structural safety of Aigas and Kilmorack dams. A check was therefore carried out making the rather drastic assumption of loss of freeboard and complete failure of all gates at Aigas and Kilmorack and, although massive over-topping of these structures would occur, resulting in extensive damage, mainly of Board property, it was established that structural failure would not occur.

CONCLUSIONS

Studies of this nature give a much greater insight into the flood control problem in a catchment and the North of Scotland Hydro-Electric Board acknowledge that with the publication of the Flood Studies Report a major step forward has been made by enabling such studies to be carried out on a much firmer footing than previously possible. The Board are indebted to the Flood Studies Team and the Meteorological Office for making information available to the Board prior to official publication of the Report.

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2 Reynolds G (1975) *Extreme Rainfall Estimation for Flood Studies in the Scottish Highlands.* Paper 4.4 BNCOLD/Univ. Symp. Newcastle upon Tyne

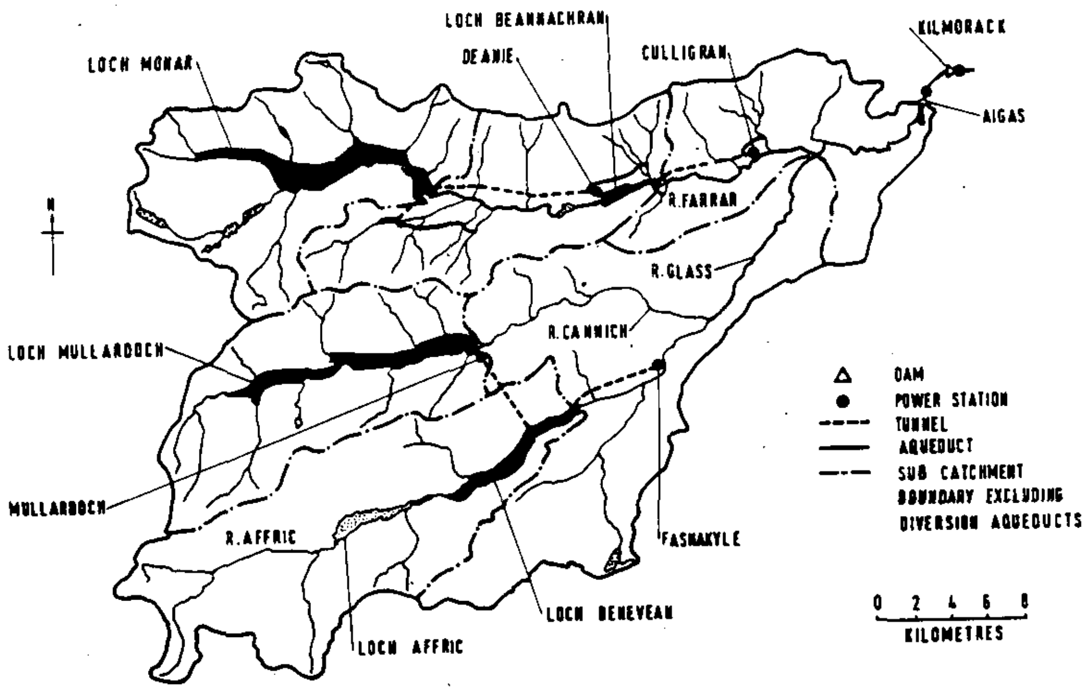


Fig. 1 — Plan of Beauly Basin

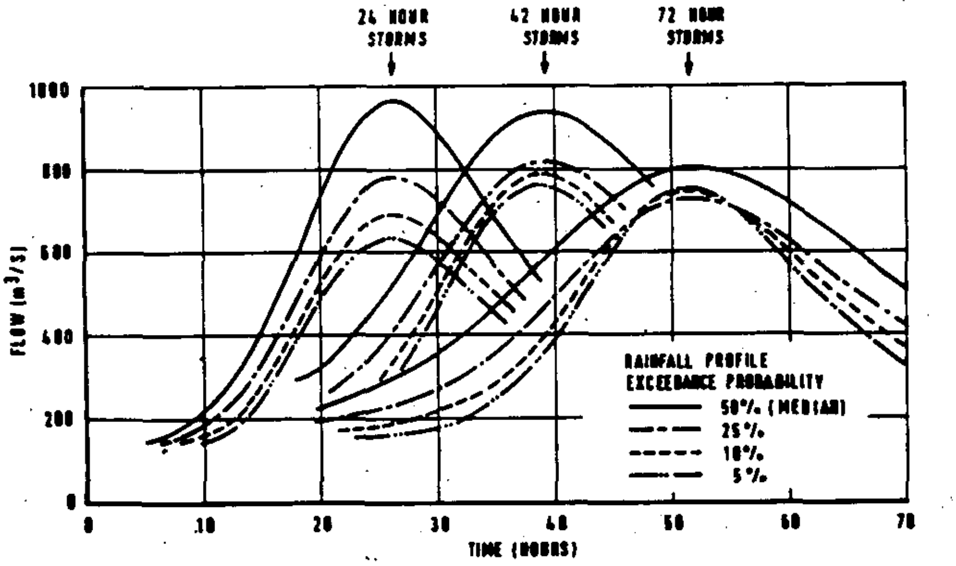


Fig. 2 — Inflow to Aigas Headpond for several 1000 year storms with no spill from Monar and Mullardoch Reservoirs

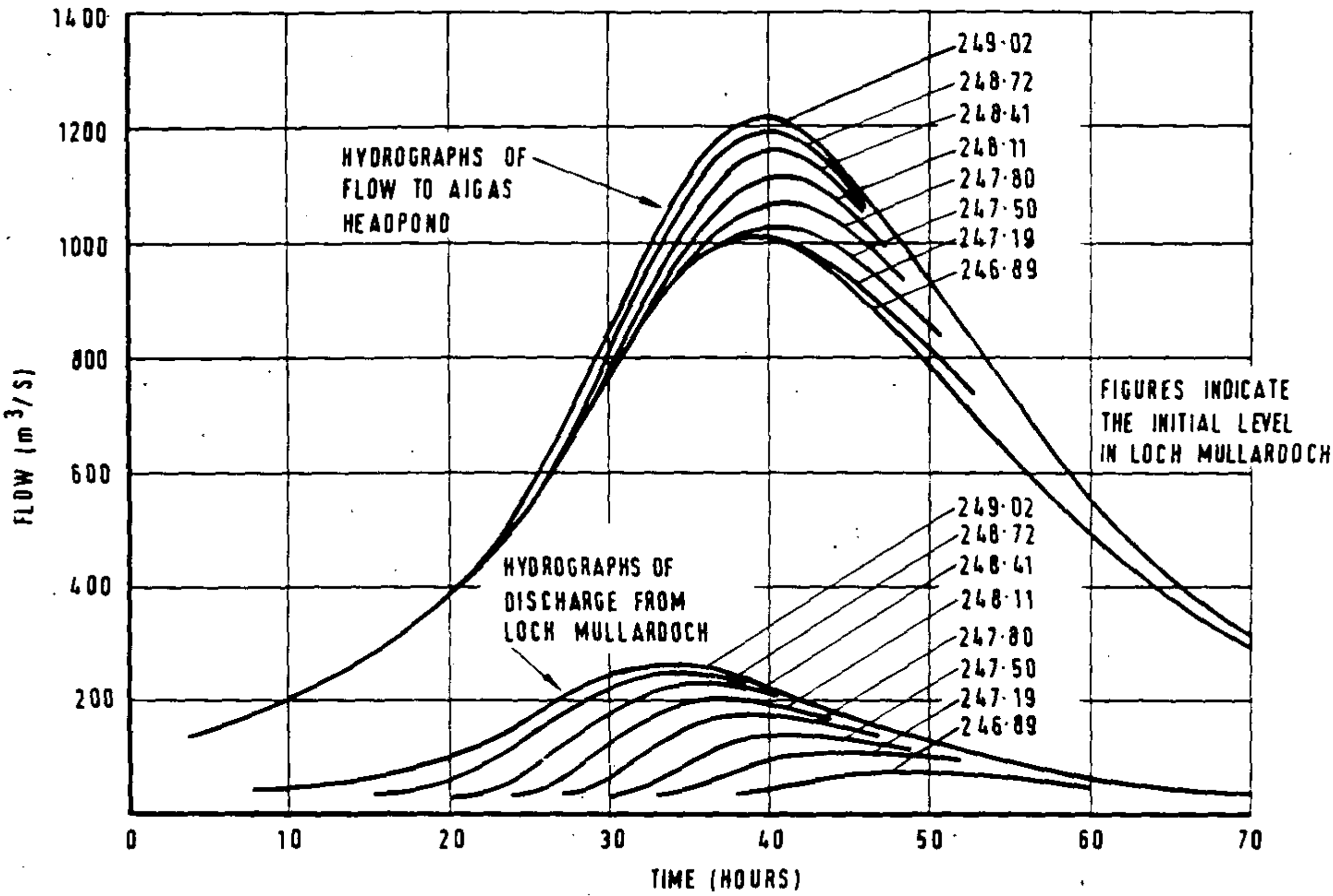


Fig. 3 — Inflow to Aigas Headpond for a range of initial levels in Mullardoch Reservoir and no spill at Monar Reservoir

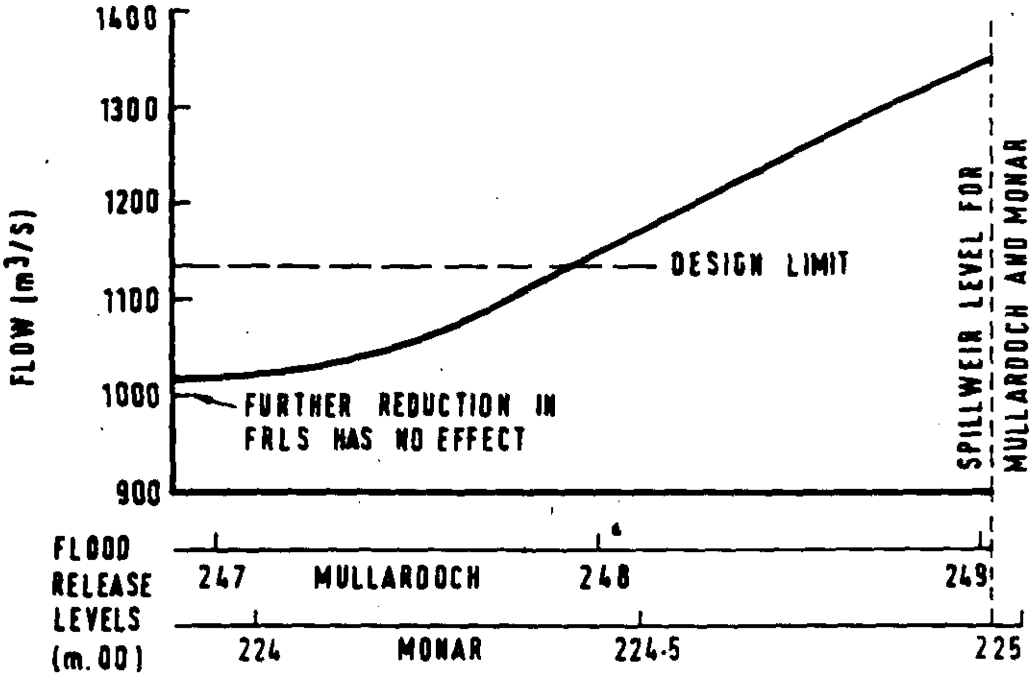


Fig. 4 — Inflow to Aigas Headpond in relation to Flood Release Levels

DISCUSSION : TECHNICAL SESSION 4

FLOOD ANALYSIS, PREDICTION AND DESIGN IN RELATION TO RESERVOIRS, DAMS AND SPILLWAYS.

Session Chairman : J PATON BSc CEng FICE
Senior Partner,
Babtie, Shaw and Morton

General Reporter : F M LAW BSc CEng MICE
Hydrologist,
Binnie and Partners
(Technical Secretary, ICE Floods Working Party)

CHAIRMAN : J PATON

Good morning Ladies and Gentlemen. If I may introduce myself, I am John Paton, senior partner in a firm of consulting engineers. I have been engaged in the dam business for 35 years and I am a member of Panel I under the Reservoirs (Safety Provisions) Act.

The subject we are to discuss, namely 'Flood Analysis, Prediction and Design in Relation to Reservoirs, Dams and Spillways' is of particular relevance at this time, having regard to the recent publication of the Flood Studies Report. This Report was, of course, followed by a discussion paper from the Institution of Civil Engineers entitled 'Reservoir Flood Standards'. In the foreword to that discussion paper the Institution declared their intention to release during 1976 a new publication entitled 'Floods and Reservoir Safety - an Engineering Guide'. This publication will supersede earlier publications dealing with floods in relation to reservoir practice.

It is further suggested in the foreword to the discussion paper that the new guide will only be as good as the breadth of experience that underlies it. With that in mind the Insitution organised a Flood Studies Symposium in London in May last with a view to drawing contributions from a range of engineers. It is the intention that contributions submitted then and at this Symposium should be considered when finalising the 1976 Engineering Guide. I trust that these remarks will serve to impress on those present the special importance of this Session.

The quite formidable task of summarising the contents of the papers and bringing forward the highlights for discussion rests with our General Reporter, Mr Law. He is eminently suited to undertake this duty, however, as he serves as Technical Secretary to the ICE Floods Working Party, in addition to which he is chief hydrologist with Binnie and Partners. I have very much pleasure in calling on him to present his General Report.

REPORTER : F M LAW

The publication of the 'Flood Studies Report' in early March and the consequent publication of the Discussion Paper on 'Reservoir Flood Standards' later that month has led to a new situation in dam inspections.

I think, tongue in cheek, that it may well be that, as in 1933, there will not be unanimous agreement that we have had a great leap forward in the right direction! However, Technical Session 4 was designed to attract papers that would not only forward knowledge with respect to existing dams but would also help the Floods Working Party of the Institution of Civil Engineers formulate standards which can be commended to Panel Engineers for general guidance in the future. The papers that have been prepared I have found to be commendably readable as well as eminently useful to us.

When I approach this topic I am conscious of not being a Panel Engineer, but perhaps a few brief statistics will help provide the setting for the problems that the Panel Engineer is up against. Here I would draw attention to the title of Paper 4.1 by Mr Bass - 'Selection of Design Flood - the Engineer's Dilemma' - the dilemma between safety and economy:

1 There are about 3000 dams in this country, of which just over 15% are Large Dams, i.e. on the ICOLD World Register.

2 There are perhaps 90 active Inspecting Engineers with - my estimate- a 20 year working life on the Panel.

3 Paper 1.4 by Mr. Moffat has shown there is just under a two to one chance against a major incident to a UK dam in any one year.

From these points one can compute the following :

4 A Panel Engineer has about 1 in 8 chance of handling a major incident in the course of his career,

but, 5, he has only about a 1 in 50 chance of it involving a large dam, and anyhow:

6 only some 25% of events are likely to be attributable to flood or waves, the remainder being structural or geotechnical problems.

The chances are thus reasonably slim, but nevertheless real, that incidents to do with flooding will occur within a career lifetime.

If I may come to the main point of the papers, I think the other authors will not mind if I suggest that the main paper in front of us is Paper 4.1, because it brings us immediately to the problem of design standards. Mr Bass concludes that the standard of using a Probable Maximum Flood (PMF) routed through a reservoir is the only acceptable answer if a community has to be protected. He comes to that conclusion after an excellent analysis of the risks if death that we run from various causes, and he suggests that if auxiliary spillways are needed to take major floods then these spillways should not take more than the main spillway is taking at the height of a rare flood. He then comes to his main thesis, that the engineer should really be relieved of his responsibility for design flood selection. Mr Bass is, in fact, suggesting that Government should take part of the risk in major incidents by putting a statutory limitation on the size of flood that needs to be designed for at certain sites, and therefore he really puts forward an alternative to the list of standards in Appendix A of the Floods Discussion Paper. Appendix A did not receive much attention at the May ICE Symposium, but it warrants more thought because standards are there set forth which deal with the risk of breaching at dam sites as well as with the risk of overtopping. There are some difficulties in the arbitrary nature of the design risks in Appendix A, but Mr Bass has been daring enough to put his own numbers on such a categorisation of dam sites. That should lead to a lot of discussion - I notice that he feels progress could only be made if the standards were generally acceptable to Panel Engineers.

Paper 4.3, by Messrs Griffiths and Berry, concludes that spillways should be capable of passing maximum floods wherever loss of life is *foreseeable*, and apparently mean not just in a community group. They suggest that for any other dam economic analysis should prevail. They go on to suggest that economic analysis can be simplified, and they give no indication that a minimum spillway capacity need be adopted. Finally they come to the conclusion that reservoir Certificates should include something in the way of a statement of the flood hazard at and below a site, and the flood wave and freeboard design parameters that constitute the current situation for the dam. I found it delightful to note that they propose discounting their costs to eternity - I find that much more hopeful than to infinity!

The remaining five papers deal with techniques. They deal with the actual quantities that we are going to be so involved with in the coming years. Taking the papers out of order I mention next Paper 4.6 by Messrs. Johnson and Novak. In the first part there is a flood forecast formula which includes a range of accuracy on the prediction, and I would be interested to know if, for instance, the North of Scotland Hydro-Electric Board is going to find this a helpful approach to predicting operation in real time for some of its complex cascade reservoir situations. The second part of the paper gives a healthy reminder of the hydraulic changes consequent upon larger design floods being 'forced' through existing dams. This section was generally reassuring, though I note that they are generally looking to design floods being perhaps twice as large as in the past, and yet if we look forward into Session 5 and Paper 5.1 we see that in some cases spillway capacity has been increased tenfold. There could be very difficult hydraulic situations arising at some small dams.

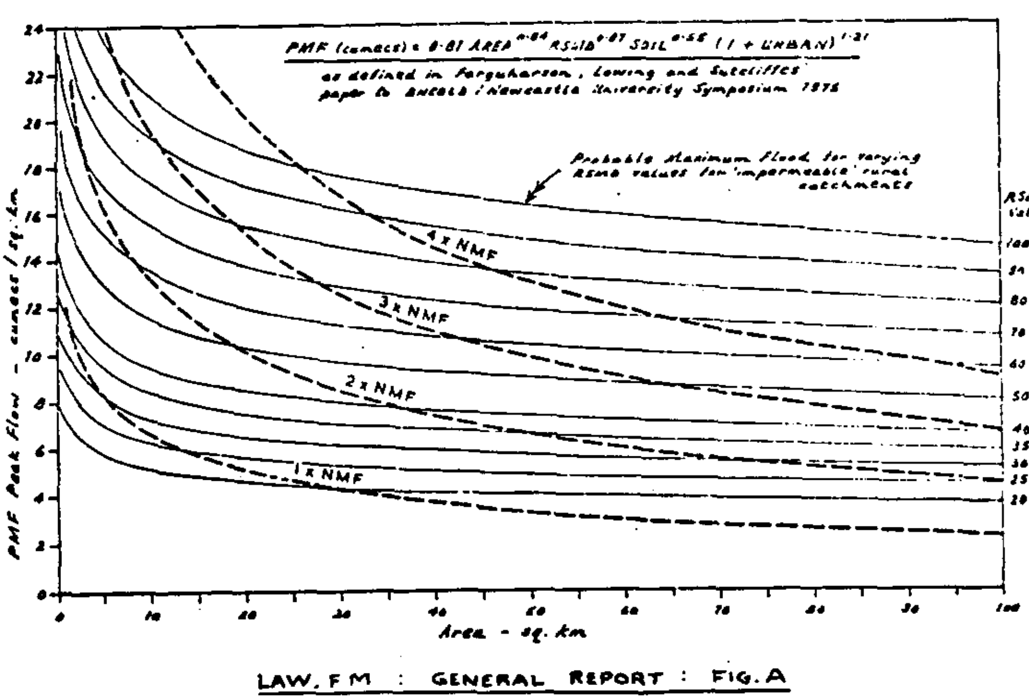
In Paper 4.2 Mr Jarvis reports on his Scottish flood calculations, and says that these were made prior to publication of the Flood Studies Report in 1975. This is useful, because it shows the transitional era in which we have all been caught up over the last four or five years. A major point to emerge in Paper 4.2 is the fact that the North of Scotland Hydro-Electric Board has been willing to adopt as a design flood the 1000 year event. This can be coupled with a low initial reservoir level, and flood release levels are calculated in the paper for some examples. We can see on Fig.3 of the paper how important the effect of low starting levels is on the outflow from various dams. Sometimes the outflows are reduced to 50% or even to 25%, because of low initial levels due to previous generation activity.

Mr Reynolds, also from the Hydro-Electric Board, shows in Paper 4.4 that he really accepts the framework of the Institute of Hydrology floods method, but clearly has doubts on the actual numbers within that framework. He brings forward some examples of very major Scottish floods in which he has found storm Areal Reduction Factors (ARF) which suggest amplification of the Flood Studies Report, but I think more examples will be required before final changes can be made. Possibly his strongest case would be his desire to see an additional and more severe index of soil-drainage in the Highlands. I must presume that this would also apply in parts of Snowdonia and in Ireland.

My connection with Mr West is a little too close for true objectivity when it comes to Paper 4.5, but I would point out that his is possibly the only mention of Reservoir Record Book flood data being used in spillway reviews. Why is this source of data so singularly ignored by all parties? Under Statutory Regulations weekly flood data has been collected over 40 years at many reservoirs, and yet very little has been done with this information. This is clearly a weakness which must be put right in the new Regulations and in our own methods of analysis. Mr West discusses possible improvements to the unit hydrograph and highlights the difficulty of settling on a critical design storm duration at a reservoir, particularly where spillway magnitude is going to be changed and therefore the routing effect is changed, the desirable critical duration changes and reiteration begins in the calculation. At the very end he points to the fact that the time taken on flood studies at spillway sites is bound to increase as the data available to the hydrologist/engineer increases. This seems at first sight to be in contradiction to paper 5.2, where it is suggested that with more data studies will be done more quickly. I think these views can be reconciled, because sometimes if you have more data you can clear up a problem, but at other times you raise new problems. Comments on the standard forms at the back of Paper 4.5. will be very welcome.

Turning to paper 4.7 by Farquharson, Lowing and Sutcliffe of the Flood Studies team, I am very grateful to the authors who kindly sent me an early copy of this paper so that I was able to prepare the grounds for my defence against some of their comments! I must draw attention to the fact that in the paper they suggest that the Probable Maximum Flood (PMF) peak may have a 100 000 to 1 chance against occurring next year at a site - it may even in fact be less; some calculations I have done from the Flood Studies Report suggest it might be 10×10^6 to 1 against at certain lowland sites in any given year. That is a very slim chance indeed, particularly compared with the figure of 35 000 years quoted as a return period in Paper 4.3; 35 000 years is the nominal return period of probable maximum precipitation, and the way in which the figures in the two papers can be reconciled is by seeing that to get PMF you are really compounding awkward conditions on the catchment as well as taking very rare rainfall. It is the compounding of probabilities which leads to such slim chances as 100 000 to 1 against. That might sound very rare at any one site, but it must be remembered that there are thousands of dam sites in the country, thus still giving us a possible problem within the next few years. The difficulty is that a failure involving fatalities can unsettle a whole country and unsettle the whole profession for some time, and I think we need to have some idea of risks so that we may resist over-reaction if some incident does occur. We can still reasure ourselves with the fact that flood induced failures since the passage of the 1930 Act have been very minor.

Paper 4.7 goes on to give to us a new way of quickly finding the peak of a PMF. This was something that the discussion paper attempted to do in Chapter 3 and which got heavily criticised at the ICE Floods Symposium in May, but it has now borne fruit with an additional equation produced in Paper 4.7. It makes it possible to show how PMF designs relate to those designs used in the past (Fig.A).



LAW. F M : GENERAL REPORT : FIG. A

I would add that to assist those who complained after publication of the Discussion Paper that it required about four of the maps from the Flood Studies Report in order to get RSMD - this awkwardly named rainfall parameter - the Met Office has produced a new map giving RSMD directly for any point within the UK. The map will appear in the published proceedings of the May 1975 ICE Symposium on the Flood Studies Report.

When one contrasts the PMF values with once, twice, three times or four times the Normal Maximum Flood by the original definition of the Institution, one finds that there is in the centre of Fig.A a broad measure of agreement, and one can understand why reservoirs designed to twice Normal Maximum Flood have stood so successfully over the past 40 years. What is worrying is that in very large catchments the Normal Maximum Flood was too low, and therefore this posed particular problems for owners like the North of Scotland Hydro-Electric Board. On catchments which are very small I think it is now clear that the Institution Committee of 1933 over-reacted to certain events and had flood intensities which were too high and some small park lakes may even be over-designed. There are some weaknesses in any quick method of getting to a maximum flood peak; the error in this equation may be understandably correct to ±30%. Nevertheless it is a step forward, and I only wish that we had a similar equation which dealt with flood volume or that enabled us to go from flood peak inflow to flood peak outflow.

The main questions involved in this group of papers are going to be echoed in a questionnaire that will be circulated by the Floods Working Party to Panel Engineers.

Just a small introductory question: who is it these days who is really prescribing the flood calculations? Is it really the Panel Engineer himself, or is it his technical staff? Is it, say, the Institute of Hydrology, or even the staff of the owner of the dam?

A more important question: Would participants agree that reservoirs threatening community life should be able to pass the PMF after it has been routed through storage without any fundamental damage that might threaten life downstream because of the flood wave that could be created? There seems to be a consensus towards this point of view, but I would like to have it confirmed so that the Working Party can at least have its upper limit standard ratified.

Would participants also agree that wave surcharges have got to be compounded, as the Discussion Paper suggests, with the level that would be reached by flood water during a PMF - that is the dam freeboard must contain not just the still water level from the flood, but also the wave run-up from associated high wind speed? Freeboard has been quite clearly a neglected subject in the past. I think even the papers here show rather scant attention to waves, and yet these have caused very severe damage in the past. It is a pity that such events have very rarely been discussed and that error may be rectified a little today.

A fourth question: Do participants agree with economic analysis in this situation? Having done that economic analysis, do participants feel that nevertheless there ought to be a *minimum* capacity to spillway size? Alternatively, participants may wish to put forward the view that a set of probability standards, as in Discussion Paper Appendix A and as in Paper 4.1, can be reached by the profession. Finally, I wonder whether anybody has actually used the minimum spillway capacity cormula that the Discussion Paper gives in one of its sections? Besides the idea of the 150 year flood being routed through as a minimum case for spillway design, coupled with minimum freeboard, a formula has been suggested to make sure that if the routing is so successful as to give almost no flood outflow at all, one nevertheless still has a reasonable spillway to allow for blockage by debris.

It may be that some of these alternatives can be reconciled. It would be possible to reconcile differing views by rewriting Appendix A in terms of proportions of the PMF, i.e. instead of taking a Normal Maximum Flood and multiplying up, or taking a 150 year flood and crudely multiplying up before doing a refined calculation, we could start with the PMF and then, in a low hazard case, work as a proportion of that top limit. People would then know the margins that had been eaten away when designing their spillway. There is some precedent for this approach in the United States of America, and also I noticed that in Paper 5.4 Mr Poskitt's work in Northern Ireland goes in this direction.

J G ELDRIDGE (Binnie and Partners) :

I felt that there were many valuable papers in this Symposium and I welcome particularly Paper 4.1, because it did raise the problems which so many Panel Engineers, I among them, must have high in their priorities at the moment.

Under the British system the law places the responsibility for deciding what is safe and what is not safe on the Panel Engineer, and because therefore the administration of the Act is dependent upon individuals, the guidance given by the Institution on this point is tremendously important. I think one must acknowledge the enormous debt that all Panel Engineers and designers owe to the Floods Working Party.

I think we ought also to acknowledge the debt we owe to the original committee that produced the 1933 Interim Report. It was a splendid document for its purpose and one could easily arrive at a spillway size which it was fairly easy to demonstrate was consistent with the accepted views of the profession as to what was safe. If needed it provided a splendid defence against the charge of negligence if unhappily a disaster should occur. Now we have a much better based scientific study produced by the Institute of Hydrology and, paradoxically, I think the question of what would be regarded as safe by the community at large becomes much more difficult to decide. I think this is so because the extent of the uncertainties and the scarcity of data in relation to the problems is very much more obvious than it was before, and also it seems possible that two different people could arrive at different values of the PMF of a particular catchment if they are working independently. I would welcome the views of those who are hydrologists.

Lastly, and perhaps most importantly, I think the decision of what is safe is more difficult because in the last resort it would be much easier to explain in Court the selection of a design flood using a historic enveloping curve as we did before, and multiplying it by two for safety, than trying to explain the new method. I think one must come to the conclusion that the more detailed study recommended by the Institute of Hydrology and by the Institution Working Party must be made in any important case, and then immediately one comes back to the possibility that there could be a difference of professional opinion as to what one should take to be the PMF.

Let us assume, however, that the particular problem of calculation of the PMF can be refined with the guidance of the experts. Let us assume that one can calculate the PMF which will be acceptable to all concerned. Mr Bass suggests that this is too severe a standard and, if I read him right, that it will be sufficient to design against the 1 in 100 000 year flood. Is this too severe? It seemed to me that Fig.3.1 given in the Institutions' Discussion Paper at least left this open to question, and Mr Law now seems to show that the PMF is a good deal more severe than we had previously calculated for a large catchment but the reverse for a small catchment, so one would question whether Mr Bass has a point in suggesting that the PMF is too long a chance to be taken into account. Any calculations on this for specific cases would be tremendously helpful. Mr Bass suggests that the PMF has received undue prominence in the recent Flood Studies Report and has cast a somewhat academic cloud over an already diffcult problem. One can have one's views about the academic cloud. He suggests there should be some sort of a statutory limitation and I am not sure that I would agree with that, because I think that the PMF is more easily conceived than a 1 in 100 000 year flood. One can conceive that the PMF is a figure on the borderline between what is possible and what is not, but I think one has to be a mathematician to conceive what is meant by a chance of 1 in 100 000 that a flood may occur tomorrow.

I think that it is important that we should try and keep the matter as simple as possible, and therefore I feel that it would perhaps be advisable to consider dropping the concept of 150 year flood for the same sort of reason. As a profession we serve the public, and I suggest that we need to be able to explain to the public, if need be, what we have done in terms that can be understood by the layman. We are really dealing with a matter of judgement here, just as we were between 1933 and last year. I think there is much to be said for the approach adopted before, in this case taking the PMF as the basis and dividing by a factor for the design flood. I suggest for the same reasons that any hazard classifications for dams be made as simple as possible. The more classifications we have the more scope there is for argument as to which classification applies. I make the plea for simplicity not so much so that engineers can understand the problem better, but so that engineers can, if necessary, explain it better to the public.

Dr. J V SUTCLIFFE (Institute of Hydrology) :

S Mr Bass has suggested in Paper 4.1 that the Floods Report has relieved the engineer of engineering decisions. I suggest, as has Mr Eldridge, that the opposite is the case. The emphasis throughout the Report and in Paper 4.7 is on the use of local data, records and engineering judgement to correct empirical equations.

Mr Bass also says that the PMF has received undue prominence, and Mr Eldridge has just made the point that I was going to make. I am not going to try and dissipate any academic clouds here but, in our own defence the emphasis on PMF derived from the ICE Report of February 1967, which laid down the requirements of the engineering profession, and these we have tried to follow.

Turning to the criticism of our Report from the North of Scotland Hydro-Electric Board, Mr Reynolds has three main criticisms. He criticises the derivation of the Areal Reduction Factor (ARF) on the grounds that (a) most of the data is from Southern England, and (b) that wrong methods of analysis have been applied. The object of the ARF is to derive rainfall for a return period, T, over a particular area in relation to the point rainfall; in other words the ratio of the 100 year storm over that catchment to the 100 year storm at any single point in it, and his method of getting the ratio of the areal storm to the highest rainfall within the catchment is the wrong way to approach that problem. We had a lot of discussion on this at the Steering Committee, in May at the Symposium, and at the Met Office, and I suggest that although our answer is more conservative it is the correct approach.

Mr Reynolds goes on to talk about snowmelt. I do not wish to talk about this subject — if he wants to add 1% or 3% to the T rainfall I would not quarrel with him, but he then goes on to say 'the maximum snowmelt rates which are deduced the South-East England could not occur in the North of Scotland.' Snowmelt rates have been shown to be proportional not only to temperatures above snowline but also to the accumulated snow, and I think this point must be taken into account.

Mr Reynold's main objection is to the estimates of percentage runoff. We recommend that local information should be used to adjust empirical relations, but I think Mr Reynolds has taken this too literally or taken it to extreme. His evidence is largely based on three storms and floods.

What we are trying to get is the relationship here between Q_T and R_T, and we are talking about single catchment, volume of rainfall or volumes of runoff. We have got a relationship which in fact shows that this relates to antecedent conditions and to soil and so on. By selecting the high floods in an area any relationship has a great deal of scatter in it. If one selects high floods and not high storms there is going to be a very considerable under-prediction in some cases, but our object is to get a rainfall of a given return period from the Met Office work; to turn this into a volume of runoff, and apply unit hydrograph methods to come up with a T year flood. It is not a forecasting tool but a prediction tool.

I really think that the number of storms that Mr Reynolds has got is too few to make a severer correction to the soil indices, as he has suggested. I would like to know something about the separation which he has used to come up with 95% or 96% rapid response runoff as we defined it. I readily admit that the number of stations which we used in our studies, particularly in the North of Scotland, is too few for comfort, but the fault is not entirely ours. We needed information from recording rain gauges on short period rainfall and we needed good runoff measurements, and the two unfortunately do not seem to coincide in Scotland in very many places. We have suggested improvements which might be made to some gauging stations and we are currently revising some of our estimates. We are looking at some of the anomalous catchments again, and when we have some more data which we can use we would be delighted to do some unit hydrograph studies in the North of Scotland. I think that the time is premature at the moment to talk about soil indices of 0.75.

I should point out that we did in fact use a lot of measurements in Wales and North West England, we would in fact have used more if there had been reservoirs instrumented more effectively for instantaneous levels over the spillway. I would appeal for more hydrological instrumentation so that flood records at reservoirs can be used in future.

I have not sufficient time to say anything about our own Paper 4.7. It provides a rapid approximate screening method by relating the Estimated Maximum Flood to catchment characteristics and it gives some examples of use of engineering judgement. One last point is that if people want to include slope our paper does in fact include a five-variable equation with slope in it.

Dr. P S KELWAY (Northumbrian Water Authority) :

S The proliferation of design criteria to accommodate maximum flow conditions emphasises the problem of using a non-rational method of obtaining design data. Amongst others, we now have the Possible and Probable Maximum Floods, the Estimated Maximum Flood, the Normal Maximum Flood and the '35 000 year Flood' which is assumed by some engineers to approach the ultimate.

There are three basic objections to this approach from an engineering standpoint. These are:

1 There is too little evidence to predict with any certainty flows with a return period of more than, say, 500 years. Long-term fluctuations in climate will compound the uncertainties, rendering unrepresentative statistics collected in the short-term. Very large errors may therefore result when using extrapolation to give the long return period values involved in maximisation. It is a sobering thought to consider the result of a near-maximum flood in 1975 Would we have the courage to resist re-assessing the size of a maximum flood event? Would this be a wise move in any case?

2 The risk to which a structure is exposed during its expected life is likely to be no higher for floods of very long return period than for other potential catastrophes such as landslides or earthquakes. The engineer should design for all expected events which have a risk greater than a stipulated amount.

3 The approach is non-rational as the engineer is expected to select a return period for design which the work must withstand, bearing in mind the supposed maximum conditions and the risk to life and property downstream of the structure. We are therefore not far removed from a 'rule-of-thumb' approach.

A suggested rational approach is as follows:

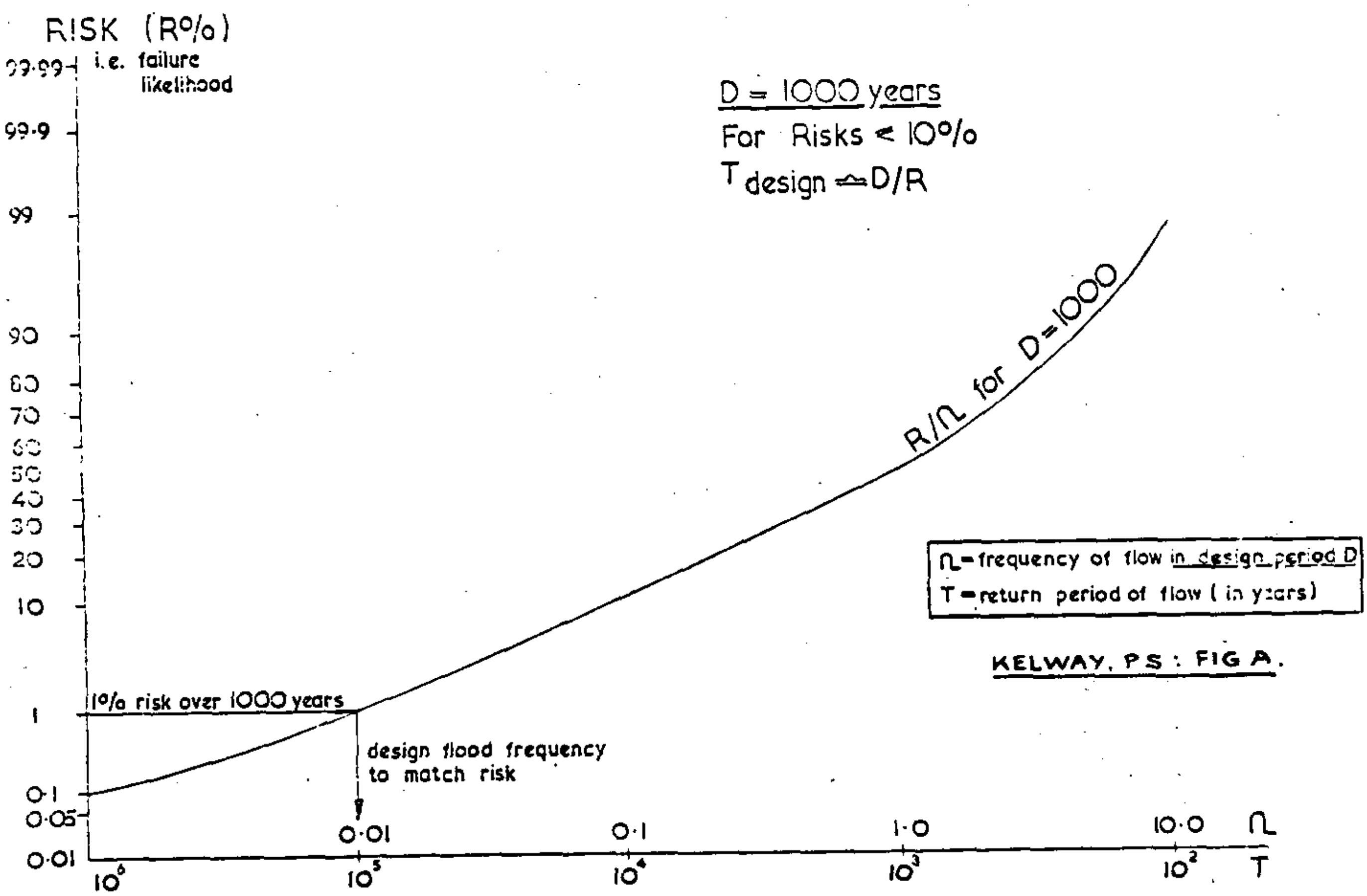
(a) Select a realistic design life for the structure, i.e. the expected length of time that the structure might be in use, thereby producing a risk to the property downstream with a finite time span.

(b) Knowing the design life, plot a curve of risk, i.e. the likelihood of failure, against frequency of occurrence of flood events. An example of such a curve is shown in Fig.A. By applying simple statistical theory a curve relevant to a particular design life can be obtained.

(c) Determine the acceptable risk level to life and property on the grounds of social or other criteria.

(d) Interpolate from the graph to determine the design flood *frequency*, to just match this risk.

(e) Determine the design flood in absolute terms of flow, knowing the frequency, using a discharge/frequency curve obtained from available catchment data or generalised information given in the Flood Studies Report.



KELWAY, P.S. : FIG A.

Advantages of this method are:

(i) The approach eliminates the controversial maximum quantity.

(ii) The risk level can be directly related to safety requirements, e.g. Category A risk 0.01%, B 0.1%, C 1%. Calculation of the *frequency* of the 'design flood' which must be accommodated to match this risk can be carried out independently of any consideration of actual flow data.

(iii) For short-life structures the same risk level can be applied as those of a longer life expectancy but the design flood obtained by this method will be less, yielding greater economy.

Dr. D E WRIGHT (D Balfour and Sons) :

There are a number of closelv inter-related philosophical issues which are raised by the papers we are considering in this Session. These issues are germane to the ICE Discussion Paper, and since Mr Law is our Reporter I am sure that what is said will be considered by the Committee that prepared the first draft of the Discussion Paper.

The points I wish to discuss are as follows :

Is the concept of 'complete' or 'total' protection a valid one?

If the total protection concept is not valid, what do we use as the criteria for selection of the design flood? Must the greatest measure of protection be provided irrespective of cost?

I propose that two criteria should be applied to all dams whose importance justifies some expense on design. The first criterion is based on the total cost of the spillway works plus damage, which is evaluated on discounted cash flow principles; and the second criterion is based on the politico-sociologically *acceptable level of risk*, which has to be ascertained and the cost of meeting it determined, There is then the possibility of synthesis.

Is the engineer responsible for selecting the design flood?

Even if we agree that the economic approach is acceptable in concept, can it work in practice? There are real practical problems in its application at present and some of these will be listed in a separate written contribution.

'Total' Safety — is this Concept valid?

Paragraphs 4.1 and 4.2 of the ICE Discussion Paper introduce the notion of 'total' or 'complete' protection against dam failure by overtopping, and although they do not say it precisely, I get the feeling that Mr Griffiths and Mr Berry (Paper 4.3) incline to this view when they write in respect of their Category 1 dams that the 'greatest measure of protection (against Expected Maximum Flood — EMF) must be provided irrespective of any economic consideration'.

In Paper 4.7, the authors remind us that the EMF implies a combination of several highly improbable events (p 1), and on p 3 'such a flood is synthesised by allowing every facet of the input data and transforming model to combine in the worst possible way while remaining physically conceivable.'

I would simply ask, how can we ever offer *complete* protection against an event which is so constructed? Our natural world knows of no finality in regard to combinations of events, and we fool only ourselves if we think that any quantitative estimate is *the* maximum. It may be emotionally comfortable to think we can provide total protection, but I suggest it is not an intellectually tenable position and is one which can give rise to a dangerous sense of complacency. To make headway in this issue I believe we have to face the fact that there is some risk of failure associated with every design, and our task then becomes one of selecting the criteria on which our judgement of acceptability is to be based.

I want to suggest two approaches: the economic and the political.

Economic Approach to Determination of Probability of Design Flood

I am well aware of, and share, the extreme distaste there is for approaching any question involving risk to human life on an economic basis. I believe, however, there are several good reasons why we have to consider this approach:

1. Before any engineering decision is made, costs should be known. Engineering economics is basically about allocation of resources and I believe that such an appraisal provides the right framework for a basic consideration of all engineering problems, including this one. The engineer's professional duty is to examine alternatives and recommend the most economic, but also to point out the extra costs of others that may be more acceptable politically. He must not pretend that only one solution exists.

2. The risks we are talking about are very low: in the range 1 in 10000 years to 1 in 100000 years (say). We do not live in a monastic community shut off from the rest of society. When risks are already low, we cannot expect national resources to be provided to lower them still further without some sort of evidence that the extra expenditure will result in a greater good to society than if it were committed elsewhere. We shall be expected, rightly, to show cause why 'our' group of people should have the risk to them of flooding reduced from the very low, from 1 in 50000 years, for example, to the negligible 1 in 100 000 years, say, when others in society may be open to much higher risks from quite different causes. We are no longer operating at a level at which one *civil engineering* risk can be compared with another, but where risks at large have to be considered. It is because I value people highly that I cannot demand resources to make 'my' people safe from negligibly small risk when others may continue to be subject to much greater risk of injury and death.

3. An economic approach, when developed, should remove a large part of the subjective element and make possible a rational approach to each situation. Thus the inevitable arbitrariness of a blanket standard applied to all dams, whether few or many lives are at stake and where damage may vary considerably, is removed. By giving appropriate values to the various economic parameters any desired degree of conservatism can be obtained, with the reasoning in clear view. An engineering economic appraisal should show the relative importance of the many factors involved and enable us to improve our judgements. I have argued the basic validity of an economic approach to the resolution of this problem. I do so not knowing the magnitude of the sums that would be required to bring all dam spillways up to a PMF standard. If this sum is negligible the practical force of my argument disappears, though not its validity.

   I know that there are many technical difficulties in the way of doing a proper economic evaluation of spillway enhancement schemes but the urge to improve technique must properly stem from a conviction that the method has an inherent validity. Unless we are agreed about that no progress will be made toward the resolution of the *real* difficulties that currently inhibit the adoption of engineering economy techniques of evaluating dam spillways, some of which will be noted in a separate written contribution.

   However, supposing there were no difficulties and that all was plain sailing on the economic point, would it be the sole answer? I think not. The other set of criteria we have to look to are the politico-sociological ones alluded to by several authors, and the basis of Appendix A of the ICE Discussion Paper.

Politico-Sociological Approach

Human society is, happily, not completely rational and its judgements are not made on wholly rational criteria. Mr Griffiths and Mr Berry make a related point when they point out that whereas river flooding is viewed as a natural phenomenon, failure of a man-made structure like a dam will be viewed in a different light. The river was there, and by choosing to live in its flood plain people implicitly accept the risks that go with it; however, they did not choose to live in a valley commanded by a new dam and they expect it to stay safe. It is a moot point whether the position is different for those who choose to live in an area commanded by an *existing* dam.

So this alternative approach would seek to establish the level of risk that a particular community would accept. There are enormous difficulties in it too - perhaps more than in applying engineering economic principles - but worth the effort. This thinking underlies Appendix A to the ICE Discussion Paper and the figures quoted, for example, by Mr Bass.

Synthesis

Assuming both approaches came up with answers, we should be in a position to determine the economic solution, *and* the *extra* cost of the scheme which was politically acceptable. It could then be decided whether, in the light of circumstances prevailing at the time, the person(s) with responsibility thought that the politically more acceptable solution was worth the extra cost. Such a procedure would mean we would safeguard life and use of the resources we have to greater advantage, because we would take a final decision knowing the full background to the various alternatives and take proper account of both the real resource costs and the political acceptability of the decision. I do not believe that progress will be made by hiding behind blanket standards, the application of which can only lead to anomaly.

Whose Responsibility is the Selection of the Design Flood?

Mr Bass suggests in Paper 4.1, perhaps with tongue in cheek, that with the advent of the methods con tained in the Flood Studies Report, life has got too complicated for the engineer used to dealing with the ICE 1933 flood envelope,and the engineer cannot really be expected to retain responsibility for this task. My answer is in the form of a question : If not the civil engineer, who is it to be? Who else should be so well fitted by training, experience and judgement to make the decision, or at least make a clear recommendation out of a number of alternatives?

How often do we hear about 'status' of the civil engineer in the columns of the NCE? Let us be quite clear that 'status' is the result of the acceptance and satisfactory discharge of real responsibility, and that it has to be earned. Asking to be relieved of what is ours to bear is the way to decline.

G REYNOLDS (North of Scotland Hydro-Electric Board) :

S I have recently had cause to examine rain-storm profiles and find that the great majority of them do not behave at all like the median profiles of Vol II of the Flood Studies Report (pp 42 - 47).

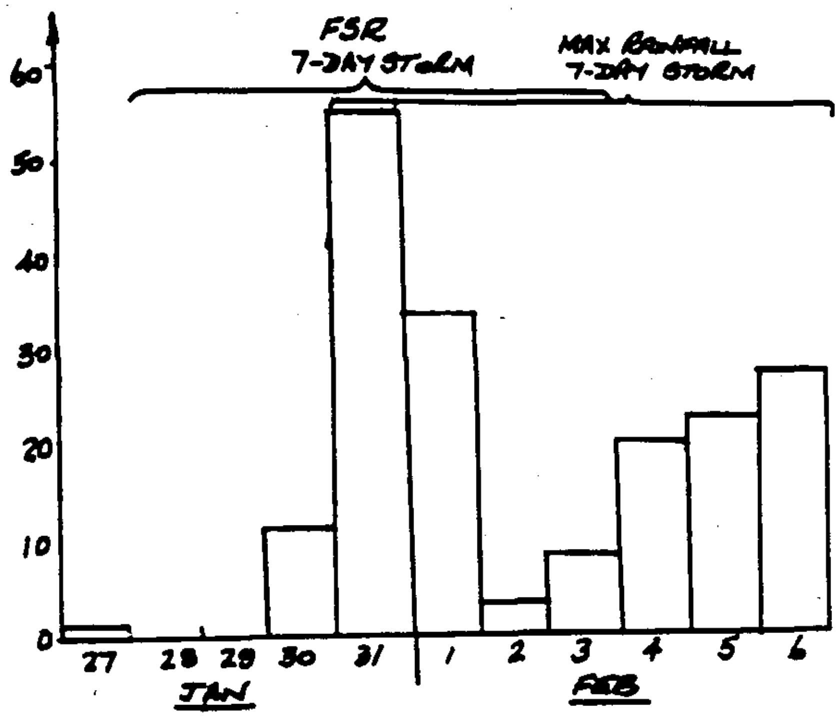


FIG. A

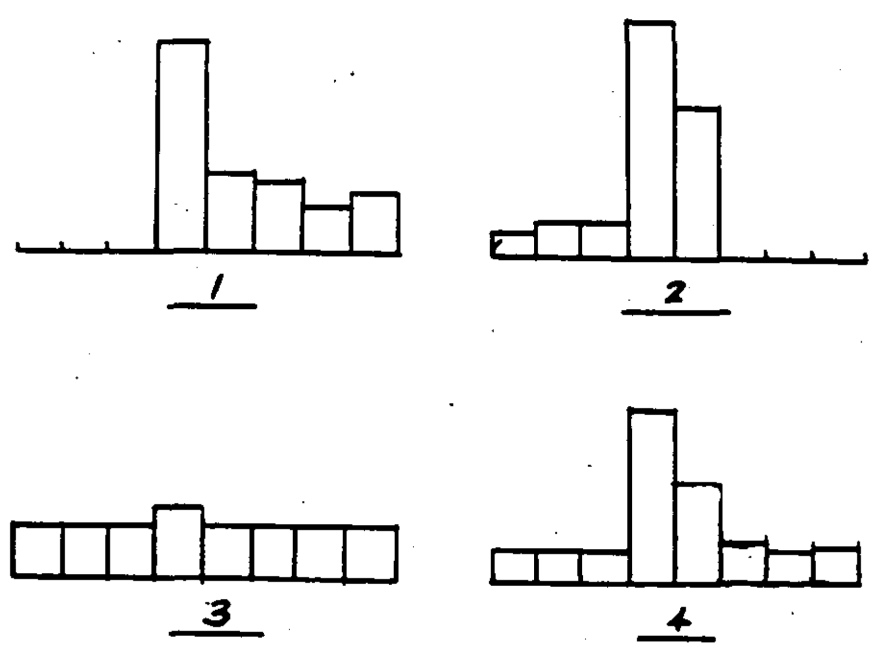


FIG B. IMAGINARY 8-DAY RAINSTORMS (1,2 AND 3) AND THEIR MEAN (4).

REYNOLDS, G : FIGS. A & B

Random unbiased examples of how they do behave show that, firstly, for 48-hour storms, in a small sample available to us seven were single storms with peaks often seriously displaced in time from their mid-point, five were double-peaked storms, and one was rather indeterminate. The same features were displayed with 5-day storms and 8-day storms. The Flood Studies Report median winter profile and the 75% winter profile recommended for use are indicated as having no such displacement and a 'normal' distribution.

Judging by the small sample available to us the median profile approaches a rectangular block as the number of storms in the sample increases. Why then did the Flood Studies Team find a very different shape? The clue lies in a sentence on page 42 of their Report : 'In the analysis storms were centred on the most intense part of the storm so that storm profiles could be compared more meaningfully.' This approach is challenged.

If an intense storm of given duration is being studied it is the period containing the maximum rainfall volume of such duration which is important. I would illustrate this from an example of a 7-day storm at Sloy Dam during the period commencing 27/1/62, shown in Fig.A. The Flood Studies Report would centre the storm on 31/1 and commence with two days without rain, while my concept of the storm would centre on 3/2 and comprise the highest total rainfall for a period of seven days around this date. Fig.B shows what would happen with the Flood Studies Report approach with three admittedly extreme cases of 8-day rainstorms (1, 2 and 3). I submit that the first two are really 24 and 48 hour storms respectively, with a little additional rainfall. Only the last is a 'real' 8-day storm. The result of averaging these profiles is profile 4, which really represents none of them at all well and is considerably more 'peaky' than 3, the only genuine 8-day storm in the set. Storm profiles can be shown to be an important parameter in many instances of flood routing, and I would suggest that more research should be done on this aspect so as to provide users with better guidance than exists at present.

The cooperation between Dr Sutcliffe and his staff and the Hydro-Electric Board has been of great benefit, certainly to the Hydro-Electric Board and, I hope, to the Institute of Hydrology. First of all on snowmelt, the purpose of putting this paragraph in my paper was really to describe what we were doing at the Hydro-Electric Board. If other people think other circumstances are more appropriate to them that is appreciated, we are not trying to convert others. I would say that I feel as a general principle that the volume of snowmelt or the rate of snowmelt is proportional to the temperature, with an upper limit set at what is actually lying on the ground. That is slightly different in detail from what Dr Sutcliffe said; he said it was proportional to the temperature and volume lying there.

On the question of small samples I have used in my Paper, particularly with areal reduction factors and with volume of snowmelt and of quick response runoff, I take Dr Sutcliffe's point entirely. I have used a very small sample, and we are very loathe to put it forward, but it is the only data available for the area in which we are interested. If you have only a small set of data I suggest that you have to accept it unless you can show that it is unrepresentative.

Dr Sutcliffe also brought up the point of base flow separation. Again I agree that our methods and the Flood Studies Report method are slightly different, but on one or two checks we have carried out we find that the integration over the storm duration of base flow or non-separated flow is more or less the same whichever method you use. We think our method is preferable.

The last point that he dealt with was the question of improvement of data from the North of Scotland. I can assure Dr Sutcliffe we are doing what we can. We are only too conscious of the need for a greater amount of data,but there is the question of the economic cost-benefit of obtaining such data.

F N GRIFFITHS (Howard Humphreys and Sons) :

The purpose of the Reservoirs Act is essentially to protect the public, and this is the first duty of the Inspecting Engineer. It is a very wide brief, to protect the public, and I think it needs a little more direction. This is why it is necessary to consider dams in categories. The ICE Discussion Paper puts up three categories, and our Paper 4.3 suggested two.

The first case is where loss of life can be foreseen, and going back to what I said about protecting the public, they are entitled to feel that they are as safe as can reasonably be expected. This generally means designing for the maximum flood that is considered possible — define it how you will. I think this is the present legal position, and that an Inspecting Engineer must insist on it. The method of estimating this maximum flood will probably vary from time to time. One could, for example, say that the maximum flood was the old envelope flood of the 1933 ICE Committee. Methods develop and will be refined, so that I am not trying to define maximum flood but just to say that design should be for some maximum flood.

I think we can deal with all other dams under a second category - they are not designed for the maximum flood. Even in this case the Inspecting Engineer's responsibility is still to protect the public.

and here he is essentially concerned with making sure that the public does not suffer unreasonable or excessive damage. I believe this is where one must consider economics.

We have suggested that the right approach is the least-cost solution, considering the community rather than the owner of the dam. It may need some direction in the Regulations to sift this out as a guideline for engineers who are designing a dam which does not comply with the stricter requirement of category 1. I think some guidance is needed in order to avoid considerable variation in the methods of approach and conclusions that different engineers would arrive at.

On the question of loss of the dam itself, in the new legislation this is something more for the Construction Engineer rather than the Inspecting Engineer, and if one adds the cost of replacing the dam our experience suggests that in many cases where design for maximum flood is not necessary because of risk to life it may nevertheless be justified on economic grounds. There are many cases where loss would be absolutely minimal, and I would suggest that in these cases there is no need to have a lower limit. If a farmer wants to build a dam of 25 or 30 tcm in some remote location with no possibility of damage or loss of life whatsoever, why should he not design it on a 10-year basis if he wishes to and is prepared to replace it on that basis?

Where a dam is designed for something less than the PMF flood, i.e. loss of life is not expected, it seems reasonable that at some future date the conditions downstream may change. This would be a matter for the Inspecting Engineer to consider, and if a new community has grown up or development has taken place there may well be a case for the Inspecting Engineer to insist on a higher standard for the spillway. This is a point that the Construction Engineer would probably have to take into account in putting forward his initial proposals for a new dam. I am skipping lightly over the question of old dams, because this is very much more difficult.

In conclusion, I wonder whether when the cost of designing a spillway to take full flood capacity is looked at in relation to taking a 1000 year or 100000 year flood we may find that that cost differential is not so great after all, and that compared with the magnitude of possible losses or as a percentage increase to the cost of the scheme it may be quite small.

D W BERRY (Howard Humphreys and Sons) :

(S) In Paper 4.3 Mr Griffiths and I refer to the use of simple economic analysis to determine the optimum size of a spillway. I would like to enlarge on this and in particular to draw attention to the fact that simple approximate calculations are adequate in practically all cases. The methods proposed by the American Society of Civil Engineers, and apparently endorsed in the ICE Discussion Paper on Reservoir Flood Standards earlier this year, appear in most cases to be unjustified. Indeed they could be misleading, as they give the impression of deriving a solution of extreme accuracy for a problem which normally has to be solved using relatively sparse data.

In our paper we suggested a very simple and rapid method for carrying out this type of calculation and recommended that detailed economic analysis should only be carried out if a simple preliminary check indicated that it was necessary in order to arrive at a sound decision. In particular we suggested that it was quite adequate to consider only a number of orders of cost for total damages, i.e. £10 x 10^6, £100 x 10^6 and £1000 x 10^6.

To illustrate this point I will refer to a dam recently completed under our supervision and which supplies irrigation water to a sugar estate in West Africa — I appreciate that this conference is concerned mainly with British dams, but a large number of those present are actually engaged on works overseas so it is not entirely irrelevant. Table A gives total costs for the spillway element of the dam estimated as described in the paper, i.e. capital cost plus Present Value (PV) of annual risk.

Other papers in this conference and elsewhere have drawn attention to the difficulty in assigning a return period to the Estimated Maximum Flood (EMF), and the costs in the table have therefore been calculated for two return periods, firstly 35000 years, which seems generally accepted, and the somewhat arbitrary period of 10^9 years, which is approximately the square of the first period.

It will be seen that even this enormous difference in the assumed return period alters the optimum design flood by only something like 25%. The capital cost of the spillway is reduced by a much smaller percentage of total capital costs. Looking now at the actual sums of money involved, the total cost, i.e. spillway capital cost plus PV of the annual risk is of the same order, i.e. between £1 x 10^6 and £3 x 10^6 for optimum design flood for each of the six cases. The additional capital costs for providing a spillway to pass 100% EMF instead of 50% EMF — the range indicated by the six cases considered — is approximately £0.6 x 10^6. The total capital investment for the whole project is of the order of £100 x 10^6, and the loss of revenue in the event of failure would be of the order of £30 x 10^6 per annum, say a total of £50 x 10^6 over the time taken to partially rebuild the dam if it failed due to overtopping.

Flood % EMF	Total Cost (£1,000)		
	EMF Return Period 35,000 years		
	D = £10^7	D = £10^8	D = £10^9
25	20,700	201,000	2,001,000
50	2,730	17,800	168,000
75	1,495	2,620	14,600
100	1,685	1,776	2,680

Flood % EMF	Total Cost (£1,000)		
	EMF Return Period 10^9 years		
	D = £10^7	D = £10^8	D = £10^9
25	3,900	32,000	313,000
50	1,100	1,160	2,060
75	1,370	1,370	1,370
100	1,680	1,680	1,680

TABLE A

In this particular case the spillway accounted for a high proportion of the total cost of the dam. It was a gated type with the further limitation of a relatively small operational head — about 2m for an EMF outflow of 2400 m^3/sec. Thus the £0.6 x 10^6 differential cost which I have quoted is probably much higher than might normally be expected. In spite of this it is only a very small percentage of the total capital investment for the project.

Although this example relates to a somewhat specialised situation overseas it is likely that similar total damages would arise in this country taking into account loss of revenue from water, consequential losses for industrial processes held up by lack of water, damage to property etc.

As we have already stated in the paper, approximate methods are adequate since, although the total estimated damages appear to be one of the more significant factors governing the selection of the design flood on an economic basis, there is nevertheless a very wide range of near-optimum solutions as indicated in the above example.

If 35000 years is accepted as the more realistic return period for EMF it is found that the spillway should be designed for about 75% EMF if damages are estimated as approximately £10 x 10^6. For anything much in excess of this figure it is economically justified to design the spillway for EMF. It may also be useful to refer to the classification of dams proposed by Snyder in 1964. Full details are given in his paper published in the Proceedings of the American Society of Civil Engineers, Hydraulics Division, May 1964, but his proposals can be summarised briefly as follows :

Major Dams :	Considerable loss of life and/or excessive damage. Design for EMF
Intermediate Dams :	Possible but small risk of life and damage within the financial capacity of the owner. Design for 40% to 60% of EMF
Minor Dams :	No possible loss of life, and damages not exceeding the cost of the dam. Design for a flood of 50 to 100 years recurrence period.

The first rule would be applicable to dams where there is a legal requirement for safety, and the second and third provide useful guidelines where design for full EMF is not mandatory.

These rules would adequately have covered the example which I have just quoted — design for EMF being justified in this case on the grounds of excessive damages resulting from a total loss of revenue. However, if the estimated maximum damages had been fixed at around £10 million — say only a partial loss of revenue which the estate owners could accept - both Snyder's rules and the simplified calculations suggested in our papers would have given comparable answers.

It is our experience that the rules would be adequate for preliminary screening. There will only be very rare cases, if any, where very complex economic analysis would radically change the decision reached on the basis of much simpler calculations.

In conclusion may I suggest that the only mandatory rule in any Code of Practice on spillway design flood which may be published should relate only to adopting EMF where loss of life may occur. Other lesser floods relating to various degrees of partial safety - safety as defined in our paper - should be excluded. Such things have a tendency to become mandatory over the years whereas they should be agreed by the dam owner and his engineer, and confirmed at the public enquiry stage.

B H ROFE (Rofe, Kennard and Lapworth) :

Mr Eldridge has already stated the problem that we now have an alternative and varied approach to selection of a flood, whereas before we had a base defined by the ICE 1933 Report, I feel, however, that he in common with one or two subsequent speakers has not really accepted the point put forward by Mr Bass - does he advocate the adoption of Probable Maximum Flood (PMF) for the worst category?

If we are to relate the capacity to a proportion of PMF, then I would suggest that it would be far more difficult to justify to a layman that you had only allowed half or a third of what might happen! The engineer's decision and responsibility remains, but must be within the framework of the categories as they are finally determined. What I believe is necessary is that the framework for such a decision should be restored, and if it is less than PMF for category A then this should be clearly understood by the Government and the Courts.

Taking up Mr Law's point about the significant effect of low reservoir levels, I would like to raise the point of what should be done if existing dams have inadequate spillway capacity and there is no money to repair them. Until the 1975 Act becomes Law there is no enforcement, but a prudent owner will nevertheless ensure that if a reasonable level of safety can be obtained by a reduction in Normal Water Level then this should be done even at the expense of yield. The new Flood Studies method enables a reasonably quick and logical calculation to be carried out taking into account the local factors, and the authors of the Report are to be particularly congratulated for providing a tool which can be used by the normal design engineer.

B W KITCHING (Allen, Gordon and Company) :

I should like to make it clear that I welcome the valuable data and advice assembled in the Flood Studies Report. I wish, however, to question Areal Reduction Factors (ARF) which I am convinced will give grossly high results for extreme events on large areas. They are the result of an analysis of autographic rain gauges sited in Southern England only but are presented as applicable to the whole country. The conclusion that the ARF is the same for an extreme rainfall event as for a steady drizzle and for any topography seems extraordanary. If we take a typical large area, say of 1000 km^2, we are given a figure in the Report of the maximum point precipitation on that area in a period of time.

Let us say we are given 200 mm in 12 hours. From the Report one gets an ARF of about 0.9 to convert this point precipitation to average precipitation over the area.

I submit that it is much more likely the real reduction factor will be something like 0.4. The 200 mm may have a probability of occurrence of about once in 35 000 years. The odds against an ARF of 0.9 applying during such an extreme rainfall event might be another 35 000 to 1, so perhaps the combined probability could be something like one in 35000^2! Perhaps it is physically impossible.

I think it might be of interest to quote some figures for the Lynmouth flood in the UK. The flood on the West Lyn at Lynmouth on 15 August 1952 has been estimated as 221 m^3/sec. The maximum possible flood for the same place using the methods of the Flood Studies Report is 280 m^3/sec, or, if one assumed frozen ground, 330 m^3/sec - unrealistic in this instance I know. In this case, with a small catchment, the method gives a sensible answer, but for very large areas I think the answers get increasingly unreasonable. I would suggest that up to about 100 km^2 we may get reasonable answers, but possibly when we get areas of 3000 km^2 the answers may be high by an order of two or three.

Mr West made the interesting suggestion that when entering Fig.5.1 of the Report we should use the area of a figure into which the catchment area just fits. He suggests an ellipse, but why not a circle? It is very much easier to draw and we would avoid debating the shape of the ellipse. The idea seems useful and it would reduce ARF's, particularly for long thin catchments.

I would like to see more research on actual historic floods - for example that on the Findhorn in 1829. There is enough data to model that valley and get a reasonable figure for the flood - I think it ranks with the Lyn flood, but was on a much bigger catchment. I believe we should find out more about events like that so that we can produce sensible answers for large catchments.

M J FEATHERSTONE (Welsh National Water Development Authority) :

My first point relates to deriving floods from catchment characteristics and, when using relationships derived from catchment characteristics, for producing flood design information. I think it is important to note that the relationships are more likely to give reliable results if used for areas with similar geological characteristics. Where there are obvious discontinuities in the geology it may be advantageous to make use of channel routing methods as covered in the volumes of the Flood Studies Report. To give two illustrations, if one has two catchments of different geology it may be better to derive values individually and route them down to where the dam is. Another example is where one has a catchment with a fault and one has the normal stream pattern, but then the stream flows along the fault-line before it gets to the dam, and here I think it would be better to use catchment characteristics to that point and then carry out the channel routing. One method of checking for discontinuities is obviously to draw a profile.

My second point relates to the categories identified for flood protection standards. If a Panel 1 Engineer decides that Category C applies and that there is no foreseeable threat to life should the responsible undertaker refer the information to the planning authorities to restrict future development? I would point out here that a farmer can use a field for camping for 28 days without planning permission, and should camp sites in these remote areas therefore be restricted? From my experience on the Wye I know that campers seem to go for the edge of the river. There could be a conflict here in that one section of the local authority may have information that other sections should also know about.

I would like to make a point on the capacity limit of 25 tcm set in the Reservoirs Act. In the Wye area we had to advise farmers on the location of trout and spray-irrigation ponds. We would point out that if it was built over 22.5 tcm capacity - under the old Act - a Panel Engineer was required. To this the farmer would reply that he had better build three at 22 tcm. The 1963 Water Resources Act requires any new impoundment to be licensed, but after a lot of internal debate the then River Authorities decided that they could not restrict an applicant or make an applicant install a spillway of particular size. You can, in some areas, thus have one dam, say of 45 tcm capacity and covered by a Panel 1 inspection; three small dams totalling, say, 50 tcm which have no specific spillway requirements and, say a further 20 tcm dam built with Ministry of Agriculture aid which has yet another set of criteria applied to it.

My fourth point is that I have been present at several public enquiries for the promotion of reservoirs. I hope that in future a council will not use the act against a promoter. In both public enquiries at which I was present the council had been employed to make sure the dam was safe for people downstream.

Dr Sutcliffe referred earlier to hydrological measurements at dam sites, and I would say that the 1963 Water Resources Act gave local authorities a statutory duty to install and operate hydrometric schemes. The 1973 Act removed this responsibility, and money for hydrological measurement has to be obtained in competition with all the other demands on the new Water Authorities. If the ICE think such data important it might be worth the Institution making a formal approach to the National Water Council, so that there is some pressure from another direction for hydrological measurements of a particular standard.

Mr Law made a point about who is going to carry out hydrological investigations, and I can see a case for the Water Authority being closely associated with the investigations if not carrying them out. In the case where one may have different Panel Engineers operating in adjacent valleys, in particular, I think the Water Authority could coordinate the results produced - presumably they are going to be the same, but they may not be.

Prof. P NOVAK (University of Newcastle upon Tyne):

I would like to stress that the basis of Paper 4.6 by Mr Johnson and myself is really given in the paragraph where it is stated that it is understandable and correct that flood estimation for reservoir design be given major priority. In the context of this Symposium, however, it is considered appropriate to draw attention to some other hydrological and hydraulic considerations resulting from spillway operation and from flood reassessment. In other words the basis of our paper was not design but the operation of existing reservoirs; problems arising from operation of existing reservoirs, and problems which might arise from the reassessment of design flood for existing reservoirs. On the operational side we dealt with the operation of flood control and the errors in forecasting flood outflows.

I would also like to correct a printing error in the preprint copies of Paper 4.6 - 1. Equation 2 on page 2 should read

$$\delta_r = \alpha\delta_{r-1} + \eta_r$$

We would be delighted if Mr Law's comment on applying this procedure under operational conditions were to be tested, and we would be delighted if, for example, the North of Scotland Hydro-Electric Board applied the suggested procedure for forecasting flood outflows on reservoirs or commented on its application. Equally we would be happy to assist in the further development of the procedure, and I hope that Mr Johnson will soon publish the complete treatment of the subject as promised in the paper. We did not treat the subject of wave run-ups as this seemed to us adequately covered by the ICE Discussion Paper and was further stressed at the ICE Flood Studies Symposium in London last May. The fact that this was not mentioned in Paper 4.6 does not, however, mean that we do not regard wave run-up as being of utmost importance.

In Paper 4.6 we tried, in the second section, to draw attention to some hydraulic considerations resulting from spillway design and operation in connection with flood reassessment.

Finally, to the questions posed by the General Reporter I would like to add one more. Is it right to design all parts of the dam outlet works with the same safety factor?

To this I for one would reply with an emphatic negative, as the example of spillway and stilling basin design quoted in Paper 4.6 shows. In that example you are bound to get erosion in the stilling basin, but we are mainly concerned with localising the erosion at a point which is not harmful to the actual structure.

M MANSELL-MOULLIN (Binnie and Partners):

I hesitate over my first point as I am not a member of Panel 1 and it is not my responsibility to carry the onus which membership implies. I have, nonetheless, been associated with the review and design of quite a large number of spillways in various parts of the world and find myself with a considerable onus that hydrological studies for those spillways are satisfactory. Let me say a few things about the philosophy of standards to which spillways should be designed.

In order to prepare a set of guidelines one has to generalise, and this is what the Discussion Paper is trying to do. Our knowledge of floods and causes of floods is extremely scanty, however, and we cannot prepare reliable estimates of floods - certainly not for long return periods - and this needs to be recognised in any proposals for Codes of Practice. For instance, in Chapter 4 of the ICE Discussion Paper one of the criteria used for small lakes and low dams is the 150 year flood. Before specifying that as the level to be considered, how accurately can we predict the 150 year flood for the 2000 or more reservoirs which have to be inspected in this country? The answer is that we cannot predict very reliably. Without being able to put a certain figure on it I would say - having discussed it with Dr Sutcliffe - that the order of uncertainty is perhaps ± 30%. Here, therefore, is a recommended standard of a flood of relatively short return period - 150 years - put forward and which we cannot in fact estimate. This becomes all the more important when one looks at Mr Bass's recommendations in Paper 4.1. His Category A reservoirs should be capable of passing at least a 100000 year flood. None of us can pretend to know what that is for any catchment in this country or elsewhere. We cannot estimate this, and to base categories on things which we are unable to estimate seems to me to be pretending that we know rather more about flood estimation and about hydrology than in fact we do.

In practice these problems, if one tries to ignore the generalisations, become much simpler than might appear, because a lot of the problems cancel one another out.

When we come to a specific reservoir - Mr West's Paper 4.5 is helpful in this respect - we have many more of the factors to be considered already defined for us. There is a community downstream and one knows how important it is; one has some idea as to whether the area is flood-prone or not; one knows the lay-out of the rivers which have got to combine in order to cause a flood. One then carries out the study and finds the inflow flood which is then routed through the reservoir. This reservoir routing is of tremendous benefit to the flood hydrologist because it smoothes out all sorts of uncertainties which he does not know, and so although there are many uncertainties in estimating routed floods, when one comes to the out-flow flood and one tries the sensitivity of the various uncertainties one finds that things are not quite so bad as they might have appeared to be when talking about them in a general fashion.

On another theme, there has been some talk about considering the way in which the reservoir is operated when assessing the safety of the reservoir and the flood for which it should be designed. I would suggest this needs to be considered very carefully. There was a paper recently in an American publication which showed that of a large number of dams which the author had studied not one was being operated in the way which had originally been intended, and most of them were serving different purposes. This was brought out in discussion yesterday when talk of changing the methods of operating the various reservoirs of the Severn-Trent Water Authority was mentioned. It is therefore important, particularly when carrying out inspections, to be quite clear as to how the reservoir is being operated and in what way this may differ from conditions under the previous inspection and also when the reservoir was initially designed. For instance, it was mentioned that there might be a need in some cases to raise summer levels for amenity purposes. This might well change the initial levels on to which design floods were imposed.

Questions which should be mentioned because a lot of engineers here today work on projects abroad are: 'How useful is the Flood Studies Report to schemes overseas? How much of it is applicable to the problems overseas?' I would say that quantitatively almost none of it is. Its value lies in having examined a large number of techniques, and these techniques are very clearly and very fully described. For people working abroad it therefore serves as a marvellous guideline or textbook, but it does not give quantitative figures which will be of much help in estimating floods overseas. It may of course be rather different for some cases in Europe.

A final point on instrumentation. In the Flood Studies work I think they were able to use two or three sets of reservoir records for estimating the inflows in large floods. It seems to me incredible that if one is inspecting 2000 reservoirs every 10 years that on none of them, apart from these two, is the data adequate to see what has happened in the previous 10 years or in the life of the reservoir and make use of this information for one's inspection. I would suggest - as some previous speakers have - a need, perhaps an obligatory need, for hydrological instrumentation of reservoirs to be such that the inflows in large floods - regardless of the initial condition of the reservoir - can be estimated reliably. I am sure this would make a great deal of difference and remove a lot of the uncertainties of trying to transpose floods from other catchments to reservoir catchments which for practical purposes are not gauged.

D M HAMILTON (Crouch and Hogg) :

The Reporter referred to the effect of wind and wave run-up. This is of interest to me, and I should like to refer to a film which was shown by the South Staffordshire Water Company to participants on a Study Tour after the 1964 Edinburgh ICOLD. That film was most graphic in showing the effect not necessarily of gradient, although that is a factor, not necessarily of surcharge, although that also is a factor, but of breaking waves. As so many of our dams in this country are earthfill embankments I feel that this aspect should be given very serious consideration. It raises doubts, perhaps, as to whether the use of wavewalls to limit freeboard is altogether wise, because that has been my experience, reinforced by an event in January 1968 when Auchendores Dam suffered almost identical damage by reason of heavy spray from waves. In the South Staffordshire case at Blithfield Dam there was a huge hole in about 30 minutes. At Auchendores the hole developed in the middle of the night. The damage was of a similar order to Blithfield and necessitated urgent action (a) to draw the water down, and (b) to increase the toe weight of the embankment. Drawing down the water was, incidentally, somewhat impeded by the fact that the main draw-off was closed with a timber bung.

There is an interesting sidelight to the Auchendores experience in that the wavewall had been caulked at the joint between lengths but that caulking had dried out, or was non-existent in certain cases. It was surprising to see how much damage was done by wind driving through these tiny gaps of about 10 mm - 15 mm, where they cut like a knife through the embankment turf for travels of something like 5 m. This once again raises the question as to whether or not somewhat less carefully manicured embankments are not possibly safer against heavy spray?

C L CLARKE (Sir William Halcrow and Partners) :

As a member of the Floods Working Party I wish to comment on something the General Reporter said about Appendix A of the Discussion Paper on Reservoir Flood Standards.

Many speakers have already referred to the design of new dams but, for every new dam likely to be built during our lifetime, there are 10 or more existing dams. Thus, I feel, the problem of existing dams is an even greater one than that of choosing the design for a new spillway. Consequently, in Appendix A in the Discussion Paper we tried to categorize existing dams according to the likelihood of causing damage, particularly loss of life, in an attempt to get a uniform risk for all the many different types and sizes of dams we have in the UK. The resulting return periods given in Appendix A were referred to by the Reporter as being arbitrary; while they may be somewhat arbitrary, they are not entirely so. What we tried to do was to arrive at an acceptable risk that society today would accept.

There seems little doubt that society would not accept a dam failure involving a loss of life happening every year. It is doubtful whether they would accept one happening every decade. On the other hand they would not accept all dams being made 100% safe, as the resources required for our 2000 to 3000 dams would be so vast. Thus an acceptable interval might be somewhere in between - that is, between about every 10 years and infinity. The return periods given in Appendix A are based on an incident happening about once a lifetime - every 50 to 100 years - which I believe would be generally acceptable in today's environment.

I have a feeling that in other spheres we may be spending too much on safety - for instance, in the medical world, some police services and so on. The public seems to attach differing standards to safety to differing aspects of our life. As an example we all apparently accept a risk of 1 in 100 of being killed on the road. On the other hand it was recently reported that the police had already spent £2 x 10^6 on looking for a particular killer. We all accept this. These cases illustrate the widely differing standards that we and the public accept with regard to safety.

As far as dams are concerned we would probably all be prepared to accept an incident involving loss of life about every 100 years, and certainly every 500 years. It was such thinking that formed the basis of Appendix A, and similar risks result from the proposals in Paper 4.1.

Dr. P S KELWAY (Northumbrian Water Authority) :

I find myself in agreement with Mr Reynolds that the Meterological Office approach to the Areal Reduction Factor (ARF) seems fundamentally invalid. Such a factor is vital as it allows areal rainfall amounts, and hence river flows, to be assessed from point rainfall values.

At the outset, the definition of ARF as given in the Flood Studies Report is imprecise and can lead to ambiguity. The ARF is more exactly defined as the factor which will give the *true areal rainfall* when applied to an *areal estimate of rainfall* derived from point rainfall values for the same return period.

The principle of using point rainfall values of return period T to derive an areal estimate of return T is invalid. The Meteorological Office ARF approach attempts to rectify this. However, no return period can be given to an areal estimate of rainfall. An analogous hydrological situation would be to attempt to define a return period for flow on a certain date in a complete river system knowing the various return periods for flow at a number of gauging stations on the same day. Any derived figure would be quite meaningless.

The application of the ARF method suffers from other defects in detail:

1 From the results available, e.g. the worked example by Jackson (available from the Meteorological Office), the method is unstable, producing considerable variation in ARF for the same duration and area. The variations only disappear on averaging values to determine points on the smoothed Flood Studies Report curves.

2 The use of annual maxima events gives rise to an arbitrary selection of storm events, biasing statistics towards less noteworthy cases.

3 Data originates from limited parts of the British Isles.

4 There is evidence that there is some dependence of ARF on return period even in the limited information given in Jackson's example. With very long return periods the ARF could become very seriously affected.

It is therefore suggested that the ARF as calculated and applied in the Flood Studies Report is unsatisfactory. It is unclear whether the use of ARF as proposed by the Meteorological Office will cause large or small errors in design and there is at present no practical way of finding out. An alternative means of deriving and applying a rationally based reduction factor for point rainfall is required, such as :

(a) Determine the peak point rainfall value of the required return period over the area of interest, using the Flood Studies Report method.

(b) Reduce the *peak* value by a reduction factor to give an areal estimate of rainfall.

It is certain that curves of reduction factors against area and duration, and probably storm type, will have to be determined for various locations. An extension of work done some years ago in the Meteorological Office by Holland would be applicable, where point to area reduction factors were studied in some detail. A fair amount of work in this direction has now been done at the University of Birmingham, being applied to storm types as well as including regional considerations.

This approach would overcome the difficulties envisaged in the Reservoir Flood Standards draft, where the determination of ARF for maximum storms is seen as a problem meriting special study. More important, it allows the engineer to recognise the fact that point rainfalls of a given return period may produce many different areal events which are quite different, as regards their rarity, from the point of view of flood prediction.

Added to the problems of ARF, the derivation of information for long return periods of 1000 years and over seems unjustified, bearing in mind the limited amount of data available and alien influences which can cause statistical instability in the region of low probability. These two factors combine to cause large uncertainties in the derivation of Probable Maximum Flood estimates from rainfall data.

Dr. J V SUTCLIFFE (Institute of Hydrology) :

I am sorry that the Meteorological Office authors are not here to answer Dr Kelway's sweeping and not very constructive criticism.

I do not find the definition of the Areal Reduction Factor (ARF) imprecise and I believe criticism has stemmed largely from confusion between areal reduction in actual storms and the ratio of areal rainfall to average point rainfall of a given return period. The latter is a statistical concept, not a physical factor, and an average value of it is rightly required in the design case. I see no advantage in Dr Kelway's suggested alternative approach, and I cannot agree that the return period of areal rainfall is meaningless.

Of the detailed comments I would accept that data originates from limited parts of the British Isles for reasons discussed earlier, and that further analysis of improved data coverage would be useful. The study of the behaviour of the ARF as the probable maximum case is approached would be worthwhile.

I would agree that estimation of rare floods involves large uncertainties, but it is more helpful to the design engineer to suggest means of estimation either by statistical analysis of flows or by using rainfall estimates than to describe the procedure as unjustified.

Mr Reynolds raised the question of storm profiles. In the design case one needs a means of distributing the rainfall estimate in time which preserves the return period of the storm through to the flood rather than a physical representation. Simulation studies, compared with flood records, showed that the flood estimate was not very sensitive to profile choice but that the recommended profile, centred about the storm maximum, together with the other parts of the procedure, could be used in this way. I agree that further research on the profiles of long duration storms, of particular interest in Highland catchments, would be useful.

J G ELDRIDGE (Binnie and Partners) :

I feel that Probable Maximum Flood (PMF) is a proper criterion when loss of life is expected because if I was asked if a 1 in 35000 year flood can occur tomorrow I would have to say that it could - there is a finite chance. If I was asked whether a PMF or a greater flood than PMF could occur tomorrow I would say that the best evidence is that it cannot. This is why I think that the PMF is the better criterion, and when referring to the use of the 150 year flood I suggested that consideration to be given to changing the expression of floods of reduced standards to a factor of the PMF for reasons of simplicity - I think it is important to be as simple as possible in the drafting of a document.

Picking up a suggestion that Mr Clarke made, he said that people may very well accept floods of 1 in 1000 years/frequency. He may be right, and in one instance that I know of this has been done, not an analogous case, but an interesting one. The new flood defences along the Thames are being designed against a return period of 1 in 1000 years, occurring in the year 2030. That is significant because the level of the land in that area is falling about 300 mm every 100 years, and the consequence of this is that the people who live there know that the chance of a flood coming over the top is probably going to be a good deal less than 1 in 1000 during their lifetime and thereafter it will be 1 in 1000 or thereabouts.

In referring to the criterion adopted for Thames Tidal Defences, I am not suggesting either that a probability basis is suitable when selecting the design flood for reservoir design, or that a 1 in 1000 year flood should be selected if it were. In the case of sea defences the works mitigate an existing hazard, whereas a reservoir creates a new hazard, and totally different considerations apply when the degree of safety to be provided is under discussion.

R G SHARP (Severn-Trent Water Authority) :

Following Mr Clarke' valuable contribution I would like to suggest that possibly different considerations could apply for the extremity of conditions provided for, as between new works to be started with knowledge of the Flood Studies Report on the one hand and,on the other, conditions that may be acceptable with existing dams. What I have in mind is that a Construction Engineer may well feel justified to design from the outset for the maximum flood as it can now be estimated, and could probably satisfy many people that the additional cost of providing for that, as opposed to some lesser severity, would be a fairly minimal extra in the context of the total cost of a major scheme. On the other hand I think I for one would take some convincing that, with an existing scheme which may be established to have a one in 10000 or one in 100000 return probability in its existing spillway provisions, that this would be unacceptable. It would necessitate great expense and operational difficulties if the scheme were to be taken out of service in order to meet a theoretical probable maximum. In conclusion I would say that to many laymen the Probable Maximum Flood appears to be a contradiction in terms. The word 'probable' implies that some time or other it will occur, and I think we are really saying that it is something that is considerably further away than one in 100000 years. I think I would like to borrow Mr Berry's nomenclature in another context, and suggest that a much better description would be the 'Eternity Flood' or EF.

F F POSKITT (Ferguson and McIlveen) :

I would make a few points regarding Northern Irish reservoirs in particular. The first thing is that we do not have any 1 : 25000 maps from which to count stream frequencies - I believe this may also apply to some areas of Scotland. We use 1 : 10560 maps. Could our Reporter confirm that this is acceptable?

Another point, with reference to the Discussion Paper on Reservoir Flood Standards. On Fig.3.3 the 150 year flood peak gives a figure of 2.05 for Ireland as a whole. Obviously there is a difference between the eastern and western seaboards, and it so happens that the majority of the dams in Northern Ireland are on the Eastern seaboard, much nearer to the Mull of Kintyre than they are to the South of Ireland. One could say the same about Donegal, so would it be reasonable perhaps to have a little regionalisation? I will not say between North and South Ireland, but possibly between East and West Ireland!

Mr Law referred to the work that I have done on reservoir examinations or assessments. They were not in fact inspections, they were a preliminary form of work. Very briefly, half of roughly 50 reservoirs I have examined, with catchments under 20 km^2 and with storages of 8000 tcm and under, have spillway deficiencies as one factor. A very big element in that spillway deficiency is the loss of freeboard that has occurred with older dams. If we corrected that and in some cases provided a wavewall we would be back to much smaller figures.

Having eliminated that factor we come back to this question of loss of life, and again roughly a half of reservoirs where spillways are a factor have got communities below them and I very much hope, Mr Chairman, that the Floods Working Party will define in more detail what loss of life is, because in the North of Ireland it would be an academic exercise to carry out an economic survey covering loss of agriclutural land on river flood plains which we know very well from previous enquiries to be negligible. It would be a trivial exercise compared to asking the first North of Ireland farmer to get up and shift up the hill a little to get out of the prospective flood plain that we have calculated might arise below some of our older dams. We have enough troubles there already Mr Chairman!

F G JOHNSON (North of Scotland Hydro-Electric Board) :

I have listened with much interest to this discussion,and I really wonder whether we should be looking at another industry to give a lead on how we should treat events, probability and risks.

Before I joined the Board I was with the nuclear industry. I believe that the nuclear industry has gone a considerable way to quantifying risks to the public. In designing nuclear power stations, the Nuclear installations Inspectorate typically require an event with a probability of 1 in 10^6 years to be designed against for a major accident involving loss of life. This 10^6 year event can be made up in many ways, and in designing a nuclear power station it is necessary to review all the combinations of accident probability and relate them to the combination of equipment required to protect the public against the accident. In my eyes, this is equivalent to analysing flood events in relation to spillway capacity and risk of dam failure. The weather conditions, which for a nuclear station dictate whether the activity will be deposited or whether it will be dispersed by wind conditions, are analogous to the effects of the flood on populations downstream. I believe that the standards set down for acceptable risks for nuclear stations should be similar to those which might be adopted for a dam, and typically not more than a few fatalities for an event of say 10^{-6} /year. Safety analyses for dams and flood studies should be undertaken on a probability basis and analysed in the same way as for a nuclear station.

SPILLWAYS – DESIGN PHILOSOPHY

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SYNOPSIS

Until the publication of the Flood Studies Report in 1975, the design flood for a dam spillway was based on the I C E Flood Committee Reports of 1933 and 1960. The methods proposed in the Flood Studies Report provide estimates of the maximum flood to be expected from the catchment, and also of floods with return periods less than that of the maximum flood. Thus there is now a means of assessing the risks involved in designing spillways with capacities below the estimated maximum.

The paper discusses the classification of dams according to the spillway capacity provided and suggests simplified methods for carrying out economic analyses to determine the optimum spillway capacity.

INTRODUCTION

Since the invitation to prepare a paper was issued, flood estimation has been changed radically by the issue of the Flood Studies Report (1) and the I.C.E. Discussion Paper (2) on Reservoir Flood Standards. In addition, the Reservoirs Bill (3) has been passed, although it has not yet come into force.

In the British Isles, for over 40 years the I.C.E. Flood Committee reports of 1933 and 1960 (4) have provided virtually the only guides to the size of flood to be used in the design of spillways. With the publication of the Flood Studies Report, there can be little doubt that the recommendations of the I C E Committee will be superseded. The Flood Studies Report is based on the principle that for any catchment there is a maximum flood which will not be exceeded and that its magnitude can be estimated. The Report proposes methods for determining the estimated maximum flood (EMF) and lesser floods Q(T) with a return period T.

Previously all that has been available has been an envelope of the highest floods recorded. This must have led, in many cases, to the provision of greater capacity than was in fact necessary.

In Britain there are extensive and reliable records of rainfall but the records of river flow and floods are very much more limited. The Flood Studies Report has developed a procedure for using the rainfall records and physical data associated with the catchment for estimating the maximum flood and the return periods of lesser floods. However, the engineer responsible for designing or inspecting a dam has still to decide whether the capacity of the spillway should be capable of passing the estimated maximum flood or some lesser flood which could be exceeded. In the latter case, particularly in the case of earth-fill dams, the result could be complete failure of the dam and the rapid release of the whole of the impounded water. Such a failure may cause loss of life, damage to property and infra-structure, loss of amenities and health risks.

In many circumstances - for example, in works for the alleviation of flooding - it is recognised that the cost of providing complete protection against loss of life is impractical and in economic analyses it has become customary to value such loss. In these cases, however, flooding is a natural phenomenon and may not be viewed by the public in the same light as the failure of a man-made structure such as a dam. In Britain, the 1930 Act (5) required that the Design or Inspecting Engineer should certify, amongst other things, that "the reservoir is sound and satisfactory and may *safely* be used for the storage of water,' and also ' whether any measures are necessary in the interests of *safety*.' The requirements of the 1975 Reservoirs Act, which replaces the 1930 Act, are similar. The problem that arises is to decide what exactly is meant by safety.

The occurrence of a maximum flood is clearly an event that can be foreseen and consequently it would seem that a dam could not be certified as being safe if failure of the embankment and loss of life could occur. It is, however, possible that these legal requirements may in time be altered, or that a definition of safe may be in some way provided, but for the moment it would seem that it would be unwise for the Engineer to accept any other less stringent interpretation of the Act. Inevitably this means that the works in some cases may be more expensive than they would be if they were designed on economic principles, even after allowing for the value of lives that might be lost, but in the present legal climate and current public opinion about dam disasters, the Authors consider that spillways must be designed to pass the estimated maximum flood without overtopping where loss of life is foreseeable in the event of failure of the dam.

It is suggested that for design purposes dams in Britain should be placed in one of two categories, as follows:

CATEGORY I DAMS

This category covers those dams where, in the event of failure, loss of life is foreseeable and must include not only those cases where people live in the area that would be inundated as the result of the rapid release of the water stored in the reservoir, but also those cases where the area is used regularly; for instance, for schools or offices or where it includes busy public roads. It should also include those cases where the sudden release of impounded water, whilst not likely to cause direct loss of life, might nevertheless trigger off events that could do so - for instance, where the stability of cliffs or hillsides might be endangered. It would, however, be unreasonable to include those cases where loss of life might result from casual or infrequent presence in the area of such people as fishermen or farm workers.

For dams in this category the greatest measure of protection must be provided irrespective of any economic consideration and the spillway should be designed to pass the estimated maximum flood. However, in cases where comparatively few people live in the area, it may be cheaper to rehouse them to provide for the discharge of the maximum flood.

CATEGORY 2 DAMS

This category incorporates the dams which are not included in Category 1 where it is proposed to determine the design flood - as defined - solely on economic grounds. Both cost/benefit and least cost analyses are frequently used in the appraisal of proposals relating to civil engineering works but in the case of dams the latter is to be preferred to the former, as discussed later. Economic considerations may indicate that the design flood should be the maximum for the catchment, rather than some lesser flood, but nevertheless the dam should fall within Category 2, rather than Category 1 since loss of life is not foreseen.

There should be little difficulty in deciding which is the most appropriate. It is probable that the majority of dams in Britain will be covered by Category 1. However, it may be that with time the public will become more prepared to accept a greater risk in the interests of economy and the number of dams designed to the lower (Category 2) standard would consequently increase. Overseas, economic conditions are frequently such that the luxury of design to the higher standard cannot be accepted as the need for the scheme is greater than the need for absolute safety. But even where the lower (Category 2) standard is applied, there will undoubtedly be many cases in which the provision of capacity for the passage of the maximum flood will be economically justified.

DESIGN FLOODS

Before proceeding further it is necessary to consider the question of freeboard in relation to both Category 1 and Category 2 dams. In the I C E Floods Report (4) the assumption is made that at least 0.6 m (2ft) of dry freeboard would be available above the level of the 'normal' flood. (In this paper freeboard, wet freeboard, dry freeboard and full supply level have the meanings assigned to them in 'Dam Terminology - A glossary of words and phrases related to dams', ICOLD, Paris, 1970). At the time most spillways were of the simple overflow type and the provision of this freeboard, together with other recommendations relating to spillway head, meant that in practice a 'catastrophic' flood could be discharged without the dam being overtopped. Since then heads over these simple spillways have been increased and the use of siphon, gated and bellmouth spillways has become more frequent so that this assumption no longer holds. Consequently there is a need for a new conception of both normal and catastrophic floods and the Authors suggest the use of the term 'design flood' to indicate the greatest flood that can be passed without the dam being overtopped - that is the estimated maximum flood in the case of Category I dams and some lesser flood in the case of Category 2 dams.

These proposals imply that there is virtually no dry freeboard under design flood conditions. This is justified by the fact that in the case of both Category 1 and Category 2 dams, the occurrence of such floods would be a rare event and that the peak would be comparatively short lived.

The occurrence of high winds or other exceptional conditions such as seismic activity are also rare events and it seems unnecessary to compound them. However, some measures - such as a wave wall or protection of the downstream slope - are probably justified to prevent any damage by waves occurring so rapidly during a flood that the level of the crest could be reduced to such an extent that the dam could fail. Alternatively some increase in crest level may be preferable.

However, where the level of the crest, in accordance with this criterion, is insufficient to provide adequate dry freeboard - as determined in accordance with Appendix D of the I C.E Report (2) - for floods that are likely to occur with a recurrence period of (say) 1:150, it is suggested that the crest should be increased appropriately.

Adoption of these criteria would mean that the well tried principles of the earlier report were maintained; that is (i) that floods likely to occur perhaps two or three times in the life time of the dam could be passed without any reduction in the dry freeboard and (ii) that the design flood could be passed without overtopping.

It is accepted that some damage could occur under 'catastrophic' conditions to both the spillway and crest and this seems entirely reasonable.

COST BENEFIT AND LEAST COST ANALYSES

There are many types of scheme - for example, irrigation and roads - where before making an investment it is normal to confirm that the present value of the benefits likely to accrue will exceed the costs. If this technique is applied directly in the case of the spillway for a new dam, the benefit almost always exceeds the cost even if the spillway is designed for the estimated maximum flood (EMF). Typically, the position is as illustrated in Figure 1 (The figures are for illustration and are based on a hypothetical case using costs derived from a number of typical schemes - details are given in Appendix A) which shows the benefit/cost ratio that is obtained if the capacity of the proposed spillway is increased from that shown to the capacity needed to pass EMF. It can be seen that:

1 The benefit/cost ratio for providing a spillway for EMF conditions as opposed to none at all is infinitely great.

2 On the other hand, the benefit/cost ratio of increasing the capacity from 25% EMF to 100% EMF would be 3.5. This would also be the position in the case of an old dam where the spillway had a capacity of 25% EMF.

3 However, the benefit/cost ratio of increasing the capacity from 50% EMF to 100% EMF would be only 0.3%.

4 Clearly, there is always a point at which the proposed or existing capacity is such that the benefit/cost ratio of increasing it to 100% EMF would be unity, and it could be argued that this capacity was the limit of economic design. However, it is almost certain that an increase from this point to a capacity of something less than 100% EMF - say 50% EMF or 75% EMF for instance, would result in a benefit/cost ratio in excess of unity. In the example used in the preparation of Figure 1 if the capacity of the spillway were to be increased from 25% EMF to 50% EMF, the benefit/cost ratio would be 10 and from 25% EMF to 75% EMF it would be 5.

Thus, the use of cost benefit analyses does not appear appropriate to the determination of the optimum size of spillway, since it is possible to obtain almost any result depending on initial assumptions. Consequently, the Authors recommend that this approach should not be used but that instead analysis should be done on a least cost basis - that is, analyses to determine the lowest discounted value of the cost of the spillway plus the annual risk cost associated with the total damages expected in the event of a failure of the dam. The results that may be expected from such a least cost analysis are illustrated by the curves in Figure 2 (a) in the case of a new dam, and in Figure 2 (b) in the case of an old dam.

If the discounted costs (present values) are plotted against capacity the resulting curve has the form of one of the two curves shown in Figure 2 (a); the upper would justify the adoption of a spillway capable of passing a maximum flood, and the lower would justify the adoption of a spillway with a capacity as indicated by point B.

Figure 2 (b) is, for practical purposes, a variation of Figure 2 (a) where there is an existing spillway having a capacity of 'X' and where the present value of the existing annual risk costs is 'Y' - point A on the figure. The curve of costs for any increase in capacity will generate from this point and is likely to be of the form illustrated by either the upper or lower curve. If point A is below the minimum of this curve, then clearly no enlargement is economically justified but if it is above - for instance, at A or A - then an increase in discharge capacity would be justified.

In carrying out least cost analyses of the type recommended, the annual risk cost should be considered as a sort of insurance premium, as recommended by Kuiper (6), equivalent to the total risk multiplied by the probability of its occurrence in any year. In Britain, it seems probable that the total risk will not vary greatly with the size of the flood, but will depend mainly on whether the dam is overtopped and breached so that the impounded water is released in a comparatively short time. Thus, the total damage is likely to be more or less the same irrespective of the magnitude of the flood causing it, and hence the annual risk factor will be dependent solely on the return period - providing that conditions in the area below the dam remain unchanged.

A dam may be expected to have an extremely long life and to determine the present value (PV) of the annual risk costs, the simplest procedure is to discount to eternity in accordance with the following formula :-

$$PV = \frac{A}{Y}$$ where A is the annual risk factor and Y is the discount rate

i.e. for a 10% discount rate $PV = \frac{A}{0.1} = 10A$

Alternatively, at present day discount rates it would be reasonable to consider a stream of costs extending over 40 or 50 years. The present value of these risk costs should then be added to the cost of the works, whether they are new or additions, to increase the capacity of an existing spillway, to obtain the total discounted cost - as illustrated in Figures 2(a) and 2(b).

THE IMPORTANCE OF THE RETURN PERIOD

Since the total value of the damage that may arise in the event of failure is likely for any particular dam to be constant, the annual risk cost is extremely sensitive to the return period of the conditions likely to cause overtopping and failure.

A return period of 1:35,000 is suggested for use in connection with the EMF. Clearly, and by definition, this must lie close to the upper confidence limit obtained by normal statistical techniques,and for the purpose of economic calculations the Authors suggest that the curve shown in Figure 3 should be used for determining annual risk costs.

DISCOUNT RATES

Economic analyses of the type proposed are also sensitive, but to a lesser extent, to the discount rate selected; in dealing with problems overseas, the choice of the appropriate discount rate poses a problem of some magnitude,but in Britain there can be little doubt that the rate fixed by the Treasury for use in the evaluation of projects in the public sector - the Treasury Discount Rate - should be adopted. At the moment this stands at 10%, having been raised to this level in 1969. Suggestions that lower rates should be used have been put forward several times in the past and the American analysis quoted in the Discussion Paper is based on 6%. However, the Government have stressed the need for water authorities and other industries in the public sector to adopt a commercial approach, and in these circumstances, the rate of return required from capital investment would probably be of the order of 35% in the conditions of inflation that apply at present. The Treasury rate of 10% thus implies that inflation is ignored, and that all computations are carried out at steady prices. It is unlikely that it will change greatly since it is comparatively stable and not affected by the level of inflation. In consequence, a design flood based on it is unlikely to be changed at frequent intervals.

SENSITIVITY TESTS

Much of the raw data used in the economic design of spillways is likely to be moderately unreliable. For this reason, sensitivity tests should be carried out using different values within reasonable confidence limits to determine what extent decisions are likely to be affected by changes in the value of data used in the analyses. This applies particularly to the return period associated with any particular flood, to the total value of the damage (total risk cost) which may occur following complete failure, and to the discount rates used. Before adopting any design flood or capacity, therefore, it is necessary to test what effect changes in the basic assumption would have on the decisions to be made - that is, to test the sensitivity of the proposals to alteration of the data on which they are based.

The Authors suggest that in the first instance it will generally be sufficient to test the effects of assuming damages to be 10, 100 or 1000 million pounds. For reference, Table 1[7] shows the total damages involved in a number of recent dam failures.

Table 1 - Damages Resulting From Dam Failures

Dam	River	Country	Year Failed	Damages £ m
Malpasset	Le Reyan	France	1959	29.0
Bab-i-yar	Dneiper	USSR	1961	1.7
Baldwin Hills	Owens	USA	1963	21.0
Mayfield	Cowhitz	USA	1965	1.0
Wycoming	Sybille Creek	USA	1969	0.6
Pardo	Seco de Frias	Argentina	1969	8.5

After testing sensitivity to variation in total damages, a decision can be taken as to whether or not a more detailed study is needed to give a more precise estimate of damage.

CONCLUSION

In Britain, in view of public opinion, use of the economic approach should be confined to projects where loss of life is not foreseeable, and it is probable that the majority of these will be comparatively small and of moderate importance. Consequently, major studies to determine the damages that would arise in the event of failure would not be justified and estimates will have to be approximate. When making a decision on whether or not to carry out an investigation to determine the extent of the damage, the cost of the study itself must always be borne in mind in relation to any saving that may accrue in the cost of the works to be constructed.

It is possible that, as the result of changes in public opinion and consequent changes in the legislation, economic design may become more widely used in the future in the design of dams. If this occurs, and particularly if this approach is applied to existing dams designed on the basis of 100% EMF, or in accordance with the 1933 Floods Committee report, it may well be found that the levels of the overflow sills can be raised and hence the storage capacities increased at a very low cost.

However, these are considerations for the future. For the moment, it is suggested that spillways should be capable of passing the maximum floods to be expected wherever loss of life is foreseeable. Where loss of life cannot be foreseen, the spillway should be designed in accordance with the economic principles set out in this paper. These principles may also apply in special cases, especially overseas, where it is felt that the possible loss of life does not warrant designing for a maximum flood and where instead an allowance for such loss is included in the analysis.

First indications are that where total present values of costs have a minimum value, as shown by the lower curves in Figures 2 (a) and 2 (b), near-minimum values will apply over a fairly wide range so that the final choice of design flood will not be critical from the economic point of view, and may justifiably be selected to satisfy other criteria.

Lastly, it is suggested that the Construction Engineer's final certificate should contain, in addition to the details normally recorded, the following. (Terminology used in relation to the Engineer is that employed in the Reservoirs Act 1975).

1) Category in which the dam has been placed.
2) The flood for which the spillway has been designed and the amount of dry freeboard under these conditions.
3) If the dry freeboard under these conditions is less than that required as the result of considerations of fetch and maximum wind speed - both of which should also be recorded - the flood that can be passed without encroaching on this freeboard should also be recorded.
5) Likewise, if the maximum flood cannot be passed, the size of the flood that can be passed with normal dry freeboard (as defined in 2) should be recorded.

It is felt that it is essential that these details should be available to enable Inspecting Engineers in the future to review the situation.

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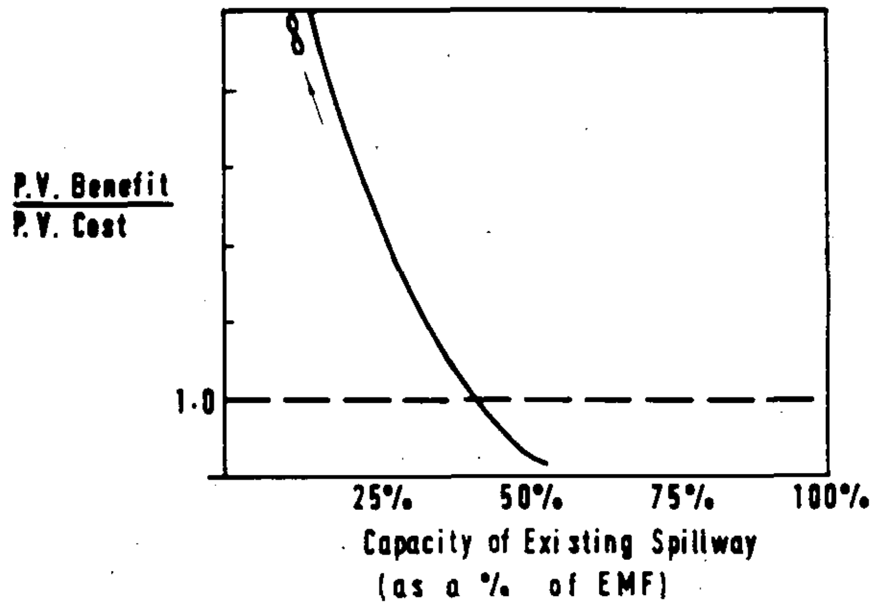

Fig. 1 Typical Benefit : Cost Ratios for increasing Spillway Capacity to EMF

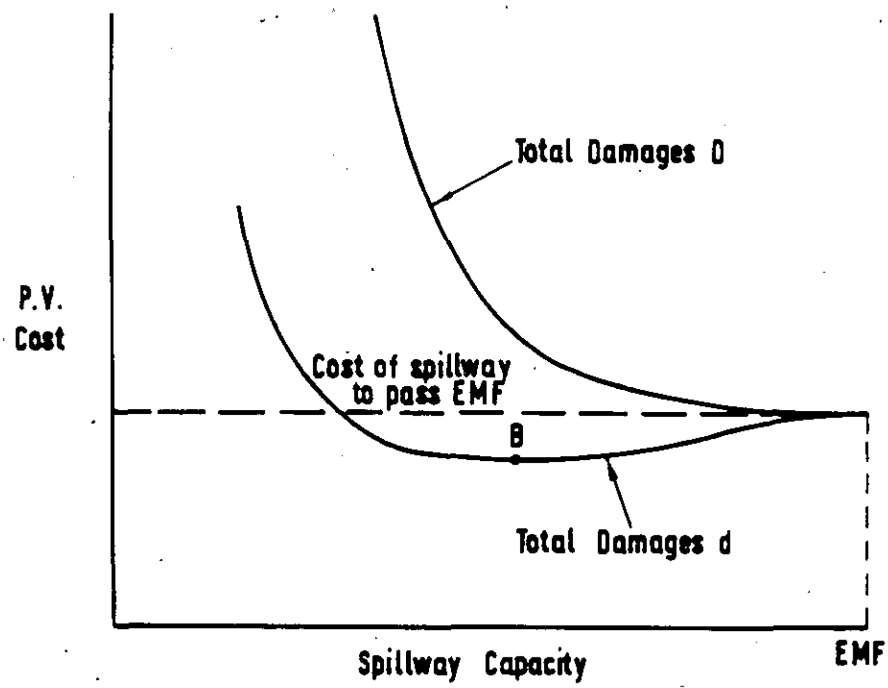

Fig. 2 (a) Typical Curves of PV (Spillway + Annual Risk Cost) for New Dams

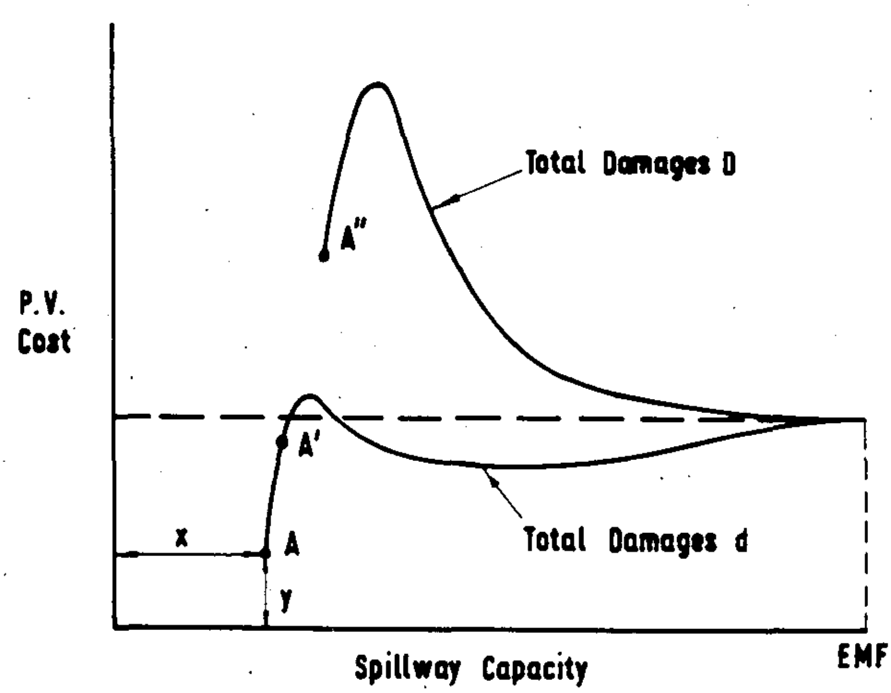

Fig. 2 (b) Typical Curves of PV (Spillway + Annual Risk Cost) for Existing Dams

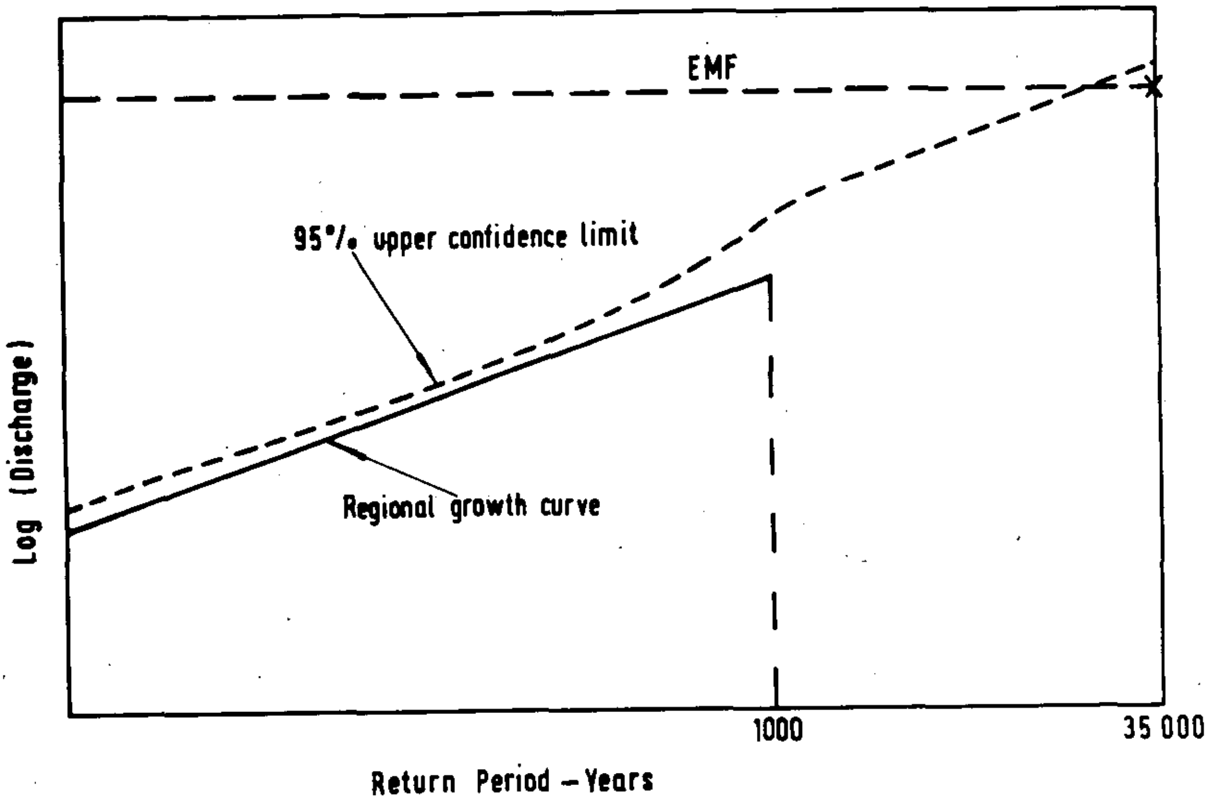

Fig. 3 Typical Confidence Limit Curve

APPENDIX TO PAPER 4.3 FOLLOWS :

APPENDIX TO PAPER ON SPILLWAYS - DESIGN PHILOSOPHY

INTRODUCTION

This Appendix has been prepared in order to illustrate the method of determining the design flood on an economic basis. The examples quoted are all hypothetical but they have, in general, been based on typical data derived from actual projects. In connection with the passage of floods of any given size it is usual to think of the capacity of the spillway only, but costs may arise from other works, and these must be included in an economic study.

For instance, instead of increasing the length of spillway to accommodate a greater flow, it may be possible to increase the depth over the crest by increasing the height of the embankment. The cost of this increase in height would have to be included in the cost of measures to permit the passage of the larger flood. The raising of the embankment would also increase the lag effect and this too would enable a greater flood to be accommodated. In the Appendix that follows the cost in all cases includes these works, together with the costs associated with the spillway itself.

PROCEDURE

Figure A1 shows the relationship between the cost of the spillway works and the size of the flood that can be discharged as a result of them. Obviously, it is possible to plot such a curve in connection with the proposals for any scheme but the result will, of course, be unique to the scheme. Figure A1 is, in fact, based on a siphon spillway where the number of siphon units could be varied quite readily.

In order to calculate the damage in the event of a flood occuring greater than the design flood, it can be assumed that such a flood would overtop the dam and would result in complete and rapid failure. It is then necessary to estimate the value of the damage that would occur. This should include:

1 The replacement value of all buildings, houses and other structures that may be destroyed.

2 Secondary or consequential losses that would arise as the result of the non-availability of water (or power) for industry and other purposes, including the actual loss of revenue.

3 An estimate to allow for intangible losses, such as disturbance, loss of amenity, ill health etc.

The mean return period for floods of various magnitudes less than EMF can be determined from the regional curves produced as part of the Flood Studies Report (1). However, it must be appreciated that they are mean values for a complete region and that variations above or below the mean can be expected. For this reason, the Authors suggest that where possible actual records relating to the catchment area should be examined and modifications made to the regional curves using standard statistical techniques.

The methods described by Gumbel (8) are relatively simple to apply and for a given set of data involve calculation of means and standard deviations only. All other factors can be obtained from published tables. The width of the upper confidence band can be calculated and the Authors propose, as an interim measure, that for economic analyses the regional curve should be shifted upwards by this amount to produce a curve of flood v return period.

Whilst the regional curves are limited to return periods of 1000 years or less, it is stated (Volume 2, page 468 of the 10th report) that for periods of 1000 years or more mean flood return period and mean storm return period can be equated, which implies a return period of about 35,000 years for EMF.

The Authors have examined data published recently (2) for ten rivers in the British Isles and suggest that a plot of log Q/probability will approximate to a straight line with a discontinuity at about 1000 years return period. It is suggested that for preliminary economic studies the relationship between Q and return period can be deduced as follows :

1 On log/probability graph paper plot Q(10) and Q(35000) i.e. EMF and join by a straight line.

2 If data is available, calculate the width of the upper 95% confidence band using Gumbels equations.

3 Read off combinations of "economic design flood" and return period from 1 and increase by the value obtained from 2 if the necessary data is available.

It is considered by the Authors that the above simple method produces results of an acceptable degree of accuracy. The regional curves reproduced by the I O H Study approximate to a straight line up to return periods of about 1000 years, but when extrapolated give a flood flow of less than EMF at 35,000 years return period. Typical examples are shown in Figure A2. Figure A3 shows the comparison between the extrapolated I O H regional curve, the calculated 95% confidence limit and the proposed approximation for a specific case. Whilst further work is necessary to verify the validity in all cases, it is clear from Figure A3 that the use of Curve 1 or Curve 2 is unlikely to give results that differ greatly. Thus even if data is not available for step 2, the effect of carrying out step 1 only (i.e. a straight line joining Q(10) and EMF plotted at 35,000 years return period) may be expected to provide results that are acceptable for economic studies.

A typical calculation is shown in Table A1 for two assumed values of maximum possible damages. It will be seen that if these could amount to £100 million, it would be economically justified to provide a spillway capable of passing EMF. However, if damages are unlikely to exceed £10 million the provision of this spillway capacity could not be economically justified, and for a Category 2 dam it would be adequate to provide a spillway to pass approximately 60% EMF.

Maximum Damages D	Spillway Capacity % EMF	Return Period R Years	Annual Risk A = D/R	PV Risk P = 10A	Cost of Spill-way S	Total PV (P + S)
100	17	100	1.000	10.000	0.350	10.350
	38	450	0.222	2.222	0.465	2.687
	58	1800	0.055	0.055	0.545	1.100
	79	8000	0.012	0.125	0.620	0.745
	100	35000	0.003	0.029	0.700	0.729
10	17	100	0.100	1.000	0.350	1.350
	38	450	0.022	0.222	0.465	0.687
	58	1800	0.006	0.055	0.545	0.600
	79	8000	0.001	0.012	0.620	0.632
	100	35000	0.0003	0.003	0.700	0.703

Table A.1 Typical Economic Evaluation of Spillway Capacity.

Notes: (a) Costs in columns 1, 4, 5, 6 and 7 are in millions of pounds.

(b) A discount rate of 10% has been assumed. For other rates, the factor 10 in column 5 will be different.

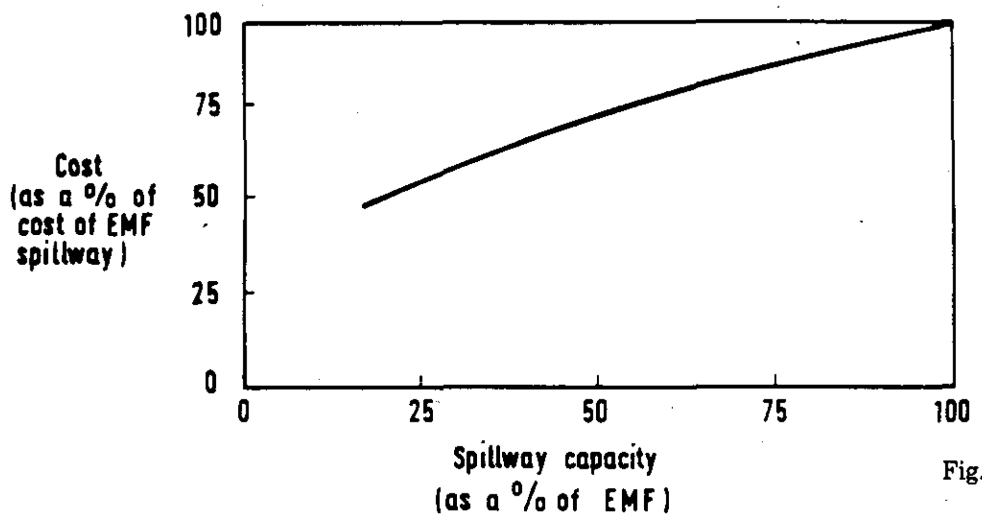

Fig. A.1 Relationship between Spillway Cost and Capacity

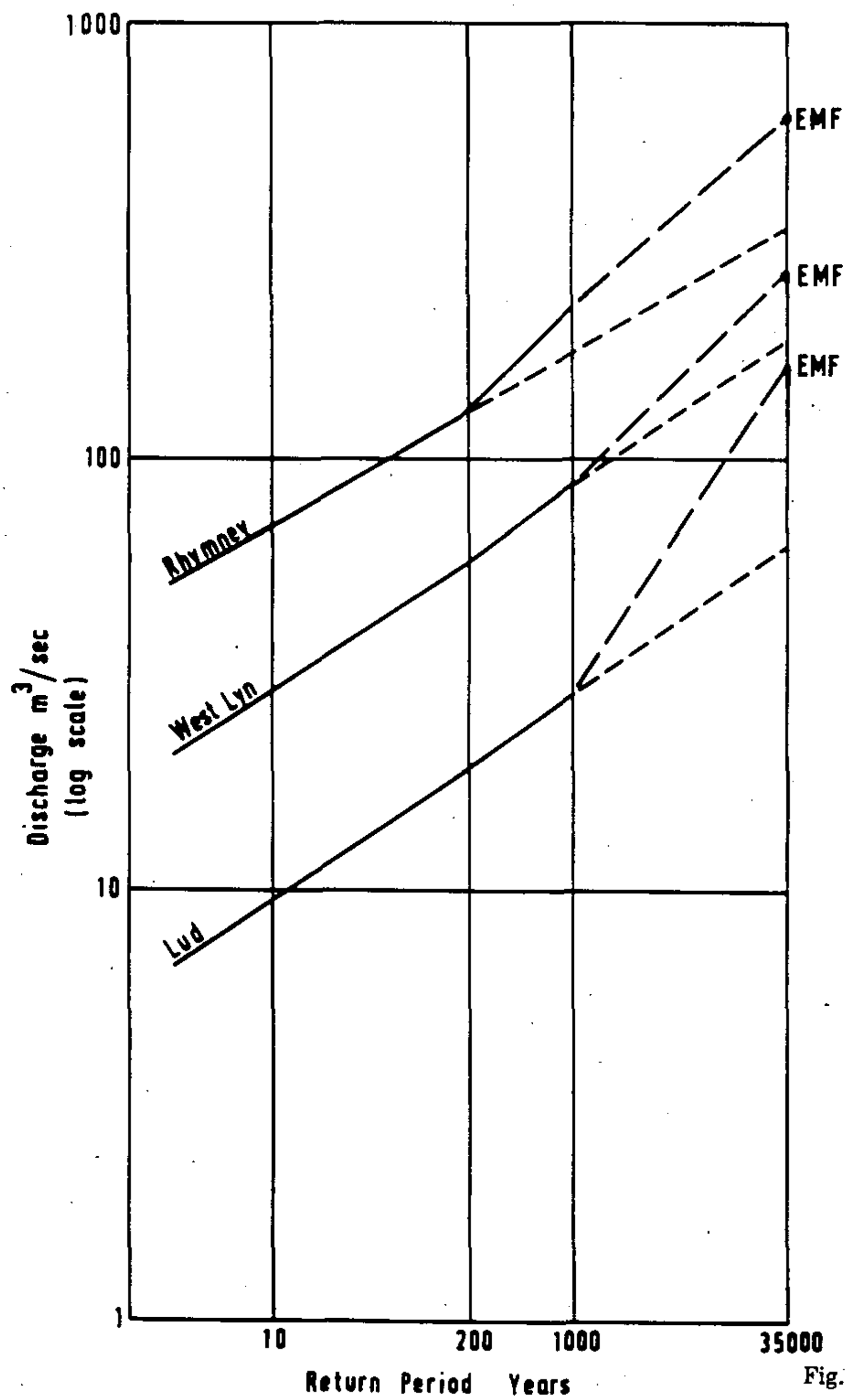

Fig. A.2 Flow : Frequency Curves for three British Rivers

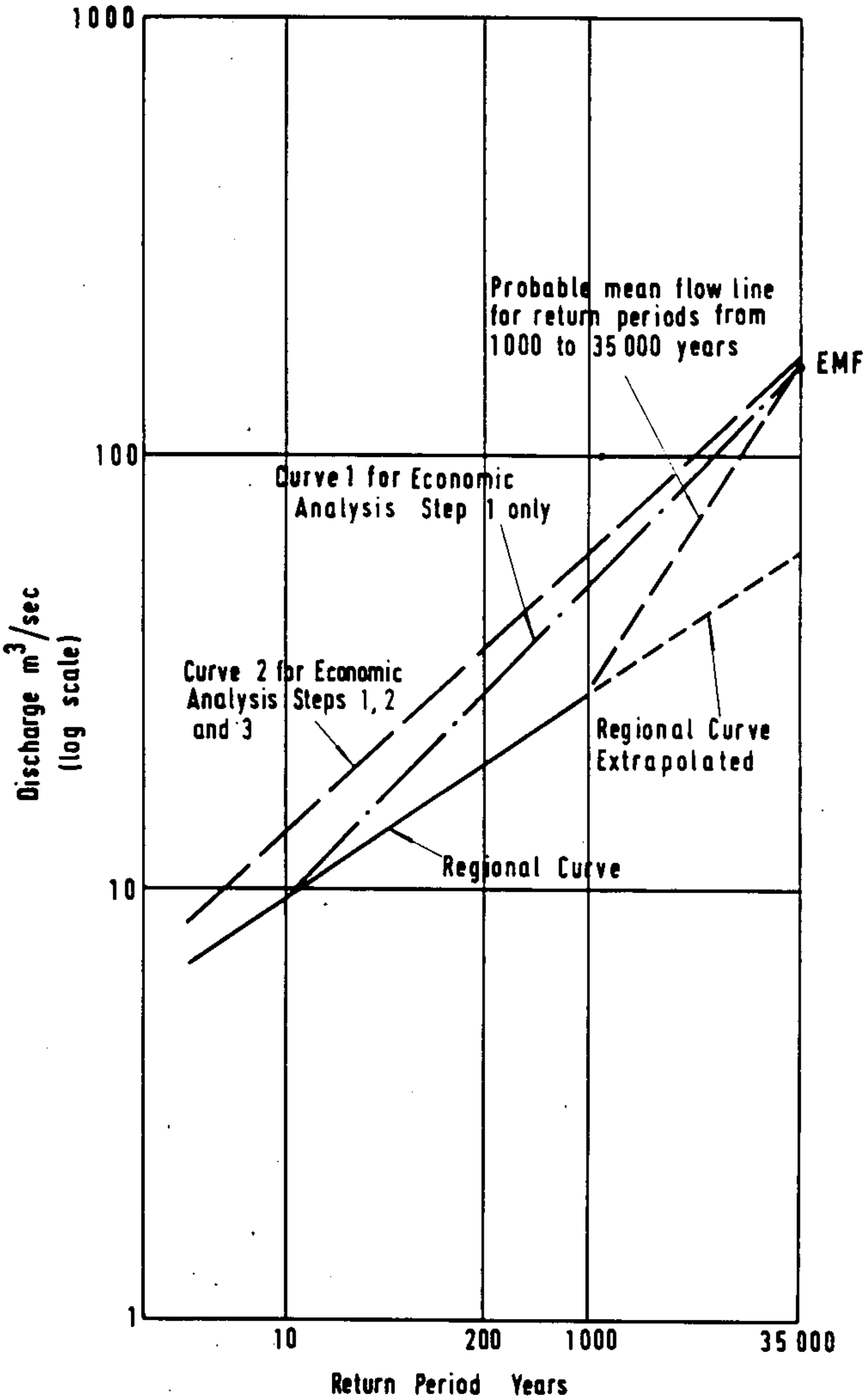

Fig. A.3 Proposed Construction for Flow : Frequency Curves for Economic Analysis

EXTREME RAINFALL ESTIMATION FOR FLOOD STUDIES IN THE SCOTTISH HIGHLANDS

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SYNOPSIS

A brief review is given of the historical approach to flood estimation in Scotland and reference made to the early work of the North of Scotland Hydro-Electric Board. This developed into more sophisticated methods in line with the Flood Studies Report (1). In some respects, particularly areal reduction factors for rainfall and volume of quick response runoff, this has been found inappropriate for Scottish conditions and the development of alternate approaches and the reasoning behind them is described.

HISTORICAL BACKGROUND

Floods have long been recognised as a problem in Scottish rivers. An account of historical floods is given in Highland Floods (2) and other well-documented floods occurred in the western Highlands in 1921/22, 1962 and 1966. Several studies of the relationship between rainfall and runoff were made but none proved really helpful for flood estimation although knowledge of the basic hydrological relationships was extended.

FLOOD ESTIMATION FROM RUNOFF

Initially attempts were made to estimate extreme floods from the runoff data but this proved difficult on a number of counts. Most of the river flow records in west Scotland were very short and many were unreliable - only two in the Hydro-Electric Board's area were of more than 20 years' duration and most were shorter than 10 years.

Reliance was placed on the I.C.E. 'Interim Report on Floods in Relation to Reservoir Practice (3) but this only provides estimates of flood peaks in a natural river and guidance with flood hydrographs on catchments up to a maximum of 104 km^2. The latter are necessary for flood routing through the reservoirs of a hydro-electric scheme, the catchments of which are almost all larger than 100 km^2 and often arranged in a complex cascade system. The flood peaks represent an envelope of what had been observed in the past over the British Isles as a whole, with no quantitative indication of rarity or regional variation. The return period of normal maximum flood, as defined in this work, has since been computed to vary between six years and infinity in eight catchments in the Board's area.

In an attempt to gain additional data the daily flows in the River Moriston were correlated seasonally with antecedent rainfall and linear regression equations derived with five terms involving a constant and the four preceding days' rainfall considered separately. The resultant expression gave quite a reasonable fit to observed flows but it both under-estimated peaks and failed to explain a number of isolated events, although it could have been used to generate a time series of flows having similar statistical properties to the original from a long period rainfall record.

Twenty-six years of River Moriston flows were the best guide to the specific problems of the Highland floods before the start of the Flood Studies Team's work at the Institute of Hydrology. The seven highest floods had been analysed to give flood peaks and associated rainfall conditions, and thus to derive rainfall thresholds which needed to be exceeded before serious flooding in this river could occur. Unfortunately such thresholds were occasionally exceeded, unassociated with flooding, and considerations of snow-cover variations did little to solve the discrepancies.

FLOOD ESTIMATION FROM RAINFALL

In view of the sparsity and frequent unreliability of the river flow records and the much better coverage of the rain gauge network much of the flood estimation was based on observed extreme rainfall figures and the effects these would have on river flows and loch levels if all the water ran off the catchment within subjectively chosen time limits.

Use was made of Jenkinson's paper (4) on the statistics of meteorological extremes in computing probabilities of rainfall and runoff amounts. Considerable experience had been gained in this direction by estimating catchment rainfalls and routing these through the Board's works for the February 1962 and December 1966 floods on the Rivers Conon and Beauly.

Two intense storms in West Scotland had been studied; that of December 1966 by Reynolds (5) and that of March 1968 by Bleasdale (6). A feature of these storms was the vast area over which they raged; an area of 1350 km^2 experienced more than 200 mm in 1966 and 2650 km^2 had the same amount in 1968. No comparably-sized previous storm over two days had ever been recorded, the largest being the Norfolk storm of 1912 with 178 mm over 670 km^2.

Since 1865 there have been 38 occasions when 178 mm or more have been recorded in a rainfall day in the British Isles; only five of these have been in Scotland and 19 of them have been in the four most south-westerly counties of England, so the Scottish Highlands are not as prone to this kind of weather extreme as is often imagined. The occurrence rate for Scotland is 0.07 occurrences per 1000 km^2 per century compared with 1.60 occurrences for Dorset and Somerset alone.

FLOOD STUDIES TEAM'S METHOD

It was at this stage that the results of the Flood Studies Team's investigation began to become available and extreme rainfall estimates were based on the methods later published in Vol II of the Flood Studies Report (1). Only in three particulars did the methods used by the Hydro-Electric Board differ - in areal reduction factors, snow melt and quick response runoff, and these differences will now be discussed.

AREAL REDUCTION FACTORS

One stage in the estimation of extreme rainfalls over specified catchments is the reduction of point rainfall estimates of a given return period to areal estimates. This is necessary because all storms must have a centre of maximum severity and the rainfall, both in absolute terms and in terms of the return period of the storm, becomes relatively less intense as one moves away from this centre. Because the design storms are basically derived from two day M5 rainfall, areal reduction factors (ARFs) should be calculated in relation to a similar parameter.

The Flood Studies Report Vol II gives tables and a graph to determine ARF's, found to be dependent solely on the catchment area and storm duration and independent of return period, but these suffer from three deficiencies if applied to the Board's area:

(i) All the data on which they are based are taken from Southern England but in certain aspects of extreme rainfall analysis there are fundamental differences between the rainfall regimes of Southern England and Scotland. For instance the meteorological conditions giving rise to severe floods are very different, widespread developing warm sector depressions contrasting with more convective systems, and the more diverse terrain. Results derived from England should not be used in West Scotland, at least without local testing.

(ii) The specific method of analysis is not thought to be valid in that point rainfalls of a particular storm are compared with the highest fall of that duration at the same gauge in the same year. This is often a totally different rainfall event. When it is not, it produces an ARF of unity which is not very helpful. The parameter which should be examined is the ratio of observed rainfall at each gauge in the catchment to the extreme rainfall of a known return period at that site so that the relative storm severity can be determined. These ratios could then be plotted and ARFs for specified areas determined.

(iii) Because the largest rain storm of each year of record was included many events which are not real storms in the true sense were used to derive ARFs.

When this matter first arose neither the alleged incorrect approach nor the modification which it was believed should be used could be applied in the Highland area because the Board's data could not readily be translated into a suitable form, and therefore a distinctly different method had to be derived.

Isohyetal maps of the distribution of the 48 hour rainfalls for two intense and widespread storms in Scotland, those of 16—17 December 1966 and 26—27 March 1968, were available. An overlay of the Beauly catchment outline with 17 roughly equidistant spot points drawn on it was prepared. The Beauly catchment of 878 km^2 was selected because it was of special interest at the time and it is of reasonably typical shape so as to have a more general application, see Jarvis (7) and Johnson and Cooke (8).

Six non-overlapping areas from Sutherland southwards to Argyll were selected for areal reduction factor analysis for each storm. It might have been possible to choose other areas but it would have proved difficult to get independent coverage over the western Highlands without using something like the areas chosen.

The overlay was placed on the map of the particular storm over each area in turn and the rainfall interpolated for each of the 17 spot points. The average of these values was expressed as a ratio of the maximum spot value. This gave a crude ARF which was compounded of both the real ARF and the natural fall-off of rainfall as one moves from normally wetter to drier areas.

To a first approximation the MT rainfall (maximum rainfall with a return period of T yrs) of any place is a function of the AAR (average annual rainfall), and the overlay was used in a similar fashion on the AAR map to determine the mean annual rainfall over the same area. Maps of two day M5 rainfall were not then available. Maps were available showing the M5 rainfall expressed as a percentage of AAR, and in West Scotland they show that this ratio is higher in dry areas than in wet. As an example, over Sutherland it varied from 5.5% where the AAR was 1000mm to 4.8% where the AAR was 2300mm.

To correct for this each spot value of AAR should have been multiplied by the ratio for its particular location and the results summed and meaned to determine the ARF in M5 over the catchment, but a shorter method was deemed sufficiently accurate. The M5/AAR ratio appropriate to the mean catchment AAR was multiplied by its M5/AAR ratio. This last ratio was the ARF due to the fact that storms of the same absolute rainfall total are not equally likely at all points in the catchment. The ratio of the crude ARF to the ARF of the M5 storms give the true areal reduction of the particular storm under examination.

Using six areas for both storms gave 12 estimates of the ARF for a catchment of 878km^2. There was no apparent difference in this when the storms were analysed separately, and no suggestion of location differences. The mean ARF was computed as 0.837 with a standard error of the mean of 0.030 for storms of 48 hours on a catchment of 878km^2. By comparison the Flood Studies Report graph would give 0.92 under the same circumstances.

On this basis it was decided to use this ARF for areas of 750km^2 or more, to use Flood Studies Report ARFs for very small catchments, and for catchments in between derive a smooth transition from one set to the other (Fig 1). Criticism has been levelled at this method on the grounds that the catchment areas were chosen after inspection of the storm isohyets and therefore to some extent centred upon storm centres. This criticism was not accepted, and in an endeavour to refute it eighteen river basins forming sections of or the whole of hydro-electric scheme catchment areas ranging between 64km^2 and 1004km^2 were chosen and ARFs for each of these were determined for each of the two storms previously mentioned — the only ones available. In this instance ARF was defined as the average value over the catchment ratio of observed rainfall at a point to the two day M5 rainfall at that point, divided by the highest such ratio within the catchment (the centre of severity of the storm). These 36 ratios are plotted on Fig. 2 along with the Flood Studies Report ARF for 24 hour storms and the curve originally derived by the Board. It will be seen that the latter is still conservative, being closer to unity than all but three points. For sake of clarity and ease of comparison with the Cruadach storm, the 24 hour ARFs only have been plotted, but the 48 hour ARFs differ by 0.02 or less.

The Flood Studies Report states that ARF is independent of return period. This may not be true of very extreme storms, which are those most interesting to water engineers. ARFs were calculated for a storm centred on Cruadach, at the western end of Loch Quoich, Inverness-shire, in 1954 when 259mm fell in 22½ hours. For circular catchments centred upon Cruadach the ARFs are shown as the crosses on Fig 2 and the curve represents a free-hand best fit line to this data. If individual river catchments in the area are computed the ARFs are even lower. The circles represent, from left to right, the catchments to Quoich Dam, Loch Morar, Loch Arkaig, River Garry alone, and River Garry including Quoich.

Clearly one storm alone cannot be used as a complete basis for ARFs, but if this is the only evidence, it may be that with such abnormal rainfalls the ARFs of the Flood Studies Report are inappropriate.

QUICK RESPONSE RUNOFF

The experience of the North of Scotland Hydro-Electric Board indicates that the Flood Studies Report formula to determine the percentage of quick response runoff very much under-estimates the resultant storm runoff in the Highlands of Scotland, despite a sparsity of data with which to check it. The Board has carried out detailed analyses of the December 1966 flood on the River Beauly, where computations based on rainfall and runoff data from five sub-catchments, all supporting the final results, can be combined at the lower end of the catchment to give the first line of the following table:-

Catchment	Date	Storm Rainfall mm	Quick Response Calculated %	Runoff Observed %
Beauly	December 1966	155	69	95
Allt Uaine	September 1962	90	58	82
Allt Uaine	January 1974	350	88	96

The Allt Uaine data were derived from two periods of heavy rainfall over a $3km^2$ mountainous catchment near Loch Lomond and would appear to suggest that the formula performs worst with moderately heavy rain-storms. With very extreme storms its performance is better, the January 1974 incident including the highest rainfall-day observation ever recorded in Scotland. The relevant parameters used to compute quick response runoff in the Highlands are the catchment wetness index, the storm precipitation and the soil type. If the catchment wetness index or the storm precipitation are to blame for the discrepancies shown very large changes in the relevant constant terms are required, and this would upset the regressions derived for other regions in the British Isles.

If all the error is thrown into the soil type term, and it is shown in Volume 1 of the Report (Table 4.5) that a great many differing soils are included in Soil Type 5, then an estimate of the appropriate Soil Index which would bring the Flood Studies Report quick response runoff into line with the observed quick response runoff can be made as below:-

Catchment	Date	Revised Soil Index
Beauly	December 1966	0.79
Allt Uaine	September 1962	0.75
Allt Uaine	January 1974	0.58
Average	—	0.73

In determining an average double weight has been given to the Beauly observations because of their more complete nature, the catchment size and their study in greater detail. All the evidence in the Board's possession suggests a Soil Index of 0.70 to 0.75 for the peaty, rocky and often steep Highland catchments of Scotland.

The U.K. Soil Survey field handbook divides the drainage potential of soils into six classes of decreasing water conductivity, of which Classes 5 and 6 are very common in the Highlands though less common elsewhere. Classes 4, 5 and 6 have been condensed into one group, 3, by the Flood Studies Report (Vol 1, Table 4.4). This, it is felt, is the principal source of error in the quick response runoff computations. The relatively large number of Class 4 catchments placed in drainage group 3 compared with the almost total absence of Class 5 and 6 catchments in the same drainage group has biased the Soil Index towards low values. In addition, slopes of tens and twenties of degrees are very common in Scottish glens, yet no provision is made in the slope categories for this kind of terrain. The combination of the absent, yet required, drainage classes 4 and 5 and a slope class 4 would be to render necessary Soil Types 6 or 7 with markedly larger quick response runoff percentages.

SNOW MELT

The Flood Studies Report recommends that no allowance should be made for snow melt when computing a design flood of specified return period, but that when considering probable maximum precipitation, melting rates of up to 42mm per day should be applied, based on computations and observations in the English lowlands.

Neither of these recommendations are appropriate for the Scottish Highlands. While the median snow cover at the time of the storm might be nil, snow cover on the mountains is sufficiently frequent to merit some consideration. An allowance of 10% of the rainfall of the storm was considered to be a realistic maximum. The average of 10% and nothing, ie 5%, was felt to be too high, so 2% was subjectively chosen to represent probable snow melt during the period of storm rainfall. This was varied to 3% on the most upland catchments and 1% on the lower catchments.

The maximum snow melt of 42mm per day was based on computations from areas where winter temperatures can rise as high as 12°C. In the Highlands they are unlikely ever to exceed 9°C at an altitude of 300 metres or 7°C at 600 metres, above which much of the snow cover is lying. It seems probable, therefore, that these figures taken in conjunction with the diurnal temperature range would lead to much lower maximum snow melt rates in the mountainous areas of Scotland.

The Conon floods of 1921/22 and 1962 are known to have been augmented by snow melt, but this was not so in 1966 on the Beauly or the Shin.

CONCLUSIONS

The Flood Studies Report has put solutions to problems of rainfall and runoff extremes on a firm foundation but has highlighted areas of doubt requiring further research such as those referred to in this paper. The areal reduction factors (ARFs) of the Flood Studies Report need to be modified before being applied to the Scottish Highlands because they are much closer to unity than all the local data would suggest. Their method of derivation is believed to be inappropriate, at least for extreme storms.

The quick response runoff obtained from the Flood Studies formula is much too low in terms of local experience. Here it is believed that the Soil Index is at fault.

The Flood Studies recommendations on ARFs are about 10% higher than North of Scotland Hydro-Electric Board figures, but there is evidence to suggest differences of 30% with extreme rainfalls. The quick response runoff differences can be 30% to 40%. These areas of doubt are of similar magnitude to differences between 100 and 1,000 year return period rainfalls and approach the differences between 1,000 year rainfall and probable maximum precipitation.

The Flood Studies recommendation of using no snow melt except for cases where possible maximum precipitation is approached is inappropriate for Highland conditions; conversely the recommendation of 42mm per day under these circumstances is considered too high.

No one yet appears to have studied quantitatively the inter-action between snow melt and heavy rainfall.

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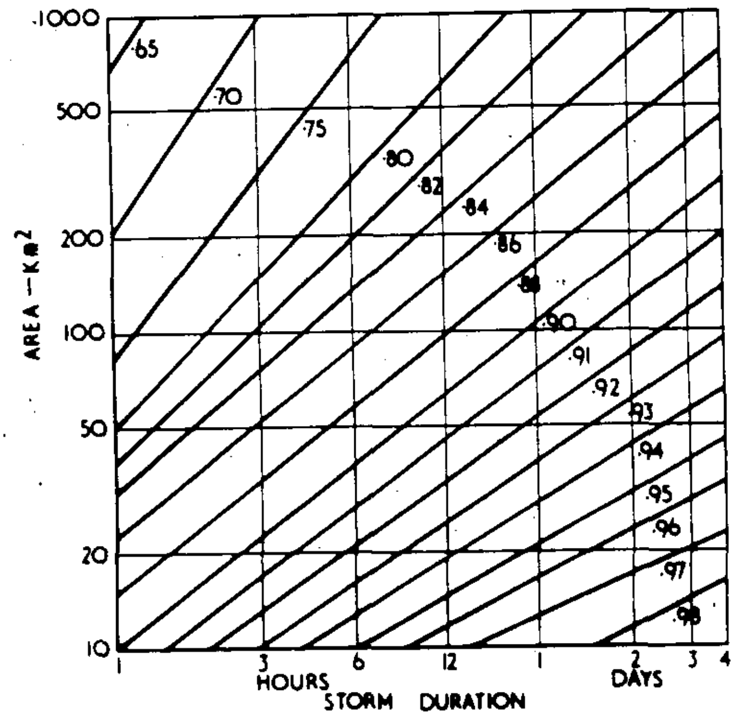

Fig.1 — Areal reduction factors used by North of Scotland Hydro-Electric Board.

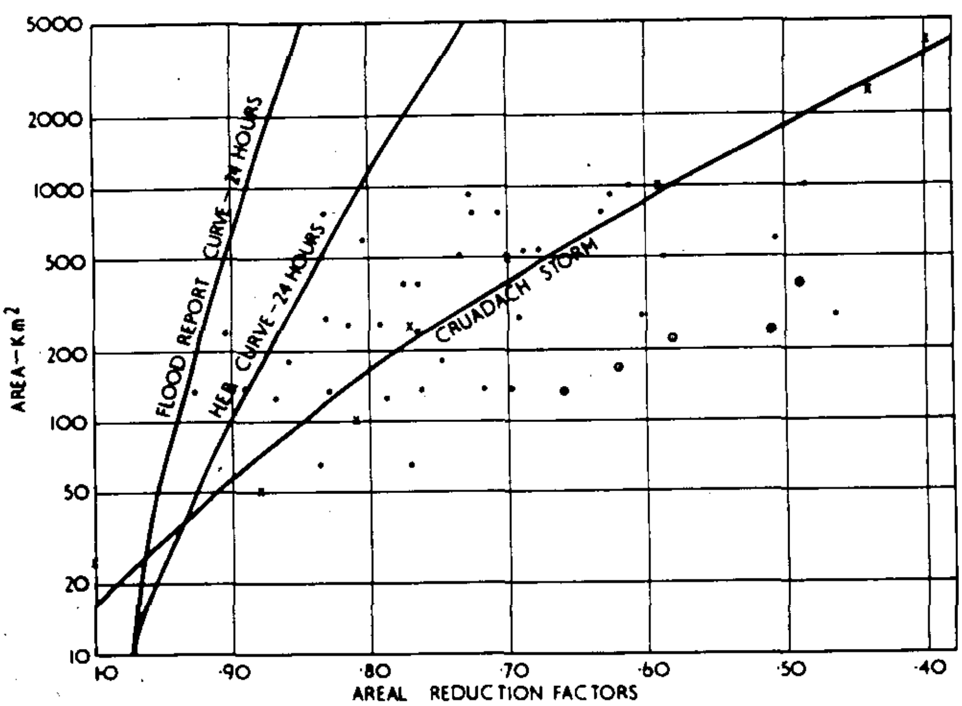

Fig. 2 — Areal reduction factors for Scottish Storms.

EXPERIENCES IN USING THE NERC FLOOD STUDIES REPORT OF 1975 FOR RESERVOIR INSPECTIONS

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SYNOPSIS

Methods are described of using the results of the studies of storm rainfall and associated runoff, published in 1975 by the Natural Environment Research Council, to assess the adequacy of reservoir spillways. Worked examples are presented on proforma designed to systematize the calculations, and to emphasise the importance of refining them by incorporating local data, the results of field visits, and experience of other similar catchments. Some lessons from recent reservoir inspections are summarised.

INTRODUCTION

Early in 1975 the Natural Environment Research Council (NERC) published the results (1) of a comprehensive study of floods and storm rainfall in the United Kingdom and the Republic of Ireland. The study, first propounded by a committee of the Institution of Civil Engineers, is a great advance on the empirical envelope curves of flood intensity used in the Institution's 1933 and 1960 reports 'Floods in Relation to Reservoir Practice', and therefore permits more assured assessments of the adequacy of reservoir spillways, such as are required by the Reservoirs Act of 1975. The figures, tables and statistical analyses presented in the NERC study are sufficient for the meteorological, hydrological and morphometric parameters for any catchment to be estimated without a visit to the site, and for a desk-top estimate to be made of the "maximum" flood or a flood of a given return period. Nevertheless it is acknowledged therein that if local runoff records are available they may improve the estimates if incorporated in the calculations for a particular site.

The purpose of this paper is to illustrate the use of the results of the NERC study in assessing the adequacy of spillways, presenting worked examples on proforma designed to make the calculations consistent and systematic; and to show how experiences in dam inspections have justified the extra time and effort involved in field visits and use of local data.

JUSTIFICATION FOR FIELD VISITS

Previous experiences have shown that a field visit to a site may avoid serious errors and uncertainties. Questions, some of which are so simple and basic as to be overlooked altogether in a desk study, may include those listed below, and some of the reservoir inspections for which they have been particularly relevant are named in brackets after the respective questions.

DIRECT CATCHMENT SIZE AND LOCATION

Is the actual location of the dam in accordance with its published map co-ordinates or location plan? Are the actual stream patterns and watersheds as shown on the 1:25, 000 map, and may these be changed by the construction of roads or other works? Are there other dams upstream whose condition or location are such that failure would suddenly increase the direct catchment and the inflow to the dam under consideration? (Lower Greggan).

INDIRECT CATCHMENTS

Are there catchwaters or other conduits leading water into the natural catchment? If so, are their alignments and catchment areas as indicated on the map, and are their discharge capacities limited by channel size, stoplogs, culverts or relief channels? (Rivington, Green Withens). Are there catchwaters or conduits leading out of the natural catchment? If so, do these channels have weak sections which may be breached during extreme flows and change the pattern of runoff to the dam?

CATCHMENT COVER

What is the present soil and vegetal cover? May these be changed in the period before the next inspection by urbanisation (Daventry, Brent) or deforestation? Is the depth of the surface cover shallow enough for sub-surface strata to become relevant at times of prolonged infiltration (Rufford)? Into which of the NERC categories do the catchment soils fall?

SPILLWAY HYDRAULICS

Does the stage-discharge relationship conform to the present dimensions of the spillway? May this relationship be affected by increased water levels downstream caused by constrictions, (Stocks, Withens Clough), other inflows or blockages ? May spillway discharges be reduced by debris (Combe Abbey Pool), stoplogs or failure of equipment such as gates? May the downstream face of the dam be damaged by very high spillway flows? (Withens Clough).

FREEBOARD

What is the actual freeboard between the reservoir water level at which spillage starts, and embankment top level, and does this vary from one place to another? Is the spillway or dam by-passed at higher water levels? Is the water level in the reservoir held down seasonally? If so, what are the most rigorous conditions likely in time of flood?

WAVE PROTECTION

What is the slope and the type of the upstream face of the dam? Are there vegetation or other obstructions which may affect wave fetch or height? What is the condition of the dam crest and of the downstream face of the dam having regard to wave splash and what is the condition of the wave wall (if any) to withstand wave impact at flood surcharge?

DOWNSTREAM CONDITIONS

Would failure of the dam threaten life in a community downstream and cause extensive damage, or would failure not add significantly to the effect of a flood?

COLLECTION OF LOCAL DATA

RAINFALL	Are any of the stations listed in 'British Rainfall' (2) within or near the catchment? Are there any other local rainfall gauges? If so, are they properly installed and read? Have rare events been recorded by these stations?
RUNOFF	Does the reservoir record book show rises in water level and/or depths of flow over the spillway which can be related to any of the rare rainfall events? Are seasonal trends in reservoir water level evident, and do records of annual peak outflow follow a frequency distribution? Are there stream gauging stations within the catchment or in similar catchments which would yield any of this runoff information?
WAVES	Are there local records or recollections of wind or wave behaviour in the reservoir, and are there any wind records in the region?

APPLICATION OF THE NERC STUDIES:

A. 'ESTIMATED MAXIMUM' FLOOD

Table 1 shows a proforma designed to make calculations of the 'Estimated Maximum flood' consistent and systematic. The symbols used are those in the NERC report (1), and the use of the form is illustrated by a worked example for Withens Clough Dam near Halifax, Yorkshire. On the right hand side of the page are two columns, one for a desk-top computation using data from the figures in the NERC report, and the other modifying the NERC figures as a result of the field visit. This table is used for the derivation of the profile of effective (runoff-producing) rainfall, and the unit hydrograph : the convolution of the two to obtain the runoff hydrograph is shown on Table 2, together with modifications to allow for a large reservoir area and calculations of wave allowance. Some detailed comments on the derivation of some of the NERC parameters are given below.

CATCHMENT DATA

MSL, S1085 and AREA were adjusted after a field visit because one catchwater which would otherwise have reduced the catchment area was considered likely to breach in an extreme event, whereas other catchwaters flowing into the catchment might still contribute to a flood:

SOIL was calculated assuming that all the catchment surface had poor 'winter rain acceptance' - NERC Table 1.4.5, except for the reservoir surface, which comprises 0.06 of the total area.

SAAR was adjusted in the light of a long local record. Other parameters were taken from the quoted figures in the NERC report and from the I C E discussion paper (3).

UNIT HYDROGRAPH

Tp, as calculated using the expression derived from recorded events in the NERC study, was reduced by a factor of 0.67 for the 'estimated maximum' event as recommended by NERC.

Tb was reduced in proportion to Tp for the 'estimated maximum' event by using the expression Tb = 2.525 Tp. However, there is not as strong a theoretical or empirical basis for reducing Tb as there is for reducing Tp (4) (5).

Qp is inversely proportional to Tb and is therefore increased by 50% due to the reduction of Tb. This was found for this example, and others, to cause a *significant increase* in the peak of the inflow hydrograph. However the volume of the design inflow remains unchanged, or will even be a little less if the shorter Tp causes a shorter design storm to be used. Thus if the routing effect of the reservoir is significant, as may be the case for water surfaces greater than 5% of the catchment, then the effect of the increased Qp on the outflow peak is small.

The initial values of Tp, Tb and Qp are for a one-hour 10 mm unit hydrograph but the optimum time interval for data input has been found to be Tp/5, and if this is different from one-hour then Tp, Tb and Qp have to be revised as illustrated on Table 1, preferably to multiples of the basic data interval.

DESIGN STORM

Duration D, where runoff is routed through reservoir storage, may be longer than given by the basic NERC expression. To take account of this it has been recommended (6) that the Tp used to derive D should be increased by the lag of the reservoir itself. This lag may either be obtained by a trial inflow /routing calculation or estimated by Lowing's method (6) as the slope of the upper part of the curve relating reservoir storage to spillway outflow. In the example, the trial routing gave a 'lag' between inflow and outflow peaks of 1.2 hr, and the slope of the storage/outflow curve gave 2.3 hr. This at first appears to be a significant difference, but the effects on the final hydrographs were not significant.

The 2 hr and 24 hr maximum point rainfall (see under 'Catchment Data') were reduced to maxima for other durations by applying factors from NERC Tables 4.1, 4.2 and 4.3, and converted to a real rainfall using NERC Fig. 5.1. For 'estimated maximum' storms on irregular-shaped catchments it would seem reasonable to enter Fig. 5.1, not with the area of the catchment, but with the area of a regular figure such as an ellipse which covers all or most of the catchment. The resultant areal rainfalls are plotted against duration to obtain the total rainfall during the design storm period, and also the profile (see under 'Runoff' below).

One of the most intense storms ever recorded in Britain occurred at Hewenden, near this site, and the measured figure of 154 mm in 105 minutes (later disputed by the Meteorological Office) falls very close to the derived rainfall/duration curve.

Figure 5.1 of the NERC study referred to above was based on only a few storms and probably gives conservatively high factors for 'estimated maximum' storms. More data is needed to produce improved curves, possibly on a regional basis.

RUNOFF

The local records of rainfall and runoff gave 69% runoff or less, and it was therefore considered safe to use the figure of 80.7% derived in Table l. The profile was then built up as 'nested' maxima from the rainfall/duration curve, to a total of (190 x 0.807) ≈ 154 mm, of which 6 mm is snowmelt. This effective rainfall was then applied to the unit hydrograph to obtain the inflow to the reservoir, and base flow was added - see Table 2. Direct rainfall on the reservoir water surface was converted to a simultaneous inflow rate and added in place of that part of the inflow hydrograph which was originally assumed to come from the unsubmerged reservoir area. The effect of this was to increase the rates of flow for the early part of the hydrograph, but to reduce the peak slightly.

WAVE ALLOWANCE was calculated in accordance with the recommendations of the I.C.E. Discussion Paper (3).

APPLICATION OF THE NERC STUDIES:

B. FLOOD OF GIVEN RETURN PERIOD

Table 3 shows a proforma for the calculation of floods of given return periods. The 'Catchment Data' section is similar to that of Table 1 except for the calculation of catchment wetness index. The 'Unit Hydrograph' section is also similar to Table 1 except that no reduction factor is applied to Tp., and the duration of the design storm is calculated as before.

The total design precipitation 'Pdes' is calculated using the results of the statistical analyses in the NERC study: growth factors for the respective return periods and for the design duration were applied to the five-year two-day value already obtained (see under 'Catchment Data'). The percentage runoff, calculated as before, varies slightly according to Pdes, and the rainfall profile, as recommended by NERC, is taken as the 75% winter profile (NERC Fig. 6.65). The NERC statistical analyses were based on data for total runoff, without distinguishing between snowmelt and storm runoff, so no separate allowance for snowmelt need be made.

LESSONS FROM RECENT INSPECTIONS

The studies by the Meteorological Office in recent years, in particular those for the NERC Floods Study, are such that the 'maximum' or rare rainfall events can be estimated within narrower limits of confidence than most of the other parameters in the calculation of a design flood. When these calculations are for a dam inspection report there is usually a reservoir routing effect such that refinements in calculating the shape of the inflow hydrograph have little effect compared with total volume of runoff. Therefore assuming that rainfall can be fairly well defined, the variable which most affects the requirements for spillway capacity is the percentage runoff, and this is an aspect which should be carefully considered during a field visit. On the one hand, can a local estimate of soil index 'SOIL' be obtained? (Perhaps in some areas it may exceed the NERC maximum of 0.5). On the other hand, can percentages of runoff be derived from records of rainfall and rapid rises in reservoir level?

At sites where there is little or no storage or routing effect the hydrograph shape is important. Where a triangular unit hydrograph is used, the peak Qp is governed by the base length Tb, and changes in Tp (Tb remaining constant) have relatively little effect on the peak of the convoluted inflow hydrograph. The present procedure in the 'Estimated Maximum' Flood calculation of reducing both Tp and Tb lead to a 50% increase in Qp, which may be very significant for small reservoirs. Careful consideration therefore needs to be given, both in further research and on field visits, as to whether Tb should be reduced for the 'Estimated Maximum' event.

Field inspections of catchwaters have nearly always revealed important features such that the catchment areas used in design have been different from those which might have been assumed in a desk study of maps and drawings. Similarly obstructions such as bridge piers or culverts - sometimes installed after record drawings have been prepared - have occasionally reduced the capacity of the channel downstream from a spillway such that backwater or even submergence reduces the discharge capacity.

TIME REQUIREMENTS

An engineer who is unfamiliar with these methods may take two or three days for the desk study, but with experience this time may be reduced to less than half a day if the necessary maps are to hand. The field visit and working up of records may take two days to ten days or more depending on the amount and quality of data available and large amounts of unprocessed data may take several weeks to analyse. The extent and detail of the studies should be adjusted to the degree to which the accuracy of the answers is critical to the safety of the reservoir.

SUMMARY

The NERC Flood Study of 1975 provides the basis for improved flood estimates, particularly on ungauged catchments, but the need for a field visit and analysis of any relevant records remains. Further research is needed, particularly in regard to percentage runoff, base length of unit hydrograph, and areal reduction factors for short intense storms. This will require further data on large storms and floods.

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"ESTIMATED MAXIMUM" FLOOD BY NERC UNIT HYDROGRAPH METHOD

Location Withens Clough Coords. 53° 42 N, 2° 02 W/E. Region N° 10

				LOCAL DATA
CATCHMENT DATA: Length of main stream, km:	MSL	=	1.5	2.4
Main stream slope between 10% & 85% of M.S.L.: m/km:	S1085	=	61	53
Fraction of urban catchment:	URB	=	Nil	Nil
Area of catchment, km²:	AREA	=	3.2	4.2
Soil Index (0.15 x 0) + (0.3 x 0) + (0.4 x 0) + (0.45 x 0) + (0.5 x 94) / 1 - .06 :	SOIL	=	0.5	0.5
Annual average rainfall, (NERC Fig II.3.1), mm:	SAAR	=	1400	1330
Maximum 2 hr point rainfall, (NERC Fig II.4.1), mm:		=	160	
Maximum 24 hr point rainfall, (NERC Fig II.4.2), mm:		=	300	
5 yr 25 day rainfall: ___ % of SAAR (NERC Fig II.3.4), mm:	M525D	=	not needed	
5 yr 2 day rainfall: 5.85 % of SAAR (NERC Fig II.3.3), mm:	M52D	=	78	
5 yr 60 min rainfall % of M52D (NERC Fig II.3.5),	r	=	25	
Effective soil moisture deficit (NERC Fig I.4.19), mm:	SMD	=	2	
Areal M52D less soil moisture deficit (I.C.E. Fig 3.2), mm:	RSMD	=	55	
UNIT HYDROGRAPH				
Time to peak (hr) = $46.6\ (MSL)^{0.14}(S1085)^{-0.38}(1+URB)^{-1.99}(RSMD)^{-0.4} \times 0.67$ = 31.2x(1.13) x (0.22) x (1.0) x (0.20) :	Tp	=	1.55	
Base of 1-hr, 10 mm unit hydrograph = 2.525 Tp, hr:	Tb	=	3.91	
Peak of 1-hr, 10 mm unit hydrograph = 2.2(AREA)/Tp m³/s:	Qp	=	5.95	
Basic data interval, Tp/5 to nearest round figure, hr:	t(say)	=	0.2	
Revised Tp = old Tp + (t-1)/2, for t-hr, 10 mm, U-H:	Tp	=	1.15	say 1.2
Revised Qp = old Qp (old Tp)/(revised Tp), for t-hr U-H:	Qp	=	8.02	say 8.0
Revised Tb = revised Tp x 2.525, for t-hr, 10 mm, U-H:	Tb	=	2.9	say 2.8
DESIGN STORM				
Duration of design storm: (1 + SAAR/1000) Tp, hr:		=	2.88	2.8
For reservoirs, add 1.2 / 2.2 hr lag, inflow-outflow peaks Ø: total D (say)		=	5	4
5D = 25 / 20 hr. Number of rainfall intervals D/t		=	25	20

OR: D = Reservoir rise time Ø + 2 = 4.4 hr, 5D = 22 hr, D/t = 22

Maximum rainfall for various durations (NERC tables II.4.1, 4.2, & 4.3)

Duration d:	10	15	30	60m	2hr	24	48	96	192hr	25 day	RECORDS
Reference	NERC table II.4.1					NERC table II.4.3				II.4.2 & 4.3	154 mm in 105 min
% or factor	36	47	65	83	**100**	**1.0**	1.11				&
Rainfall mm	58	75	104	133	**160**	**300**	333				___ mm in
ARF (Fig. II.5.1)	88	90	92	93.5	94.5	98	99				
P=Area r/fall	51	68	96	124	151	294	330				___ min

From log-log plot of d vs P : Rainfall during design storm:	P(d = D)	=	194	183
Longer-period rainfall:	P(d = 5D)	=	303	286
Snow melt @ 42 mm/day = 42D/24 = 1.75D mm		=	9	7
Total design precipitation = P(d = D) + Snowmelt: mm	Pdes	=	203	190
Antecedent 2D hr precipitation: $(P_{5D}-P_D)/2 + 2\ (1.75D)$:	Pa	=	72	65
RUNOFF				
Catchment wetness index: $125 + (Pa \times 0.5^{D/24})$: mm	CWI	=	187	183
Percentage runoff: 95.5 (SOIL) + 12 (URB) + 0.22 (CWI-125) + 0.1 (Pdes-10) =	PR	=	80.7	69 max
Net rain running off catchment: (Pdes x PR)		=	164	154

PROFILE of net rainfall = (nested values from plot of d vs P) x PR

Duration:hr	t	·	3t	·	5t	·	7t	·	9t	·	11t	·	13t	D	
R/fall from plot	46		82		100		110		118		126		132	148	
R/fall increment	46	18	18	9	9	5	5	4	4	4	4	3	3	16x1	
Bell profile (mm)	8x1	3	4	4	5	9	18	46	18	9	5	4	4	3	8x1
Snow (42t/24) (Total 6)	1				1		1		1		1			1	
Total Pt	5	5	5	5	6	9	19	46	19	9	6	(4 x 5 mm)			

(convolute Pt/10 with t hr, 10 mm unit hydrograph)

Ø lag, peak-to-peak may be estimated from the slope of the reservoir storage-discharge curve

Reservoir rise time is approximately Tp + Lag, peak-to-peak

TABLE 1

A. CONVOLUTION OF t hr, 10 mm UNIT HYDROGRAPH
B. Modification to allow for direct rainfall on reservoir surface where that surface area exceeds 0.05 AREA
C. Calculation of allowance for waves.

unit hydrograph ordinates, m^3/s, at (t = 2 hr) intervals:

A

cm net rain	1.3	2.6	4.0	5.4	6.7	8.0	7.0	6.0	5.0	4.0	3.0	2.0	1.0	0														
0.5		.65	1.3	2.0	2.7	3.35	4.0	3.5	3.0	2.5	2.0	1.5	1.0	0.5	0													
0.5			.65	1.3	2.0	2.7	3.35	4.0	3.5	3.0	2.5	2.0	1.5	1.0	0.5	0												
0.5				.65	1.3	2.0	2.7	3.35	4.0	3.5	3.0	2.5	2.0	1.5	1.0	0.5	0											
0.5					.65	1.3	2.0	2.7	3.35	4.0	3.5	3.0	2.5	2.0	1.5	1.0	0.5	0										
0.6						.78	1.56	2.4	3.24	4.02	4.8	4.2	3.6	3.0	2.4	1.8	1.2	0.6	0									
0.9							1.17	2.34	3.6	4.86	6.03	7.2	6.3	5.4	4.5	3.6	2.7	1.8	.9	0								
1.9								2.47	4.94	7.6	10.26	12.73	15.2	13.3	11.4	9.5	7.6	5.7	3.8	1.9	0							
4.6									5.98	11.96	18.4	24.84	30.82	36.8	32.2	27.6	23.0	18.4	13.8	9.2	4.6	0						
1.9										2.47	4.94	7.6	10.26	12.73	15.2	13.3	11.4	9.5	7.6	5.7	3.8	1.9	0					
0.9											1.17	2.34	3.6	4.86	6.03	7.2	6.3	5.4	4.5	3.6	2.7	1.8	0.9	0				
0.6												.78	1.56	2.4	3.24	4.02	4.8	4.2	3.6	3.0	2.4	1.8	1.2	0.6	0			
0.5													.65	1.3	2.0	2.7	3.35	4.0	3.5	3.0	2.5	2.0	1.5	1.0	0.5			
0.5														.65	1.3	2.0	2.7	3.35	4.0	3.5	3.0	2.5	2.0	1.5	1.0	0.5		
0.5															.65	1.3	2.0	2.7	3.35	4.0	3.5	3.0	2.5	2.0	1.5	1.0	0.5	
0.5																.65	1.3	2.0	2.7	3.35	4.0	3.5	3.0	2.5	2.0	1.5	1.0	0.5
TOTAL		.65	1.95	3.95	6.65	10.13	14.78	20.76	31.61	43.91	56.60	68.69	77.99	85.44	81.92	75.17	66.85	57.65	47.75	37.25	26.50	16.50	11.10	7.60	5.0	3.0	1.50	0.5

Add ANSF = Average non-separated base flow = [(0.00033 CWI) + (0.00074 RSMD) −0.038] x AREA = 0.28 to each ordinate to give total stream flow (TSF)

TSF		.91	2.21	4.21	6.91	10.4	15.0	21.0	31.9	44.2	56.9	69.0	78.2	85.7	82.2	75.4	67.1	57.9	48.0	37.5	26.8	16.7	11.4	7.9	5.2	3.2	1.8	0.7

B Where reservoir area RA exceeds 5% AREA, consider separately: RA = 26 km^2: Revise stream flow by a factor of (1−RA/AREA) = .94 :

: add instantaneous response per 1cm direct rainfall on RA in t hours = RA/0.361 = 3.6 m^3/s x cm rain = Qr

Revised TSF		.9	2.1	3.9	6.5	9.8	14.1	19.7	30.0	41.5	52.5	64.8	73.5	80.5	77.2	70.8	63.0	54.4	45.1	35.2	25.2	15.7	10.7	7.4	4.9	3.0	1.7	0.7
Qr		1.8	1.8	1.8	1.8	2.2	3.2	6.8	16.5	6.8	3.2	2.2	1.8	1.8	1.8	1.8												
TOTAL		2.7	3.9	5.7	8.3	12.0	17.3	26.5	46.5	48.3	55.7	67.0	75.3	82.3	79.0	63.0	54.4	54.4	45.1	35.2	25.2	15.7	10.7	7.4	4.9	3.0	1.7	0.7

C WAVE ALLOWANCE

Longest straight line fetch = 85 km: Effective fetch = 0.5 x straight line fetch = 42 km (~~alternatively~~ tabulate as I.C.E. table D.1)
Wind force (Gale or Hurricane) = Hurricane: upstream slope = 1 in 3: Vertical run-up H = .8 m (I.C.E. Figure 7.1)
Design wave height (I.C.E. Table 7.2): Class of dam = C , Wave height = 1.2 x H. = .96 m
Factor for slope roughness (I.C.E. Table 7.3) = 1.0 : Required wave allowance = 1.0 m.

TABLE 2

ESTIMATE OF Q(T) BY NERC UNIT HYDROGRAPH METHOD — **LOCAL DATA**

Location ________ **Coords.** __°__' N, __°__' W/E. **Region No** _______

CATCHMENT DATA: Length of main stream, km: MSL = ______ ______

Main stream slope between 10% and 85% MSL, m/km: S1085 = ______ ______

Fraction of urban catchment: URB = ______ ______

Area of catchment, km^2: AREA = ______ ______

Soil Index $\dfrac{(0.15 \times \_\_) + (0.3 \times \_\_) + (0.4 \times \_\_) + (0.45 \times \_\_) + (0.5 \times \_\_)}{1 - \_\_}$: SOIL = ______ ______

Annual average rainfall	(NERC Fig II.3.1), mm:	SAAR	=	______	______
5 yr 2 day rainfall ___ % of SAAR	(NERC Fig II.3.3), mm:	M52D	=	______	______
5 yr 60 min rainfall as a % of M52D	(NERC Fig II.3.5), mm:	r	=	______	______
Effective soil moisture deficit	(NERC Fig I.4.19), mm:	$\overline{SMD}$	=	______	______
Areal M52D, less soil moisture deficit	(I.C.E. Fig 3.2), mm:	RSMD	=	______	______
Catchment wetness index	(NERC Fig 6.62), mm:	CWI	=	______	______

UNIT HYDROGRAPH

Time to Peak $= 46.6(MSL)^{0.14}(S1085)^{-0.38}(1+URB)^{-1.99}(RSMD)^{-0.4}$

$= 46.6(\_\_) \times (\_\_\_) \times (\_\_\_) \times (\_\_\_)$, hr: Tp = ______ ______

Base of 1-hr, 10 mm unit hydrograph = 2.525 Tp, hr: Tb = ______ ______

Peak of 1-hr, 10 mm unit hydrograph = 2.2(AREA)/Tp, m^3/s: Qp = ______ ______

Basic data interval, Tp/5 to nearest round figure, hr: t(say) = ______ ______

Revised Tp = old Tp + (t-1)/2, for t-hr, 10 mm, U-H: Tp = ______

Revised Qp = old Qp(old Tp)/(revised Tp), for t-hr, 10 mm, U-H: Qp = ______

Revised Tb = Revised Tp x 2.525, for t-hr, 10 mm, U-H Tb = ______

DESIGN STORM AND RUNOFF

Duration of design storm: (1 + SAAR/1000) Tp, hr: = ______ ______

For reservoirs, add ___ hr lag, inflow-to-outflow peaks Ø: total D(say) = ______ ______

5D = ___ hr. Number of rainfall intervals: D/t = ______ ______

OR: D = Reservoir rise time Ø + 2 = ___ hr, 5D = ___ hr, D/t = ___ — **RECORDS**

Return period(s) of design flood ___ or ___ or ___ yr — ___ mm

Return period(s) of design storm (NERC 6.61), T: ___ or ___ or ___ yr — in

(i) Ratio M5(D)/2 day M5 (NERC Table II.3.10): ___ — ___ min

(ii) Growth factor MT/M5 (NERC Table II.2.7 & 2.9): ___ or ___ or ___

(iii) Area reduction factor (NERC Fig II.5.1): ___

Total design precipitation (M52D)(i)(ii)(iii), Pdes = ___ or ___ or ___ mm

Percentage runoff PR = 95.5(SOIL) + 12(URB) + 0.22 (CWI-125) + 0.1(Pdes − 10) — max:

PR = ___ or ___ or ___ % according to Pdes — ____ %

Net rain running off catchment, (Pdes x PR) = ___ or ___ or ___ mm ____

PROFILE of net rainfall (NERC Fig 6.65). ___ intervals of (t = ___) hr.

Interval No.	___	___	___	___	___	___	___	___	___	___	___	___	___
% time (cumulative)	___	___	___	___	___	___	___	___	___	___	___	___	___
% rainfall (cumulative)	___	___	___	___	___	___	___	___	___	___	___	___	___
% rainfall (increment)	___	___	___	___	___	___	___	___	___	___	___	___	___
mm rainfall (increment)*	___	___	___	___	___	___	___	___	___	___	___	___	___
mm bell profile	___	___	___	___	___	___	___	___	___	___	___	___	___

* for (Pdes x PR) = ___ mm. For other (Pdes x PR) reduce U-H ordinates in proportion.

Ø lag, inflow-to-outflow peaks may be estimated from the slope of the reservoirs storage-discharge curve; reservoir rise time is approximately Tp + lag, peak-to-peak.

TABLE 3

SOME HYDROLOGICAL AND HYDRAULIC CONSIDERATIONS IN SPILLWAY DESIGN AND OPERATION

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SYNOPSIS

The advantages of gate controlled flood spillways over uncontrolled flood spillways is briefly described. A method is outlined which shows how forecasts of flood outflow hydrographs are made up of deterministic and probabilistic components and how these may be calculated to update forecasts at regular intervals. The usefulness of this method to flood warning procedure and control is discussed.

The effect of increasing the design discharge on existing spillway and stilling basin capacity is discussed with particular reference to overfall spillways; shaft and gated spillways are also briefly discussed. The possibility of using different safety factors in relation to floods for different parts of outlet works is demonstrated.

INTRODUCTION

Control of river floods by reservoir storage is a well established technique. Design of flood spillways to effect release of flood waters from reservoirs in a manner both safe to the reservoir, dam and spillway and to river sections downstream of the dam has long been a topic of critical debate. In Great Britain, the publication of the discussion paper on '*Reservoir Flood Standards*' by the Institution of Civil Engineers in 1975 (1) is bound to stimulate interest, not only in methods for the derivation of design floods for new reservoirs, but also in the reassessment of the capacity and safety of outlet works of existing dams.

The ICE discussion paper deals almost exclusively with the hydrology of design flood estimation. The only hydraulic aspects discussed are those of surface wave and run-up calculations in connection with dam freeboard. Detail consideration of other hydraulic problems and of operational aspects of hydrological control is not given, presumably since such topics have received a great deal of attention in research and publications in the past.

It is understandable and correct that flood estimation for reservoir design be given major priority. In the context of this symposium on *Inspection, Operation and Improvement of Existing Dams*, however, it is considered appropriate to draw attention to some other hydrological and hydraulic considerations resulting from spillway design and operation, and from flood reassessment.

OPERATIONAL FLOOD CONTROL

Once a design is realised there exists the problem of forecasting flood flows into and out of the reservoir and of operating reservoir outlet works in such a way that hazard and damage of flood waters (mainly downstream of the dam) are minimized, and water resource benefits maximised. Few reservoirs in Great Britain can be considered exclusive to flood control. In multiple-use reservoirs efficient operation of outflow may demand the establishment of monthly or seasonal reservoir level control rules, to ensure that demand for water is met at all times and that flood storage space is available to attenuate flood discharges entering the reservoir(2). In a country like Great Britain, however, where the occurrence of major flood events is not strongly seasonal and where the demand for water grows and conflicts in part with a desire for environmental control, variable storage control rules are difficult to determine.

To a large extent flood control reservoir attenuation storage is in this country incidental, but nevertheless still effective. Without seasonal storage control it can become more effective by the use of gate controlled spillways. Increased benefit of control depends largely upon volume of reservoir storage effectively controlled by outlet works (3).

Fig. 1 is a simple illustration of typical low head installation control,

a) without gated spillway

b) with gated spillway but where the gates are operated only in response to monitored inflow to the reservoir

and c) with gated spillway but where the gates are operated in response to forecast inflows.

(Simple gate control is assumed in this illustration and effectiveness is relative depending on size of ungated and gated spillways.)

The advantage of (b) and (c) over (a) is that since outflow is very much higher during, or before, the early part of the flood, the volume of water in the reservoir either decreases or remains the same, to give greater capacity during later peak inflows for larger attenuation. Clearly the earlier (c) can be operated the more efficient attenuation becomes, subject to at least the following two conditions : Since operation is based upon a forecast hydrograph, subject to more uncertainty the earlier in time it is given,

> it will be important that reservoir levels be not drawn down prior to the flood to such an extent that the reservoir cannot be refilled by flood inflows due to an error in forecast,
>
> and further it will be necessary to update the initial forecast of flood outflows at regular intervals during the flood event so that an efficient flood warning programme can be operated for river reaches downstream of the dam.

ERROR OF FORECAST IN FLOOD OUTFLOWS

Flood forecasts are all the more meaningful if hydrologists not only give an estimate of expected peak flows, durations, etc but also estimates of probable ranges in these values. The significance of this sort of information to flood warnings is fairly clear, and it is as important to reservoired catchments as it is to un-reservoired catchments. Estimates of probable ranges in flood flows downstream of a gate controlled spillway can also be of assistance in determining operational procedure for the remainder of a flood event, especially if such estimates can be updated at regular intervals.

A method which provides quick solution to these problems has been developed and is briefly summarised here. (A more complete treatment of the subject is to be published elsewhere)

BASIC ERROR RELATIONSHIP

If the relationship between flood outflow (Q) from the reservoir and its storage (S) can be treated as linear over discrete intervals of time (an adequate approximation for all reservoirs) then the error in forecast outflow (εr) due to errors in forecast inflow ($\delta_1, \delta_2 \ldots \delta_r$) is given by

$$\varepsilon r = Nr\ \delta r + N_{r-1}\ \delta_{r-1} + \ldots\ N_1\ \delta_1 + M\ \varepsilon_1 \qquad \ldots\ldots \quad (1)$$

where suffix r refers to (r - 1) units of discrete time intervals after an initial point for which the difference betwwen observed inflow and forecast inflow is δ_1

$Nr, N_{r-1} \ldots M$, are coefficients which depend upon the constant of proportionality in $S = K.Q$ and also therefore on gate control if gates exist.

FORECAST INPUT ERROR

As previously described by Jamieson et al (4), differences between observed and forecast inflows (δ_r, δ_{r-1} etc) can be analysed as a time series such that for example

$$\delta_r = \alpha\,\delta_{r-1} + \eta r \qquad \ldots\ldots \quad (2)$$

where α is 1st order autoregressive constant

ηr is a random variable

(Different autoregressive structures for (2) make only theoretical differences to ensuing relationships.)

FORECAST OUTPUT ERROR AND TOLERANCE

Combining equations (1) and (2) gives

$$\varepsilon_r = \underbrace{M\varepsilon_1 + \delta_1 \sum_{i=1}^{r} Ni\ \alpha^{i-1}}_{\text{ERROR COMPONENT WHOLLY DETERMINISTIC}} + \underbrace{\sum_{i=2}^{r} \sum_{j=2}^{i} \alpha^{i-j}\ Ni\ \eta j}_{\text{ERROR COMPONENT DUE TO RANDOM COMPONENTS OF INPUT ERROR}} \quad \ldots\ (3)$$

This simple relationship enables the determination of forecast output error εr, r-1 units of time after observed differences $\varepsilon 1$ and $\delta 1$ have occurred for α and η determined by analyses of previously forecast inflow hydrographs.

It is worth noting that α may also vary between events but that for a given event it is asumed constant.

It is useful to be able to estimate the possible range of εr and hence to produce estimates of expected values of flood discharge, and their probable *maxima* and *minima*. If α is assumed constant, then the relationship between the frequency distribution of εr, $g(\varepsilon r)$, and the frequency distribution of $\eta 1$, $f(\eta)$, is given by

$$g(\varepsilon r) = \frac{\partial \eta}{\partial \varepsilon r} \cdot f(\eta)$$

from which

$$g(\varepsilon r) = \frac{f(\eta)}{\left(\sum_{i=2}^{r} \sum_{j=2}^{i} \alpha^{i-j}\ \eta_i\right)} \quad \ldots\ (4)$$

Using equations (3) and (4) the relationship between the variance of εr, $V(\varepsilon r)$, and the variance of η, $V(\eta)$, can be shown to be

$$V(\varepsilon r) = \frac{\sum_{j=2}^{r} \left(\sum_{i=j}^{r} Ni\ \alpha^{i-j}\right)^2}{\left(\sum_{j=2}^{r} \sum_{i=j}^{r} Ni\ \alpha^{i-j}\right)} \quad V(\eta) \quad \ldots\ (5)$$

Use of equation (5) gives an estimate of uncertainty in forecast outflows.

EXAMPLE OF USE (Fig. 2).

Suppose during a flood an engineer wishes to estimate the probable error of forecast peak discharge of outflow from a reservoir five hours before it occurs. The reservoir has a value of K= 4500 secs. Previous analyses indicates $\alpha = 0.8$ and $V(\eta) = 40\ (m^3/s)^2$. Five hours before forecast peak discharge occurs, observed differences are $\varepsilon_1 = +2\ m^3/s$ and $\delta 1 = +5\ m^3/s$ respectively.

Estimated error in forecast peak discharge $Ep = +0.014\ \varepsilon_1 + 0.42\ \delta_1 \pm 2\ \sqrt{0.588\ V(\eta)}$

$$Ep = (+2.13 \pm 9.68)\ m^3/s$$

This answer means that the engineer could expect the forecast peak discharge (which may not of course remain the peak value) to be 2.03 m^3/s less than the actual value, but that due to uncertainty of forecast the value could range by $\pm$ 9.68 m^3/s about that new estimate. (See Fig.2)

IMPROVED FLOOD WARNINGS AND CONTROL

The foregoing example and the structure of equation (3) suggest a number of possible uses of the analytical technique :

1. Analyses can yield regular updating of the outflow hydrograph and probably *maximum* (or *minimum*) values. These analyses, especially if carried out using on-line computing facilities, may be of value to flood warning procedure particularly as forecasts obviously improve as peak outflow is approached.
2. Results of single-reservoir computations may be readily combined to evaluate multiple-reservoir uncertainty in flood control and hence to promote more information for flood warning within complex reservoir systems.
3. Since early forecasts of flooding are often important if timely flood warnings are to be made, it is important that more effort be given to improving the accuracy of synthesising hydrographs through catchment models and even to synthesising precipitation through 'Qauntifiable Precipitation Forecasting'. Greater uncertainty lies with earlier forecasts. The technique outlined provides a means by which tangible benefits of reducing uncertainty of forecast can be evaluated.
4. Cost-benefit analyses of flood control are frequently determined using methods involving full simulation of procedure. The basis of the technique described may be useful in this context since uncertainty is part of the analytical solution and may be used in determining expected values of costs and benefits. Such expected values can be adopted as statements of objectives to determine control rules of gated spillways using dynamic programming or similar procedure to optimise expected cost/benefit ratios, especially when control relies upon early flood forecasts.

FLOOD SPILLWAYS

OVERFALL SPILLWAY

The hydraulics of overfall spillways is well known. Assuming an ungated spillway to be formed according to the lower surface of a jet flowing over a sharp crested weir (which will vary with the head, approach velocity and the inclination of the upstream face of the overflow) the coefficient of discharge C_d in the standard equation :

$$Q = \frac{2}{3} C_d \sqrt{2g}\, b\, H^{3/2} \quad \ldots\ldots \quad (6)$$

can be taken as $C_d = 0.75$ for $H = H_d$ (design head corresponding to the design discharge). For lower heads (and discharges) the coefficient decreases until it reaches the value $C_d = 0.58$ which is valid for a broad crested weir.

For $H > H_d$ the coefficient of discharge increases until it reaches a value $C_d \simeq 0.805$ for $H = 1.65\ H_d$ (5) This increase in C_d is accompanied by pressures on the crest which are smaller than atmospheric. For $H < 1.33\ H_d$ these negative pressures do not exceed $0.5\ H_d$(6). $H = 1.65\ H_d$ is considered to be a safe limit for *negative pressure* spillways to avoid dangers of cavitation and vibration. A condition for their use is of course that there should be no access of air into the sub-atmospheric zone on the spillway to avoid vibrations of the jet (e.g. there must be no piers on the spillway).

For various values of $1 < \frac{H}{H_d} < 1.65$ we can interpolate linearly the coefficient of discharge between 0.75 and 0.81. Thus assuming that a new flood assessment on an existing reservoir results in 100% increase of the design discharge ($Q = 2Q_d$) the ratio of the heads above the spillway crest $\frac{H}{H_d}$ would be 1.585 for a constant coefficient of discharge. Taking the increase of C_d into account the corresponding value of $\frac{H}{H_d}$ is 1.520 (by iteration the ratio of the coefficient is 1.064 and $\frac{H}{H_d} = (\frac{2}{1.064})^{2/3} = 1.52$). The actual increase of water levels is thus $\frac{0.065}{1.585} = 4.1\%$ less than in the case when the variation of the discharge coefficient is neglected; when considering storage of flood water and flood routing this difference is quite substantial.

Similar considerations apply to the capacity of a side channel spillway for as long as the flow over the spillway crest remains unaffected by the flow in the channel (i.e. the control is exerted by the spillway crest and not by the channel).

SHAFT SPILLWAY

For a shaft spillway the opposite trend applies as the coefficient of discharge decreases with an increase of the head over the design head. This is due to the fact that the coefficient is a function of $\frac{H}{D}$ where D is the diameter of the spillway crest[6]. A well known limit used in design for the shaft spillway to operate as a free overfall (not submerged) is $\frac{H}{D} \leq 0.225$. Taking e.g.the initial design value $\frac{H_d}{D} = 0.15$ an increase of H to 1.4 H_d, resulting in $\frac{H}{D} = 0.210$, gives a 5.5% decrease of the coefficient of discharge; this decrease cannot be neglected and the effect of a substantial increase of the flood discharge above the original design discharge in case of a dam with a shaft spillway has thus to be investigated extremely carefully. (In the above discussion other important considerations e.g. the tunnel capacity etc have not been taken into account.)

GATED SPILLWAY

The hydraulics of gated spillways, whilst not complicated, brings a new variable - the gate opening- into the discharge equation. The coefficient of discharge is a function of this opening and the ratio of the head above the crest and the gate opening.

The profound effect of gated spillways and of their operation on flood routing and control have been referred to in the first part of this paper.

One not very often appreciated hydraulic aspect is that for partially opened gates there is a zone of negative pressures downstream of the gate on the spillway face. This can be troublesome and its contribution to increasing the discharge coefficient is quite negligible. A substantial improvement of the pressure distribution can be obtained by a small shift of the gate seat in the downstream direction (i.e. downstream of the crest). The shift should be small as otherwise the required increase of the gate height is marked. On the basis of laboratory experiments it has been suggested that optimum distance for the gate sill downstream of the crest avoiding negative pressures on the spillway at flows through partially opened gates is about 0.2 H_d[7]. This point could be borne in mind not only in design but also in any gated spillway reconstruction following new flood assessment.

STILLING BASINS

The effect of increasing the discharge through an existing stilling basin designed originally for a smaller discharge can be assessed from the standard stilling basin design using the following equations:

$$E = y_1 + \frac{q^2}{2gy_1^2\phi^2} \quad \ldots \quad (7)$$

$$y_2 = \frac{y_1}{2}\left(-1 + \sqrt{1 + 8\frac{q^2}{gy_1^3}}\right) \quad \ldots \quad (8)$$

$$y^+ = \sigma y_2 - y_o \quad \ldots \quad (9)$$

where E is the height of the reservoir water level above the stilling basin bed, $q = \frac{Q}{b}$ (m^2/s), y_1 and y_2 are the hydraulic jump conjugate depths, $\phi = \frac{1}{\sqrt{1+\xi}}$ where ξ is the head loss coefficient between the spillway crest and the stilling basin entry, y_o is the depth in the downstream channel corresponding to the discharge Q, y^+ is the stilling basin depth (below the downstream bed level) and σ is a safety coefficient (usually taken as about 1.1 - 1.2). The stilling basin length can be taken as

$$L = K\,(y_2 - y_1)$$

where $K \simeq 4$ for $\frac{y_2}{y_1} = 20$ and $K \simeq 5.5$ for $\frac{y_2}{y_1} = 3$ (8).

An increase in q results in a higher head above the spillway crest and therefore an increase in E. Neglecting this increase and assuming ϕ to remain constant, we would find that an increase of q results in an increase in y_1 (by about the same percentage) and y_2 (by a much smaller percentage). A trial computation by the authors showed that a 100% increase in q resulted in a 100% increase in y_1 and a 40% increase in y_2 under these conditions for a wide range of discharges.

As the stilling basin safety factor $\sigma = \frac{y^+ + y_o}{y_2}$ and as y^+ is clearly not affected by a change in discharge the safety coefficient will decrease for an increase in discharge unless the downstream channel depth y_o increases faster than the conjugate depth y_2. This is usually not the case and therefore a decrease in the stilling basin depth safety factor must be expected.

The effect of an increase in discharge on the required stilling basin length is even more pronounced as $(y_2 - y_1)$ will increase but $\frac{y_2}{y_1}$ decrease and therefore K increase. The stilling basin length is therefore even less likely to be adequate for an increased discharge than the stilling basin depth (assuming that the basin was designed economically for the original design flood). Taking into account the increase in E and probable decrease in ϕ (resulting from increase in Q) would slightly improve the situation and lessen the decrease in stilling basin safety.

SAFETY CONSIDERATIONS

The above considerations lead to a reassessment of the need of an equal factor of safety for the stilling basin and spillway. Basically the purpose of a stilling basin is to localise the resulting scour in a position where it cannot affect the safety of the dam whilst at the same time reducing it to 'acceptable' values. It is practically impossible or certainly uneconomical to design a stilling basin for no scour to occur under any condition. A well designed stilling basin should thus be able to pass for a short period of time a discharge bigger than the original design discharge without too serious damage. It is, therefore, reasonable to use a more stringent flood criterion for the design and reassessment of spillways than for the stilling basins and accept in some cases the need for occasional expenditure on maintenance downstream of the structure always assuming that the stability of the dam itself is not endangered.

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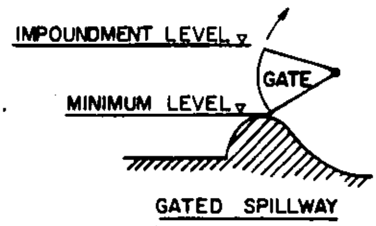

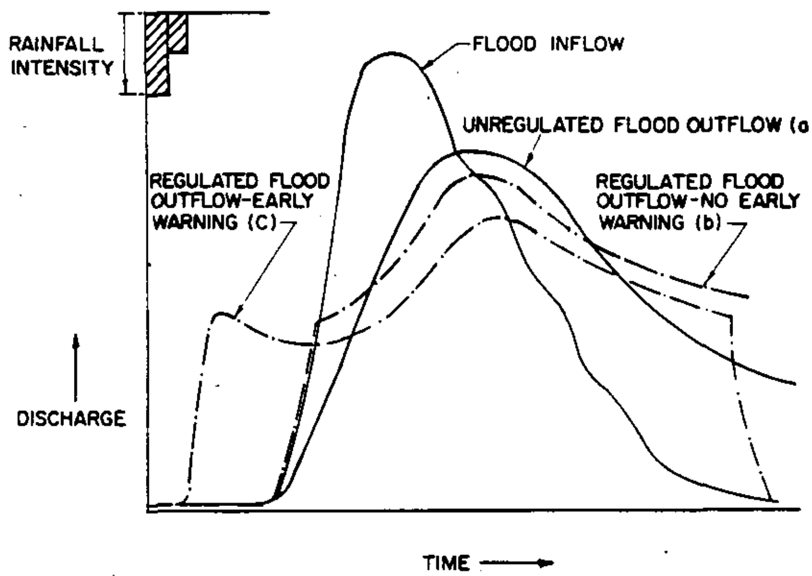

FIG. 1 REGULATED AND UNREGULATED OUTFLOWS

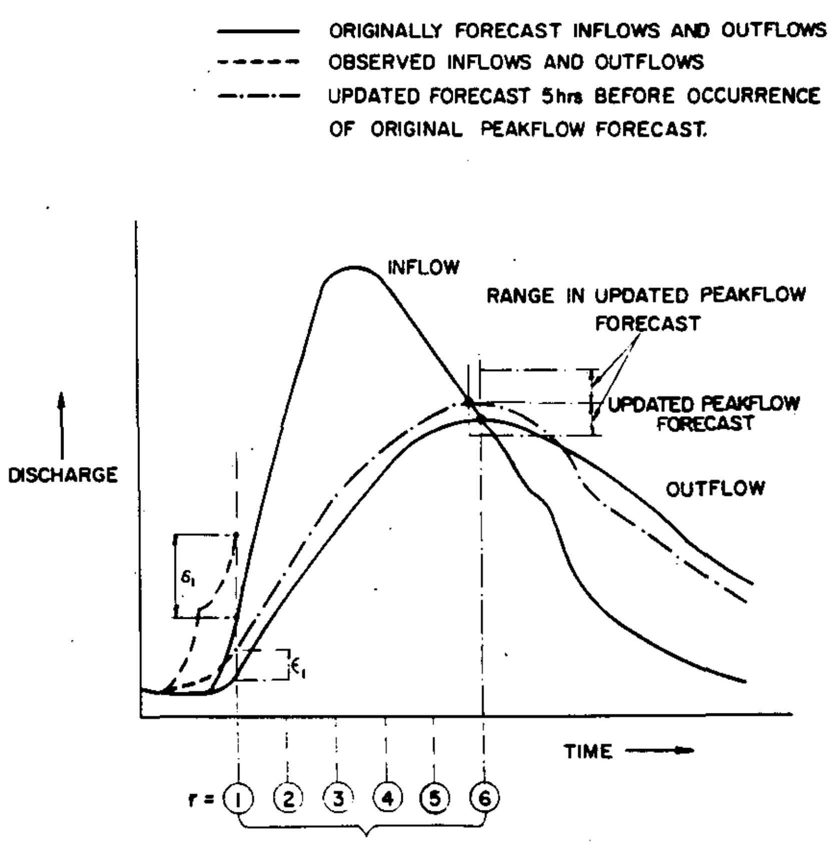

FIG. 2 ILLUSTRATION OF EXAMPLE OF FORECAST

SOME ASPECTS OF DESIGN FLOOD ESTIMATION

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SYNOPSIS

The Flood Studies Report (1) describes a technique of flood hydrograph synthesis for spillway design. The annual exceedance probability (or return period) of the peak may be specified at, say, 10^{-5}(100,000 years) or unspecified but near zero (estimated maximum). Since the Report was completed, the Institute of Hydrology has made a number of studies on behalf of consulting engineers and Water Authorities. This paper draws on the experience gained to give further illustration of the techniques. The problems of designing against a specified return period for outflow peak are discussed; the use of local data to modify predictions from regression equations is also demonstrated. A further regression equation is presented which enables a preliminary rough estimate of the maximum hydrograph peak to be made directly in terms of catchment characteristics derived from maps.

INTRODUCTION

Flood estimates are required for different engineering purposes in a wide variety of forms. The aim of the Flood Studies Report (1) has been to provide a portfolio of methods which should enable an estimate to be made of the flood corresponding to a given specification and in different circumstances. The design flood may be specified as having a given frequency or return period and may consider shape as well as peak, and estimates may be required for a site at which there are long, short or no records available.

The context of large dam design limits the range of topics to be considered. The flood requirement is for the spillway design flood, and in some cases the diversion flood, and except for the feasibility stage of an investigation one should be able to assume that at least a short period of records of rainfall and runoff will be available at the site.

If one takes the criteria put forward in the ICE discussion paper 'Reservoir Flood Standards (2), then the spillway design requirement is a flood between 150 year return period and the estimated maximum. Because this assessment should take account of reservoir attenuation, not only the peak inflow but also the shape of the hydrograph is required for routing through the reservoir. Thus we are considering only the flood of large return period, and the peak and shape are both required. Therefore the unit hydrograph approach to the problem of estimation is indicated, and because records at the site should be available there is need for considerable judgement by the design engineer to incorporate site information into the estimate. This information may include records of the time to peak of unit hydrographs derived from actual events, or simply the 'lag' of the catchment deduced from rainfall and river level records. A second source of local information which may prove more difficult to derive and interpret but which may well have more influence on the result is evidence on the runoff to be expected from a given rainfall. An empirical equation is given in the Report which is based largely on a soil map compiled for the study. This may be overriden by local records of rainfall, antecedent conditions and runoff which would lead to an assessment of the normal response of the catchment at about field capacity, which is termed the standard percentage runoff. An example of this is presented below.

It is stressed in a number of places in the Flood Studies Report and in the ICE draft manual (2)that the responsibility for the spillway flood estimate must remain with the engineer. Certain aspects of flood estimation can be standardised, and indeed there is advantage in economic comparisons if standard techniques have been used to estimate the mean annual flood and the growth factor for certain return periods. With spillway design floods where minimum risk can be accepted there is considerably more scope for engineering judgement as the estimated maximum flood implies a combination of several highly improbable events. To obtain a balance between absolute safety and economic realism some form of compromise will be necessary. To take an extreme example, it would be absurd to assume that a world maximum tropical storm would fall on frozen ground at the same time as an extreme snowmelt event. Some guidance is given in the Report on the antecedent conditions which might reasonably be assumed to occur at the beginning of the design maximum storm. It should also be recognised that such conditions imply a more conservative design in the South East than in upland Britain; to express this another way, it is likely that the estimated maximum flood (EMF) would be approached more frequently in wetter than in drier parts of the country. Some decisions, like the inclusion of a rare snowmelt rate in the design event, may depend on individual investigation, while the incorporation of local knowledge in the design procedures cannot be reduced to rule of thumb.

Thorough assessment is thus required for a final design, but in view of the large number of existing reservoirs it has been suggested that there is a need for a rapid screening method to isolate those cases where detailed examination is necessary: a procedure for deriving Q_{150} (flood with 150 yr return period) and a ratio EMF/Q_{150}, (EMF = estimated maximum flood), was proposed by the ICE Working Party. Although the need for a *separate* method may have been overstated by the Working Party - the unit hydrograph procedure can itself be rapidly applied with approximations in place of the careful examination required later - it is here suggested that a ratio method gives particularly poor estimates of EMF. A number of individual estimates of EMF have been made using the full procedure, and these have been related directly to catchment characteristics with some success. Essentially, a flood estimate is derived from an estimate of the relevant rainfall amount, the fraction of this which becomes rapid runoff, and the distribution in time of this runoff. Because the estimated rainfall for 24 hour and two hour durations is much more conservative than the M52D, (a two day rainfall with a five year return period) and still more so than an index of net rainfall like RSMD (a net one day rainfall with a five year return period), the variation of EMF over the country is less than $\bar{Q}$, the mean annual flood. In fact the two hour duration maximum is greater in S.E. England than in the west of the country, unlike 24 hour and longer duration rainfall. Thus for certain intermediate durations the maximum may be relatively constant, and the flood magnitude may depend on the fraction of response runoff (related to soil type), and the shape of the hydrograph. This constancy explains the high variation in ratio EMF/$\bar{Q}$ or EMF/Q_{150} but may be exploited in a relatively stable relationship with catchment characteristics. This may provide an alternative means of rapid screening if this should prove essential. The estimate does not take into account the attenuating effect of an existing reservoir.

EXAMPLES OF ESTIMATED MAXIMUM FLOOD PROCEDURES

Two examples are used to illustrate points of detail which frequently arise in estimated maximum flood (EMF) computation. The discussion is widened beyond the specific examples to cover comments and criticisms received from engineers who have tried the methods at other sites.

Following an approach from the Derwent Valley Water Board (now part of the Severn-Trent Water Authority), the Institute was asked to produce a flood frequency curve for Ladybower Reservoir and, in particular, to examine the likely effect of the July 1973 rainstorm had it occurred on a wet catchment with reservoirs full rather than on a dry catchment with reservoirs drawn down.

Figure 1 shows the three reservoirs in the Upper Derwent Valley with the major tributaries and gauging points. In assessing the input to Ladybower reservoir it is necessary to consider the attenuating effects of the other two storages and, therefore, to synthesise separate inflow hydrographs for the three catchments.

USE OF LOCAL DATA

The data available were such as to allow unit hydrograph derivation for only one of the three - the catchment to Ladybower reservoir below Derwent Dam - and, even here, the runoff from the land catchment had to be calculated from eight variables. Thus, natural runoff from Ladybower catchment below Derwent Dam :

= storage change in reservoir	+ outflow from reservoir (F1)
+ Ashop diversion flows (F3)	+ abstractions from reservoir (F7)
- Noe diversion flows (F4)	- Limestone pumping (F8)
- outflow from Derwent reservoir	- rain falling directly on reservoir surface

The first storage change is the dominant item and also the least accurately measured. Quite large flows are implied by relatively small storage changes which are highly sensitive to the accuracy of level measurement. However, with a certain amount of smoothing and joint interpretation of the several sets of flow data and rainfall it was possible to produce reasonable hydrographs.

It has been suggested that many reservoired catchments are 'abnormal' because input to the reservoir may be from a number of short streams rather than a single channel. It is questioned, therefore, whether a synthetic unit hydrograph based on the analysis of records from 'normal' catchments can be applied. Figure 1 shows that the Ladybower below Derwent catchment is dominated by the Ashop tributary but there are some minor streams on the north east side. Figure 2 shows the average of six *derived* unit hydrographs together with the synthetic triangular unit hydrograph which would have been predicted, from the Flood Studies Report in the absence of any local data. (The Ashop was used for channel length and slope estimates but the whole of the Ladybower below Derwent catchment used for area and RSMD estimates.) The agreement is quite good and leads to the suggestion that where there is a single tributary which drains more than half of the total catchment, the several subcatchments contributing directly to the reservoir can safely be lumped together and considered as a single catchment (see I.7.4.2 in the Report [1]).

Despite the good agreement of time to peak and peak flow between the average observed and the predicted synthetic unit hydrographs the former was obviously used for design purposes on the Ladybower catchment. For the Howden and Derwent reservoir catchments, however, there were no inflow hydrograph data and the predicted triangular unit hydrographs were used.

The comparison of observed and predicted precentage runoff was also good; when compared to the observed events, the Report's prediction equation gave values only 4% higher on average. Greater differences occurred in individual events and are always to be expected; the errors of measurement of both rainfall and runoff are such that at least five events must be analysed and an average value of standard percentage runoff extracted before any sensible comparison with a prediction can be made. As a single soil type (class 5) applies to the whole study area, predicted values of percentage runoff on the two ungauged areas can be used with more confidence in the light of the good agreement.

In applying the unit hydrograph technique to the Derwent system, the recommended procedure described in the Flood Studies Report, Section I.6.8.2/3, was followed as far as possible. The presence of three reservoirs in series necessitated a departure from the usual method of choosing design storm duration. Any one catchment considered alone would require a different design duration but it seemed logical to consider a single design storm as applying to the *total* catchment area. To test sensitivity to the choice of duration, two values were used, namely 8½ and 17½ hours; differences were small as can be seen in Figure 3.

When aiming to find the reservoir outflow of specified exceedance probability or return period it is necessary to consider the frequency distribution of reservoir water level. Figure 3 illustrates the application of several features of the Flood Studies Report. Twenty-nine years of annual maxima of Ladybower outflow were ranked and plotted (Section I.1.3.2 of the Report); the growth curve for Region 4 was applied to the mean annual flood (I.2.6.2). Also, the procedure of I.6.8.2 was applied for return periods of 6, 60 and 1000 years with various starting levels for the three reservoirs (expressed as a percentage in storage). The unit hydrograph technique gave peaks of the right order with an assumed initial state of about 95% full. Such a combination of techniques increases the confidence with which the frequency curve is extrapolated.

DESIGN DURATION AND ANTECEDENT CONDITIONS FOR THE EMF

In any reservoir study there is always an interest in the maximum flood. Whether or not it is sensible to design against this flood it may be prudent to compare the magnitude of finite risk floods with that of the infinitesimal risk flood or estimated maximum flood. Such a flood is synthesised by allowing every facet of the input data and the transforming model to combine in the worst possible way whilst remaining physically conceivable in the light of current meteorological and hydrological knowledge. This means that reservoirs are assumed to be full at the start of the event, that severe snowmelt will coincide with rainfall of maximum intensity, and that response runoff will be particularly rapid. The method of determining the antecedent condition (I.6.8.3 (e) p 471) has been criticised as arbitrary. Why, it is asked, should a preceding period of twice the design storm duration be used to provide initial wetting? The difficulty arises from the decision to assume a symmetrical nested profile for the design rainstorm. By 'nested' is meant that the one hour estimated maximum rainfall occurs in the central one hour of the storm, the three hour fall occurs in the central three hours and so on. With such an assumption, the total rainfall of the 'storm' clearly increases with duration but with no compensating reduction of maximum intensity. Consequently, for a given percentage runoff prediction, the highest peak discharge would be obtained from a storm of infinite duration.

Figure 4 illustrates the way in which predicted percentage runoff increases with design storm duration and how lower antecedent catchment wetness index (CWI) partially compensates for the effect of increasing rainfall. The Report recommendation is that the design storm duration (D) is determined as in the case of a flood with finite return period but that the extra rainfall before the event contributes its effect by increasing the CWI. It was thought unnecessary to go beyond a period of 2D before the event for this purpose as the effect of extra rainfall on the CWI is usually small.

This can be illustrated by another Institute study which was carried out for Messrs Rofe, Kennard and Lapworth and concerned the site of the proposed Ardingly Reservoir in Sussex. Figure 5 shows estimated maximum rainfalls in various durations for this site. Table 1, derived from Figure 5, shows rainfall and CWI for various duration assumptions.

Assumed storm duration D (hours)	Rain in D hours (mm)	(a) Rain in preceding nD period and (b) CWI at end of it							
		n = 1		2		3		5	
		a	b	a	b	a	b	a	b
2	150	25	149	40	163	50	171	65	<u>181</u>
5	190	30	153	50	168	60	173	72	<u>175</u>
10	230	35	155	50	<u>162</u>	55	161	60	154
20	270	32	<u>149</u>	40	147	45	144	48	136

(Underlined value is highest in row.)

Table 1 : Rainfall and CW1 for various duration assumptions (see Fig.5)

It can be seen that a 2D wetting period produces the highest CWI for D = 10 hours. For smaller D a longer antecedent period is worse and vice versa. It was thought that 2D was a suitable choice in most situations but it was an arbitrary choice and one made in the cause of simplicity.

A RAPID METHOD OF PREDICTING THE ESTIMATED MAXIMUM FLOOD

Engineers who have grown accustomed to the simplicity of the 1933 envelope curve as a means of estimating the spillway design flood may feel somewhat overawed by the apparent complexity of Section I.6.8.3 of the Flood Studies Report. However, the estimated maximum flood (EMF) is not a simple function of area and so it is unlikely that any envelope curve can produce more than a very approximate estimate, but there are occasions when some rapid estimate of the EMF is required and shortage of time precludes a rigorous flood study.

It has been suggested that the EMF could be estimated from some index of a catchment's flood potential such as the mean annual flood ($\bar{Q}$) or the flood with a 150 year return period. The problem with such approaches is one of determining the correct regional multiplying factors for the EMF. In order to examine various rapid estimation methods, the EMF was derived from 80 catchments where an observed unit hydrograph and mean annual flood were available. The catchments are listed in Table I.6.18 of the Report and their location is shown in Figure 6.

A computer program was developed to calculate the EMF, requiring input of estimated maximum rainfall for two and 24 hours duration for the catchment (from maps II.4.1 and II.4.2 in the Report) either an observed unit hydrograph or predicted Tp from catchment characteristics, and either an observed or predicted standard percentage runoff. The program, an extension of that given in I.6.8.6 on p.482 of the Report, derives the design rainfall profile, determines the percentage of this that will produce rapid response runoff, and combines the rainfall and unit hydrograph. A snowmelt allowance is introduced where appropriate and baseflow added to yield the total runoff hydrograph.

The EMF was firstly computed using the observed unit hydrograph and standard percentage runoff, which will be referred to as EMF_{obs}. It is felt that EMF_{obs} is the best estimate of the likely design flood peak for the "probable maximum" case short of a rigorous hydrological study of each individual site. The mean annual flood ($\bar{Q}$) was converted using the regional growth curves of Figure I.2.14

of the Report to the peak with a 150 year return period, referred to as Q_{150}. The scatter of this ratio is large, indicating that a simple multiple of Q_{150} is a poor estimate of EMF. The ratio $EMF_{obs}/Q150$ was mapped but no obvious regional distribution was apparent, although the higher ratios tended to occur in the lower annual average rainfall areas of East Anglia and the south coast.

It was thought that any regional pattern that did exist could possibly be explained by regressing the ratio on catchment characteristics. Only two characteristics appeared to be significant, SOIL (an index of the winter rain acceptance potential of the catchment) and RSMD (a net one day rainfall with a five year return period). These two variables predicted the ratio $EMF_{obs}/Q150$ but with very large factorial standard error of estimate (f.s.e.e.) of +41% - 29%. A plot of the residuals from the regression did show some evidence of regional grouping but a preliminary allowance for this reduced the f.s.e.e. to only just below + 40%,−28% and much more data would be needed to divide the country intc reliable regions in the way suggested by the Flood Studies Report (Section 1.4.3.8).

DIRECT PREDICTION OF EMF FROM CATCHMENT CHARACTERISTICS

As an alternative to either the rapid application of the unit hydrograph method or estimation of EMF from either $\overline{Q}$ or Q150, a direct relationship between EMF and catchment characteristics was sought. This was done by multiple regression of EMF on the best 1 to 5 characteristics of a chosen set, 'best' implying the maximisation of R^2, the multiple correlation coefficient. The results of the regressions are summarised in Table 2

The resulting f.s.e.e. show that estimating EMF directly from catchment characteristics is better than using Q_{150}. There is a significant gain in the fitting by increasing the number of variables up to four but the f.s.e.e. only decreases from +27% to +26% by increasing the number from four to five. It is suggested therefore, that the four variable equation should be used. This equation has one less variable than the Floods Working Party suggested equation for EMF peak and should, from comparison with the EMF_{obs}/Q_{150} ratio regressions, yield better results in most cases. A study of the residuals from the regression suggested that there was no advantage in regionalisation.

	Independent variable	Coeff b	s.e.b.	R^2	R	s.e.e.	f.s.e.e.
1	Const	11.2	0.129	0.717	0.847	0.223	+67%, - 40%
	AREA	0.821	0.059				
2	Const	0.324	0.151	0.900	0.949	0.132	+36%, - 26%
	AREA	0.795	0.035				
	RSMD	1.036	0.088				
3	Const	0.917	0.153	.929	.963	.112	+29%, - 23%
	AREA	0.818	0.030				
	RSMD	0.885	0.080				
	SOIL	0.618	0.113				
4	Const	0.668	0.152	.937	.967	.105	+27%, - 22%,
	AREA	0.825	0.029				
	RSMD	0.941	0.078				
	SOIL	0.573	0.108				
	URBT	1.255	0.409				
5	Const	0.835	0.152	0.941	0.970	0.101	+26%, - 21%
	AREA	0.878	0.035				
	RSMD	0.724	0.116				
	SOIL	0.533	0.106				
	URBT	1.308	0.397				
	S1085	0.162	0.066				

Variables are as defined in Notation Table at front of Vol. I of Flood Studies Report but note that URBT = (1+URBAN)

TABLE 2 Regression of log (EMF_{obs}) on log (Catchment Characteristics) *

However, both SOIL and RSMD are themselves strongly regional being, higher in wet, upland areas. A map of RSMD has been prepared by the Meteorological Office as a result of discussion at the ICE Conference. The RSMD value derived from the map must be adjusted by the appropriate areal reduction factor (ARF) and a rapid method of achieving this for small upland catchments is given below:

$$ARF = \exp(-A^{1/4}/50)$$

For larger catchments it may be necessary to take into account the point that the rainfall component but not the soil moisture component of RSMD is subject to the areal reduction factor.

It is suggested therefore that the EMF may be rapidly predicted from catchment characteristics with reasonable precision by the four variable equation below:

$$EMF = 0.67\ AREA^{0.82}\ RSMD^{0.94}\ SOIL^{0.57}\ (1+URBAN)^{1.25} \quad *$$

This estimate of the EMF appears to be preferrable to one based on a multiplying factor of $\overline{Q}$ or Q_{150} although it lacks the precision that can be obtained by rapid application of the unit hydrograph method even where predicted Tp and standard percentage runoff are used. However, the regression equation approach has some merit in its simplicity.

CONCLUSIONS

The Flood Studies Report method of computing estimated maximum flood (EMF) has been applied in a number of cases. Two examples have been used to illustrate certain features of the method. The EMF may not necessarily be the chosen spillway design flood but it provides a realistic upper bound below which to constrain the flood frequency curve. Rapid estimates of EMF based on a multiple of the mean annual flood or Q_{150} are generally poor. A much better estimate can be obtained from a direct regression on catchment characteristics,but it is suggested that a computerised version of the complete method, less the individual assessment required for a final design, may be the optimum solution for initial screening purposes.

ACKNOWLEDGEMENTS

The authors would like to thank the Severn-Trent Water Authority and Messrs Rofe, Kennard and Lapworth for permission to quote from the Ladybower and Ardingly reports respectively. They are also grateful to their colleague, M A Beran, for comments on the draft of this paper and for his advice on statistical interpretation, and to Dr J.S G. McCulloch for permission to publish this paper.

REFERENCES

1 *Nat Env Res Council* (1975) — *Flood Studies Report* 5 *Vols* NERC London

2 Instn of Civil Engrs (1975) — *Reservoir Flood Standards* Discussion paper ICE London

* Ed. Note — Symbol denotes data and/or equations amended or added to the preprint papers in the light of further research work by the Authors.

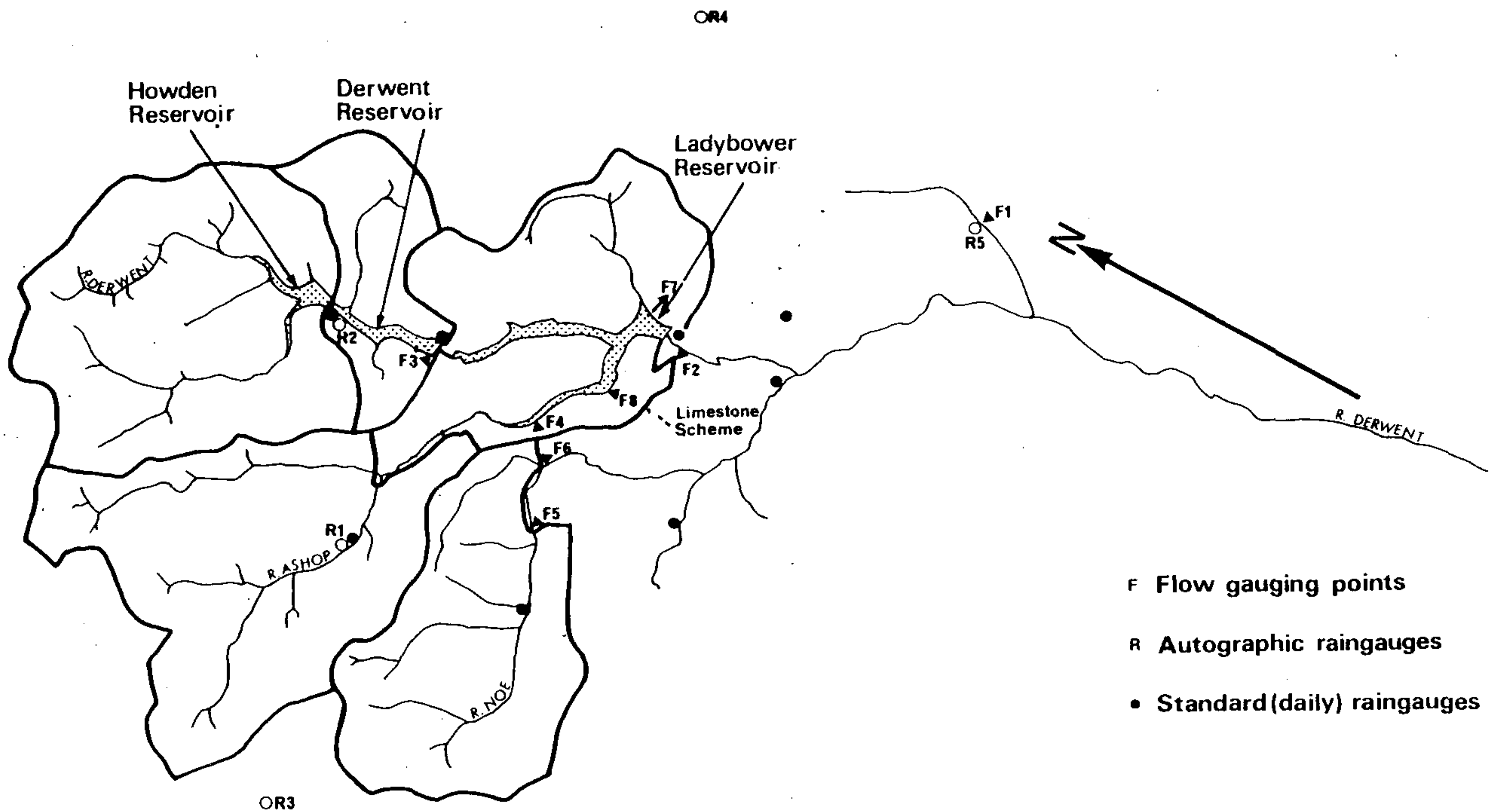


FIGURE 1. THE UPPER DERWENT

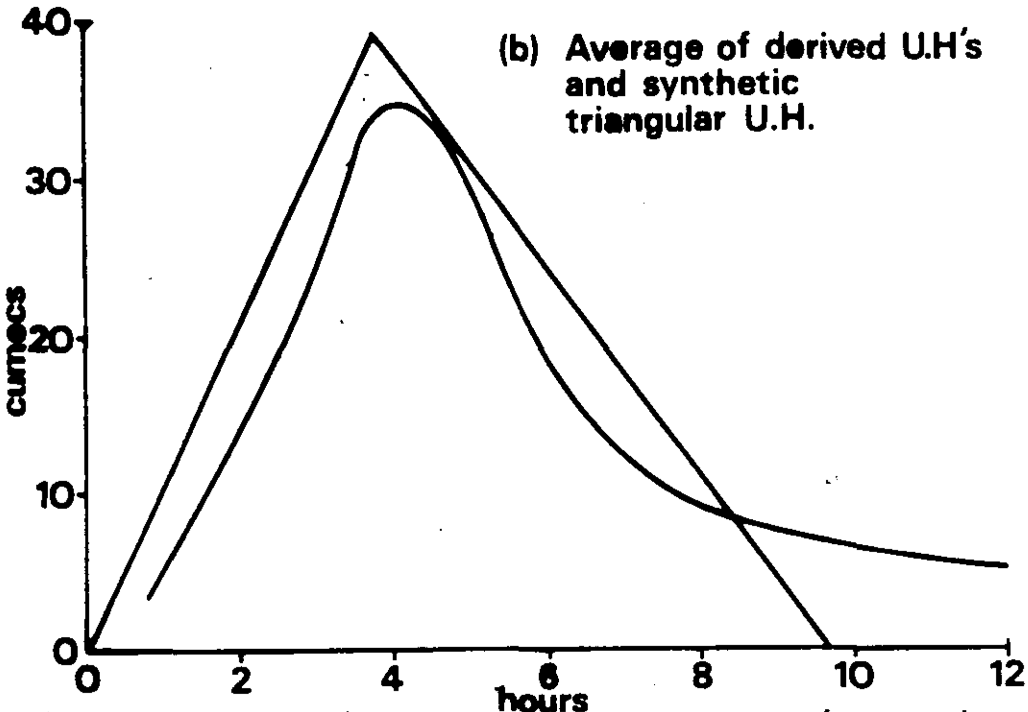


FIGURE 2. UNIT HYDROGRAPHS FOR 'ASHOP' CATCHMENTS(69.5 SQ. KM.)

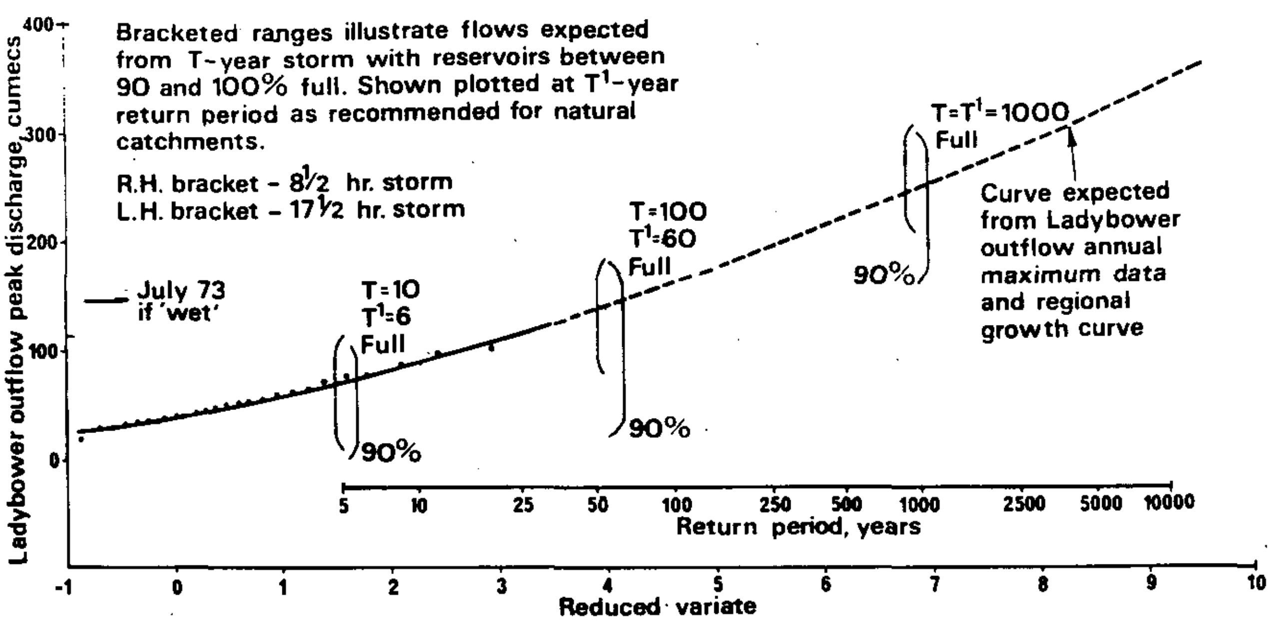


FIGURE 3. ESTIMATED FREQUENCY DISTRIBUTION OF LADYBOWER OUTFLOW

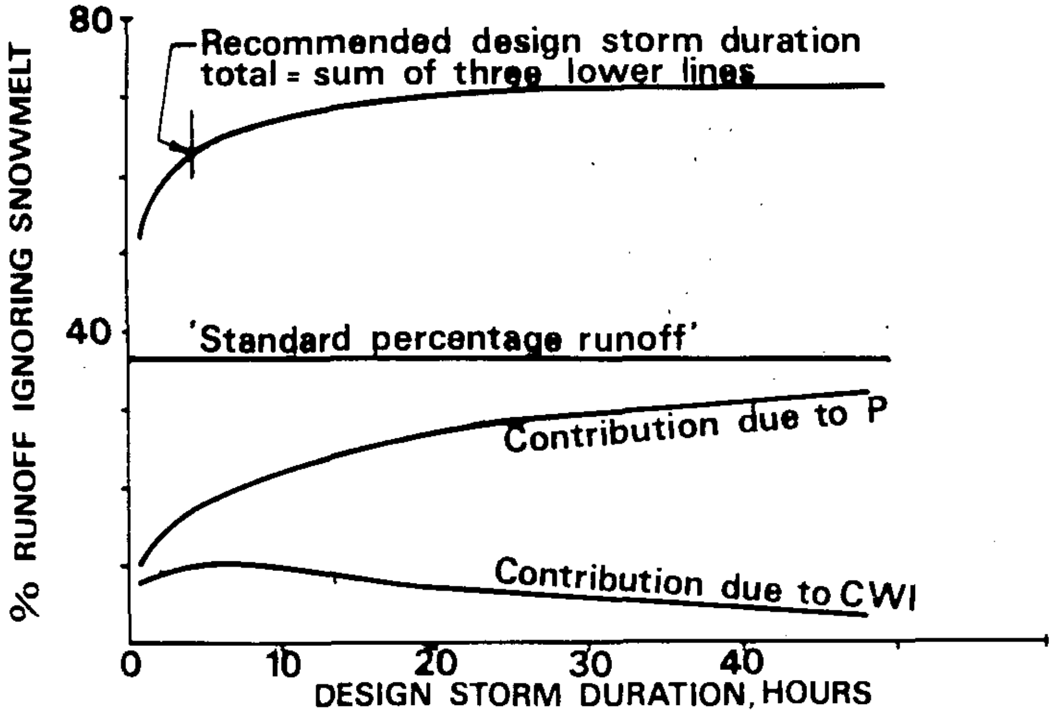


FIGURE 4. VARIATION OF PERCENTAGE RUNOFF WITH DESIGN STORM DURATION

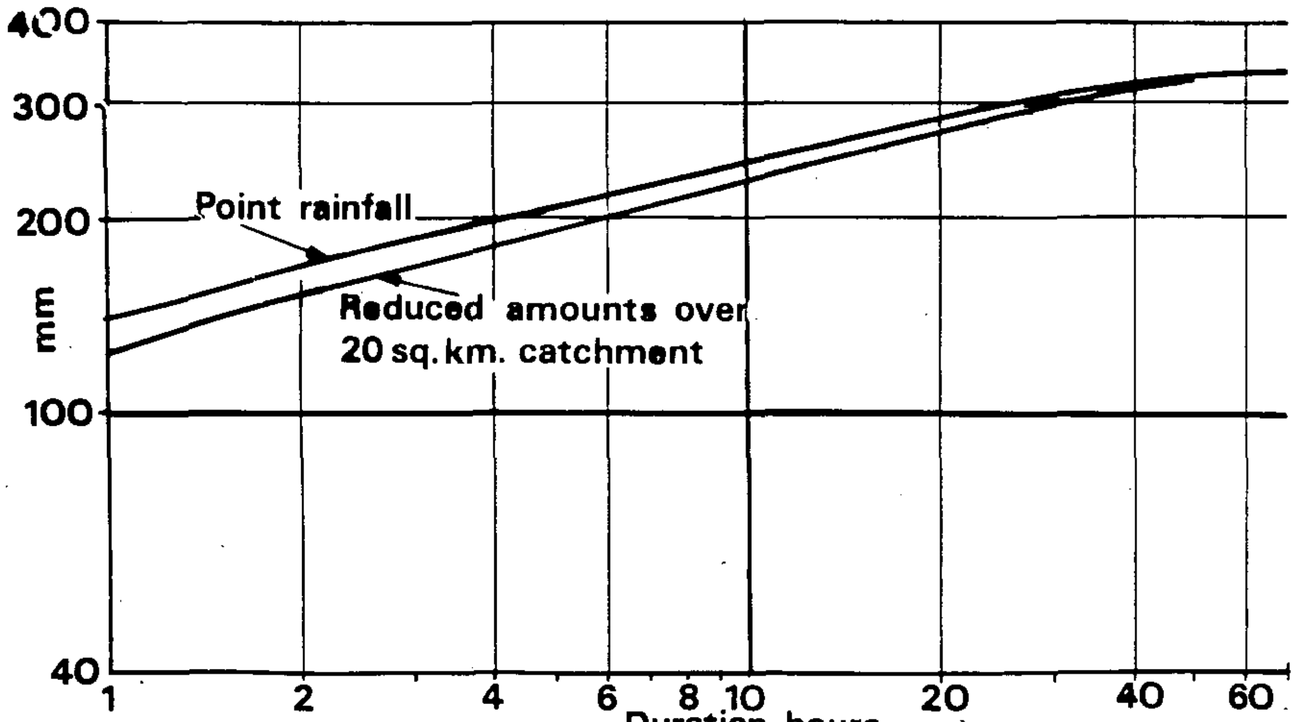


FIGURE 5. ESTIMATED MAXIMUM RAINFALLS IN VARIOUS DURATIONS SHELL BROOK (41/24)

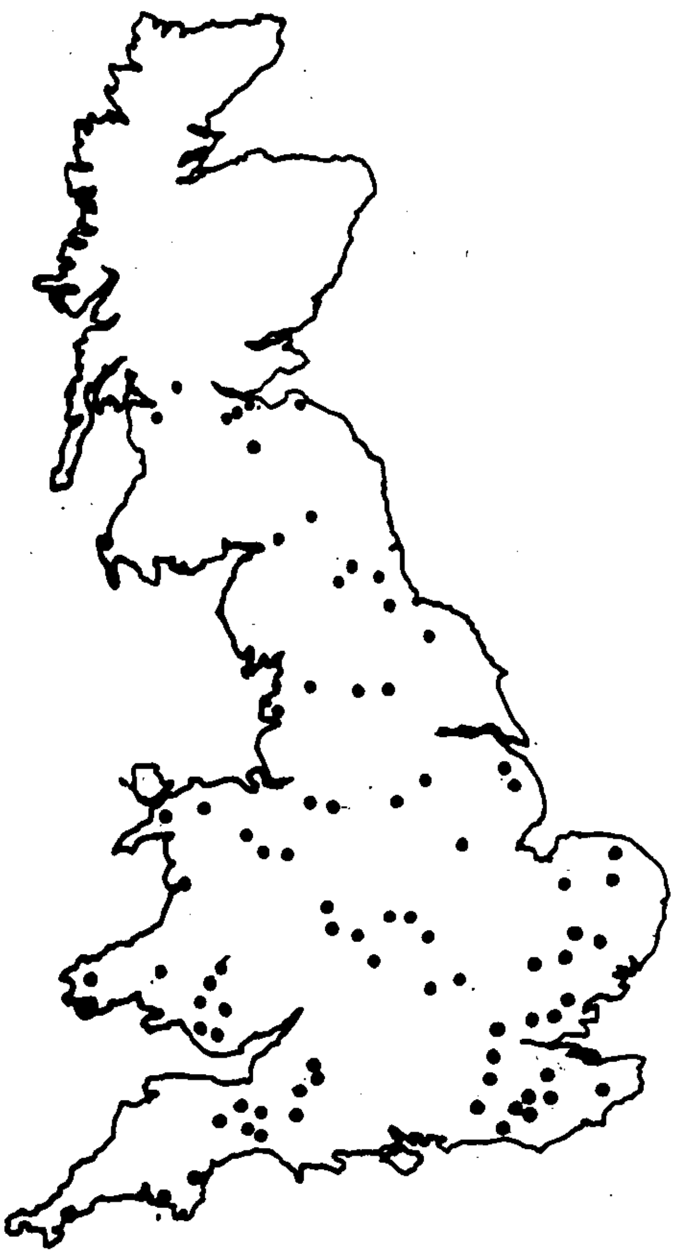


FIGURE 6. LOCATION OF CATCHMENTS USED IN E.M.F. ANALYSIS

REPORTER : F M LAW :

Early on in our discussion Dr Wright suggested that it was not possible to define a Probable Maximum Flood (PMF) because, in his words, 'our natural world knows no finality'. I felt I could not let that pass, because people do not accept this when it comes to human ageing. We accept that a certain percentage of people exceed 100 years of age, but when figures of an age of 135 years are quoted, as for a gentleman in Iran recently, serious doubts are raised. Certainly nobody has suggested that even with all the probability statistics on all the world's inhabitants anybody anywhere is going to reach age 1000. I do feel, therefore, that there is an upper limit to age and similarly that there is an upper limit to a flood as well.

Now, of course, each individual will get a different PMF value, but I think this is no worse than the fact that if you take different Panel Engineers to the same dam sites they will come up with proposals for different dam types, never mind different dam profiles; so this is only part of the engineering judgement that we are constantly exercising. People lay different weight on different factors.

I was very interested at the honesty with which the last paper was written by Dr Sutcliffe and his colleagues, because they stressed the arbitrary nature of some of their decisions in order to get an answer inside the project period. They themselves mentioned the value of engineering judgement. For that we are grateful.

Now if we are to proceed to any sort of Table like Appendix A of the Discussion Paper, for which very cogent arguments have been prepared, I do think that we have got to be more careful about the risk that we are talking about. Very often we are only talking about the probability of flood inflow to a reservoir, not the flood outflow, and this I think links in very well with Mr Johnson's comment about taking overall probability. At many dam sites the total freeboard between spillway crest level and the top of the wave-wall is half for the flood water and half for waves. We spent a great deal of time calculating the floods and not as much on calculating the waves. I do think that if a Table like Appendix A is to be finalised we must come to a better integration of the probabilities of floods and the probabilities of waves in order to get to the sort of total design risk which I can see some people would like us to adopt.

If I might move on to Mr Featherstone's contribution, I think he highlighted a very interesting point when he mentioned that campers could be on a farmer's field for quite a period of time without planning permission. Do they or do they not constitute a community threatened by a dam? To my way of thinking they do not; they are, as it were, a somewhat random selection of people, just as random as the people in a railway carriage on top of a bridge over a river at the time that it is swept away. I think that society accepts the incredible toll on our roads merely because the risks are thought to be somewhat random. Each time you set out in your car you are running a random risk, but once we come to a dam threatening a community we have a stable situation. People can see the dam, they can see the community. They can, if it fails, challenge those involved, and whether there is legislation that is adequate or not the Government has the power to set up special tribunals and take people to task for what they have done. I thought that his case for actually getting Water Authorities to bring together the work of different Panel Engineers in adjacent valleys was a very valuable suggestion, and I think it should be more widely publicised.

I am afraid I must disagree with Mr Mansell-Moullin, because I feel that although there are uncertainties about making estimations of flood magnitudes there are two quite separate points here. I accept that it is impossible to define the absolute truth about flood magnitudes. It cannot be done, but I suggest we are not actually after absolute truth. We are after a reasonable calculated estimate - and Mr Griffiths stressed this - an estimate that is reasonable within the profession and can be justified to the interested lay public. If, therefore, we do an honest calculation then I think we have done as much as can be expected of us professionally.

I am grateful to Mr Hamilton for his extra example of wave damage at a dam, and the seriousness of it just adds to my feeling that we could do with an entire Session on this one topic at some future BNCOLD meeting.

I have also met up with errors in Reservoir Record Books; I have met the one where there is no record of a serious reservoir rise if it is from, say, a drawdown of 8 m to a drawdown 2 m. It is all logged, as 'under spillway level', so again knowledge of a major event has been lost. I think that these errors will in future be stopped by the Supervising Engineers, and this will be a great help.

On the detailed point from Mr Poskitt about Ireland, I would entirely agree on the basis of some work we have done recently in the Wicklow Mountains; the multiplying factor of 2.05 is only really suitable for Lowland Ireland. When one gets into the mountainous areas I would suggest that if one takes the South of Scotland curve one would find that a much more reasonable analogue.

I would like to stress, though, that the Discussion Paper intentionally concentrated on philosophy and not on exact techniques or numbers. Therefore the only help that I can give to many of the contributors of this morning is to say to look again at our Appendix E, where we suggest items for research. High on the list were things like Areal Reduction Factor during the Probable Maximum Precipitation events. That is not really for the Floods Working Party to be settling at this time. The Working Party has a rather wider remit than that, and we have to leave it to other agencies to do the research to solve many technical points. If the framework of these analyses are accepted, then I believe eventually reasearch will be done to get the numbers as near correct as we can reasonably expect.

I close by saying that all the members of the Floods Working Party apart from Mr G M Binnie are here, so if you have got something that is burning into your soul that you would like to get across to the Working Party, would you please see Mr Seddon, Mr Chapman, Mr Clarke, Mr Kennard or Mr Lander, and I am sure that your views will be taken into account.

WRITTEN CONTRIBUTIONS

D W BERRY (Howard Humphreys and Sons) :

During the discussion of Session 4 a number of speakers referred in general terms to 'wasted resources' as a result of designing a spillway for Estimated Maximum Flood (EMF). In my own contribution to the discussion I attempted to show by means of an actual example that there was very often a sound economic argument for adopting EMF as the design flood, i.e. that resources were not necessarily being wasted. The example which I quoted was deliberately chosen as it involved a very high cost spillway which might have been expected to be sensitive to variations in capacity, and was located overseas.

To complete the picture I would give similar details for a dam in the British Isles - Ballyshonock Dam near Waterford, Eire. Ballyshonock Dam was designed in 1968 and the spillway capacity was based on the Report of the 1933 ICE Floods Committee. The spillway was designed to pass three times Normal Maximum Flood as defined in that Report, and this is what is meant by EMF in the table detailed below.

Although the scale of the project is smaller than my first example the conclusions are similar, as will be seen from the attached table. Accepting 35000 years as a realistic approximation to the return period of EMF, it is economically justifiable to provide a spillway having EMF capacity if possible damages are estimated to be approximately £10 x 10^6 or more.

The total cost of the dam was of the order of £0.5 x 10^6 (1970 prices) and the spillway accounted for about one fifth of this. The reduction in capital costs for providing a spillway to pass 50% EMF instead of 100% EMF - the range indicated by the six cases considered - is approximately £38000, or 7½% of the cost of the dam. The percentages relative to the whole project cost would obviously be still less. Even this saving can only be economically justified by the joint assumptions of an unreasonably long return period for EMF and low estimated possible damages. In these circumstances design for EMF can hardly be described as wasting resources.

The main Consultant with whom we were associated for the design of the Ballyshonock Dam was Nicholas O'Dwyer, Son and Partners of Dublin. I am indebted to Mr Mehigan, a Partner of that firm, for providing me with a priced Bill of Quantities on which to base this note.

EMF Return Period 35,000 years			
Flood % EMF	Total Cost £1,000		
	D = £10^7	D = £10^8	D = £10^9
25	20 040	200 040	2 000 040
50	1 720	16 810	167 050
75	200	1 320	12 570
100	105	195	1 095

EMF Return Period 10^9 years			
Flood % EMF	Total Cost £1,000		
	D = £10^7	D = £10^8	D = £10^9
25	3 160	31 290	312 540
50	65	150	1 050
75	70	75	75
100	95	95	95

A I B MOFFAT (University of Newcastle upon Tyne) :

With respect to abnormal rainfall records, details are given in the reference of a rainstorm of 'unprecedented severity' which extended over some 6500 km^2 of South East Scotland on 12th August 1948.

Twenty-four hour figures averaged some 125 mm of rain, of which it was estimated that 90 mm to 100 mm fell in the space of eight hours. Two earth embankments (Spott Lake and Thorters) were severely damaged by overtopping, the former to the extent that it was abandoned, and a gravity dam (Stobshiels) suffered downstream erosion from the same cause. Estimated peak runoffs for the reservoirs ranged from 35 m^3/s/1000 ha to 95 m^3/s/1000 ha.

In addition to the damage referred to, seven major washouts of the main London-Edinburgh railway embankment took place over a length of eight km, and in one instance a debris dam retaining some 450 tcm of flood water threatened the village of Eyemouth for several days.

Reference:

Baxter, G (1949) *Rainfall and Floods in South-East Scotland, 12th August 1948* Jn. Instn. Wat. Engrs.

R M ARAH (Binnie and Partners) :

Dr Wright described the case where diminishing returns in terms of human safety require a calculated risk to be taken by a design engineer in order to free resources so as to allow a designer in another field to reduce the risk in that field with a net gain in safety to the community. The concept is that of movement towards an equal degree of risk in all structures.

I think that this implies a degree of rational control of resources which is quite impossible in our present society, where a designer who takes an extra risk to save expense on a spillway will find either an enlarged sailing club or a minimally reduced water rate as the end result, rather than a length of safety barrier on a motorway. Certainly the legislation requires reservoirs to be 'sound and satisfactory', without qualification, and this must be taken to represent public feelings related to this particular class of structure. It is very difficult to see how designers as a whole could possibly be required to accept unnecessarily greater risks of fatal accidents on economic grounds and then be held responsible for any form of failure.

The engineer designing a spillway can now use his judgement to choose a design flow lying between an upper limit based on theoretical considerations and a lower limit based on events which have already occurred. No doubt these limits will continue to change with experience, but the situation seems very satisfactory when compared with the structural aspects of safety. Bearing in mind that structural failures are more common than overtopping, and that the structure usually costs far more than the spillway works, there must be more scope for economies resulting from better identification and control of the safety margins in the structural design process than from further refinement of the hydrology. After their success with rainfall and runoff, should the hydrologists now be asked to apply similar techniques to the uncertainties of site investigations, of representative strength parameters, of drainage patterns, of the effects of control during construction and of observation of the completed dam?

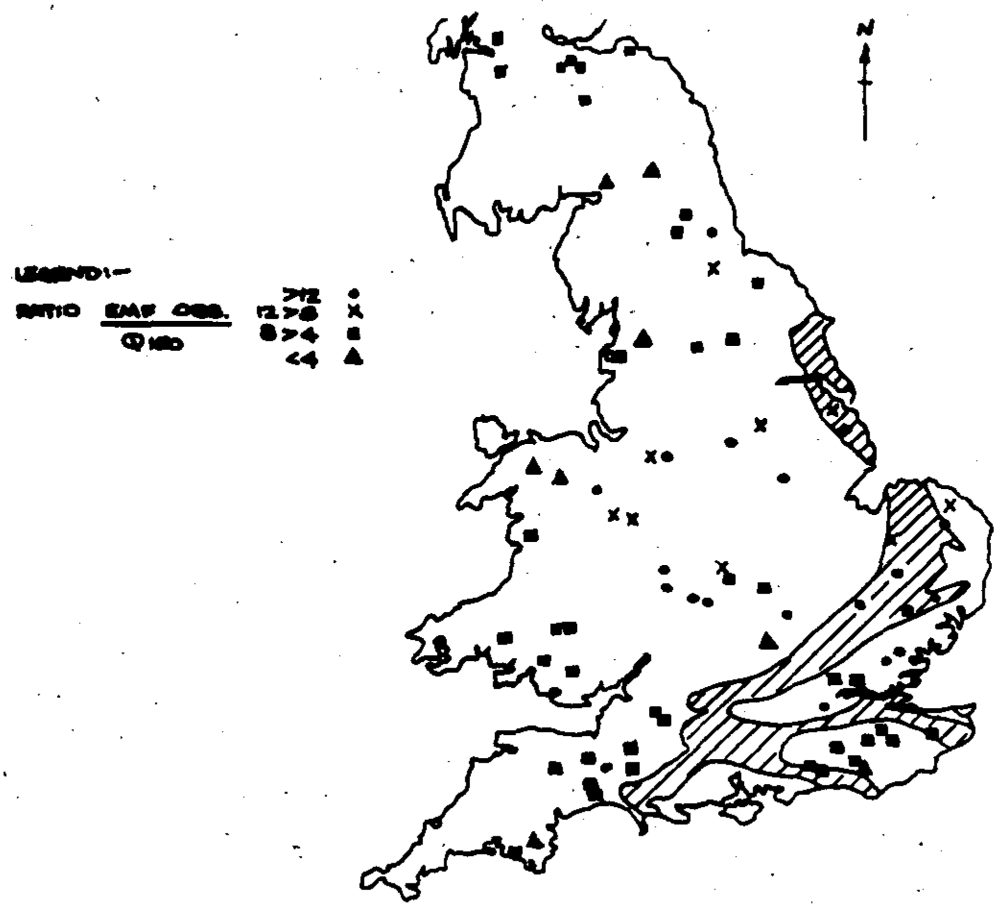
Dr. D E WRIGHT (D Balfour and Sons) ;

In my verbal contribution to the discussion of Session 4 I referred to the difficulties in the way of an economic approach. For completeness this written contribution lists some of the technical difficulties in the way of an application of engineering economy.

1 The rate of failure of dams, which is of course a prime determination of the consequential flood wave and damage. I believe there are very real difficulties of prediction here and it would be useful for all available data to be brought together. Messrs Griffiths and Berry's assumption that the dam fails suddenly once spillway capacity is exceeded, irrespective of the magnitude of the flood and the shape of the peak, may be defensible in the light of present knowledge, but it is hardly a situation that can continue to be accepted. We know of dams that have successfully withstood overtopping for some hours.

2 The cost of damage wrought by fast flowing water. The Flood Hazards Team at Middlesex Polytechnic is making progress on the costs of damage due to river flooding - on the whole, slow-moving water. It may be possible to adapt some of their material.

3 The sensitivity of the cost of damage to the time of discharge or failure. One example is the difference in damage that occurs before and after harvest. This is the conjoint probability problem in another guise.

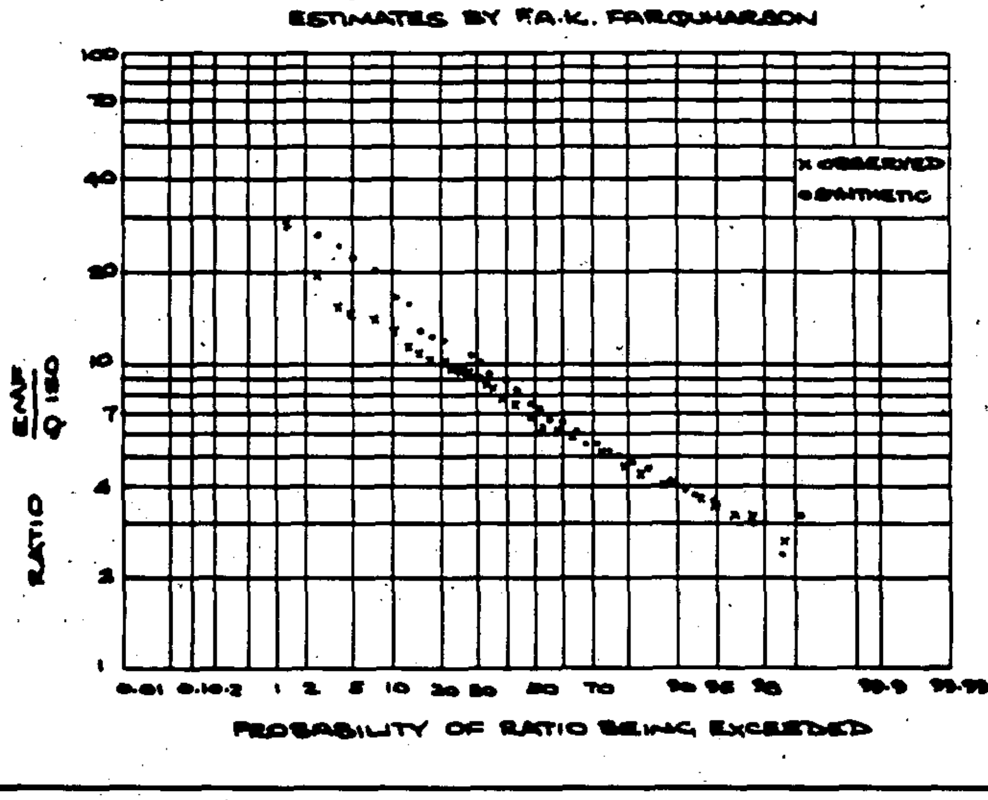


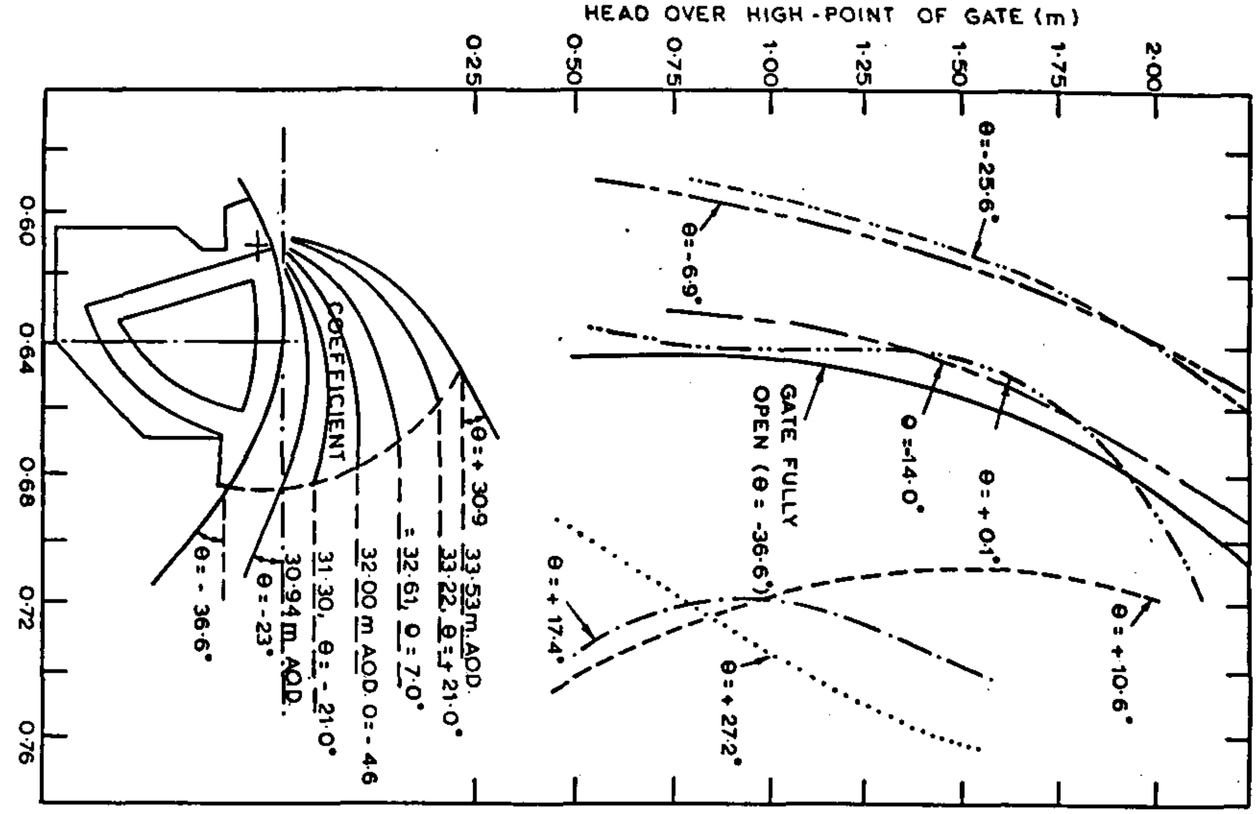
K.T. BASS.: FIG. A.

REGIONAL DISTRIBUTION OF RATIO EMF. OBS. /Q150
REFER TO IN 4.7.

K.T. BASS. : FIG. B.

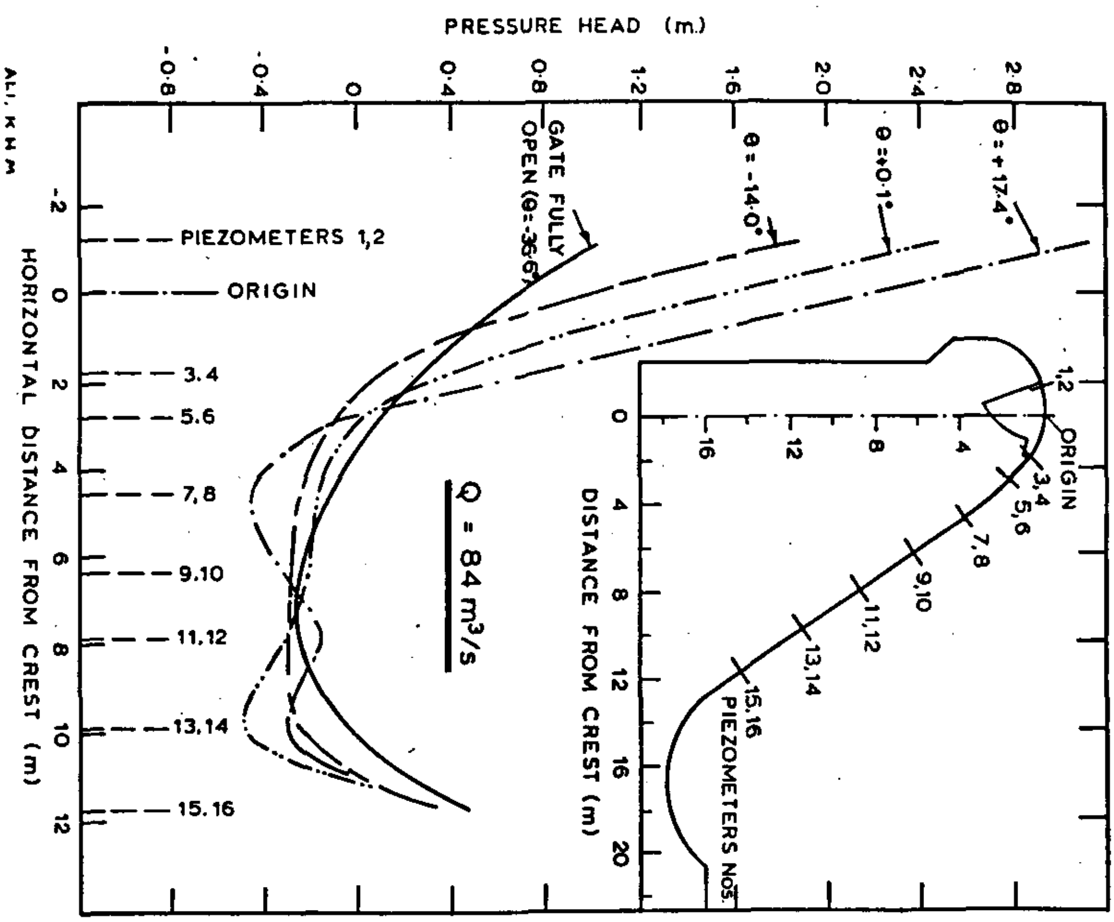
COMPARISON OF EMF AND Q150 FROM
ESTIMATES BY F.A.K. FARQUHARSON





ALI. KHM

FIG.A VARIATION OF C_d WITH GATE OPENING.



ALI. KHM

FIG. B VARIATION OF PRESSURE WITH GATE OPENING

They have the attractive feature of passing large flows for very small increases in reservoir water level.

Some designers, mainly in the United States, have adopted the practice of assuming H_d to be 75% of the maximum expected head because of the resulting increase in C_d. When such a spillway is passing the predicted flood, sub-atmospheric pressure will occur. This procedure, taking advantage of the increase in C_d with H, allows the spillway to be higher for a given length than would be possible if sub-atmospheric pressures were to be avoided during the flood discharge. This practice increases storage but does not take into account the pressure which will actually occur on the spillway during the flood event. If this pressure is low enough, cavitation and possible separation could occur. The effect of possible increase in discharge resulting from new flood assessments upon the pressure distribution of such spillways must be examined carefully.

References:

1	Amer. Soc. Civ. Engrs. (1961)	*Task Committee on Air Entrainment in Open Channels,* Committee on Hydromechanics, Jour. of Hyd. Div., Proc. ASCE 87, May.
2	Ali, K H M (1964)	*Some Flow Characteristics of Drum-Gated Spillways,* M.Eng. Thesis, Univ. of Liverpool.
3	Whittington, R B and Ali, K H M (1971)	*The Llysyfran Spillway and Stilling-Basin: Scale Model Studies,* Jour. Instn. Water Engrs. 25, No 4.
4	Whittington, R B and Ali, K H M (1969)	*Convergent Stilling Basins,* Disc. Instn. Civil Engrs. 43

C L CLARKE (Sir William Halcrow and Partners) :

It was a pity that only one paper, Paper 4.3, dealt with spillway design based on economic analysis, since this is the main method proposed for the majority of United Kingdom reservoirs in the ICE Discussion Paper. The authors of Paper 4.3 concluded that 'the use of the economic approach should be confined to approaches where loss of life is not foreseeable, and it is probable that the majority of these will be comparatively small ... Consequently, studies ... would not be justified'. They also noted that, in economic analysis, optimum values 'will apply over a fairly wide range so that the final choice of design flood will not be critical from the economic point of view, and may justifiably be selected to satisfy other criteria.'

These seem damning conclusions from the only paper on the proposed method of spillway design. The main difficulty with such a method is the evaluation of damage likely to be caused in the event of a failure following overtopping. The extent of such damage is largely dependent on the mode of failure, e.g. whether the dam takes 10 mins or two hours to wash out - and to a lesser extent on when the flood occurs, e.g. on agricultural land, when it has just been sown or is lying fallow - or in residential areas, whether at night or during the day.

I would thus support the author's conclusions that the economic approach is not yet a practicable one to use on the majority of dams in the UK. Rather, the approach outlined by Paper 4.1 would appear simpler and more certain. This approach is similar to that outlined as an alternative to economic analysis in Appendix A of the ICE Discussion Paper, though the author omits the lowest category of reservoir. It does seem that there is a case for a category having a lower standard than suggested in Paper 4.1. There are many small 'reservoirs' which, if the 'dam' failed, would cause no noticeable increase in the downstream flow. For such cases, heavy expenditure would hardly be warranted to upgrade the spillway from say, a 100 year flood to a 1000 year flood.

I mentioned at the Symposium that the return periods suggested in Appendix A of the ICE Discussion Paper were not entirely arbitrary, but were based on risks thought to be acceptable to society today. The table illustrates the maximum risks in each category suggested in that Appendix. It can be noted that, over a 100 year period, the highest risk of loss of life is around 1 in 2000 - for reservoirs in all categories except those threatenting a community, where it is less. Since there are some 2000 to 3000 dams in the UK this would mean that, on average, there is likely to be, during a lifetime, one failure caused by flood and involving loss of life.

(Clearly, the figures on the table cannot be regarded in any way as accurate but they probably have the decimal point in the right place, which is the most that can be hoped for in estimating such events.)

TABLE ILLUSTRATING POSSIBLE RISKS IN DIFFERENT CATEGORIES OF RESERVOIRS

CATERGORY OF RESERVOIR IN APPENDIX 'A' *	A		B		C		D
New Reservoirs	All (Except D)		–		–		No loss of life can be reasonably foreseen
Existing Reservoirs Where a breach in the dam	Would endanger lives in a community		May endanger lives		Would pose a negligible risk to life		
Risk of breach by overtopping ?	Yes	Negli-gible	Yes	Negli-gible	Yes	Negli-gible	Yes
Return Period, in years, of flood that can be passed ("Design Flood")	PMF	10,000	10,000	1,000	1,000	150	150
Risk of "Design Flood" being exceeded in 100 years	0.1%	1%	1%	10%	10%	50%	50%
Risk of flood greater than the "Design Flood" causing a breach	20%	1%	25%	1%	50%	1%	100%
Risk of such a breach causing loss of life	100%	100%	20%	20%	1%	1%	0.1%
Combined risk of loss of life through flood	0.02%	0.01%	0.05%	0.02%	0.05%	0.005%	0.05%
Or : Odds against loss of life over 100 years	5,000	10,000	2,000	5,000	2,000	20,000	2,000

Notes (a) "Negligible Risk" is intended to represent a probability of 1% or less, i.e. there is less than 1 chance in a 100 of the event taking place.

(b) The risk of a PMF being exceeded is unknown. Theoretically, it is zero. On the table a risk of 1 in a 1,000 is given, making it equivalent to a 100,000 year flood.

(c) The same risk, 1 in a 1,000 is shown against an event which 'cannot reasonably be foreseen', i.e. loss of life in Category D.

(d) The risk of a breach being caused by a flood exceeding a design flood is clearly greater for a low design flood than for a higher one, as the higher design flood is only likely to be exceeded by a smaller amount. Thus, a risk of a breach being caused by a flood exceeding the design flood varies from 20% (PMF) to 100%(Q 150).

* Appendix A, ICE Discussion Paper : Reservoir Flood Standards

If reservoirs are to be classified, the question arises whether the classifications should refer to a return period, or to fractions of Probable Maximum Flood, or to multipliers of, say the 100 year flood. While the latter alternatives may appear simpler than the return periods, I do not think they would be so acceptable to the public. Would one rather be told that the reservoir above one's house was 'capable of safely passing a flood estimated to occur once in 10000 years', or only 'capable of passing rather less than half a probable maximum flood'?

PROCEEDINGS : TECHNICAL SESSION 5

PROBLEMS OF EMBANKMENT DAMS AND REMEDIAL MEASURES

PAPERS :

DISCUSSION :

Cont.

Index 5 (cont)

EIGHTEENTH CENTURY DAMS IN ENGLAND

M F Kennard BSc CEng FICE FIWES

PARTNER

ROFE KENNARD AND LAPWORTH

SYNOPSIS

Many reservoirs were constructed for landscaping purposes in the latter part of the eighteenth century, several of which are still standing and have been in use for about 200 years. These were the forerunners of later reservoirs, firstly for canals and then for town water supplies. A number of the early reservoirs come within the Provisions of the Reservoirs Act, and remedial works have been carried out in some cases. In the paper some of these dams are discussed.

INTRODUCTION

In the latter half of the eighteenth century in England, the landscaping of country estates developed rapidly from formal layouts to areas where a freedom of nature was expressed. Stourhead, Wiltshire was one of the first examples. The gardens were laid out in 1741 by Flitcroft, an architect for the owner Henry Hoare, and the meres in the valley were converted into a series of lakes.

This was followed by a large number of landscaping schemes where areas of water were formed, and this led to the construction of many dams, several of which are still standing and are therefore about 200 years old. Apart from some mill dams, these landscaping dams are the earliest dams in Great Britain, and they were the forerunners of reservoirs for water supply to canals constructed from the end of the eighteenth century.

The part played in the history of dams by landscaping reservoirs is often not generally realised. The leading exponent of landscaping in Great Britain in this period, Lancelot 'Capability' Brown (1716 - 1783). was responsible for the formation of a large number of lakes. In 'Capability Brown and Humphrey Repton' by Edward Hyams (1) there are mentioned over 50 places where he was responsible in whole or in part for the landscaping of estates. In more than 20 of these, lakes were involved. These works are listed in Appendix 1. Other references show that he was involved in some way in over 150 places, although in some of these the work was not carried out, or others subsequently changed the proposals considerably.

Others, including Humphrey Repton (1752 - 1818), were also responsible for some similar lakes, and a number of these are listed in Appendix 2. It is not always known who was responsible for the lakes, as several persons were often involved, and some schemes are attributed to Brown by one authority and not accepted by another. Petworth, Sussex, is one such example.

These Appendices are not necessarily complete, and the lakes were not all constructed in the eighteenth century. Some of the apparent lakes were not formed by construction of dams. They were enlargements of rivers or formed solely by excavation. Burton Constable, Humberside, is an example of excavation in a flat area to create a lake, where an apparent dam is a bridge with a low weir beneath.

Some of the dams and reservoirs are not significantly large from an engineering point of view, but others are substantial structures, especially when considered in the light of technical knowledge some 200 years ago. For example, the lake at Blenheim has a capacity of 620 t.c.m. and the earth dam has a maximum height of 9 m and a length of 140 m.

Norman Smith in 'History of Dams' (2) does not refer to these landscaping works, but states that modern dams in Great Britain started with canal reservoirs in the last decade of the eighteenth century with dams such as Todd Brook Dam and Coombe Dam, near Chapel-en-le-Frith, completed in 1794.

These were of a greater magnitude than the landscaping reservoirs and they were designed to store and release water, compared with solely forming a sheet of water. Nevertheless there are similarities in the designs, and the fact that many dams of this period are still standing is a tribute to these early designers, who were architects and gardeners and were not engineers in the sense of Brindley and Telford who were engaged on canals and reservoirs. Some of these early reservoirs have been inspected under the Reservoirs (Safety Provisions) Act, 1930, but there are probably some larger than 25 t.c.m. capacity which have not been inspected.

In Parliament, (Second Reading, Reservoirs Bill, 22nd January 1975), it has been stated that there are 230 reservoirs, which are 'mill dams, fishing lochs and lakes, ornamental lakes and so on' of more than 25 t.c.m. capacity in private ownership. The author does not know if this number is complete and includes all the examples in this paper. Several examples of dams for landscaping are given in the following sections. The information has been obtained from several sources, including some statutory inspections, by a number of engineers, but the author cannot personally confirm the accuracy of all the information.

In a letter published in the Journal of the Institution of Water Engineers in 1972 (3) the author commented that he considered '. . . that Cability Brown was probably one of the country's leading dam engineers . . . ', and that this paper enlarges on this aspect of his achievements.

ASHBURNHAM, Sussex (4)

Brown drew the plans in 1762 for a series of lakes cascading in front of the house. There are three lakes with capacities of 3 t.c.m. and 15 t.c.m. respectively. The total catchment areas at each dam are 120 h, 150 h and 200 h. The lower dam is 260 m. long.

Discharge from the upper lake used to pass through a low level brick outlet, but this was replaced, (probably earlier this century), by a high level culvert. Discharge from the centre lake used to pass down a dropshaft in a purpose built island situated in the centre lake. Flow was then conducted along a brick culvert about 90 m long into the lower lake. The cross-section of this culvert is generally about 0.75 m x 0.55 m. wide; but at one point it enlarges into a gallery about 1.5 m. square. The alignment in plan also varies, and this suggests that the culvert may have been extended on more than one occasion as landscape development proceeded. An overflow weir and chute is now used. Flow from the lower lake discharges over an uncontrolled weir and down a chute built on the downstream shoulders of the dam. Flood capacity has recently been augmented by a second weir.

There is some circumstantial evidence that the discharge capacity of the system has been exceeded on at least one occasion during its life by floods.

BLENHEIM, Oxfordshire (5)

In the building of Blenheim Palace, Vanbrugh had constructed an elaborate bridge over the River Glyme out of all proportion to the size of the stream. Brown, by the construction of a dam during the 1760's transformed the stream into two lakes joined at a point crossed by the existing bridge. The shape of the lakes obviously owes much to excavation.

The catchment area is 124 km^2 and the lake area is 51 h. and capacity is approximately 56 t.c.m. The earth dam is 6 to 9 m. high and 135 m. long. The slopes of the embankment are 1:3.5 upstream and 1:3 downstream. There is a limestone facing on the upstream slope. The embankment is believed to possibly have a puddle clay core. The spillway is a chute excavated in rock on one flank, able to pass approximately 10 m^3/s and there are low-level hand-operated sluice gates with a capacity of 4.3 m^3/s.

The defects have included internal erosion due to piping through the core and around the sluice gate structure, both leading to depressions in the crest. Remedial measures comprised a filter system constructed at the downstream toe and the internal cavities backfilled from the crest with filler material. The cavities around the gate structure have been backfilled with clay. The freeboard was considered inadequate, and the crest had been raised with earthfill.

In 1893 (6) it was reported that the dam had been repaired about 20 years previously, and that there were leaks in the bank at that time. It is possible that leakage and internal erosion has continued from before that time until remedial works were carried out in 1967. Fears had been expressed about the water holding capacity of the Great Oolite strata, but the evidence of the existing lake for over a century led to consideration of other sites in the area by the Royal Commission on Water Supply of the Metropolis.

CASTLE HOWARD, Yorkshire

The dam was built in 1798 by W. Chapman, and has a maximum height of 6 m. Its length is 520 m. and the reservoir has a capacity of 45 t.c.m. with a catchment area of 32 h.

In a previous paper (7), the Author quoted a specification of the proposed form of construction as:-

'The embankment should be, at least, 9 ft. wide at the top, so as to form a roadway; and the outward face of the bank to slope with two feet base to each foot in height (or even flatter) which will admit a horse to go up or down by passing aslant.

"The inner face should slope the same, down to a foot below the surface of the water; and below that it may be as far upright at 16 or 18 in. base to a foot in height.

"The mean height of the embankment should be a foot above the water; and, consequently, the inner edge more; the road-way should slope outwards.

"If the soil will hold water without puddling; the excavation and embankment, including the sodding of its face, may cost from ten to fifteen shillings a yard in length, according to the width of the opening, and the consequent space the earth has to be moved.

"Besides an overflow for the waste water; you should have, under the highest part of the embankment, an offtake or drainage sluice."

This shows the insufficient freeboard and lack of upstream slope protection that is typical of some dams of the period.

Although still standing and forming a reservoir, the Castle Howard dam has needed remedial work over the years. Leakage and settlement probably occurred before 1925, but in that year clay puddling was carried out to deal with leakage and in 1938 some sheet piling was driven. Further steel piling was added in 1945, and in 1948 cement grout was injected which stopped the leaks in the limited area treated. In 1962, further work was thought to be necessary. Grouting was considered, but was rejected because of the cost.

An inspection in 1967 showed the top of the bank to be only a minimum of 0.3 m. above the overflow sill, which itself had to be exposed as a visual inspection first failed to locate an overflow. It was obvious that water must have flowed over the dam. There was leakage at many points, generally at 1.8 m to 2.1 m below the top of the bank. The leaks were reduced by excavating along the line of the leaks, refilling with puddled clay and protecting the clay face with bagged concrete.

DANSON PARK, Kent

Danson Park Lake was constructed by Brown in 1762 as part of the landscaping of the grounds of Danson Hall. These grounds are now a public park, and the surrounding catchment area is mostly built up with residential property, which also covers the natural flood plain below the dam.

The dam consists of an earth bank constructed from material excavated from the reservoir area, and is a clayey part of the Blackheath Beds. During the construction of a new overflow in 1964 possible traces of a clay core were found.

The dam is about 180 m. long, 6 m high at its deepest point and has a crest width of 4 m., though the lake is only 2 m. deep. The upstream face of the dam has a slope varying from 1.5:1 at the top to 4:1 at the bottom, whereas the downstream slope is more constant, between 1.5:1 and 2:1. The lake holds about 9 t.c.m. of water and has a surface area of 8 h.

It is impossible to say what was originally provided as an overflow, but immediately before 1964 the overflow consisted of a long weir leading into a 1.15 m. dia. concrete pipe which discharged into the water course below.

Prior to 1964, the freeboard was about 0.4 m. between top of bank and overflow levels. This freeboard proved inadequate and the embankment would have been overtopped in 1958 if an emergency wall of sandbags had not been constructed along the crest. This occurrence stirred the local authority into authorising the construction of the present overflow arrangements.

The present overflow structure is a somewhat complicated structure comprising a lower weir probably capable of passing about 3 m^3/s through pipe culverts, and an upper flood relief spillway capable of passing another 1.5 m^3/s.

The engineer who designed the overflow was given three criteria to work to by the then River Board and his clients, the Borough of Bexley. These were:-

a) the overflow level should be kept at about 38 m to leave enough water in the lake for boating;

b) The maximum outflow should not be more than about 2.25 m^3/s as flooding would occur of houses downstream of the dam;

c) at least 85 t.c.m. of flood storage should be provided.

To meet these ends, the overflow level was kept at 38 m the present overflow structure was provided, and the embankment was raised by 1 m.

Opinions vary on the maximum flood for design purposes because of the complexity of the catchment due to housing and a railway line which now bisects it, but reasonable design flows range from 20 m^3/s by the Engineer carrying out the last statutory inspection to 26.5 m^3/s. by the Institute of Hydrology. To provide overflow arrangements to cater for such floods would probably include both lowering the spilling level, which would destroy the amenity value, and enlarging the stream channel for miles downstream of the dam if property is not to be flooded.

The dam structure appears to be generally sound, but the Council have planted young forest trees on its downstream slope. These they have been recommended to remove before they become established.

HAREWOOD, Yorkshire

Brown worked on the landscaping of Harewood between 1772 and 1782. The lake was formed by a dam about 7.5 m. high and 116 m. long. The crest width is about 6 m. The slopes are approximately 1:2.5 upstream and 1:2 downstream. The upstream slope has stone pitching in steps surmounted by a coping, now covered with grass. The freeboard is now about 0.6 m.

The overflow, at one end of the dam, and in original ground, consists of three sections of weir, each 4 m wide, surmounted by a bridge and leading to a cascade. This cascade had rocks built in as an energy dissipator, and has similarities with Sherborne and Staunton Harold. The maximum capacity would appear to be about 25 m^3/s for a catchment area of about 17 km^2. Since the lake was formed, Eccup Reservoir was constructed by Leeds Corporation upstream of Harewood, and this probably accounts for the lack of any changes to the original overflow works. There appears to be no leakage from the dam. Brown planted a chestnut tree on the crest near the middle of the dam which is still standing.

NOSTELL PRIORY, Yorkshire

The house was begun in 1733 and added to in 1766. There are three lakes, and the middle lake and upper lake is separated by a bridge carrying the A638 road . The bridge was constructed in 1761, and the lakes may have been formed then or at a later date, probably following mining subsidence, to ensure water over an unsightly area.

The dam forming the middle and upper lakes is about 100 m long with a maximum height of 8 m., and is formed of clay throughout. The slopes are generally 1:2, but at the overflow near the centre of the dam the lower portion has been cut into at about 1:1, and a heavy stone revetment provided to form what was intended to be a pleasant cascade. A sluice with a pipe was built in the embankment adjacent to the overflow.

The dam is heavily overgrown with trees, rhododendrons and other shrubs and has suffered due to mining subsidence over the years. There are leaks in the downstream slope which have been picked up in drains from inverted filters.

The reservoir has a capacity of about 6 t.c.m. and the main inflow is through the upper lake which has a marked lag effect on floods. The catchment area is 300 h. and is very flat.

Subsidence due to mining with resulting modifications has made it impossible to know exactly what everything was like at the beginning, but it is believed that the maximum capacity of the overflow with the water level at bank crest level was originally about 8.5 m^3/s. The present capacity under the same conditions is 6 m^3/s, which is roughly 5 times Q150 routed through the two lakes or twice Q150 similarly routed with 0.3 m.freeboard above flood level.

SHERBORNE, Dorset

Sherborne Lake was landscaped by Brown in 1756. It is said that the then owner of Sherborne Castle was so impressed by the sight of his estate, following on exceptionally heavy flood on the River Yeo, that he asked Brown to turn the flood area into a permanent feature.

The lake has a capacity of about 55 t.c.m. and an area of about 20 h. The dam is so well landscaped that it is difficult to define its location and hence its size. It appears to be about 7 m high, 150 m. long and 45 to 60 m. wide at the base. The downstream slope is quite irregular and on this slope and on the crest of the dam there are many large trees, some up to 3 m. girth and over 35 m. high. The embankment was constructed mainly of clay on a clay foundation. The fill contains pieces of limestone and traces of organic material and some thin bands of hard packed limestone occur in the foundation. There is no evidence of a core.

The overflow was 4 m. long and was designed as an ornamental cascade. This appears to have been the only overflow provided originally, though at some later stage a culvert was constructed through the narrowest part of the dam and flow was controlled by two sluices. Although attractive to see, the discharge capacity of 1.5 m^3/s was hopelessly inadequate. Some minor leakage had developed both at the culvert and at the cascade weir. At the weir there was also evidence of scour where floods had overtopped the embankment on either side.

Though the lake was privately owned, the Somerset River Authority were interested in it for the regulation of the Yeo. They have undertaken dredging operations to increase its capacity from 60 t.c.m. to 570 t.c.m. To meet the requirements of both parties, additional overflow capacity was provided so that there would always be a discharge over the cascade and so that all discharges from the lake could be measured. These conditions were met by providing a low head air regulated siphon in reinforced concrete on the line of the old culvert, and by extending the cascade weir to 10.67 m. and providing it with a proper crest for measurement. The siphon has three barrels, each 3.66 m. wide and with a 1.22 m throat. The overall head is 2.13 m. and the crests are set 150 mm above the crest of the cascade so that the latter will always be discharging. The original freeboard is maintained at 1.83 m. and the maximum discharge from both siphon and weir is 113 m^3/s. The dredging work and the new siphon overflow will restore the eighteenth century appearance and characteristics of the lake and landscaped grounds.

STAUNTON HAROLD, Derbyshire

A lake above the Staunton Harold Reservoir constructed by River Dove Water Board in 1960 is believed to have been built in the eighteenth century, possibly by Brown. Its cascade overflow is typical of his work.

The dam is about 7 m. high and 100 m. long, and has a capacity of 6 t.c.m. The drive to Staunton Harold Hall surmounts a bridge over a low dam that forms an upper lake. Two rectangular bellmouth overflows lead from the upper lake to the lower one, whilst the overflow from the lower lake comprises 5 lengths of weir under a five-span brick arch bridge, leading to a cascade in original ground at the end of the dam. The cascade has rocks built in as an energy dissipator. The capacity of the overflow is about 3.23 m^3/s and the embankment has been overtopped on several occasions. A new length of overflow is shortly to be constructed alongside to increase the overflow capacity to about 34 m^3/s.

The dam has slopes of about 1:2 upstream and 1:2 downstream, and is formed of clayey material. The upstream face is irregular, and the crest and downstream are well covered with mature trees. The trees are to be lopped to a maximum height of about 7 m. as it is considered that their removal could affect the stability of the embankment. Dredging of the upper lake has recently been carried out.

DISCUSSION

The previous sections give some information on eight representative examples. The limitation of the information and the lack of uniformity given is recognised by the Author, but the private nature of these works, and the confidentiality of the material, prevents a complete study of these and other interesting cases.

It is believed that generally no cores were used nor zoning of materials carried out. The fill material used was probably obtained from excavations within the reservoir area which improved the shape of the water surface. It is not known whether stability problems arose during construction but the modest height (6m to 8m) and light compaction probably was such that the clayey materials were satisfactory. Vegetation, especially trees, have probably assisted stability over the period of use.

Puddle clay, although not used in landscaping dams, was used extensively by Brindley and other canal builders in the latter part of the eighteenth century on the inside face of embankments or cuttings. It was placed to a thickness of about 0.9 m, applied in layers,and care was taken to work the new layer so to unite with the layer immediately beneath. Over the top course a layer of common soil was usually laid.

There was a choice of civil engineering expedients available to those responsible for embankments in the eighteenth century. In referring to the canal work of James Brindley (1716-1772) Samual Smiles wrote (8).

'The skilled men had their trade secrets, in which the unskilled were duly initiated, and the following were amongst them, - simple matters in themselves, but not with use:-

A wet embankment can be prevented from slipping by dredging or dusting powered lime in layers over wet clay or earth.

Sand or gravel can be made watertight by shaking it together with flat bars or iron run in some depth, say two feet, and washing down loam or soil, as the bars are moved about, thus obviating the necessity for clay puddle'.

These practices developed during the extensive canal building programme, and it is possible that similar practices could have been used in landscaping dams of the same period.

In the 1790's, the first of a number of reservoirs were built to supply water to the upper levels of canals. Amongst the earliest was Coombs dam, near Chapel-en-le-Frith, Derbyshire, for the Peak Forest Canal. This 16 m high dam is believed to have been completed in 1794. This is the first year for any British dams over 15 m. high recorded in the ICOLD World Register of Dams (9). The dam has an upstream masonry block face over puddle clay.

There were failures of several dams at about the end of the century including Blackbrook near Loughborough in 1799 (7), Whinhill, Scotland in 1815 (9), and several canal reservoirs.

Telford (1757-1834) is credited with moving the position of the watertight clay layer from the upstream face to the centre of the embankment in about 1820. It is therefore unlikely that any dams constructed in the eighteenth century had central puddle clay cores.

The puddle clay lining on the inside face of canal embankments is considered by Bishop (10) to probably represent the earliest use of zoned-fill construction. The use of more free draining material behind a clay zone has led to the satisfactory performance of some embankments with apparently steep slopes. It was the weathering of the clay lining from fluctuating water levels that led Telford, in about 1820, to abandon its use in preference to a central core. Dams that he built for the Birmingham Canal in 1827 were of this type.

CANAL RESERVOIRS

In addition to the Peak Forest Canal dams others in the same period include those for the Leeds and Liverpool Canal (Foulridge and Barrowford), Trent and Mersey Canal (Rudyard Lake 1797) and Grand Junction Canal (Aldenham 1795; Braunston 1795/6; Weston Turville 1797/8; Wiltstone 1802/3 and Ruislip 1808).

Aldenham dam is described in the next section. Remedial works are also known to have been carried out at the Ruislip dam.

ALDENHAM, Hertfordshire

In an Act of 1793 the Grand Junction Canal Co. were obliged to build a reservoir to supply water to the River Colne to compensate for any river water diverted into the canal. The site was at Aldenham and the reservoir was first completed in 1795. In 1802, the dam was raised to 8 m. high and 400 m. long, and the reservoir capacity to 78 t.c.m.

According to Faulkner (11), there was trouble due to slips during construction. Jessop reported in 1802 that the trouble was due to the treacherous nature of the clay which tended to crack in dry weather and he recommended covering it with a protective layer of sand and gravel about 175 mm thick. This refers to the weathered London clay removed from the basin for the embankment, but the 1:4 slopes of the dam suggest foundation problems during construction. Faulkener states that in December 1804 the water had to be lowered in a hurry to enable urgent repairs to be carried out.

The spillway works in 1933 had a capacity of about 7 m^3/s and a freeboard of only 0.65 m. It was reported that waves had passed over the top of the bank. In 1939, the three overflow weirs were replaced by a 12 m. long weir with a capacity of 17 m^3/s., and the freeboard was raised to 1.1 m.

In 1923, part of the upstream face was protected with in-situ concrete slabbing due to erosion of the face. The slabbing was placed on the eroded face lying at a slope of 1:1 to 1:2 typical of eroded slopes. In the 1930's, concrete slabbing was extended over the whole length of the embankment, and a 1.4 m. high wave wall added.

In 1959 a length of the upstream face slipped, probably due to erosion of the face beneath the level of the concrete, and it was repaired with a line of concrete sheet piling at the toe of the concrete slabbing. In 1974, the top of the bank was made up to level, a tarmac path laid, open joints adjacent to the wave wall and between slabs filled with concrete, and trees removed from the downstream slope.

Settlement records since 1932 showed continuing settlement of the crest at several locations. In January 1975, two areas of the upstream face, not involving the previously stabilised section, slipped and subsequently two sections of the downstream slope slipped to a minor extent. Remedial works comprising a gabion protected weight block to the toe of the upper steep section of the upstream face, lowering of the top of the embankment behind the wave wall, and replacing the displaced lengths of wave wall with light weight concrete have been recommended at the time of writing.

This example shows how critical is the top section of an old earth embankment when erosion has occurred. Remedial works resulting from statutory inspections could affect the stability when little or no safety margin exists.

MILL DAMS

Low Dams to impound or divert water to water-wheels, have a long history. In 1676, Trew's weir was built on the River Exe, Devon. In 1767 Smeaton designed the Larbert dam on the River Carron. The 2.5 m. high dam still stands, as does his 2.5 m. high dam on the River Coquet, which he designed in 1776, and is curved in plan to a radius of 52m. It consists of a rubble masonry core faced with large Masonry blocks. (2) Its success is due to Smeaton's care with design and construction and the uninterrupted overflow weir across the whole width of the river.

Other early dams were used in the mining industry, including some in Weardale, Co. Durham and elsewhere.

Near Ffestiniog, N. Wales there is an interesting form of construction in a dam built in 1800. It comprises two dry stone walls with the centre filled with peat and boulders and with additional peat on the upstream face. (12)

CONCLUSIONS

The number of examples discussed is not sufficient to draw any firm conclusions regarding the eighteenth century earth dams. Knowledge of the materials and cross-sections is meagre, and although there is reference to puddle clay cores the author considers this to be unlikely and the embankments to have been homogeneous.

The design of overflow works must have been on an empirical basis, and probably suggested by the original river channel. The freeboard seems to have been low, and it is very likely that several or most of these embankments have been overtopped more than once during their existence. The substantial grass and tree growth would have given sufficient erosion protection. Brown's cascades built in original ground, and of generous cross-section, have survived satisfactorily.

Upstream slope protection seems to have developed in natural stone areas, but many cases, (not by Brown) exist where erosion of unprotected faces took place leading to the upper part of the water face becoming steep. At Burton Constable, Brown used bricks to protect the natural ground around a shallow lake.

Problems of erosion, leakage, piping, settlement and local slipping have all been reported and can be overcome such that the structures, which have been accepted into the landscape, can continue to fulfil their original or present function. The example of Aldenham shows the low Factor of Safety that can exist and the need for special care in maintenance and remedying defects.

The Author considers that points of study that merit further and more detailed investigation are the resistance of old embankments to overtopping; the effect of trees; the effectiveness or otherwise of the upstream fill to prevent further erosion; and the design of original sluice gates and culverts.

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APPENDIX 1

A PARTIAL LIST OF LANDSCAPED AREAS INCLUDING LAKES WHERE CAPABILITY BROWN WAS ENGAGED

Ampthill Park, Ampthill, Bedfordshire.

Ashburnham Place, Ashburnham, Sussex.

Audley End, Saffron Walden, Essex.

Benham, Speen, Berkshire.

Blenheim Palace, Woodstock, Oxfordshire

Bowood, Calne, Wiltshire.

Burghley House, Stamford, Northamptonshire.

Burton Constable, Sproatley, Humberside.

Chilham Castle, Chilham, Kent.

Danson Park, Bexley, Kent.

Dodington Park, Dodington Ash, Gloucestershire.

Harewood House, Harewood, Yorkshire.

Heveningham Hall, Walpole, Suffolk.

Kew Gardens, Richmond, Surrey.

Longleat, Wiltshire.

Luton Hoo, Luton, Bedfordshire.

Ragley Hall, Alcester, Warwickshire.

Sandleford Priory, Newbury, Berkshire.

Sheffield Park, Uckfield; Sussex.

Sherborne Castle, Sherborne, Dorset.

Syon House, Kew, Surrey.

Weston, Weston-under-Lizzard, Staffordshire.

Wimpole Hall, Wimpole, Cambridgeshire.

Wrest Park, Silsoe, Bedfordshire.

APPENDIX 2.

A PARTIAL LIST OF LANDSCAPED AREAS INCLUDING LAKES EXCLUDING THOSE ATTRIBUTED TO CAPABILITY BROWN

Arbury Hall, Nuneaton, Warwickshire.

Blickling Hall, Aylsham, Norfolk.

Buscot Park, Faringdon, Oxfordshire.

Castle Howard, York, Yorkshire.

Corsham Court, Corsham, Wiltshire.

Crewe Hall, Crewe, Cheshire.

Culford, Bury St. Edmunds, Suffolk.

Deene Park, Corby, Northants.

Gayhurst House, Gayhurst, Buckinghamshire.

Hodnet Hall, Market Drayton, Shropshire.

Kenwood, Hampstead, London.

Melbourne Hall, Melbourne, Derbyshire.

Milton, Milton Malsor, Northamptonshire.

Nostell Priory, Wakefield, Yorkshire.

Petworth House, Petworth, Sussex.

Rousham House, Steeple Ashton, Oxfordshire.

Spetchley Park, Worcester, Worcestershire.

Stapleford Park, Melton Mowbray, Leicestershire

Staunton Harold, Nr. Melbourne, Derbyshire.

Stourhead, Mere, Wiltshire.

Wentworth Woodhouse, Rotherham, Yorkshire.

Welbeck, The Dukeries, Nottinghamshire.

Westbury Court, Westbury-on-Severn, Gloucester

Woburn Abbey, Woburn, Bedfordshire.

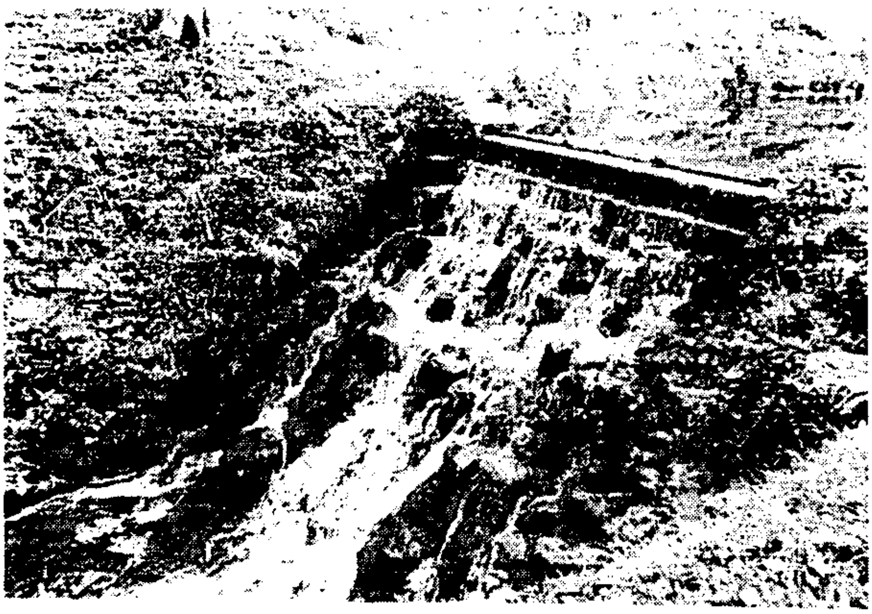

Fig. 1 — Original Cascade Overflow at Sherborne

Fig. 2 — Remodelled Overflow at Sherborne

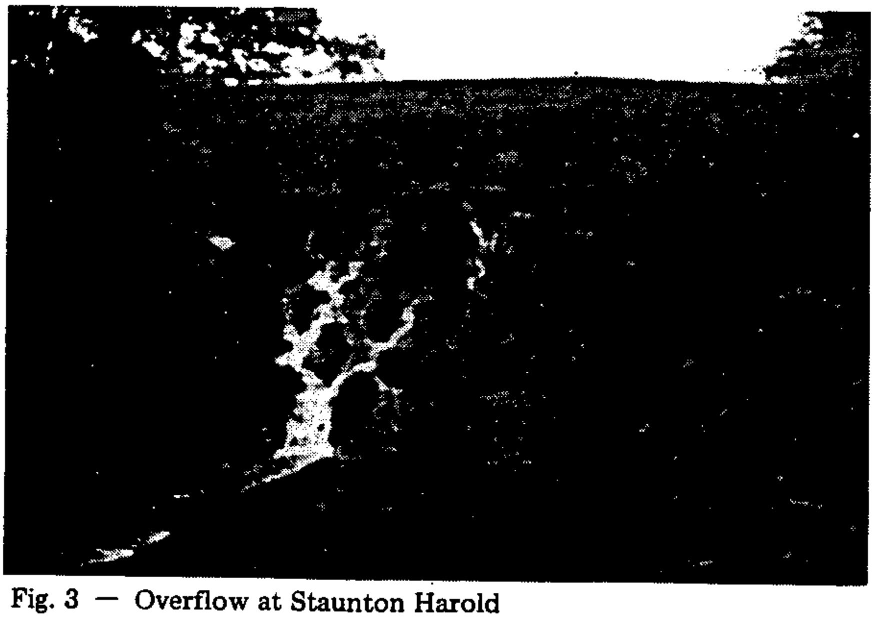

Fig. 3 — Overflow at Staunton Harold

INSPECTION OF OLD EMBANKMENT DAMS

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SYNOPSIS

The paper describes, in simple terms, what is involved in the inspection of an old embankment dam. It starts by outlining the preparatory work, usually carried out by the Inspector's staff, and continues with a description of a typical inspection, drawing attention to the usual things to look out for. The paper winds up with a summary of some of the factors affecting the Inspector's report and his recommendations. An Appendix provides a check-list of matters to consider, or look at, during an inspection visit.

INTRODUCTION

The paper outlines the engineer's work involved in an inspection with particular reference to old embankment dams, of which there are at least 500 in Britain. Many of these dams were built in the late eighteenth and early nineteenth centuries. While some are still used for supply of water to our canals and industry, many are now being transferred to local authorities for recreational use.

It is hoped that the paper will be of help to Inspecting Engineers making their first inspection of such dams, and perhaps also to Supervising Engineers. However, the main purpose of the paper is to attract criticism in the hope that the more common problems can be discussed, and attention drawn to other problems and their solutions - with the ultimate aim of a more uniform approach by all Inspectors.

(Where the 'Owner' is referred to, this can be taken to cover the 'Undertaker' if appropriate. Also, Inspecting Engineers are, for brevity, referred to as 'Inspectors').

PREPARATIONS FOR AN INSPECTION

GENERAL

The Inspector's first task following appointment under the Reservoirs Act is to collect together all the available information. The Owner may be able to provide a great deal of this but, for old reservoirs, information may be limited though he is, of course, required to provide information in the 'Prescribed Form of Record' and also the Reports of previous inspections. Often a survey or plan of the reservoir is available, and sometimes a cross-section through the dam, though these may be incomplete, and a preliminary visit to the reservoir by one of the Inspector's staff can be helpful to collect, or check on, details.

HISTORICAL DATA

Before carrying out the inspection, all inspection and other reports are studied and a note prepared of dates, descriptions of features, and recommendations which have been made. A study of early correspondence may be helpful in seeking out information on incidents before 1930. Searches of archives and early Ordnance Survey maps may not, in themselves, throw light on unusual events, but they may reveal something which helps to explain some feature of the reservoir. For instance, the date of construction, when considered with other works in the area, will provide useful background and it is helpful to know if the reservoir has been enlarged, and whether feeders have been added; also the presence of quarries shown on maps may help to date repairs or improvements. If a preliminary visit is made, discussion with elderly inhabitants can be helpful.

A historical resume is then prepared with dates, incidents and sources of information; (this should later be attached to the Inspection Report, so that it is available to future Inspectors).

THE RECORD BOOK

The statutory form of Report (under the 1930 Act) requires the Inspector to state whether the particulars in Section A of the Record are correct at the time of the inspection; to check this may entail some survey work. Sections B, C (i) and C (ii) are not specifically called upon to be checked, but it is usual to examine them as they cover major repairs, water levels. and leakages; (any measured leakages can be checked, against water levels and antecedent rainfall).

GEOLOGY

An appreciation of the geology of the reservoir is essential; for instance, seepage may occur through strata and settlement of the reservoir banks may result from mining activities. If such activities are possible, a brief geological report should be prepared and this may later be attached to the Inspection Report. Where there has been mining activity near the reservoir copies of mine plans are obtained and may be used to confirm the positions of faults and for checking settlements and strains in embankments. The quest for geological data may often bring to light other features such as old shafts or tunnels below the resevoir.

HYDROLOGY

It is helpful for the Inspector, when he visits the site, to have an idea of the maximum flood that may have to be passed. Often not all the characteristics of the catchment area are known, and one of the Inspector's staff may visit the site as mentioned previously - for instance, if feeders and other inflows to the reservoir are likely to be important and there is insufficient detail to assess their effects on floods, (e.g. in urban areas, the surcharging of stormwater sewers or, in some reservoirs, the function of catch-drains in diverting floods).

At this stage a quick and simple method of flood assessment is nevertheless desirable. The Authors consider that such estimates should be arrived at by at least two separate methods - either as a check, or to throw up possible inconsistencies. Thus, in addition to the 'envelope' method of the 1933 ICE interim Report, they also use a 'rational' approach (a modified B.D. Richards' method requiring about one engineer-hour and a computer). From these two estimates, the Inspector has some idea of the flood to be expected - or, at worst, of the uncertainties.

Hydraulic calculations then follow to route the flood and to estimate the capacity of the spillway. This frequently involves estimates of discharges through constraints such as culverts and bridges and in feeder channels. Where a reservoir lies alongside a river it is sometimes necessary to estimate the flood rise in the river if this could overtop the embankment and flow into the reservoir.

When the estimated floods have been routed, the Inspector should be in a position to judge whether the freeboard is adequate or marginal and hence whether a more sophisticated flood study is warranted.

THE INSPECTION

THE ENVIRONMENT

Having noted, from the preparatory work, things which should be looked at, or inquired about, the inspection normally starts with a drive by car to look at the country surrounding the reservoir. The object of this is to form an idea of the catchment area, particularly with respect to any recent changes which could affect the runoff. Such changes would include, for instance, the construction of a motorway, which could form constraints to a runoff during a flood and hence reduce the inflow into the reservoir. Large areas of new building developments, on the other hand, could reduce the time of concentration and increase the peak inflow. At the same time an idea can be gleaned of the general topography and possibly of the superficial geology of the area.

The Inspector must also assess during this stage the development downstream of the reservoir and the ease of access to the dam. The downstream development will have an influence on the standard of safety that the Inspector will apply. For example there are dams built near the foreshore which, if they failed, would result in little damage. Those located upstream of a thickly populated area,on the other hand, must clearly have a higher standard of safety. The accessibility of a dam could also influence the Inspector's findings. If access is easy it could make additional works comparatively cheap; if difficult, it is unlikely that the dam will be frequently inspected by the Owner's staff.

DISCUSSIONS WITH OWNER

The next stage of the inspection is normally a discussion with the Owner's representative, who should be present during the inspection. He is usually an Engineer, though frequently a Mechanical or Electrical Engineer. There is also commonly a waterman or reservoir keeper who either lives near the reservoir or inspects it regularly. Such men have sometimes been there for thirty years or more, and their verbal reports are invaluable. During discussions with the Owner's representatives the Inspector can find out his attitude towards maintenance and surveillance of the dam. This varies widely - from a reservoir with a resident keeper, with weekly visits by a foreman, to others which are not looked at between statutory inspections. During these discussions the usage and operation of the reservoir can also be clarified and the Inspector can look at any records kept by the Owner.

INSPECTION OF EMBANKMENTS

The inspection then takes place - ideally, on a dry (and warm!) day, with the reservoir full. This usually involves walking along the downstream toe of the embankments looking for signs of seepage or slips, and noting any provisions for drainage. At the same time, potential sources of damage to the embankment can be noted, e.g. trees, burrowing animals or paths worn by the public. A walk along the crest will indicate whether provision for wave protection is likely to be adequate and will give some idea of where the freeboard may be critical. Since a high proportion of dam failures have occurred through the abutments rather than the embankments particular care should be given to the inspection of the ends of the dam.

ANCILLARY WORKS

From an inspection of the spill provisions it is usually quite clear what is critical, e.g. capacity, downstream erosion, or blockage by trash. The Inspector may also have to consider the effects of overtopping of the embankment. Most grass or tree covered embankments can put up with some spill over them for a limited period provided this is not concentrated in one or two low points.

The facilities for drawing-off water from the reservoir must be inspected and, though probably not affecting the safety of the reservoir, it is customary to see them operating.

Particular attention should be given to any culverts or pipes through the dam whether these are for spill or draw-off, as these are a potential source of leakage and eventual wash out. It is not always practicable to inspect inside such culverts, either due to their small size or to the flow of water during the inspection — but it is certainly desirable. (One of the Authors failed to do this on one dam and it was later discovered that the brick culvert had collapsed and a large portion of the fill above it had been washed out, leaving the embankment as a hollow shell.) If the discharge through culverts is shut off, a leak can often be detected by ear.

MISCELLANEOUS

Before leaving the site it can be helpful to run down a check-list such as that given in the Appendix and to look at any items that may have been forgotten. The Inspector usually decides during the inspection what his findings will be and what action he will recommend. There may, however, be matters, such as spill capacity that he will want to study again before making up his mind.

Photographs taken during an inspection not only act as an aide-memoir to the Inspector when writing his report but help his assistants who may be doing the hydraulic calculations etc., but have never seen the dam. A few photographs attached to the report lend life to it and prove useful for the succeeding inspection.

THE REPORT

SOME SHORTCOMINGS

Apart from spill capacity, the more common problems met with seem to be the following:

(a) Seepage through the embankments or appearing somewhere downstream.

(b) Leakage around pipes, culverts or spill structures constructed through the embankment.

(c) Settlement of the embankment, especially in mining areas.

(d) Damage to the embankment by waves, animals or the public; and potential damage by tree roots or trees liable to be blown over.

Perhaps the most important factor affecting the Inspector's decision is the degree of attention paid to the reservoir by the Owner. The next most important factor is the potential damage that would be caused by the failure of the dam, especially possible loss of life. The Inspector will take into account the extent and reliability of data available and also the Owner's budget. Whilst it can be argued that this last factor should not influence the findings of the Inspector, in many cases it undoubtedly does. The Inspector would for instance be svmpathetic towards a private owner of a hammer pond and would do his utmost to avoid recommendations involving heavy expenditure unless there was a very real potential danger.

USUAL REMEDIES

The normal solutions to problems met on old embankment dams include the following:

(a) The collection and monitoring of seepages.

(b) Enlarging spillways and, perhaps, making the bank ant its toe safer for overspill.

(c) Adding weight, or drainage, at the downstream toe.

(d) Sealing off culverts or pipes which may cause leakage through the embankment.

It is sometimes necessary to have further investigations carried out (surveys, pits, borings, soil tests, etc.) before recommending a solution, especially in cases of potential instability, piping or settlement.

Though it does not appear necessary under the Act for the Inspector to include in his report matters of maintenance which do not affect safety, it is usual to do so. If such maintenance is continually neglected, it could eventually affect the safety.

PERIOD BEFORE NEXT INSPECTION

The Inspector, having written his findings, must then decide whether the reservoir should be inspected within the statutory maximum period of ten years. If he recommends work to be carried out on the embankment it is usual to ask for an inspection when these are complete, say, in one or two years. If there are doubtful matters on which the Inspector is not recommending any immediate work, or if the owner pays little attention to the reservoir, a period of three to five years is often recommended. In other cases, the full ten year period is allowed to run.

COST OF AN INSPECTION

A description of an inspection would not be complete without a reference to the cost. At today's salaries and under the ACE time scale a normal inspection would cost in the region of £300 — £500. With no data available, the cost can be considerably more, as geological and hydrological studies will be involved. Given ample data the cost should be less. Under the new Act, the Inspector will have available not only drawings but the reports of the Supervising Engineer and thus the inspection itself should, it is hoped, require less time.

APPENDIX — INSPECTION OF OLD EMBANKMENT DAMS

CHECK LIST FOR USE DURING INSPECTION

Environmental	Catchment Area	Type of Country
		Changes e.g. development, drainage
		Constraints: new or removed
	Downstream Situation	Extent of building
		Liability to flooding during spill
		Possible effects of dam failure
	Geology	General assessment
		Settlement records
		Locate faults
		Mining: past, future, location
	Low points in reservoir rim	Secondary dams
		Spill points
	Access to dams	For inspection
		For maintenance
General (Discussions with Owner)	History	Changes in ownership
		Changes in reservoir
		Past and present usage
	Inspections	By owner - quality and frequency
		Previous statutory inspections
		Other inspections
	Attitude of Owner towards	Staff responsible for reservoir
		Regular inspections, maintenance
		Expenditure on repairs
		Expenditure on investigations
		Possible reduction in TWL
		Possible abandonment
		Recommendations by Inspectors
		Liaison with Regional Water Authority
	Records	Reservoirs Act; Form F: Section A
		Previous Inspection Reports
		Other Reports
		Other records; Form F: Sections B and C
		Settlement records
Embankments	General	Overall condition
		Material used
		Abutments
		Crest settlements, displacements
		Crest width
		Effect of overtopping
		Orientation, exposure
	Upstream	Wave protection and erosion
		Pitching movement
		Shape of face and slope
		Freeboard
		Crest width
		Siltation
	Downstream	Shape of face and slope
		Seepages in face and downstream
		Vegetation
		Drainage
		Abutments
	Drawdown facilities	Culvert levels, sizes, capacities
		Culvert condition incl. settlement
		Control arrangements
		Condition and operation
		Safety of operator
		Leakage around culverts
		Other means of emptying reservoir
Flood Provisions	Culverts	Levels, sizes, capacities
		Condition
		Controls incl. operation
	Open channels	Levels and dimensions
		Condition
		Downstream erosion
	Possible Blockage	By flotsam, trash, rubbish
	Emergency spillways	Regularity, size, strength
Miscellaneous	General	Water level during inspection
		Weather during and before visit
	Feeders	Controls and diversion
	Photographs	Embankments
		Spillways
		Changes
		Damage

THE INVESTIGATION OF OLD EMBANKMENT DAMS IN GLACIATED VALLEYS

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SYNOPSIS

The paper reviews the problems of carrying out internal investigations to determine the condition of embankment dams with central puddle-clay cores constructed in glaciated valleys. General characteristics of fill and foundations are discussed in relation to defects likely to arise from the nature of the design and poor construction techniques.

INTRODUCTION

A full diagnosis of the condition of an old embankment dam cannot be obtained by external examination alone. Such a diagnosis requires a knowledge of the internal structure of the dam, of the nature and geotechnical properties of the fill and foundations and of the pattern of seepage within, beneath, and around the dam. Some information on the construction of the dam may be supplied by record drawings, but these have often been lost and even when they exist give little geotechnical data. More precise information, which must include observations of piezometric levels, can be obtained only by sub-surface investigation of the dam and its surroundings.

In the past, internal investigation of dams has usually been undertaken only when the dam is already showing signs of distress or when modifications to the dam or its ancillary structures are under consideration. Recent experiences with old embankment dams in this country suggest that investigations should be undertaken as a routine measure to obtain the geotechnical data required for proper care and maintenance.

Every site investigation, whether of a virgin site or of an existing structure, needs individual planning. Some general principles can however be formulated for a single type of dam in a particular geological environment. Many of the reservoirs constructed in the United Kingdom for public water supply are sited in upland glaciated valleys. Embankment dams built in this situation in the 19th and early 20th centuries were usually constructed of glacial soils with a central core of selected puddled clay extended down in a vertical-sided trench into bedrock.

This paper reviews the long term performance of these dams and the problems of carrying out investigations to locate and identify possible defects. Specific examples are drawn mainly from the investigations of the Withens Clough Dam which is also described in paper 5.5 of this symposium.

A full description and history of the dam are given by Arah (1) and are not repeated here. The dam, which is approximately 35 m high, was completed in 1894 and little is known of its early history or construction. The condition of both the draw-off works and the embankment had caused concern and an investigation was carried out in 1970 with the following objectives:

1. To establish the seepage pattern in the dam and foundations, in particular to examine the possibility of unprotected drainage into the foundations.
2. To determine the properties of fill and foundation material for stability analyses.
3. To examine a thick deposit of peat of unknown origin on the downstream slope.

DAM CHARACTERISTICS

THE FOUNDATIONS

Geological records of cut-off trenches, as given for example by Lapworth (2), combined with investigations of modern dam sites give us a clear picture of the foundation problems to expect.

The valleys under consideration may have been subjected to one or more glaciations. Bedrock may have been wholly or partially covered by glacial soils and it is not uncommon to find that neither the present topography nor the buried rock profile conforms to the text-book U-shaped cross-section. The predominant soil type is usually till, more generally known, but incorrectly so, as boulder clay, which often consists of silty sandy clay with gravel cobbles and boulders, but whose particle size distribution may range between clay and boulders. The drift sequence may also include lenses or beds of sand, gravel or laminated clays.

Watertightness of many reservoirs is dependent on the continuity of the glacial clays and may have been impaired by the excavation of borrow pits. 'Insitu' measurements commonly show that the permeability of the rock immediately below rock head is at least one order of magnitude higher than the overlying drift cover. The drift thus tends to form a confining layer to the rock aquifers and full artesian pressures may exist below the valley floor even before impounding. Many of the English dams of this type were constructed on sites underlain by Carboniferous rocks whose alternations of sandstones, siltstones and shales further complicate the pattern of groundwater flow.

The rock below the drift often shows evidence of shattering and drag effects of the passage of ice, together with the more deep seated effects of cambering, landslides and valley bulging. Foundation conditions in glaciated valleys are thus complex, and it is hard enough to conduct a satisfactory investigation of a virgin site without the added difficulty of probing it through an elderly water retaining structure of doubtful composition.

THE CUT-OFF

Graphic descriptions of cut-off trench construction are given by Watts (3) Excavation, which might well occupy two years, was carried through the glacial drift into rock until, in the opinion of the Engineer, a satisfactory watertight horizon was reached. In view of the geological complexities this objective was often found at considerable depths, in some cases down to 60 m, and the resulting trench had an irregular stepped profile. Substantial quantities of water flowed into the trenches and high piezometric levels led to heave of the floor and difficulties with backfilling. Contemporary accounts speak of spring flows being piped up the downstream side of the trench to be led into drains, sometimes silting up, presumably with puddle clay, and sometimes being grouted.

Towards the end of the 19th century concrete began to be used at the base of the trench and later as a substitute for puddle clay. The quality of this early concrete may, however, be suspect. Watts (4) for example, advocated that it should be placed as wet as possible without compaction.

THE CORE AND SEEPAGE

The central feature of the dam, both physically and in importance, is the puddle clay core. Contemporary discussions of dam design show that this was relied upon as the sole watertight membrane which must not and, if properly constructed, would not leak. Seepage was not expected to reach the downstream shoulder and drains were installed only to dry out the site at foundation level or to remove surface water from the downstream slope.

Although the principle of graded filters was put forward in 1872 by Latham (5) and successfully employed by Jacob (6) in Indian dams, it did not seem to find favour with Engineers in Britain. Stone toes were commonly used in clay embankments but neither these, nor drains nor the original river bed seem to have been protected from seepage erosion. Engineers did of course add their own individual features . J.F. Bateman (7), for example, advocated placing a layer of peat 375 mm thick adjacent to the core so that if a leak occurred the fibrous particles would be drawn into it and help to seal it.

BOULDER CLAY FILL

Till, or boulder clay properly compacted at a suitable moisture content, can make excellent fill but its use is not without problems. Excavation even with modern plant is impeded by boulders and the high undrained strength when dry, while wetting produces rapid softening. Glacial deposits are often highly variable in composition and it is therefore likely that fill was often extremely heterogeneous. It was common practice to attempt to zone the shoulders by placing material with the highest clay content nearest to the core with the more granular material on the slopes. Whilst sound in theory, success in practice depended on the materials available and the quality of supervision. Leslie's specification for earthwork published by Conybeare (5) shows that some Engineers took great care with their fill, while others were criticised for making dams like railway embankments.

Clay for the puddle core was selected with more care, even to the extent of mixing and passing material through pug mills.

The grading of many boulder clays lies with the zone regarded by Sherard et al. (9) as being susceptible to cracking. Such cracking is most likely to occur when fill is placed too dry. The natural moisture content of many boulder clays is near the Plastic Limit, and if placed as-dug or allowed to dry out the fill may be brittle.

Cracking may be induced by differential settlement of the foundations or zones in the fill, by irregularities in the core trench or foundations, by arching of the core on the shoulders or sides of the core trench and may be assisted by hydraulic fracture. Water passages through the fill may also result from the incorporation of granular material or from the inadequate compaction of lumps of stiff or hard clay. Not all properties of boulder clay fills are undesirable. The tendency for fill to crack is counterbalanced by the material's resistance to erosion by piping. Fill also has a low brittleness index (Bishop (4)) and this means that when slips occur down-slope movements tend to be slow and limited in extent. Shear strengths in terms of effective stress usually show low or zero values of cohesion, and this implies that for a given pore pressure ratio within the bank shallow rather than deep-seated slips are most likely to occur.

INVESTIGATION TECHNIQUES

RECORDS AND SURVEYS

Although many dams have been constructed to the same basic design, record drawings of the actual structure are vital to planning investigations. Without drawings, for example, the full geometry of the core and puddle trench cannot be determined. In the absence of drawings old editions of Ordnance Survey maps and plans may indicate original stream positions and ground levels. Foundation levels can be estimated from upstream and downstream ground levels but may be obscured by siltation and ancillary works respectively. Since fill and foundation are essentially the same material it may be difficult to identify the junction in boreholes.

Even where records exist it is important to carry out a detailed surface survey before any subsurface work is done. This should not only show the essential features of the dam and its surroundings but also the precise positions and levels of any known or suspected defects such as wet patches or slip scars. Borehole logs should be plotted onto transverse and longitudinal sections as the work proceeds so that results can be interpreted and changes made in the investigation programme without delay.

METHODS OF BORING

It is well known that tills which combine rock boulders in a clay matrix are amongst the most difficult of materials to drill and sample. If the larger boulders have been removed from the fill shell-and-auger methods may be satisfactory, particularly if 'U4' (100 mm dia) open drive samples are taken continuously or at frequent intervals. The method has the further advantage that the hole can be cased continuously and any sudden influx of water sealed off. For this reason flight auger boring is less satisfactory although faster.

Trial pits or shafts provide the best opportunity for examination of the fill but may well cost of the order of ten times more than boring to equivalent depths. Close timbering provision for pumping and careful supervision are essential.

Rotary cored drilling is essential to prove and investigate rock, and NX or 76 mm dia.holes should be regarded as the minimum size acceptable. Rotary coring in larger sizes can be an effective means of obtaining continuous samples, provided that double or triple tube core barrels and suitable bits are employed.

INSTRUMENTATION

Determination of internal pore pressure should be a major objective of every investigation. The standpipe or Casagrande piezometer is quite suitable for most applications except the upstream shoulder. The instrument is cheap, and reasonably simple to instal and read. With care and suitable sealing two or even three can be installed at different levels in a single 150 mm dia hole. For near-surface observations the driven type of piezometer described by Parry (11) avoids the expense of setting up a drill rig. A further advantage of the standpipe piezometer is that it acts as a simple movement indicator, the tube being bent or sheared as the slope deforms. Lowering a straight and closely fitting rod down the tube will indicate the zone of movement.

THE CONDUCT OF INVESTIGATIONS

THE CORE

Investigation of the puddle core should be designed to establish whether the core is functioning correctly, and, if not, why and where it is leaking. Measurement of piezometric levels is most likely to provide the answers to these questions since there is a low probability of identifying localised defects without a very large number of boreholes. If the core is satisfactory piezometric levels upstream should be high but respond to reservoir level, while piezometric levels downstream should be low and steady. Such observations require boreholes placed outside the core; observations within the core may be ambiguous. It is, however, more convenient to drill within the core, particularly if there are no record drawings and it is not clear exactly where the limits of the core may be. A hole drilled from the centre of the crest is fairly certain to encounter the full depth of the core.

Continuous undisturbed sampling of the core is advisable and since the samples are not usually required for triaxial testing they should be split and examined for signs of erosion and cracking. Boreholes should be continued into rock, particularly on the abutments where seepage may be passing around or below the core, and information will be gained on the general rock succession under the site.

A useful indication of the condition of the core can be obtained from measurement of its insitu moisture content and Atterberg Limits. Puddled clay had to be sufficiently plastic to be workable but not too sticky. Clays were often both wetted and 'toned down' by the addition of sand and gravel. In practice the liquidity index of the clay as placed would be about 40%. With seepage and consolidation the moisture content may have changed, and Bishop (12) has drawn attention to the possible gain in strength of puddle clay due to thixotropy. The liquidity index nevertheless gives a guide to the present state of the core.

At Withens Clough variations in the appearance and consistency of the core were confirmed by variations in liquidity index. The top 6 m of the core consisted of recognisable puddle clay with liquidity indices of 15 to 40%. The base of this zone coincided with a layer of boulders which was not identified in the site investigation but was picked out in the closely-spaced grout holes. The core at this level also tended to be very soft and wet. The core below this zone included another boulder layer and much more varied water contents, many of them below the plastic limit. The clay was stiff to hard and split samples showed that it contained narrow water passages up to 6 mm diameter with smooth mud-lined walls. This brittle section of the core was clearly leaking, and it seems doubtful whether this clay could ever have been puddled correctly. At greater depths and within the puddle trench water contents were consistently higher and similar to the upper zone.

Boreholes and hence piezometers at Withens Clough were installed in what was believed to be the centre of the core. Unfortunately the brittle zones were not identified until after piezometer installation. Some of the instruments, however, indicated high piezometric levels which varied with reservoir level.

THE DOWNSTREAM SHOULDER

Investigation of the downstream shoulder will usually be directed towards identifying the nature and source of movement and seepage. Piezometric levels in both fill and foundation are required and undisturbed samples should be taken for effective stress triaxial testing.

The boreholes should be sunk into foundation strata and specifically into the old river bed. At Withens Clough holes sunk to the known line of the river encountered what appeared to be boulders similar to those in the river downstream of the dam. Water injected into these holes appeared almost immediately at the downstream toe drain. It seemed likely that this free-draining layer extended up to the core and that the core was thus subjected to a very high hydraulic gradient without the benefit of filter protection.

Because the rock, and in some cases soils in the foundation, are highly permeable they tend to act as underdrains and piezometers installed at this level may show very low piezometric levels. This practice was followed in the 1957 investigations and meant that the pore pressure ratios used in stability calculations were far too low. Seepage will tend to follow any layering in the fill and it is important to instrument both median and near surface layers. Observations at Withens Clough showed that in one area, at least, flow was virtually horizontal through the bank and that here a representative phreatic line would be one drawn from reservoir level to the downstream toe.

In carrying out stability analyses the chief difficulty is obtaining and interpreting pore pressures. A fairly small number of triaxial tests will define effective stress parameters that are reasonably representative but in practice one can never have too many piezometers.

At Withens Clough studies of plasticity and liquidity indices were made to see whether the zoning of the fill indicated on the only surviving Engineer's drawing had any meaning. Little difference was detected between the inner and outer zones except that the former (and the core) contained less peat. Much of the fill and all of the foundation boulder clay was below the plastic limit.

A special problem of Withens Clough concerned the peat deposits on part of the downstream shoulder. To avoid sinking large numbers of boreholes the peat was sounded with a Mackintosh Probe. This revealed that the surface of the underlying fill was irregular and it appeared that a slip had occurred in the past. It was not clear however whether this took place before, after or as a result of placing the peat.

THE UPSTREAM SHOULDER

Apart from the state of the wave protection the upstream shoulder generally gives cause for concern only under drawdown conditions or when serious piping through the core is occurring or suspected. Modification of draw-off arrangements may lead to unacceptably high rates of drawdown.

Investigations of the upstream shoulder may be limited by these considerations and by the special physical difficulties that arise if the reservoir level cannot be kept down. The contractor must choose between using staging for his rigs which can be erected in shallow water, or pontoons which can be used in deeper water but are difficult to anchor on a pitched slope. Boreholes must be cased through the water and divers will usually be required to remove and replace pitching. Clients may impose strict conditions on the use of plant in order to safeguard water quality.

Instrumentation of the upstream slope also presents difficulties. Piezometers installed below water level must be of a remote reading type and must not be placed too deep in the slope if they are to be used to measure drawdown pore-pressures. Air bubbler tubes were used successfully at Withens Clough in standpipe piezometers but required close supervision in installation and some patience in reading.

Under normal conditions the nature and condition of the upstream fill may be considered of little importance. If the fill is clay, properly placed and compacted, it may in fact relieve the core of full responsibility for water-tightness for many years. The fill at Withens Clough, except in the peaty zones, was surprisingly dry. All the material tested was below the plastic limit and initial pore pressure measurements on undisturbed samples suggested that the clay was not fully saturated, even some 75 years after construction.

INTERPRETATION AND CORRELATION

One of the major difficulties in the investigation of dams is that of interpreting and correlating the information provided by boreholes. This seems to be no easier in the works of man than in natural strata. It appears that the fill of an embankment may consist of stratified layers which record not only the material dug from the borrow pit but the weather, the method of placing and the degree of supervision.

At Stocks Reservoir over 30 years after construction Kalaugher (13) found that wet patches in the fill could be correlated with the periods of winter shutdown recorded by the Engineer.

In one area of Withens Clough peat in the upstream and downstream shoulders, a change of puddle clay consistency and a belt of reeds on the downstream face all occurred at the same level, suggesting that a layer of poor quality fill extended right through the bank. One might speculate as to whether this zone represents the Inspector's week off.

A common problem is to find that a single borehole encounters anomalous material, the extent and significance of which is difficult to assess. In an investigation of the Lower Silent Valley Dam to determine its behaviour on rapid drawdown, one hole in the upstream shoulder penetrated a 600 mm layer of clay in predominantly granular material. An extensive layer of clay would have posed problems but would have been very difficult to trace and it was assumed, perhaps optimistically, that it was an isolated barrow load. At Withens Clough pockets of peat up to 2 m thick were found. Since there was no undue settlement these were also assumed to be of limited extent, except in the case described above.

CONCLUSIONS

A successful investigation of an old earth dam is one that will correctly and economically determine its composition and locate and identify any defects. Such an investigation requires not only a knowledge of the design and external appearance of the dam but also an awareness of what problems may have been caused by complex foundations, defective construction and the long term behaviour of the dam.

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9 Sherard, J L et al. (1963) *'Earth and Earth-Rock Dams,'* Wiley, New York.

10 Bishop, A W (1973) *'The stability of tips and spoil heaps.'* Q.Jnl.Eng.Geol. 6.

4 The value that is put on life in the analysis. It is worth noting, in passing, the point that appears from time to time to the effect that all you get out of an economic analysis is a fairly flat curve which does not provide much guidance to choice of flood. We must not judge a method by the answer it gives. If the procedure is valid and the numbers used are approximately right, this is a highly relevant piece of information.

K T BASS (Rofe, Kennard and Lapworth) :

In Paper 4.1 I refer to the ratio Probable Maximum Flood (or Estimated Maximum Flood) to Q_{150} covering a range of 2.8 to 8.9. Since the paper was written Mr Farquharson kindly supplied me with the examples he had worked out and refers to in Paper 4.7. To illustrate the large scatter mentioned Fig.A (p D4/24) shows the ratios EMF $_{obs}/Q_{150}$ and EMF $_{syn}/Q_{150}$ plotted on log/probability paper, which shows a distribution from 2.5 to 30, albeit skewed. Figure B shows the results broken down into four groups and plotted using a separate symbol for each group to demonstrate that no obvious regional distribution is apparent'.

C C PARKMAN (Ward, Ashcroft and Parkman) :

There has been an excellent discussion on this theme and I have confidence that all the relevant technical points of view have already been expressed, and I do not at this stage propose to repeat any of these. As a matter of policy, however, I should like to state the following points:

1 That engineers should remain fully responsible for the decision on this most important subject. If not, engineers could well see another inroad into their territory.

2 To this end I would hope that the final ICE Report would be clear, concise and definite.

3 The 1975 Reservoirs Act has been drafted, and places responsibility clearly on the Engineer. Let us see to it, therefore, that we do not just retain *responsibility* and *somebody else retains the* power.

Dr. K H M ALI (University of Liverpool) :

Paper 4.6 very properly drew attention to some hydrological and hydraulic problems resulting from spillway design not covered by the ICE Discussion Paper.

The authors considered the effect of increasing the head over gated and ungated spillways upon the coefficient of discharge and the pressure-distribution. One other important problem is that of air-entrainment on the downstream face of a spillway. It is generally known that the point where air-entrainment starts moves downstream with an increase in head. Approximate calculations can be made to predict the increase in depth resulting from the bulking of flow due to air[1]. This is important in assessing the adequacy of the side-walls of the spillway. Gated spillways complicate the calculations greatly.

The authors considered spillway gates of the Tainter type only. In some cases drum gates are very practical and are widely used in the United States. The drum gate usually floats in a chamber and is buoyed into position by regulating the water level in that chamber and the gate can be set to pass a desired discharge in a few minutes.

Model tests are usually required to study the characteristics of drum-gated spillways and the energy dissipators. The writer was involved in such study on a 1/36 scale three-dimensional model[2]. The variation in Cd for a given head as shown by that study is rather complex and would not have been obtained through calculations. It is interesting to note that the use of this drum-gate would have resulted in an increase in reservoir storage by 10% at an added cost of only 5%. Figures *A* and *B* refer. (p D4/26)

Recently some dam designs have tended to take long-term future storage demands into account. This was the case in designing the Llysyfran Dam, where construction of the dam was planned in two phases - in phase 2 the spillway crest level will be raised by 12.2 m. Several scale models were built and tested for discharges of twice the design discharge.Extensive studies were conducted on a 1/42 scale three-dimensional model of the spillway, stilling-basin, downstream channel, and about 300 m of the river valley[3].

Model tests are always advisable if the shape of the overflow spillway or the energy dissipator is not of a standard type, such as those recommended by the United States Bureau of Reclamation. This is true, for example, in the cases of drum-gated spillways, air-regulated siphon spillways, flip-bucket dissipators and convergent stilling-basins[4]. Lately, air-regulated siphon spillways have come into favour in the United Kingdom.

11 Parry, R H G (1971) — *'A simple driven piezometer.'* Geotechnique 21.

12 Bishop, A W (1946) — *'The leakage of a clay core wall'* Trans.Instn.Wat.Engrs. 51

13 Kalaugher, P G — *Personal communication.*

ASSESSMENT OF SELECTED DAMS IN NORTHERN IRELAND

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SYNOPSIS

Fifty-two public reservoirs comprising 56 dams, and four private reservoirs comprising seven dams, are generally described with respect to their location, geology, and broad foundation conditions, together with the outline topography of the areas in Northern Ireland where they are sited. A few more unusual dams are also described, with the methods adopted to overcome particular difficulties in each case.

General findings from the investigation of 12 large public dams and many other public and private dams in the Province are described, together with the method adopted to classify risks on a rapid preliminary basis from an assessment of data collected and external examinations made, pending complete and more detailed investigations in appropriate cases.

INTRODUCTION

The original Reservoirs (Safety Provisions) Act, 1930, together with consequent Statutory rules and Orders was not extended to Northern Ireland, although it is fairly certain that a Northern Irish Act will now follow soon after the Reservoirs Act of 1975. Nevertheless since the passing of the 1945 Water and Sewerage Act construction of all new reservoirs of any size in N. Ireland has either been carried out by, or supervised by, a Panel engineer as prescribed by the 1930 Act.

In certain cases Panel engineers have also been employed to make recommendations regarding modification of or repairs to older existing public reservoirs, and in Northern Ireland there are now approximately 60 publicly owned dams which include 17 large dams over 15 m high constructed between 1868 and 1970. With the exception of one rockfill embankment and seven concrete gravity type structures up to 40 years old all of the others are earth embankments which vary in age up to 135 years.

The author has been responsible for the inspection of various dams forming private reservoirs in Northern Ireland and also for the external examinations of 12 large dams and a number of others for the Northern Ireland Government prior to the introduction of new legislation. Some of the problems revealed by such inspections and examinations, and the methods used to classify results from the examinations so as to emphasise cases where urgent action was needed, form the subject of this paper.

EXISTING DAMS IN NORTHERN IRELAND

PUBLICLY OWNED

Twenty-two reservoirs with 24 dams have been constructed to the west, north and north west of Belfast on the tertiary Antrim basalt lavas or overlying boulder clays, and generally close to the coastal edge of the Antrim plateau. The basalts occur over practically all of Co.Antrim and extend into Co. Londonderry and into Co. Down and Co. Armagh. They are characteristically discontinuous, individual flows being lenticular in form and limited in lateral extent. While the cores of the flows are usually hard, compact, well-jointed rocks the top and bottom of each flow is usually more or less vesicular and decomposed, while sub-aerial weathering during deposition has often formed a layer of reddish lateritic clay at the top. Such variations make grouting difficult, and require extensive borings to determine the extent of precautions against leakage.

Due mainly to the suitability of local material as fill, and to the weathered and discontinuous nature of the foundations, all of the dams constructed in this area have been earth embankments, with the exception of one earth/concrete composite bank and one rockfill embankment.

Fourteen reservoirs comprising two mass concrete gravity and 13 earthen embankment dams have been constructed to the east south,and south west of Belfast on the Lower Palaeozoic rocks which are mainly slates and grits or, in a few cases, on overlying boulder clays. These rocks underly most of Co. Down with the exception of the Mourne and Newry granites, and extend over into the southern half of Co. Armagh. The bed rock is uaually massive strong and dense and has negligible surface weathering, but may be strongly jointed. Where it is intruded by Mourne granites, the rock may contain veins of sand and decomposed material caused by hot intrusive gases which extend in depth as relatively narrow fissures. Such veins are difficult to clean out, and the clayey material in them does not receive grout readily. An additional difficulty due to the nature of the rock is that of excavating any form of foundation trench to profile, and more extensive grouting may provide a cheaper and more effective cut-off than trench excavation.

Eleven reservoirs comprising one mass concrete gravity, two earth/concrete composite embankments and nine earthen embankment dams have been constructed to the south of Lough Foyle in Co. Londonderry and Co. Tyrone on the Pre-Cambrian schists and quartzites or on overlying boulder clays. The bedrock is usually hard, strong and durable and difficult to drill, and has joints generally tight close to the surface.

Two earthen embankment dams have been constructed in Co. Fermanagh east of Lough Erne on Old Red Sandstone conglomerates and sandstones, which are essentially impermeable except where deeply weathered or extensively shattered by joints or faults. Some areas contain weathered basalt dykes and old copper mine workings, increasing the difficulties of effective grouting.

Three single earthen embankment dams have been sited in Counties Tyrone and Armagh, in the area lying south-west of Lough Neagh, on hornblende schist, andesite and dolerite respectively.

PRIVATELY OWNED

Many small earthen embankment dams were originally constructed for power purposes in the valleys of major rivers such as the Bann, Main and Blackwater when the linen industry was active, and are now used for industrial purposes. A larger example, however, is Loughislandreavy Reservoir, west of Newcastle in Co. Down, which was formed in 1839 on the Mourne granites with four earth embankments, and impounds nearly 8000 ML. It was constructed to regulate the summer flow of the upper River Bann for the benefit of mill owners and fall holders with premises situated 20 km to 30 km downstream. Local sands and clay gravels were used to form the embankments, but no trench was excavated into the foundations to form a cut-off. There are also examples of substantial private reservoirs formed by single earthen embankment dams at Clandeboye and Corbet in Co. Down, and at Shaw's Lake in Co. Armagh, all of which are sited on the Palaeozoic slates.

TOPOGRAPHY

In the various areas described above reservoirs formed by dams have top water levels in the range from 7 m to 339 m.o.d. Belfast, and impound catchments rising to 670 m.o.d. Belfast. There are no natural lakes used as reservoirs with elevations outside these ranges.

Many sites tend to be blanketed with boulder clay which thins out and disappears on valley sides, and in some cases it completely fills and conceals a buried valley in the underlying bedrock. The boulder clay is generally thickest in Counties Tyrone and Londonderry, and reaches a thickness of about 20 m in the Lagan Valley south west of Belfast. Generally speaking no dams are sited immediately south and west of Lough Neagh, where sand and gravel deposits are extensive, or (with one exception which is heavily blanketed with boulder clay) on the Carboniferous limestone in Co. Fermanagh and the south west of Co. Tyrone.

The depth of peat cover on different sites is variable, and on the Co. Antrim basalts and also on the Co. Tyrone schists depths of up to 8 m have been encountered.

UNUSUAL DAMS

There are a few instances in Northern Ireland of more unusual forms of dam construction. The earth embankments completed in 1839 at Loughislandreavy Reservoir were originally intended to be formed up to 11 m above ground level with puddle clay cores, but such material was not available locally, and internal investigations have revealed little difference between core material and that used for general filling. However the design provided for waterproofing by the placing of a 1 m thick layer of finely cut and trodden peat, worked at natural moisture content against the upstream face of the proposed core and also beneath the gravel layer on which the pitching stones of the upstream face are bedded. Although there is a high phreatic surface in some areas and certain remedial works are required, the banks generally are still in sound condition with little deterioration. They are fully described in Volume 1 of the Proceedings of the Institution of Civil Engineers (1841) in a paper by J F Bateman.

The earth embankment of the Silent Valley Dam, completed in 1933, is underlain by a concrete cut-off wall constructed in fine silts and sands, gravels and boulders, which overlie the rock floor in places to a depth of nearly 60 m. To construct the wall to such an extreme depth a series of shafts lined with cast iron segments was sunk to rock under compressed air, and then used as pumping sumps in free air, following which the intermediate sections of trench were excavated in free air and lined with further segments. The trench is fully described in Volume 239 of the Proceedings of the Institution of Civil Engineers (1935) in a paper by G. McIldowie.

The rockfill embankment of the Dungonnell Dam, sited in a wet area where no clay was available and completed in 1970, was rendered impermeable by means of a multi-layer asphaltic lining laid on the dam's upstream face, and connected to its underlying cut-off wall and grout curtain. To restrict the degree of settlement of the under-lying embankment to limits permitted by the lining, material sluicing and compaction of the rockfill with heavy vibrating rollers was adopted. It was the first case of an asphaltic lining being used to seal an impounding reservoir in the United Kingdom, and it is fully described in Volumne 51 of the Proceedings of the Institution of Civil Engineers (1972) in a paper by the Author.

SELECTED DAMS IN NORTHERN IRELAND

EXAMINATION OF RECORDS

Some, though very rarely all, of the following Northern Ireland records were found to be available for each dam which was examined externally:-

a) Parliamentary, contract, and working drawings, and drawings of subsequent repairs or modifications, on paper or linen.

b) Design calculations, technical papers, and specialist reports on soils, foundation grouting, earlier repairs, modifications and inspections.

c) Information regarding levels, flows, water quality etc., measured on site or recorded from site instruments, on tape or charts, and in log books.

There were relatively few examples recorded where pressure grouting had been used to assist the sealin of foundations. Ten of the seventeen existing large dams, however, which date from 1934 to 1970, definitely did have curtains under-lying their concrete cut-off walls pressure grouted with Portland cement. Of the remaining seven dams, six dating from 1868 to 1909 were built with cut-off trenches sunk to rock and filled with puddle clay, and one completed in 1933 was constructed with a deep concrete cut-off wall only.

Where cut-off walls had been formed in concrete it had apparently usually been possible to fill them out to the sides of the excavated trench by leaving intermittent bays as pump sumps In one of the ten instances which were grouted, however, this had not been possible owing to the ingress of water and side shuttering had been used with pumping between the cut-off wall being formed and the sides of the trench. In this case the excessive water in the excavation was largely due to layers of water-bearing strata at relatively shallow depths of about 10 m in a highly discontinuous and stratified basalt foundation, and grout holes drilled into the base of the trench for a further 15 m had shown artesian effects, with water under pressure rising from the bottom of such holes into the trench itself. The remedy adopted was to stop the ingress of the artesian water by caulking in pipes and plugging them, with their grouting postponed until the cut-off wall had been carried well up above the base of the trench.

EXTERNAL EXAMINATIONS

These were carried out if possible with grass recently cut, and with each reservoir at or near top water level, but with the proviso that a further upstream examination was desirable at a low water level. The principal headings which created problems in deciding the extent of comparative deterioration of each dam were:-

a) Structural Condition

b) Leakage

c) Settlements

d) Growths and Vermin.

e) Unrecorded Modifications.

STRUCTURAL CONDITION

There were frequent examples of dislodgement of isolated stones in upstream pitching, and of loss of fines through open joints from the underlying filter layers. In only a few cases had concavities developed in the pitching which showed up as hollows in the waterline at or near top water level.

Some of the instances of crest settlement correspond to concavities in the upstream pitching, and some occurred where embankments turned through an angle. No case was found where there had been any significant upstream slip.

A few instances of downstream movements of varying degrees of severity were detected, none of them involving large dams, which ranged from slight surface cracks and outward bulges of the downstream slope to complete failures where shallow circular-type slips had developed. None of the latter had penetrated back to the upstream side of its embankment, and in no case showed a run of water in or at the toe of the downstream slope of the embankment. A lack of recorded data made it difficult to assess externally the severity of each movement, and such cases were generally recommended for continuous monitoring and internal investigation so as to decide further action. The structural condition of spillweirs, spillway channels, eduction towers, access bridges, outlet culverts, bywash channels and ancillary works was generally found to be good so far as cast iron, concrete and masonry items were concerned, but there was often severe deterioration of associated timber and mild steel structures remote from dams, such as at inlet sluices, and in these situations such items might be better constructed with more durable materials.

LEAKAGE

At a few of the smaller dams there were surface areas of standing water with growths of rushes some distance out from the downstream toe, which in one or two cases showed a flow into an adjacent drain. The water flowing in such cases was clear and not near any part of the adjacent embankment, but all such flows were recommended for investigation and continuous gauging in relation to reservoir level. It was also recommended that proper surface drainage should be ensured in the vicinity of the downstream toes of some embankments which had flat areas, or areas with reverse falls which could grow rushes and remain damp, thus disguising any leakage which might commence in the vicinity at a later date and reduce the available factor of safety.

SETTLEMENTS

Vertical settlements were found to have ranged from about 10 mm in a recent rockfill embankment 16 m high to approximately 350 mm in an earthen embankment 12 m high, and in the latter type of bank had often resulted in a considerable loss of original freeboard. In several of the higher earth embankments it was found that metal channels supporting the access bridge out to the eduction tower had been anchored to the top of the tower at one end, and to a heavy pad stone on the embankment crest at the other, and also had been supported rigidly on top of intermediate piers founded on rock. In some of these cases the channels were found to be bowed downwards, corresponding to the crest settlement at what was the hightest section of the embankment, and in one case had broken the surface plating on the top of the tower to which they were attached. These conditions could be easily remedied by restoring the original freeboard, and in new designs by avoiding rigid connections to the embankment crest.

GROWTHS AND VERMIN

In addition to grass in the ungrouted pitching of overflow channels and any open joints of slabs, various trees and ornamental shrubs had become established on a few banks and grown to considerable size when left undisturbed. The root pattern of trees commonly found, such as the willow, alder, ash, hawthorn, sycamore etc. spread over an inverted cone typically 3m to 6m in diameter with a main tap root lm to 2m deep, the diameter of the cone and depth of roots below the ground surface depending on the type and age of tree concerned. With the exception of the willow and alder the trees found required aerated soil for their roots, which did not therefore tend to penetrate into any permanently saturated layer in an embankment. Trees had frequently become established near crests and waterlines of certain reservoirs, where their limited root penetration had increased the danger of their falling during a gale. The life of such trees is limited to 100 to 150 years and they will eventually die and rot, but if cut down when young their roots may last in the ground for 60 to 70 years before they are disrupted.

There were some private dams with extensive growths through their upstream pitching and on their crests which had resulted from plantings many years ago. When deciding whether to cut down or pull out such growths and stumps it was essential to identify the species concerned, since a tree such as the ash generates a springy tenacious and wide-spreading shallow root system under pitching, which will disturb a very large area if any attempt is made to pull the tree out with mechanical plant. In the author's opinion all growths through pitching seem to progressively disrupt it unless they are arrested, and, they cannot easily be dealt with chemically, but on downstream slopes it may often be possible to assess a root pattern as lying entirely in the relatively flexible porous general filling, and by cutting down and poisoning the stump with a chemical like 'Amcide' to allow it to rot away rapidly in about ten years with little real harm to the embankment. There was little evidence found of troubles arising from the activities of vermin on earthen embankments, although one or two rabbit holes were detected near the crests of some dams where extraneous growths provided cover. It would appear that clearance of such growths should reduce future troubles from vermin generally.

UNRECORDED MODIFICATIONS

These were found at several of the older embankment dams, and had apparently been carried out generally for the purpose of increasing storage in the reservoir concerned. There were four instances where flashboards had been placed on top of the spillweir without any other modifications to restore the consequent loss of freeboard. Such flashboards had a low coefficient of discharge compared to the original spillway profiles, and were particularly vulnerable to malicious damage compared with the original mass concrete or masonry structures. There were three examples of direct heightening of small embankment dams where the slope of the upstream face had been steepened to retain the original crest width at its new level, with a new layer placed on the downstream side and at the original slope. Two of the heightenings were not shown as such on available records, but in the Author's view any marked steepening of the upper section of an upstream face is usually strong evidence that the original crest was at a lower level. There was, incidentally, only one record of a large dam having been heightened in Northern Ireland, the dam in question being a concrete gravity dam with heightening effected by a change in the type of spillway structure rather than by any direct structural alteration.

CLASSIFICATION OF RISK

Internal examination of an embankment dam is obviously an essential part of a comprehensive and detailed investigation. However, such investigations take time, and in order to allow broad and rapid preliminary risk assessments to be made from the external inspections of 48 assorted dams they were classified as follows:

'CLASS A' DAMS, needing no obvious attention or repairs, where only completion of standard data and drawings was required, together with site records in standard form for leakages, movements, pressures, levels, overflows, compensation flows, raw water analyses, damage, repairs and previous inspections; also where standard maintenance procedures and implementation of a general policy for instrumentation were needed.

'CLASS B' DAMS, as 'Class A' above where there were no signs of rapid deterioration, but where repair or replacement of defective items or investigation of apparent deterioration was needed, or cases where spill-weirs and channels catered for more than 50% but less than 100% of the estimated maximum flood required to be accommodated with prescribed freeboard.

'CLASS C' DAMS, as 'Class A' above but where there were external signs of rapid structural deterioration and early action was needed, or where outlet pipes passing beneath embankments were liable to settlement and fracture or were controlled on the downstream side, or where the reservoir could not be rapidly emptied, or cases where spillweirs and channels catered for less than 50% of the estimated maximum flood required to be accommodated with prescribed freeboard.

'Prescribed freeboard', in the case of both earthen and rockfill embankments, was taken as 900 mm between crest and maximum flood level where no wavewall existed, and as 600 mm if a wall had been provided. Such standardization was considered acceptable for comparative purposes, since in almost all cases the maximum fetch was less than 1.3 km and almost all upstream slopes were similar, being pitched up to near crest level with slopes of 1 vertical to 2.5 or 3 horizontal.

Estimated maximum floods were computed on the basis indicated in the Institution of Civil Engineer's report on 'Floods in Relation to Reservoir Practice' and will require to be re-assessed when the proposed Institution guide 'Floods and Reservoir Safety' is issued in tangible form. However it is possible there will be little change in the comparative classifications already selected, unless it is decided there is absolutely no risk to life in the event of failure in any particular case.

Where possible spillway deficiency is a factor in Northern Irish dams roughly only half of such cases appear likely to involve a risk to small communities in the downstream valleys, but in this connection it would seem desirable to define more closely how the 'community risk' aspect of road and rail bridges and level crossings in valleys below dams should be regarded, since in some areas they occur fairly frequently, and in accidents involving any form of public transport a large loss of life could be involved.

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INVESTIGATIONS, PROBLEMS AND REMEDIAL WORKS AT WITHENS CLOUGH

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PARTNER,
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SYNOPSIS

Withens Clough Dam is an earthfill structure some 35 m high built in West Yorkshire at the end of the last century. The paper outlines its construction and history, leading to investigations started in 1957. The problems revealed by inspections and these investigations are listed. Remedial works carried out in 1971 and 1972, including provision of a new drawoff system and a new core diaphragm, are described. An account is given of factors affecting the design and construction of the works and general observations are made.

Note: This dam is also referred to in Papers 5.3 and 5.7 presented to the present Symposium. It should not be confused with two other dams of similar ages and names in the same area: Green Withens and Warm Withens. The former is currently under investigation and the latter failed at the time of the remedial works at Withens Clough.

HISTORY OF THE DAM

Withens Clough Dam was built by Sir Charles Gott of Bradford between 1890 and 1894 to provide a gravity supply of 7 Ml/d to Morley, some 30 km away. In broad terms the geology of the site consists of alternating grits and mudstones forming the Kinder Scout series of the Millstone Grits, overlain towards the bottom of the valley by boulder clay. The dam site was apparently a steep rocky gorge cut by the stream through massive grits: a short distance upstream of the dam a transverse fault replaces the grits by mudstone which have eroded to form a wide reservoir basin holding some 1400 ML.

Figure 1 shows two sections through the dam. It was constructed of fairly homogeneous boulder clay fill with a 2 m - wide cutoff trench taking the vertical clay core below the general foundation levels. The crest is 5 m wide, 250 m long and 35 m above the lowest fill. Both shoulders sloped at 1 on 3. The lowest drawoff was a masonry culvert of internal diameter 1.5 m passing under the dam 22 m below crest level: at the centreline it was closed by an iron bulkhead from which ran the 460 mm cast iron supply pipe. The first control valve on this line was housed at the downstream toe. There was also an upper siphon drawoff system passing through the core 3 m below crest level: this was removed some 30 years ago.

No construction records have survived. The earliest drawing is a copy of a proposal for raising the retention level dated 1907: there are significant discrepancies between this and the few available later drawings. There are no records of any structural problems or remedial works except for the restoration of some 0.5 m of settlement in 1951 when one of the two spillways was lengthened.

THE INVESTIGATIONS

When ownership of the dam passed to the newly constituted Wakefield and District Water Board there was growing concern at the condition of the dam and its drawoff system. This was the only supply in the system at a pressure high enough to feed the Morley area, and its failure would have been unacceptable.

Superficially the dam was showing signs of seepage and settlement, and had a tip of peaty silt of unexplained origin covering the right-hand half of the downstream shoulder to depths subsequently found to be up to 6 m. The drawoff system, by now reduced to the one low-level culvert, was giving treatment problems whenever silty stormwater was impounded and was vulnerable to failure of the culvert, the iron bulkhead, or the supply pipe in the downstream culvert. With no valve on this pipe upstream of the dam toe there was moreover a risk of serious damage by erosion of the dam if the pipe or bulkhead had failed.

Construction of a new drawoff system was considered, with the possibility of gaining the benefit of extra storage by raising the reservoir level at the same time. These were thus two justifications for investigating the condition of the dam.

In 1957 Geo. Wimpey and Co Ltd carried out the first subsurface investigations, putting down 16 boreholes and installing 6 standpipe piezometers on the downstream side of the dam. These showed a complex drainage pattern in the shoulder, suggesting seepage through the core, non-homogenous fill, and drainage downwards to the foundations with no evidence of any filter or mattress to protect the fill.

In 1959 these results and the question of the future potential of the reservoir were referred to Binnie, Deacon and Gourley, who recommended support of the iron bulkhead by a concrete plug as a matter of urgency, further investigations to resolve the questions raised in 1957, and consideration of the raising of the dam in conjunction with new drawoff works in the light of such further investigations. The concrete plug was placed during the following year, and in 1969 Binnie and Partners were asked to review the 1959 report and subsequently to supervise further investigations.

These were carried out by Nuttall Geotechnical Services in 1970 and are described in Paper 5.3 presented to this Symposium by Dr. M.S. Money.

Briefly they included 300 m of boring, 300 m of drilling, and 60 m of augering. Over 80 water tests were carried out and 33 piezometers were installed, 5 of them in the upstream shoulder to be read by air bubbling. Laboratory testing also carried out by the Contractor showed that the core contained slightly more fines and had a somewhat higher Plasticity Index (19% compared with 13%) than the rest of the fill, and that the core and inner zones shown as 'selected fill' on one drawing contained rather less peat than the outer zones. Otherwise the materials of all the fill and the boulder clay foundations were effectively alike: a uniformly graded boulder clay with $\phi = 30^{\circ} - 35^{\circ}$ and c' = o, and bulk density of 2 100 kg/m^3. There was, however, evidence of inconsistencies such as cobble layers and peat pockets throughout the fill proper, one borehole in the upstream shoulder finding a peat layer some 2 m thick within the fill. The drainage patterns were complicated by these inconsistencies, by the reservoir effect of the peaty deposits on the downstream shoulder, and by apparently unfiltered drainage to grit exposures at the foundations and to the boulders of the old river bed. These seem to have been left under both shoulders as far as the edges of the core trench. Permeabilities of the orders of 10^{-7} m/s to 10^{-8} m/s in the core and 10^{-6} m/s to 10^{-7} m/s in the shoulders and boulder clay foundations were indicated.

THE PROBLEMS

By the end of 1970 it was apparent that the following problems required attention with varying degrees of urgency:

(a) the drawoff conduit should be relocated away from the fill

(b) facilities for drawoff at different levels should be restored without re-introducing pipework into the fill

(c) an outlet for more rapid emergency drawdown and for scouring should be provided and the upstream shoulder stabilised accordingly

(d) the peaty silt tipped on and apparently concealing a slip in the downstream shoulder should be removed and the shoulder stabilised as necessary

(e) the phreatic surface in the downstream shoulder should be lowered

(f) seepage through the downstream shoulder towards the open-jointed gritstones should be reduced.

(g) piezometric gradients within the downstream shoulder (as high as 1 in 1 from the core to the old river bed) should be reduced

(h) erosion of the core, indicated by waterways of a few millimetres diameter through undisturbed samples and by the erratic drainage pattern within the downstream shoulder, should be prevented.

(i) the adequacy of the dry-stone wavewall, spillways and spillway outlet channels should be checked

(j) damage to the protective pitching at the upper levels of the upstream shoulder and to the concrete facing at the middle levels should be repaired, and the exposed earth face at the lower levels should be protected.

Seven of these problems were apparent from the superficial inspections which led to the subsurface investigations. The investigations then found four more problems, including the core erosion which was regarded as a serious threat to the whole structure, and provided the parameters needed for the design of the remedial works.

THE REMEDIAL WORKS

It was clear that the new drawoff works would require a low-level tunnel around one abutment, and the topography, road access and siting of the treatment works all indicated that this should be at the left abutment. There was then a choice available between a free-standing drawoff tower in the bottom of the reservoir or a shaft at the reservoir edge with a trench to feed two upper drawoffs and a plugged and valved low-level tunnel from this shaft to an open drop-shaft in the reservoir bottom.

Protection against internal and foundation erosion required renewal of the core. Because of the uncertain state of the original core and its foundation it was considered essential to have a positive system of excavation and refilling taken below the original cutoff trench, rather than an injection system. The core refill should be flexible in relation to the shoulders, physically stable against erosion at any crack which might form, and chemically stable against seeping reservoir water, which is unusually acidic with pH values as low as 3.5 and free sulphuric and humic acids present. These requirements led to a design in which the core would be stabilised by grouting from tubes a manchettes, excavated under slurry in panels 610 mm wide and 6 m long, then backfilled with plastic concrete tremied under the slurry.

After checking the stability of the downstream shoulder at its original slope of 1 on 3 it appeared that the 20 000 m^3 of peaty deposits overlying the right abutment could safely be removed to assist drainage and show the condition of the shoulder.

During earlier stages of the investigation it was expected that promotion of the Overwood Reservoir by a neighbouring authority would produce a surplus of treated water which could be diverted into the Morley area, allowing the Withens Clough supply to be interrupted for two years. In this case it would be possible to strip out and rebuild the entire dam, at the same time building a new drawoff tower in the dry bottom of the reservoir. When costed against the remedial works described above the rebuilding was marginally cheaper and had the advantage of removing the uncertainties about the structure which remained after the investigations. Unfortunately the Overwood promotion failed, the Withens Clough supply remained indispensable to Morley, and it was necessary to continue with the remedial works in such a way as to minimise any interference with the supply. This led to the following programme.

Work started in 1971 on three contracts at the site. Foraky Ltd began to drive the drawoff tunnel from its downstream outlet around the left abutment and to sink the drawoff shaft at the reservoir edge to meet it.

Soletanche Co.(UK) Ltd, injected some 1 000 m^3 of cement grout to stabilise the core and began to grout the rock beyond the abutments. John Swale Ltd constructed an access road across the lower downstream shoulder to meet the crest at the right abutment, removed the peaty deposits and rebuilt the slipped original slope with broken stone, put in a new system of surface drainage, took out disused pipework from the downstream shoulder and built a flood-retaining bund to increase the capacity of the channel below the spillweir, which had been lengthened in 1951. The reservoir was in normal use during this period and was drawn down by a few metres at most.

In 1972 the reservoir was emptied at a calculated rate of 1.5 m/week without causing further damage to the upstream face. As the water level fell Foraky began to push the upper drawoff trench and the drawoff tunnel into the reservoir area and Soletanche began to excavate the core panels and place the plastic concrete diaphragm. Durhing this drawdown inflow was diverted from the reservoir direct to the treatment works via the original by-washes (peripheral ditches). After the reservoir emptied it was possible within 14 weeks to complete the bottom drawoff shaft and tunnel, to divert the drainage into it, to backfill the upstream length of the old drawoff culvert with plastic concrete, to place the broken stone covering the inlet to the old culvert and protecting the lower upstream slope, and to drive the last panel of the new core diaphragm through the masonry culvert and into the bedrock beneath. The contact between diaphragm and bedrock was grouted and a grout curtain 9 m below the base of the diaphragm was injected from holes preformed in the diaphragm at 3 m centres. The alluvium of the old river bed, up to 10 m deep, was grouted for a distance of at least 7 m downstream of the core. The upstream slope protection was patched with broken stone, the diaphragm was protected by a concrete slab (level with the pre-trench kerbs required for the diaphragm construction and so providing a hard surface to the crest road) and the wave-wall was reconstructed and more substantially in concrete panels with gritstone trimmings.

The remedial works started in April 1971 and impounding for supply was resumed in December 1972. The reservoir did not refill until February 1975: after slight spilling the level soon fell again.

The piezometers in the downstream shoulder now show little response to reservoir level. Surface drainage from the downstream shoulder has fallen since 1972. The new drawoff system is reported to help treatment plant operation.

The whole cost of the investigations and work carried out from 1969 to 1972 was £770 000, including engineering and supervision. The new drawoff system cost £325 000 and the cutoff works £288 000 out of this total.

VARIOUS RESTRAINTS ON THE WORK

The investigations were affected in a number of ways by the need to keep the reservoir operational at this stage. The upstream shoulder had to be bored and drilled from pontoons and staging and the upstream piezometers had to be read by an air-bubbler device. Both these restrictions increased the cost of work on the upstream shoulder and therefore limited its scope. The response of the piezometers to rapid drawdown could not be measured until the emptying in 1972 during the second year of the remedial works.

The injection of grout into the core was restricted by the risks of fracturing or of introducing local stresses which could lead to falls during excavation for the diaphragm. Sleeve grout quantities were checked as a control on cavities found at the initial drilling stage, and grout was injected to limited pressures and quantities at each sleeve following a predetermined pattern in order to consolidate the core in a gentle progression from the base upwards. Watch was kept for heaves or other local distortions during injection: none was measured although the core accepted 1 000 m^3 of grout.

The excavation of the diaphragm was restrained by the crest width which compelled the NCK Kelly - bar excavator, the CIS reverse circulation rig for rock excavation and the tremie-pipe system to operate as a train needing access from either end of the crest. Excavation was restricted to alternate panels, not within 3 m of any freshly - placed concrete, and only when the water level in the reservoir was at least 3 m below the spillway crest. The slurry was regularly checked for density, viscosity and sand content and contingency arrangements were made for rapid back-filling in case of sudden loss of slurry.

The plastic concrete mix was determined by the properties of the dam fill and by the aggressiveness of the reservoir water. It was designed to have the best available combination of freedom from segregation, long setting time, shear strengths no greater than those of the boulder clay, high strains without cracking, low permeability and chemical stability against attack by the extremely acid reservoir water. Various constituents were tried in a wide range of trial mixes used to make samples which were tested after heat-accelerated curing, (a) in triaxial shear at cell pressures simulating half and full core depths and (b) in accelerated percolation tests using highly acidic water driven by carbon dioxide pressure. Pulverised fuel ash (PFA) was found to reduce permeability and to increase chemical stability very effectively, and the mix adopted after the tests was as follows;

Ordinary Portland Cement	3.2 % by weight
PFA	16.3
Bentonite	1.4
Aggregate (12mm down)	57.3
Water	21.8
	100.0% by weight

Control samples for triaxial testing were regularly cast during the placing of concrete, and the rate of rise of the concrete in the panel was monitored as an indication of falls from the sides of the trench. None were found. Standby concreting plant was kept available: in the one case where a joint unavoidably went cold under the slurry the panel was re-excavated and concreted afresh.

The removal of the peaty deposits on the downstream shoulder had to precede the reservoir drawdown in order to allow the access road to reach the remote end of the crest before the start of the diaphragm construction. It was therefore carried out in transverse strips of limited width, each being backfilled with stone to the final profile before excavation of the adjoining strip.

The safe rate of complete drawdown for the 1972 work was limited to 1.5 m/ week, and this caused no further damage to the upstream slope. After refilling the reservoir by 7 m the water reaching the treatment works was found to be contaminated with phenols from a small amount of bituminous paint accidentally introduced into the wet tunnel, and the reservoir was emptied in 12 hours. The new stone toe, which had been pushed forward into the silts at the foot of the dam, survived this rapid drawdown without damage but the silt deposits alongside slipped in a number of places. This drawdown and the removal of the offending paint delayed refilling by 8 weeks.

GENERAL OBSERVATIONS

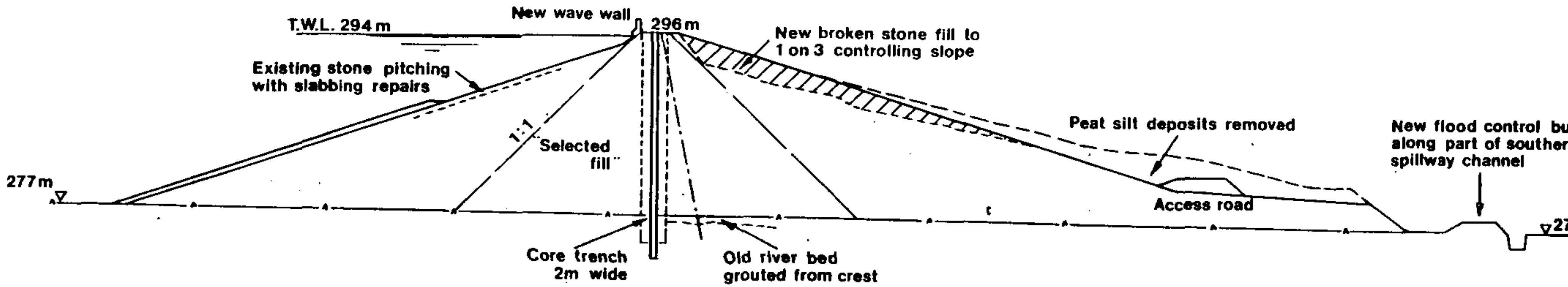
1. Investigations and repairs on old dams are subject to many restraints which do not apply to new works. Relatively slow and costly work is to be expected.
2. Even after extensive and careful subsurface investigations the designer of remedial works must accept that areas of ignorance remain. The design of remedial works must in consequence be more conservative than that of new works.
3. Reliable records of the original construction and subsequent works would have been of great value to the investigations and remedial work.
4. The investigations showed unsystematic inclusions of weak and permeable materials and unsystematic drainage patterns within the structure. The real dam differs significantly from idealised models whose stability can be calculated.
5. A major structural failure was concealed under the material tipped on the downstream shoulder.
6. Maintenance of the supply affected the investigations and works in many ways, most significantly by preventing the cheaper and more reliable expedient of complete removal and reconstruction of the dam.
7. Only subsurface sampling could have demonstrated the erosion of the core at such an early stage of its development.
8. Dams outlast several generations of their owners and therefore seem to have a unique permanence. Engineers know that this is fallacious and have a responsibility to make owners aware that regular investigations and remedial works are not aberrations but normal and essential procedures.

ACKNOWLEDGEMENTS

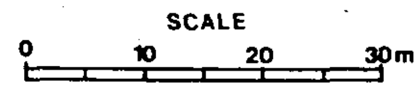
The Corporate Management Team of the South Western Division of the Yorkshire Water Authority, which now owns the dam, has kindly consented to presentation of this paper.

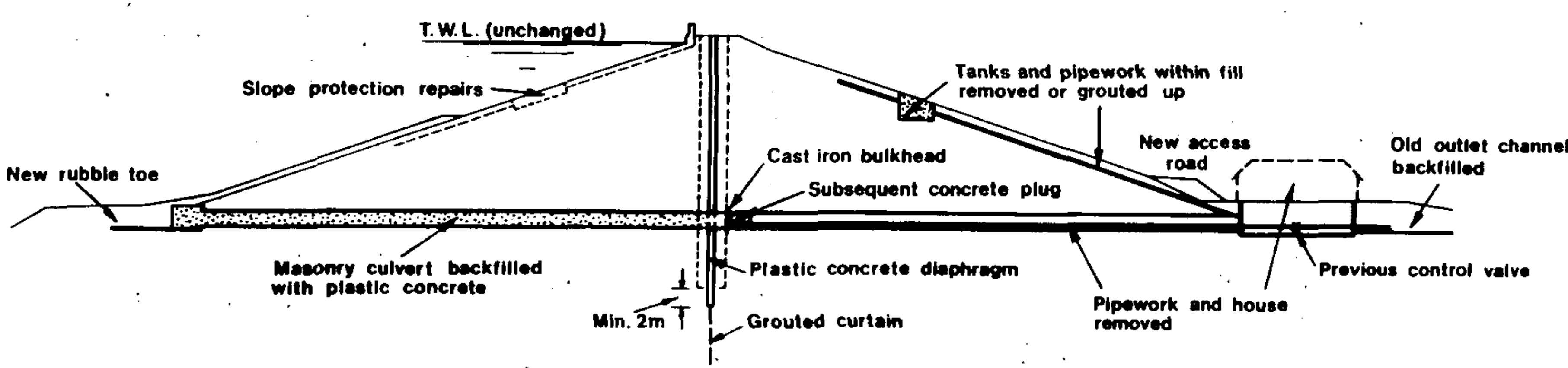
The Engineer and Manager of the Wakefield and District Water Board during the investigations was the late Mr. T.E.S White. Mr. P.J Gadd held this office during all the remedial works and until the dissolution of the Board.

The inspecting Engineer and partner responsible for the investigations and remedial works is Mr. T G Hammond, MBE.



SECTION AT GREATEST HEIGHT





SECTION AT OLD DRAW-OFF WORKS

PROBLEMS AND REMEDIES AT COWLYD DAM, NORTH WALES

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SIR ALEXANDER GIBB AND PARTNERS

SYNOPSIS

The paper describes the chequered history of this embankment dam, its state as indicated by geotechnical investigations, the problems of assessing their findings, the various remedial works required, the field and office procedures and action requirements which evolved over more than a decade for assessing the regularly received instrumentation readings, the further problems and required works, and the lessons learnt from this half-century old dam.

INTRODUCTION

Cowlyd Dam, a 14 m high earth embankment with a concrete core wall, was completed in 1921 downstream of an earlier water supply dam, to increase storage for the joint purposes of water supply for The Conway and Colwyn Bay Joint Water Supply Board and of hydro-electric power for the Aluminium Corporation Limited's power station at Dolgarrog. (Fig. 1.)

The original agreement provided for construction and maintenance of the dam being the responsibility of the Corporation, but for ownership to be vested in the Water Board. The water down to level 356.8 m.o.d. was for joint use, but below this level the water was for the sole use of the Water Board. In due course the Central Electricity Generating Board (CEGB) assumed responsibility for the power station, penstocks and dam maintenance, and in 1974 the water supply functions became the responsibility of the Welsh National Water Development Authority.

EARLY HISTORY OF DAM

The dam was originally intended to be of concrete and curved in plan, but cement supply and transport difficulties during the 1914-18 War caused this scheme to be abandoned and an earth dam to be built instead. Because, however, most of the trench had already been excavated for the concrete dam, it was decided to continue the new construction on the curve, Fig. 2.

The dam as built in 1921 was therefore constructed of moraine fill each side of a central concrete core-wall, Fig.3, which was founded partly on rock and partly on glacial drift, Fig.4.

The discussion of a contemporary paper (1) referred *'to the manner in which the banks were being formed, by dropping the contents of the four yard wagons from a considerable height'* (Fig 5), and continued: *'Well it was rather novel, one would not like to form a very definite opinion as to the soundness of such methods without actually seeing the work, but it sounded rather revolutionary to one who had been brought up in the theory that one should always spread the material of which a bank was formed in thin layers and take every precaution to consolidate it thoroughly by rolling or slewing the wagon road carrying the material on to the embankment. He could quite imagine the actual spot where the wagon load was dropped was made very solid, but one might hesitate a little as to whether they got uniform consolidation ... one would like to know whether such a form of construction gave satisfctory results, because, obviously, it must be a cheaper method than spreading the material in thin layers and consolidating it'*. The author replied that *'When the filling was tipped into the bank in dry weather, water was kept continually running on to it, in order to consolidate it further, and up to the present he had not noticed any settlement. The tipping of the filling in this manner was certainly a new venture, and he agreed ... that it departed from the usual practice, but with the very excellent filling available he did not anticipate any trouble'*. In his opinion the curve *'would add a great deal of strength to the dam, and possibly minimize some of the anticipated dangers referred to'* (i.e., fill not 'solid' and inadequate wall support).

These early opinions were destined soon to stand dramatic trial. The lake is peculiarly exposed to the prevailing and worst winds, which are funnelled through a 'neck' at the head of the valley. On the night of December 31st/January 1st 1924-1925, a storm caused overtopping of the dam and a V-shaped area of the entire downstream shell was scoured out down to foundation level, exposing the concrete core wall. The late Mr. J K Hunter, an eye-witness, told the present Author that in the morning men worked frenziedly to backfill the dam with anything to hand, whereby a disaster was narrowly averted. Subsequently the spillway crest was lowered in part and the wave-wall raised over its central length.

On 2nd November, 1925, the Eigiau dam in the adjacent valley failed, causing overtopping and failure of Coedty dam and the Dolgarrog disaster (2) (3).

In 1926, the Cowlyd dam spillway crest was again modified on the recommendation of Sir Alexander Gibb and Partners, who had been appointed to examine the causes of the disaster and to report on the performance of all the dams belonging to the Corporation, and also that year rock from the Eigiau-Cowlyd tunnel was placed at the downstream toe of the dam to give added stability. At last, in 1930, the Reservoirs (Safety Provisions) Act came into force after a number of failures and near failures, notably those referred to above. Thus within five years of completion the dam had had an already chequered history.

GEOTECHNICAL INVESTIGATIONS 1961 - 1962

By 1960 the dam showed signs of excessive distortions of the shells. The upstream shell had developed a hummocky surface which had cracked the concrete facing slabs. This appeared to be the result of differential settlement resulting from the method of construction, but stability in draw-down needed checking as a possible contributory cause. Restrictions on reservoir level and an investigation were ordered under the Reservoirs (Safety Provisions) Act.

A geotechnical investigation was carried out in 1961 to determine the cause of the distortions and to provid the basis for bringing the dam to present day engineering standards. The investigation included 19 boreholes, with indisturbed sampling of fill and foundation materials, six holes through the concrete core/cut off wall, pits, fluorescein seepage investigations, the installation of 12 stand-pipe piezometers and laboratory testing of samples. The work was executed by Soil Mechanics Ltd under the supervision of Sir Alexander Gibb and Partners. A shaft to foundation level had been envisaged just downstream of the core wall in the centre of the dam, but after attaining a depth of only 3.7 m work had to be abandoned on it due to the presence of large boulders. It was later realised that this was quite possibly the backfilled zone of the shell which had suffered the washout already described, Fig. 3.

In assessing the nature and boundary of the fill and foundation recourse was made to all versions of all borehole logs, whether the driller's, inspector's, engineer's or geologist's, preliminary or final, as well as to an examination of old drawings, as the slightest detail could assist in delineating the interface. Difficulty was experienced in determining this, although in places it was marked by a peat band, presumably representing unstripped areas, Fig. 3. The investigations confirmed the nature of the fill as being generally moraine, the greater part of which was a straight-graded mixture of clayey silty sand and gravel, but with coarser zones also evident, Fig. 6.

Pockets of peat were also found. The foundation was generally glacial drift, which varied from layers of gravel and boulders to boulder clay, as expected from the original trench excavation record. One of the holes cored through the central concrete wall produced a sample of concrete joined to stiff blue clay. The concrete wall proved to be of very variable quality, with honeycombing and porous concrete in the cores obtained.

Flurescein tests initiated in upstream boreholes to investigate the water tightness of the concrete wall, and seepage paths generally, were sometimes almost immediate in their downstream manifestation, particularly near the pipe-carrying concrete culvert which passes through the dam. During boring of one of the downstream holes it was noticed that a regular downstream water discharge, believed to emanate from a pipe left in the original cut-off trench, turned milky when boring reached a certain level. The source and pattern of seepage through and beneath the dam was then, and has continued to be, a principal problem at Cowlyd.

The results of triaxial tests on 102 mm dia. undisturbed samples to determine the shear strength of the moraine fill are shown on Fig. 7. For the foundations the clayey peat layer was the weakest. Stability analyses showed a need to strengthen both slopes.

The distortions were assessed as having been primarily due to continuing consolidation over a long period of time, following dissipation of excess pore pressures in the excessively wet loose fill subjected to the original construction process. The possibility of some local shear failure having occurred below apparent bulges still remained, however. Further distortion was expected to be minimal.

REMEDIAL WORKS AND INSTRUMENTATION 1963-1964

In order to remedy the deficiencies in stability, drainage and external protection revealed in the investigations, remedial works were designed, and executed in 1963-1964. The works, Figs 2 and 3, comprised the addition of a rockfill berm, riprap, new concrete slabs and wave breakers to the upstream slope, sealing of all cracks in, and undergrouting of, the concrete facings and wavewall at the top of the dam, the flattening of the downstream slope with additional rockfill over its central portion, the addition of downstream toe drainage, and the installation of an instrumentation system comprising 19 standpipe piezometers and 25 surface survey stations. An access road to the dam was also constructed. Higgs and Hill Ltd carried out the work. The borehole logs for the installation of the piezometers provided valuab additional information on the nature of the dam fill material.

BEHAVIOUR, PROCEDURES AND FURTHER WORKS 1964-1974

After completion of the remedial works the reservoir restrictions were lifted, but instrumentation readi were required to be submitted by the C E G B at regular intervals. Such readings have been made over t past ten years, and have shown the variable state of the cut-off, unsatisfactorily high water levels within parts of the downstream shell and satisfactory water levels within others. The regular readings comprise reservoir level, water levels in each standpipe, rainfall and V - notch weir measurements monitoring the flows at the downstream toe. All readings were plotted on one drawing, covering one complete year, on reading v. time basis. The V - notch readings, one of which is shown on Fig. 8, varied consistently with reservoir level, confirming a direct relationship. At first the individual downstream foundation and shell piezometric levels were assessed by plotting each set of water levels on two further drawings, showing cross sections and a longitudinal section. By comparing current and remedial works design water profiles a check could be kept on whether design levels were being exceeded. In due course, as data accumulated, this processing procedure was modified. The original readings-time plots were continued but it became possible, once a knowledge of the effect of the readings on stability had become established, simply to check standpipe water levels against previous maxima. Occasionally the maxima would be exceeded, an a check would be made on the effect of allowing a new maximum to become the acceptable limit.

With the better coverage by piezometers compared to those installed during the 1961 investigations it gradually became apparent that, whereas seepage profiles in the right bank and central sections of the d appeared to be satisfactory, those in the left bank section were consistently higher (4), despite the provision of a new drain, and that the maxima there were being exceeded with increasing regularity. In par icular at section E-E the water level in the shell was near the surface at downstream berm level, and conc was felt lest the phreatic surface was following a profile high within the shell from downstream of the c wall, which was there known to be particularly porous. Accordingly in 1966 three shallow, additional standpipe piezometers were installed up the slope. Readings from the augmented instrumentation confi a maximum water level of about 0.5 m below the external downstream surface and, despite the provisic in 1967 of a herringbone drain constructed in the lower half of the slope over a length of 50 m, levels remained high. More detailed analyses of the water level behaviour at this section were showing the exis tence of some strong correlations with reservoir level but also of some other patterns without obvious correlations, Fig. 9.

The possibility of leakage from the Eigiau-Cowlyd tunnel was taken into account in seeking explanatior for the source and patterns of seepage. It was eventually concluded that seepage both through and beneath the core wall, as well as seepage around the wall from the left bank hillside, were jointly respon Stability analysis for this condition showed a significantly lower factor of safety than adopted for the 1964 remedial works, and it was therefore decided to implement further remedial measures. These were constructed in 1974, when additional rockfill was placed over a length of 115 m of the left bank downstream shell, Fig.2, and 7 further piezometers were installed, Fig.10.

Throughout, previous seepage patterns and water levels have been used in conjunction with stability checks to interpret new readings, and every few years a planimetric and level survey of the dam has beer made. The processing system that evolved began and continues with a standard office 'action note', whi emphasises the uncertainties still pertaining to this old dam, lists the plots to be made, details the methc of their assessment and specifies the actions to be taken, and requires a report, to named senior enginee of any apparent untoward behaviour. In consequence, since the main remedial works, two further sets of works and instrumentation have been executed to remedy further suspected deficiencies. Monitoring continues.

LESSONS

1 The past is the key to the present.

2 An internal physical investigation of fill, foundation and seepage was a prerequisite to the proper diagnosis of, and treatment for, the unsatisfactory behaviour of this old dam. This probably applies to other dams also.

3 The shear strength of an unconventionally placed moraine fill has been measured.

4 For such a dam it is important to continue regular seepage monitoring, to evolve a workable standard procedure for processing and interpreting readings, and to provide a guide to consequential action.

ACKNOWLEDGEMENTS

The Author is grateful to the Conway Valley Water Division of the Welsh National Water Development Authority as Owners, to No. 4 Group, North Western Region, Central Electricity Generating Board, responsible for the Dam's maintenance, and to Sir Alexander Gibb and Partners, Consulting Engineers to the Board, for permission to publish this paper. He is also grateful to colleagues who have assisted him in its preparation.

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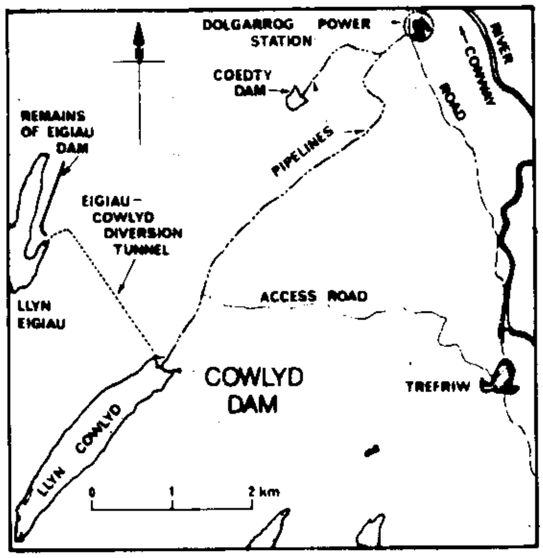

Fig. 1 — Location Plan

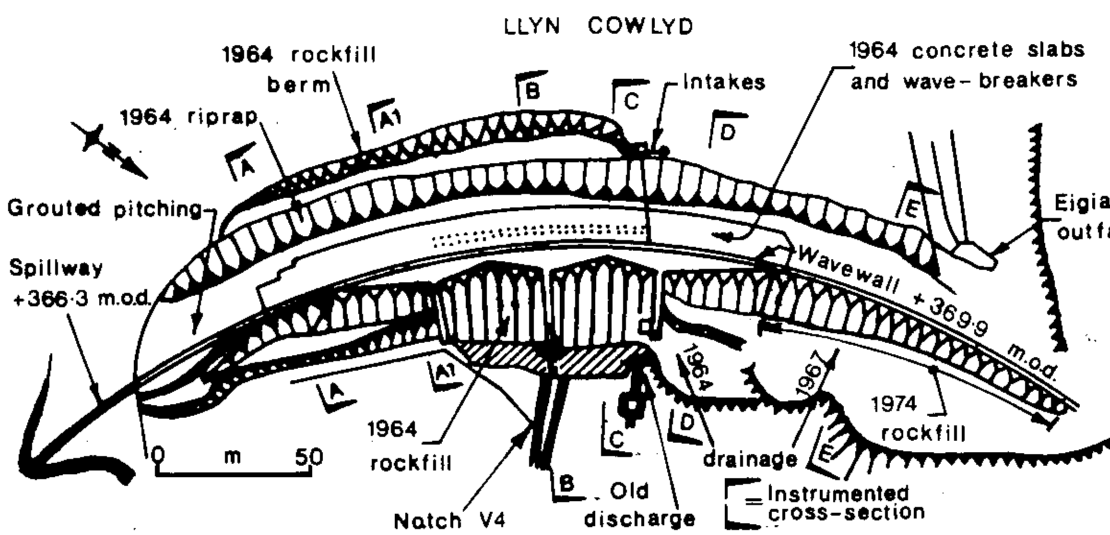

Fig. 2 — Plan of the Dam

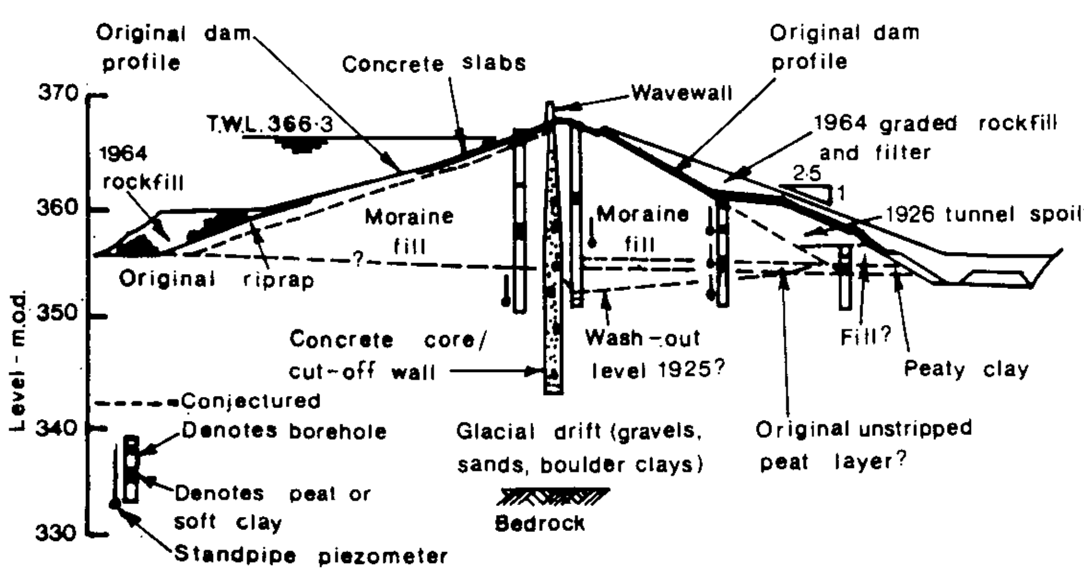

Fig. 3 — Dam Cross-Section C—C, showing Original and Subsequent Profiles

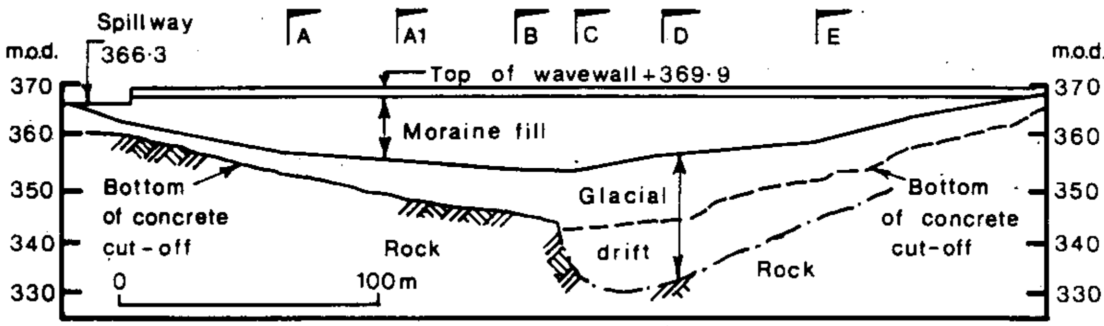

Fig. 4 — Longitudinal Section of Dam

Fig. 5 — Dam during Construction (from Farrington, 1921)

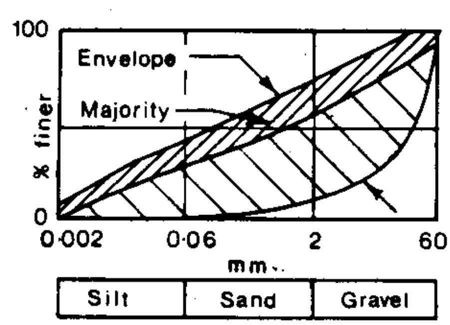

Fig. 6 — Grading of Moraine Fill

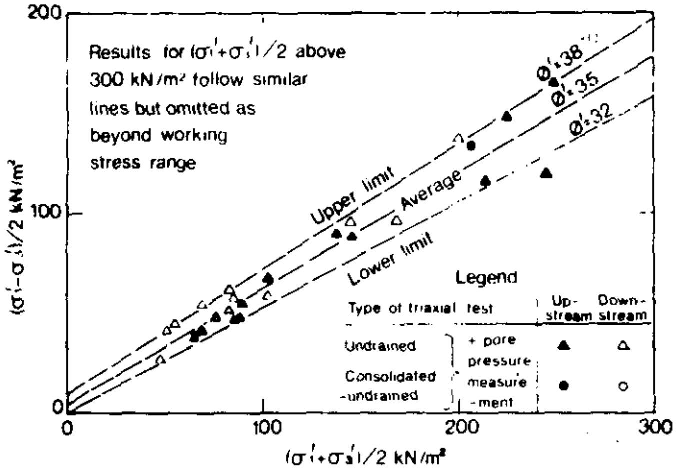

Fig. 7 — Moraine Fill Shear Strength, Effective Stress

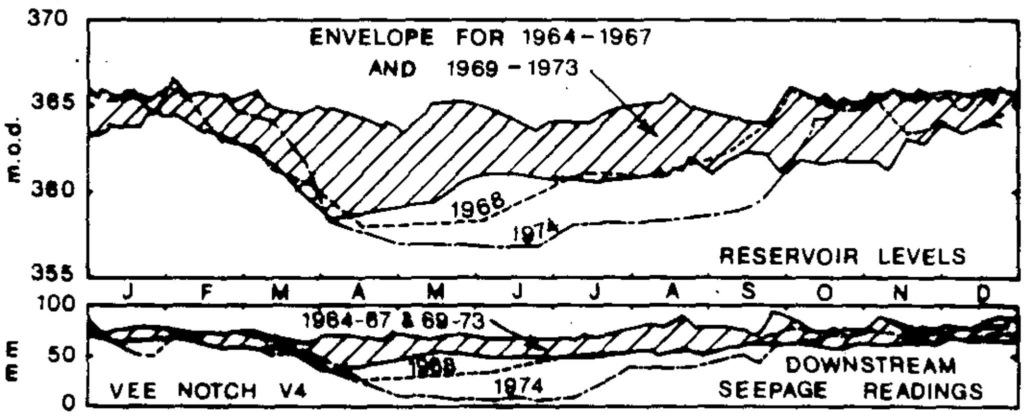

Fig. 8 — Downstream Seepage Relationship, 1964-1974

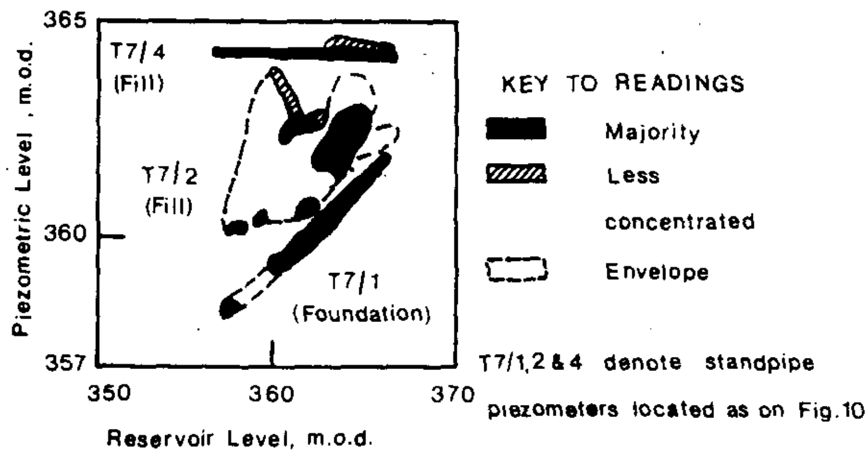

Fig. 9 — Piezometric Level Patterns 1964-1974 at Section E-E

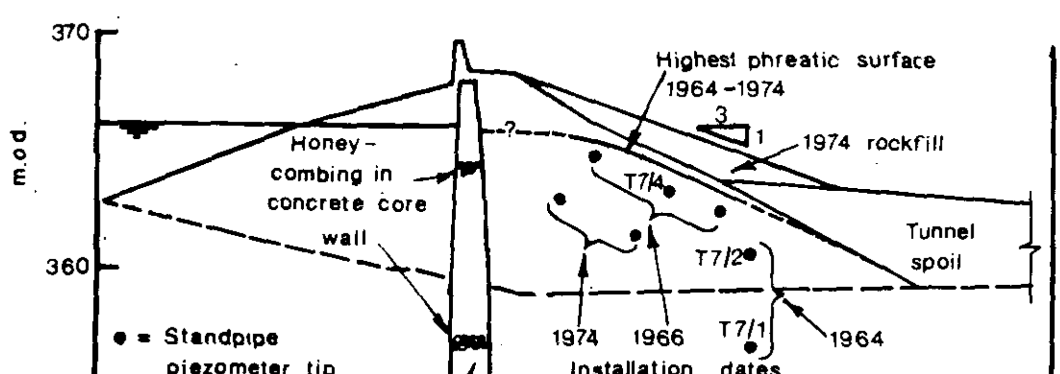

Fig. 10 — Left Bank Section E-E Water Levels and Additional Works

MAINTENANCE AND REMEDIAL WORKS AT SOME EMBANKMENT DAMS

A L Little, B Sc (Eng) C Eng FICE FI Struct E FGS MConsE

GEOTECHNICAL CONSULTANT
BINNIE AND PARTNERS

SYNOPSIS

This paper deals with four different classes of maintenance and remedial works on existing dams:

(i) Grouting

(ii) New cut offs

(iii) New earthworks (e.g. slope flattening, additional berms

(iv) Alterations in drainage

Examples are given of each class.

GROUTING

Grouting has been a traditional way of reducing leakage beneath dams since Thomas Hawksley used cement injections to seal rock fissures beneath the Tunstall Dam during its construction in 1876. With the modern development of clay/cement injections, grouting has begun to be used extensively for remedial works on existing dams which show signs of weakness or leakage. However, grouting for remedial purposes had been used as early as 1915-16 when 50 tons of plain cement grout were injected into the Lluest Wen Dam when it showed signs of weakness about 25 years after construction.

The Cwmtillery Dam, South Wales, had been severely damaged by mining subsidence. Although standing on a pillar of coal, this protection proved to be inadequate when deeper mining began in the 1950s; the culvert beneath the dam was damaged by mining subsidence and had to be supported with steel arches, the puddle core was damaged by differential settlement and began to erode into downstream drains. Clay/cement injections were made on a temporary basis in 1954. In 1972 when mining was complete and subsidence was substantially over, more permanent works were put in hand. These included a new lining to the culvert, replacement of the spillway and more extensive grouting using clay/cement, with pure cement injections in the bedrock.

Remedial works at Balderhead, Lluest Wen and Withens Clough have been described elsewhere [4] and the last named dam is the subject of a separate paper to this symposium (1). The works included grouting and the installation of plastic concrete diaphragms (see below).

At Balderhead Dam, swallow holes which had developed on the upstream side of the dam were filled with flyash/cement filler grout injected under gravity only. The main treatment along the line of the rolled clay core was from tubes à manchette at 3 m centres, injecting 0.42 m^3 of cement/bentonite grout at each sleeve(Fig.1). The object of the grouting was two fold. At the less damaged areas, grouting was intended to fill cracks and voids in the core, reducing its permeability to acceptable values and increasing the vertical stress in the core; at badly damaged areas, a new plastic concrete diaphragm was installed in a slurry trench and the function of the grouting was to limit slurry losses. The amount of grout injected in these areas was reduced [6]. The work was completed in 1968.

Piezometers were installed at about 20 m centres in the shale fill, just upstream of the core, to check the performance of the remedial works. Two years after the completion of the remedial works, depressions of the piezometric level of between 2 m and 3 m were observed at two of these piezometers. These depressions were traced to development of further cracking of the clay core and leakage which was cured with further grouting [5].

NEW CUT-OFFS

This has been interpreted in the widest sense to include the insertion of a new impermeable diaphragm into the core of an existing dam as well as in the ground beneath the dam.

Plastic concrete diaphragms were introduced into Balderhead, Lluest Wen and Withens Clough dams when signs of damage and/or excessive seepage developed. Briefly, the object of the new diaphragms was to limit seepage through damaged areas of the existing cores and cut offs.

At Balderhead, the new diaphragm was installed along part only of the length of the dam, because investigation had established that although about half the dam required treatment in the form of grouting, only about a quarter of the length of the core was so badly damaged as to require a new plastic concrete core (Fig.2).

The plastic concrete was placed by tremie in a 0.6 m wide trench excavated in 6 m long panels under bentonite slurry. The short panel lengths were chosen to minimise the effect of a collapse, should one occur because the reservoir behind the dam still held 10.4×10^6 cu m of water; stop ends were formed with 0.6 m steel tubes. A remarkable feature of the work was the verticality of the trenches, which the contractor was able to maintain in spite of the presence of quite large boulders in the rolled clay core. Except for one panel which was 150 mm out of line, all the rest landed on the 0.6 m wide flattened top of the concrete spearhead which formed a cut off in the valley bottom(Fig.1)at a maximum depth of 36 m.

At Lluest Wen, on the other hand, the investigation disclosed so much damage along the length of the puddle core that a new plastic concrete diaphragm was provided along the whole length of the dam. During the course of the remedial work, an exploratory shaft, 6 m (20 ft) in diameter, was put down on the centre line of the dam in the neighbourhood of the swallow hole which had developed in the crest. This shaft disclosed that the tunnel which passed through the puddle core of the dam merely butted against an upstream valve shaft and was not bonded into it. Movement of the dam had caused a 50 mm gap to develop; clay from the puddle core had passed through this crack and via another crack in a 150 mm (6 inch) dia drain to the downstream side of the tunnel plug (Fig.3). Presumably the process had gone unnoticed for years until serious erosion of the puddle core had caused the swallow hole. After the investigation, the plastic concrete diaphragm was completed across the shaft which was backfilled with compacted material. Piezometers were installed in the dam and discharge from drains measured to monitor its performance, which has so far been satisfactory.

Opportunity was also taken to improve the drawoff and the overflow works, and a downstream rockfill berm was added.

NEW EARTHWORKS

A dam across a small river near Bristol is believed to have been a mill dam dating from the eighteenth century. At later dates additions had been made to it until it was 10 m high providing water for industrial purposes in a works situated wholly in the valley below the dam. Although nominally a masonry dam, with a central spillway and chute, investigation disclosed that beneath a mortar skin the hearting consisted of loose stone blocks in a gritty matrix - all that remained of the original mortar; in fact, the dam was effectively a rockfill and gravel dam with masonry dam slopes - 57^{o} downstream, upstream near vertical. On two occasions between 1960 and 1968 the dam had been overtopped, and probably some 20 years previously as well, inundating the works. On the most recent occasion, part of the left abutment had been eroded, leaving the dam with only a tenuous connection to the left bank although the 'masonry' was undamaged. Following an inspection, the abutment was reinforced with tipped rockfill and the decision was taken to lower the upstream water level by 4 m to reduce the impounded volume and the head. Once the mortar skin was broken through, workmen had no trouble in *shovelling* off the 'masonry' of the dam to excavate the spillway down to the new level; the spillway chute was refaced with reinforced concrete.

Ruislip Reservoir is retained by an embankment about 6.5 m high which was probably constructed in the early part of the nineteenth century. During works on the crest to reinstate the freeboard, a slide of the upstream slope took place, although the slide was not obviously connected with the new works.

As at many of these old embankment dams, part of the upstream slope was very steep and this was the section which had failed(Fig.4a). Remedial works consisted of filling to produce an average upstream slope of 1 on 4.75. Analyses of the new slope using parameters obtained from a back analysis of the failure gave a satisfactory Factor of Safety. Even assuming a residual value for shear strength gave a Factor of Safety above unity (Fig 4b). When designing the new upstream slope protection, attention had to be paid to its vandal resistant qualities. The opportunity was also taken to improve the stability of the downstream slope by flattening it and raising the level of an existing berm.

Not infrequently, dams to retain industrial waste are built on a haphazard *ad hoc* basis as the occasion demands. Rarely are they designed by civil engineers; all too often they are the responsibility of an overworked, harassed plant engineer whose main preoccupation is to keep the industrial process going at all costs.

One such dam in the Midlands was 30 m high with a 1 on 2 downstream slope, built of local shale by an excavator driver with occasional visits from the plant hire firm's foreman by way of supervision (Fig.5). The dam retained sludge from an industrial sand washing process with a shallow depth of water to prevent the sludge drying out and blowing away. The overflow consisted of a 405 mm dia pipe with 150 mm dia tees which were blanked off as the sludge level rose. In 1963, the 60,000 m^3 dam cost only £8,000 to construct. It should be no matter for surprise that as the sludge level rose the dam began to show signs of distress about 2½ years after construction and developed small slips on the downstream slope. A stream of water, fortunately clear, emerged from a hole about 50 mm in diameter which developed in the fill near the downstream toe. It was also established that the 405 mm dia pipe was broken beneath the dam. The situation was rectified by placing a drainage layer and more fill on the downstream slope, flattening it to 1 on 3, and providing a substantial rockfill toe. A properly designed concrete overflow was also provided and the 405 mm dia pipe grouted up. About two years later, the dam was raised by 3.6 m. The junction between the old and the new work was not well made and, following a rainstorm, the water level on the upstream side rose well above sludge level; water began to leak through to the downstream slope. A plastic sheet was laid on the upstream side over the junction between old and new work and no further trouble has been reported.

Another dam to retain industrial waste in the west of England was rather better designed(Fig.6a). Unfortunately, the supervision resulted in the partially finished product differing from the Engineer's intentions. Waste water from the lagoon was to be collected at two 0.91 m square section culverts upstream, parallel to the dam axis, and brought down to a circular section tunnel consisting of a 1.83 m (6ft) dia pipe running from upstream to downstream beneath the dam. A vertical 1.83 m (6ft) chimney was provided at the junction(Fig. 6).There was also an emergency overflow.

Construction started in 1968 at the same time as the lagoon was being filled with waste; by a misunderstanding the shoulder fill encroached on the area which should have been occupied by the core. Some attempt was made to rectify this but was not wholly successful. Whereas the downstream slope was supposed to be 1 on 1.75 (30°), the shale fill was loose tipped and took up the much steeper slope of 38°. An upstream berm of shale was provided for access to the 1.83 m dia chimney. Early in 1969, the berm was found to be pushing the chimney out of the vertical and unsuccessful attempts were made to pull it back using wire ropes. This movement becomes significant in the light of subsequent developments although the most serious shortcoming of the structure was the flimsy nature of the square section culverts. These had 230 mm concrete block walls with 114 mm thick precast concrete top slabs reinforced with 13 mm bars at about 114 mm centres. In February 1970, when the embankment had reached a height of just under 18 m, a slip took place in the upstream slope and a suspension of waste began to discharge from the downstream end of the 1.83 m dia pipe into a stream. It is not certain if the slip in the upstream slope preceded or followed the escape of water.

It was strongly suspected that the upstream slope, like the downstream, was much steeper than the design (1 on 1.5, or 34°). It also seemed probable that the slope had been constructed Christmas tree fashion (Fig.6b), resting partly on the waste, but inspection was impossible because of the accumulated waste. The incident may have been caused by a stability failure of the slope which then disrupted the waste water culverts either at the junction with the 1.83 m pipe or elsewhere. Alternatively, as seems more probable, one of the square section culverts might have collapsed first, the subsequent erosion of waste removing the toe support to the slope and causing the slip.

As a first aid remedy, the downstream end of the 1.83 m pipe was covered with a graded filter which held up waste but permitted drainage; a plug of concrete was poured down the chimney. The flow of waste was stopped and it became possible to continue with more permanent remedial works. The upstream slope was restored by tipping material over the upstream edge of the dam and a 15 m (50ft) wide berm was added to the downstream slope, the 1.83 m dia pipe being extended beneath this berm.

A larger spillway was also provided. Alternative arrangements were made to deal with the supernatant water and storm overflows. It was considered advisable to clear out the 1.83 m dia pipe and backfill it with gravel. The downstream part of the pipe was found to be filled with waste, fragments of the culvert and shale from the upstream shoulder. About half way along the pipe, it had deformed by 150 mm so that its vertical diameter was only 1.67 m, and it was so badly cracked that its collapse was feared to be imminent; removal of material from within it was stopped. A concrete plug 6 m long was provided and gravel was packed into the pipe downstream of this plug. A second, permeable concrete plug was placed at the downstream end of the pipe.

It was necessary to finish the dam to its designed height of 21.3 m (70ft). In view of the previous occurrences, it was decided not to continue using hydraulic waste fill for the core but to complete with compacted sand which was sufficiently fine to retain the waste. For slope protection, a section of the upstream shoulder was of compacted shale.

Whilst this work was in progress, on 7th July, 1970, more waste began to discharge from a 150 mm dia land drain which had been used as an extension, through the new berm, of a 300 mm dia drain laid outside the 1.83 m pipe. On 9th July, the waste discharge stopped and clear water began to emerge from the downstream toe of the new berm on either side of the 1.83 m pipe outlet. On 16th July, a vortex developed in the supernatant water above the waste on the upstream side. This vortex was above the submerged inlet of the right hand square waste water culvert. About 127 m^3 of sand were tipped into this vortex and eventually plugged it. This greatly reduced the discharge of water downstream. Some observation standpipes had been installed in the embankment and the water level in one of them had risen 11 m on 15th July, but dropped to its previous low level on 16th July when the downstream discharge of water was stopped by the sand. On 20th July, a 1 m depression appeared on the dam crest immediately above the centre of the tunnel.

It was concluded that these phenomena were due to the following events:

1. Because its condition was so bad, it had not been possible to plug the whole of the 1.83 m pipe and part of it still contained debris and voids from the previous incident.
2. On 7th July, part of the core of the dam began to pipe into the damaged upstream part of the 1.83 m pipe; this piping extended back to the lagoon and waste from the lagoon also began to enter the tunnel.
3. After travelling along the pipe until it reached the concrete plug, the piping material found a path to the 300 mm drain. This drain had originally been provided to a tap-off box used in connection with the hydraulically placed core.
4. The piping material travelled along the 300 mm drain and escaped to the downstream toe via the 150 mm land drain.
5. After two days, the flow of waste clogged the paths by which it had been escaping sufficiently for the waste flow to be stopped except intermittently. A sufficient channel was left for water to continue to flow, however.
6. Removal of the waste and the consequent flow of water from the lagoon created the vortex; this path was choked by the 127 m^3 of sand tipped into the vortex. The loss of core material by piping caused the depression which appeared at the crest.

To remedy the situation, the new berm was excavated back for 6 m and the 150 mm land drain removed. A graded filter was placed over the end of the drain where it was cut and the berm replaced with compacted shale fill. Attempts were also made to grout the 1.83 m pipe from the surface, but only 5.2 m^3 was injected although the theoretical volume of the pipe is 100 m^3. There is no doubt that much of the pipe has now collapsed completely and the void filled with dam material, possibly with some waste and debris included as well.

ALTERATIONS IN DRAINAGE

Clydach Dam, South Wales. 9 m high, was built in about 1900. Following an inspection in 1970, drive-in type piezometers (3) were installed, disclosing a rather high water table in the downstream slope. Four equally spaced trenches were then dug into the downstream toe, filled with filter material and the resulting flow piped to a manhole where it could be measured and fed into an existing drainage system. Although the flow from the drains is small, it is considered that they will prevent shallow surface slips, especially near the downstream toe. Calculations based on parameters from tests on the embankment fill show that the overall stability is acceptable. Although coal mining has taken place beneath the reservoir and to within 220 m from the dam, it does not appear to have been affected.

An operation of a different and it is believed rather unusual kind was performed at Thorpe Malsor Dam in Northamptonshire. The dam, which was built in 1905, suffered a partial failure during construction. As a result, the upstream slope was flattened, a downstream berm was provided, extensive foundation drainage was installed and continuous pumping instituted from a 6 m deep sump near the downstream toe. In 1971 boring and sampling was carried out and piezometers were installed and observed for some time. These showed that the groundwater level was low and a stability analysis showed that the Factor of Safety of the dam had risen to a satisfactory value. The pumping was discontinued in June, 1972. Should the water level rise and the Factor of Safety begin to fall to an unacceptable level, the piezometric observations will give ample warning. The sump remains available and pumping can be resumed at short notice.

ACKNOWLEDGEMENTS

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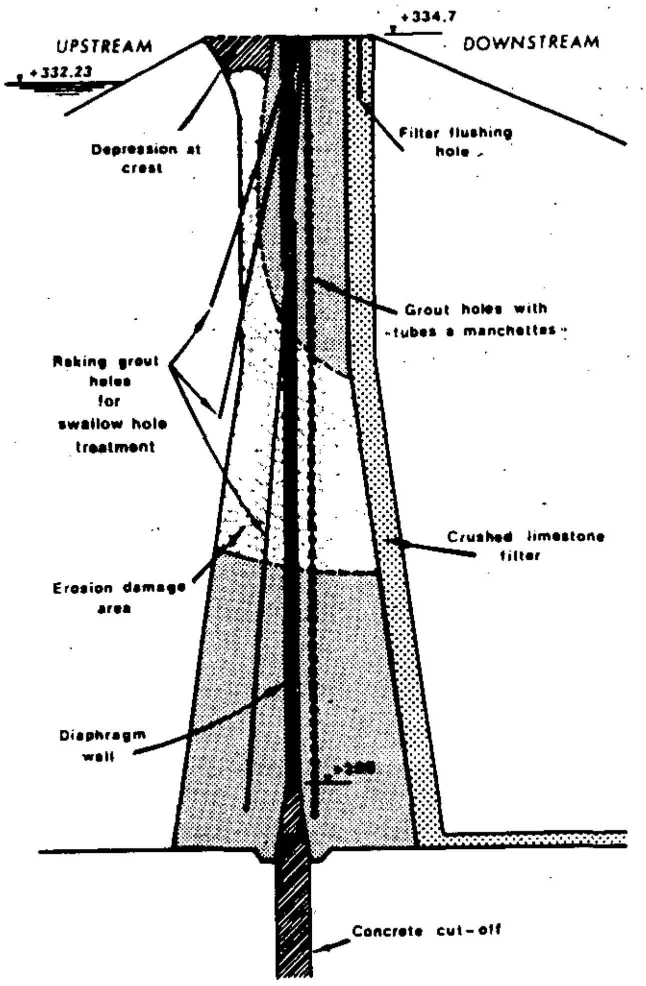

Fig. 1. Balderhead: cross-section of core

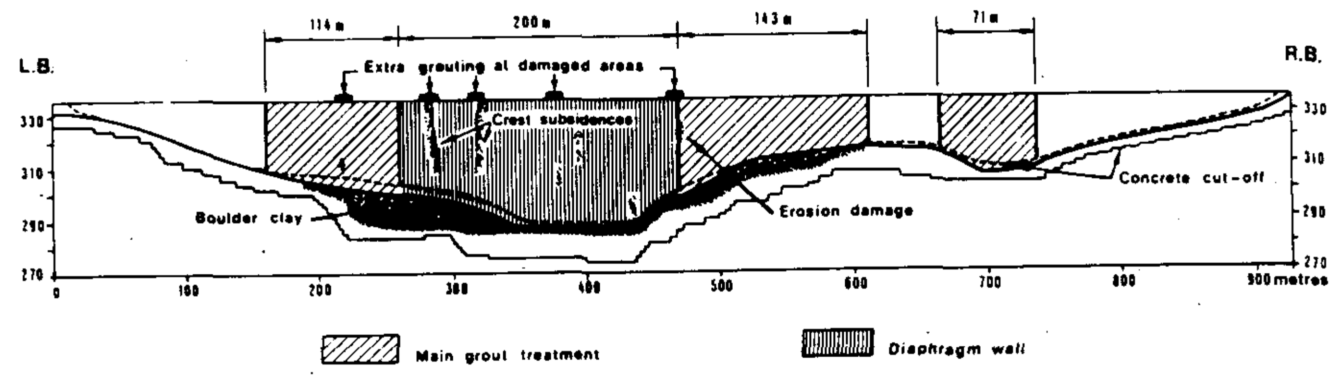

Fig. 2. Balderhead: longitudinal section

Diaphragm walls and anchorages. Institution of Civil Engineers, London, 1975, 23–26.

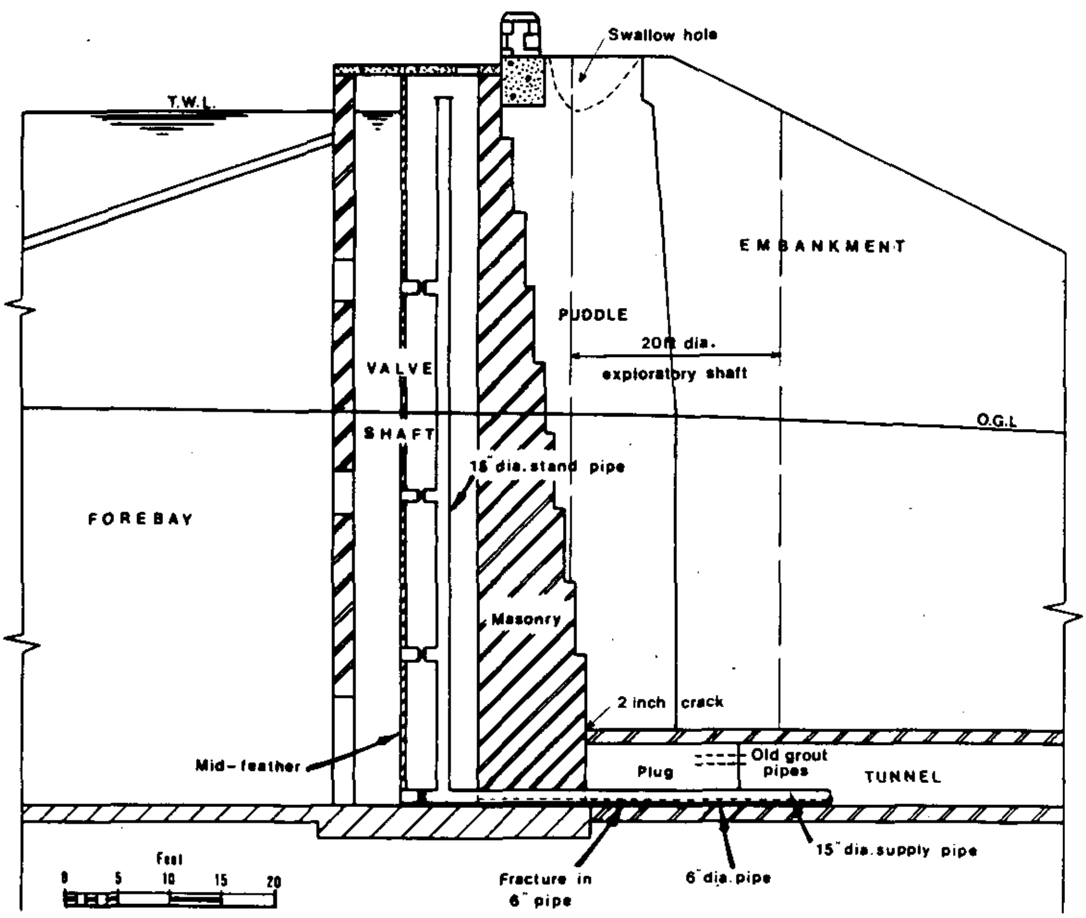

Fig. 3. Lluest Wen: conditions at valve shaft

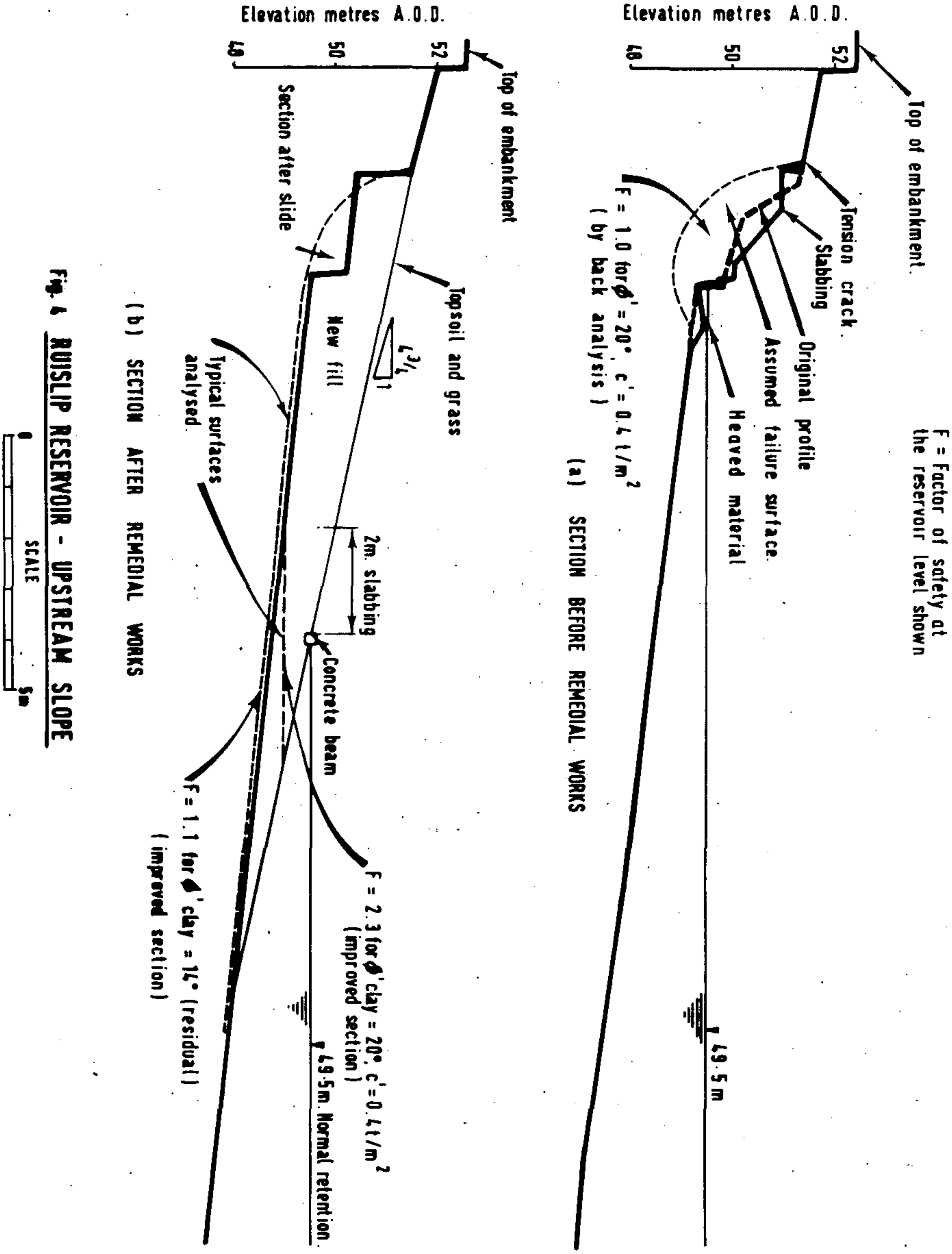

Fig. 6 RUISLIP RESERVOIR - UPSTREAM SLOPE

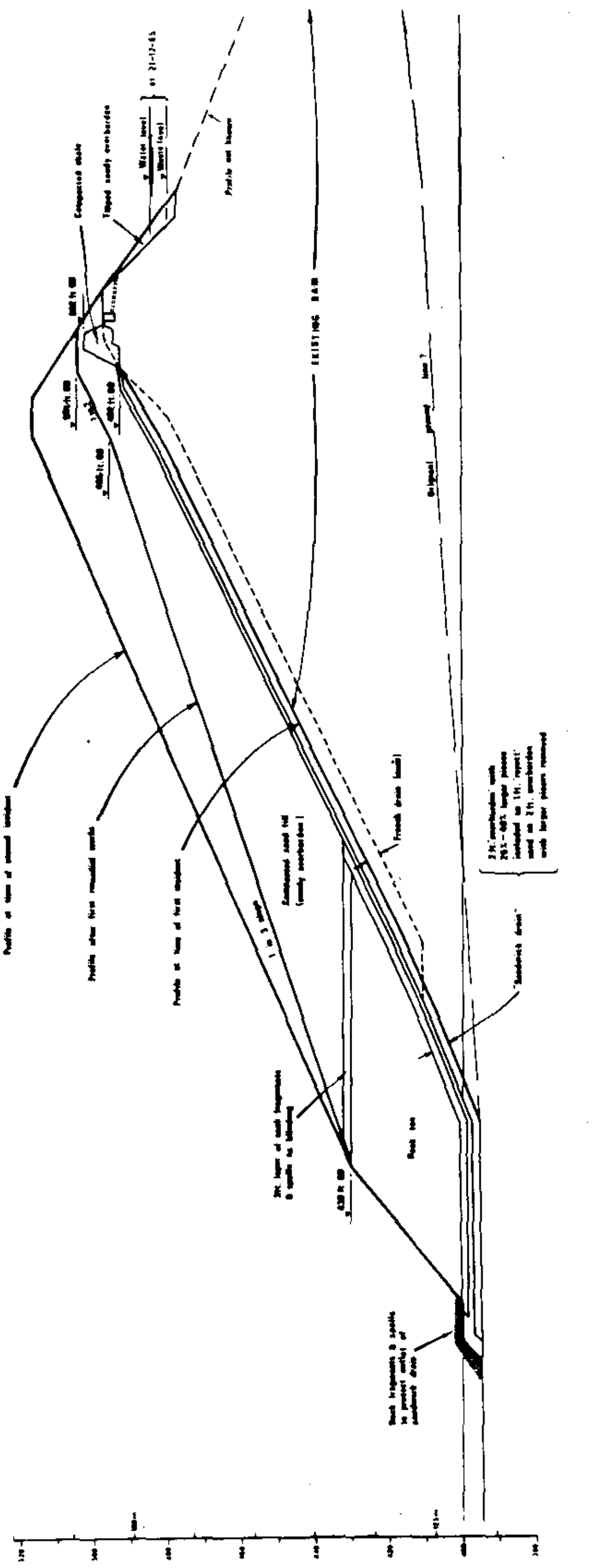

Fig. 5 MIDLANDS INDUSTRIAL DAM — APPROXIMATE MAXIMUM CROSS-SECTION

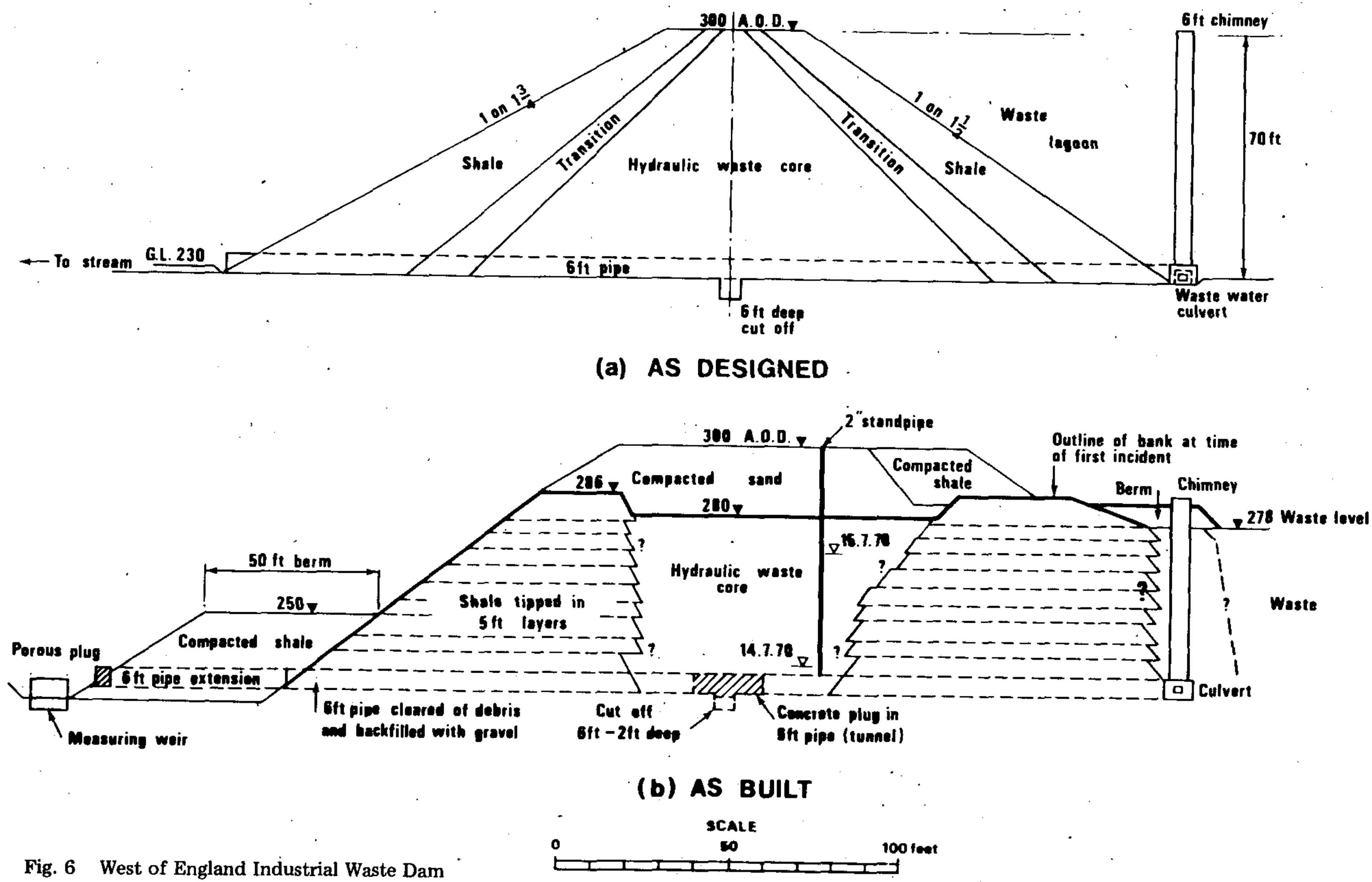

Fig. 6 West of England Industrial Waste Dam

REMEDIAL WORKS TO THE LOWER RODDLESWORTH AND RAKE BROOK RESERVOIRS OF THE LIVERPOOL WATER SUPPLY UNIT, NORTH WEST WATER AUTHORITY

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SYNOPSIS

The two reservoirs which form the subject of the paper are part of the system of eight impounding reservoirs with earth embankments on the 4000 h. Rivington Catchment Area in S. Lancs which was developed for the supply of water to Liverpool between 1850 and 1875.

The works consisted of: investigation of the embankments by means of bore-holes; the sinking of new valve shafts and the installation of pairs of new 610 mm valves in tandem in the scour tunnels; the removal of existing scour pipes and the lining of the tunnels upstream and downstream of the new valves. Ancillary works included the improvement of existing roads and the construction of new ones, and the laying of a 305 mm dia main to by-pass the reservoirs.

A major factor in the work was the controlled draining of the reservoirs with the attendant problems connected with the silt which both contained, and the necessity of keeping inflows to a minimum.

GENERAL DESCRIPTION OF LOWER RODDLESWORTH AND RAKE BROOK RESERVOIRS

The two reservoirs principally impound the waters of three rivers - River Roddlesworth, Rake Brook and Birch Clough.

They were designed largely for the purpose of supplying compensation water to the River Roddleswoth and they are connected by a channel, the invert of which is 3.25 m below the original T.W.L. The outlet from Rake Brook Reservoir to a channel known as the Goit is at the same level. They have a common overflow weir from the connecting channel, at a level of 168.361 m.o.d. The general layout of these reservoirs is shown in Figure 1.

Following an inspection in 1965 by Mr. R le G Hetherington under the Reservoirs (Safety Provisions) Act 1930 it was recommended that,when suitable opportunities for emptying occurred, the reservoir scour valves should be renewed. It was also recommended that the next inspection should be five years later.

The Reports prepared following the 1970 inspection by Mr. T G Hammond recommended that, in the case of Lower Roddlesworth, the existing scour valve should be replaced or new scour works constructed as a matter of urgency, and in the case of Rake Brook that a new scour be provided. Another recommendation contained in the 1970 Reports was that the access to both reservoirs should be improved.

ORIGINAL WORKS

Due to the absence of construction records, which were lost in a fire many years previously, nothing was known of the submerged parts of the dams.

LOWER RODDLESWORTH RESERVOIR

The dam is an earth embankment 182.9 m long and assumed to have a clay core which appears to be carried up to within 0.76 m of the top. The upstream face of the embankment is covered with hand placed stone pitching. The maximum height over the original ground level is 25 m and the upstream and downstream slopes are 1:3 and 1:2 respectively. The catchment area is 65.6 h and the maximum quantity of water which can be impounded is 415 t.c.m.

The scour tunnel was 2.44 m int. dia. lined with brick. A 460 mm dia. C.I. scour pipe ran throughout its length, being supported on brick piers upstream of the valve shaft and surrounded by brickwork in the invert in the downstream length. The details of the upstream tunnel were unknown until exposed during the contract. On exposure, the inlet was found to have a heavy wrought iron grille across the tunnel mouth, and was protected by a headwall and by dwarf wing walls.

The original 460 mm dia. guard and control valves were sited adjacent to each other and anchored in mass concrete in the bottom of the 21.7 m deep valve shaft. This shaft was lined with brick and the 9.45 m long plug upstream of the shaft was found to be constructed mainly of brickwork, but with sections of clay, presumably to effect a seal with the overlying rock.

The downstream tunnel had a disused ventilation shaft 49 m from the valve shaft. The construction of this had presumably been found necessary because of fumes when burning out the lead joints during re-laying of the original scour pipe which had risen when first put under pressure. There was a valve on the outlet of the 460 mm pipe at the downstream portal to control the flow of compensation water without having to operate the main valves.

RAKE BROOK RESERVOIR

The dam is an earth embankment 461.9 m long, in three lengths angled to each other. The embankment is 26 m above original ground level and has five berms on the downstream slope, each 4 m wide. The upstream slope is 1:3. The embankment is believed to have a puddle clay core. The catchment area is 367.3 h and the quantity of water which can be impounded is 331 t.c.m. There was no valve shaft to this reservoir, access to the valve chamber being from the downstream portal of the tunnel. The upstream tunnel was mainly 1.83 m dia as was the downstream, but a length of 31 m, commencing 9.14 m downstream of the portal, had a secondary brick lining of 1.27 m int. dia. In this case, the scour pipe did not extend upstream from the plug. The inlet details were similar to those at Lower Roddlesworth except that there was no head wall. There was a heavy C.I. grille across the tunnel mouth. The 460 mm dia. scour pipe was embedded in brick infill in the invert downstream of the valve chamber and terminated at the downstream portal with a control valve.

The plug in the compensation tunnel was formed principally of brickwork but with sections of clay acting as seals, the total length being 13.72 m.

PRELIMINARY WORKS

Prior to the start of the main Contract, preliminary work was completed as follows:

The improvement of the existing access road from the A 675 to the overflow channel between the two reservoirs, and the construction of a new access road from this channel to the Lower Roddlesworth valve shaft, including the forming of an 'Irish Bridge' across the channel by ramping down to it on both sides. This work was carried out under contract at a cost of £4750.

The laying of a 305 mm dia. u P.V.C pipe line 1705 m long from Upper Roddlesworth Reservoir direct to the Goit, by-passing Lower Roddlesworth and Rake Brook Reservoirs. This was in order to maintain a flow in the Goit of sufficient quantity to enable the compensation water and supply commitments from that channel to be met. This work was carried out by direct labour at a cost of £18,890.

Site investigation by means of boreholes, which were sunk under a separate contract with Geo Wimpey & Co Ltd, at a cost of £3,967. At Lower Roddlesworth the boreholes were sunk near the junction of the dam with the natural embankment adjacent to the channel which joins the two reservoirs. At Rake Brook, boreholes were sunk on the line of the assumed position of the scour tunnel.

GROUND CONDITIONS

The five boreholes in the vicinity of the new valve shaft at Lower Roddlesworth showed brown/grey sandy clay to a depth of between 3 m and 4 m followed by boulder clay down to mudstone rock at approximately 9 m. The standard penetration test values were 40 in the boulder clay.

At Rake Brook, the two boreholes in the vicinity of the new shaft, downstream of the assumed clay core, showed brown/grey-blue clay, with occasional sandstone boulders, to depths of between 3 m and 14 m. A greater number of boulders appeared down to 15.5 m at which depth shaley mudstone was reached.

HYDROLOGY

Records indicated that the run-off figures for the total Rivington Catchment Area could be applied to the individual catchments of both reservoirs, and also to that of Upper Roddlesworth Reservoir. The maximum monthly run-off recorded was 289 mm in August 1956 and higher short term run-offs can occur.

The moorland catchment of the River Darwen, into which the River Roddlesworth flows, suffered a flood of approximately 14 m^3/s per 1000 h in July, 1964. The records indicated that the contract works might be subject to difficulties as a result of high short-term run-offs, especially at Rake Brook. where no means of upstream control existed. As the Lower Roddlesworth catchment on its own is not large, a degree of control was afforded by keeping the water in Upper Roddlesworth at a low level.

THE NEW WORKS CONTRACT

The Contract was awarded to C V Buchan Ltd for the sum of £293 590. The Contract included provisional sums for new works at both reservoirs, the actual requirements being unknown prior to them being emptied.

The Contract period was specified as two years, but a rider was added offering sympathetic consideration to a tenderer who could complete the work in a shorter period as it was considered that Rake Brook especially should not undergo a further winter with its original operating arrangements. From the site investigation results both embankments were considered as having small Factors of Safety. Buchan's initial programme envisaged completion of the work in 49 weeks.

The Contract specified that the new valve shaft at Lower Roddlesworth should be sunk, the new valves, pipework and concrete plug completed, the downstream tunnel lined and the existing valves broken out, under water if necessary, before the reservoir was emptied. At Rake Brook, the reservoir was to be emptied prior to the commencement of any other work.

Special measures were to be taken during the progress of the work to avoid the imposition of any unduly harsh conditions which might impair the safety of the embankments and work commenced on site on 1st January 1972.The Certificate of Substantial Completion was issued on 21st March, 1973 the work having taken 64 weeks.

NEW VALVE SHAFTS

GENERAL

Both shafts were designed to be constructed of pre-cast bolted rings 610 mm deep by 4.57 m int. dia.for a depth of 25 m then lined with concrete to give an int dia.of 4.10 m. In order to limit possible ground movement during shaft sinking, the Contract envisaged the use of a shield for the excavation through the overlying clays to depths of approximately 12 m at Lower Roddlesworth and 9 m at Rake Brook. It was specified that excavation should not advance more than 750 mm beyond the last precast ring grouted, the whole of the void outside the extrados of the ring having to be filled with a grout of sulphate resisting cement.

C V Buchan Ltd proposed an alternative method for the excavation of the shafts through the clays to rock which was adopted. It was based on the use of a steel cutting edge bolted to a precast segmental ring 4.57 m int dia and 4.98 m ext dia known as the choker ring. On top of this, the standard precast rings of 4.93 m ext dia were built as the shaft was sunk. The cutting edge was forced down through the ground by the imposition of kentledge weights on a frame at the top. The cutting edge and choker ring being 50 mm larger than the standard rings, the sinking resulted in an annular space 25 mm wide between the standard rings, and the ground surrounding them. This space was filled with bentonite slurry which supported the surrounding ground and at the same time provided a virtually frictionless surface which allowed the precast segments to settle freely as excavation progressed. This method was effective for obviating ground movement during shaft sinking, which was especially important in the case of Rake Brook shaft where excavation was carried out in close proximity to the position of the assumed clay core of the dam. The kentledge weights had to be removed and replaced on each occasion when it was necessary to build additional precast rings on the top of the shaft as sinking progressed.

Once rock was reached and further sinking under kentledge became impossible shaft construction was completed by underpinning methods, the rock being excavated by pneumatic picks and the rings built and grouted one at a time.

LOWER RODDLESWORTH SHAFT

Excavation for the Lower Roddlesworth shaft commenced within a previously constructed concrete collar, circular in plan 5.18 m int. dia. and 1 m wide by 1 m deep at ground level. The collar provided a template within which the cutting edge and the initial precast rings could be positioned prior to the commencement of sinking under kentledge, and also provided a solid surround through which the bentonite slurry could be placed in the annular space below. Sinking under kentledge continued to a depth of 8.3 m below ground level, the maximum superimposed load being 62.6 tonnes. After the first 3 m in soft brown sandy clay, the soil conditions became stiffer as a brown boulder clay with a pocket of soft brown puddle clay in the east side of the shaft was reached. The boulders and the stiffness of the clay prevented the cutting edge sinking into the ground and the clay around it therefore had to be excavated to maintain progress. The boulders also added to the tendency for the shaft to tilt out of plumb, the kentledge weights being moved from one side of the shaft to the other to rectify this. There was also the danger of losing the bentonite slurry out of the bottom of the shaft when tilt became unduly pronounced, and such 'blows' had to be hurriedly plugged with clay to prevent undue loss.

At 8.32 m below ground level rock was encountered on the south side of the shaft tunnel, and this gradually extended over the whole shaft area, becoming harder with depth, and with the pocket of soft clay remaining on the east side. At this point the cutting edge was removed and cement grout pumped into the cavity behind the rings from the bottom, thus displacing the bentonite, which escaped at the surface. The grouting was continued until displacement of all bentonite from the annular space above the choker ring was proved. Excavation then continued by underpinning through the mudstone down to a level of 146.54 m.o.d. at which level hard brickwork was encountered extending over the shaft area. The soft clay pocket on the east side of the shaft had persisted through the rock and it was eventually found that this constituted fill over the top of a 610 mm dia pipe extending up from the tunnel crown for a distance of 1.30 m. This is thought to have been part of an old ventilation pipe. The rock was excavated using hand-held pneumatic rock picks.

The brickwork above the existing tunnel proved very hard, and slow progress was made in excavating through this and in breaking into the tunnel. The shaft was bottomed up at 142.00 m.o.d. with a blinding layer of concrete and a reinforced concrete base 1 m thick.

Steel frames were fixed at the junctions with the existing 2.44 m dia bricklined tunnel and were subsequently concreted in with the lining to the shaft. The total time of construction of the shaft was seven weeks. The total depth of the shaft was 25.23 m, 41 no. precast rings being built and 52 t of grout used. A larger quantity of grout was used behind the rings in the section built in rock, owing to the overbreak in excavation averaging 152 mm. This shaft is shown in Figure 2.

RAKE BROOK SHAFT

This was constructed on the same principles as that at Lower Roddlesworth, but with certain significant differences.

Due to the fact that the centre-line was 3 m from the downstream side of the edge of the first berm from the crest of the embankment it was necessary to construct a deeper collar than had been provided at Lower Roddlesworth. This was therefore designed to a depth of 3 m, the formation level being 162.30 m.o.d. The int. dia. was 5.18 m as before. A working area was formed out from the original slope to accommodate the collar and the necessary plant and materials. The cutting edge and choker ring were positioned within the collar and eleven additional rings built on top before sinking under kentledge, with bentonite slurry as before, commenced. The weight of materials and plant was kept to a minimum in order to avoid causing distress in the dam. Initially, good progress was made through soft clay with few boulders down to 157.00 m.o.d. after which firm boulder clay was encountered. At 154.00 m.o.d. the shaft 'hung up' in the boulder clay and although the superimposed kentledge weight was increased to 95 t and shifted about from side to side at the top of the shaft no further progress could be made. The bentonite was again expelled by grouting with cement, the cutting edge removed, and excavation continued by underpinning.

The rock below 159.00 m.o.d. varied across the shaft as the depth increased, from hard sandstone on the south side to soft black shale on the north side, alternating with mudstone. The existing brick lined tunnel was broken into and the blinding layer and shaft base of reinforced concrete completed as before with steel frames fixed at the shaft/tunnel junction. The total depth of the shaft from ground level was 24.79 m from the top of the temporary collar, 43 no. rings were built and a total of 28 t of grout was used in grouting up behind them. The total period of shaft construction, excluding the collar was five weeks. This shaft is shown in Figure 3.

LOWER RODDLESWORTH - WORKS PRIOR TO DRAW-DOWN

On completion of the new valve shaft the remaining works preparatory to draw-down commencing were the strengthening of the downstream tunnel and the installation of the new valves and pipework and upstream plug.

DOWNSTREAM TUNNEL

It was originally specified that the existing 460 mm scour pipe and brick filled invert were to be broken out and the new secondary lining to 2 m dia. placed in lengths, but in the event the entire length of 94 m was done in one operation. An additional length, upstream of the new valve shaft as far as the rectangular passage, Figure 2, was broken out similarly.

The complete downstream length was concreted in 25 lengths each 3.78 m long at a rate of two lengths per shift. Ready mixed concrete was used for this section and excess concrete used to backfill the disused ventilation shaft. Grout holes were drilled through the primary and secondary linings into the rock beyond and the voids between the rock and the back of the primary brick lining were grouted. Drainage holes were then drilled through the linings at sections where leakages had been observed before the secondary lining was placed. The quantity of grout injected round the downstream tunnel averaged 0.62 t/m.

PIPES, VALVES AND PLUGS

610 mm and 152 mm C.I. pipes and two 610 mm guard/control valves were supplied under separate contracts. These were installed without spindles and headgear and the valves were clamped open for use during draw down, the concrete plug upstream then being concreted. The secondary lining upstream of the new plug, as far as the rectangular passage, had been carried out following completion of the downstream tunnel lining, and a water bar had been inserted at the end of the lining at the junction with the plug. The void between the crown of the plug and the existing brick lining was grouted up.

LOWER RODDLESWORTH DRAW DOWN

The preliminary works having been completed, Lower Roddlesworth draw down commenced on 9th May, 1972. To avoid employing a diver, it was decided to attempt to open the existing control valves. This was accomplished successfully after the existing valve shaft had been filled with water to 1 m above T W L, which created a head differential across the valves, the higher pressure being on the downstream face.

As analysis of the embankment had indicated that the Factor of Safety of the downstream slope during draw down might only be approximately 1.05, it was specified that draw down was to be carried out at a rate not exceeding 200 mm per day. In general, this was achieved by controlling the 152 mm dia.by-pass valve. It was also stipulated that the daily maximum draw down should not be accomplished in less than 4 hours.

Due to the fact that the new 610 mm scour valves had been installed with no extension spindles up to the working platform a temporary 460 mm valve was fixed on to the end of the 610 mm pipeline at the downstream end of the new plug, and additional discharge was achieved through this during the periods of high rainfall. From 14th June the maximum permissible daily rate of draw-down was increased to 300 mm. Delays due to high rainfall were experienced throughout early July, the total during July being 135.89 mm against the average over a 40 year period of 51.31 mm. The graph Figure 4 shows the record of water level and weekly rainfall over the period. On the occasions when the water level rose after rains it was lowered again as quickly as possible to the lowest level reached previously under controlled draw down, using the 460 mm dia valves.

Level and alignment stations were established along the centre of the crest at 50 m intervals, lined up with targets beyond the limits of the embankment at each end. Three lines of level pins 21.5 m apart were established on the upstream slope, the pins being at 10 m intervals down the slope. Level checks were carried out once or twice weekly during draw down, and at weekly or fortnightly intervals afterwards. The overall reduction in level on the crest amounted to 64 mm and 76 mm at the two stations in the centre during the period from commencement of draw down in mid-May 1972 to October 1972, three months after draw down was completed.

Levels taken on the slope were found to have reduced by 25 mm to 30 mm at the top to 40 mm near the toe over a period from the beginning of July to mid November, 1972.

RAKE BROOK DRAW-DOWN

The Contract required that Rake Brook should be drawn down and emptied by pumping, a lump sum being quoted in the Bill to cover this. The Contractor elected to commence this in May to complete over four summer months for which records indicated that rainfall might be a minimum. The work was sub-contracted and 152 mm dia. diesel-engined pumps mounted on platforms were installed on the upstream slope of the embankment, discharging round the penstock headwall into the Goit. Three pumps were installed initially, these being considered adequate to give the permissible rate of draw down of 200 mm in 24 hours. It was considered that the same number of pumps would be adequate for the whole operation. In the event, rainfall upset these calculations, the actual rainfall figures experienced over the four months May - August, 1972 being shown in Figure 4 related to the previous 38 year average for those months. A rise in water level of 7.62 m over 12 hours followed a thunderstorm on 23rd July. It became clear that more pumps would be required, and as rainfall increased during June the number of pumps was increased to six at the reservoir, pumping into the Goit, with two more installed at a chamber through which some of the reservoir inflow water passes. These also pumped direct to the Goit through a 304 mm dia P V C pipeline laid above the ground for the purpose and by-passing the reservoir.

Following the heavy rains considerable volumes of silt were washed down from the catchment area, and on 1st August Preston and District Water Board notified Liverpool Corporation Water Works that the pumped discharge down the Goit was no longer acceptable on the grounds of the unduly high suspended solids content of water drawn from the Goit about 1.3 km downstream of the reservoir. Delivery from the pumps therefore had to be directed down the spillway. Four pumps were then mounted on two pontoons in the reservoir. These pumped in tandem with four others higher up on the reservoir bank, the total head amounting to over 16 m with a total delivery distance of 215 m. As much silt as possible was removed by being allowed to settle in a basin formed by constructing a bund across each end of the channel connecting Lower Roddlesworth and Rake Brook Reservoirs. The partially clarified water then overflowed down the spillway where three barriers of small stones caused further settlement.

On 13th August, when the new valve shaft had reached the existing tunnel and access to the existing scour valve in the chamber was possible from the shaft, the valve was opened three turns, the water level in the reservoir being at 153.28 m.o.d. or 4.28 m above estimated bed level. A small flow passed, which ceased overnight. The following morning, the valve was fully opened and the reservoir emptied quickly, followed by large flows of silt. These passed for over one hour and then ceased due to blockage upstream following collapse of the inlet grille, evidently from high suction pressures.

INLET WORKS AND UPSTREAM TUNNELS

At Rake Brook it was estimated that the crown of the upstream tunnel at the inlet would be about 3 m below the silt level left at draw down. Excavation of the silt by drag line in the vicinity of the inlet commenced as soon as possible. It was found that owing to the fluid nature of the silt excavation had to be augmented by pumping, and a 152 mm submersible sludge pump was found to be capable of pumping silty water into the spillway channel against a total delivery head of 20 m. The Rake Brook inlet was eventually exposed to view on 3rd October 1972, after three weeks excavation.

The Lower Roddlesworth inlet was just visible when draw down was complete, the general silt level being some 1.2 m below the coping of a masonry headwall at about the level of the top of the inlet grille at the entrance to the tunnel. Sheet piles were driven into the underlying rock to form a cofferdam, the piles being supported by 203 mm x 127 mm semi-circular arch ribs as walings.

At Rake Brook there was no headwall adjacent to the inlet and the rock was at too great a depth for the piles to be toed in, but a similar cofferdam to that at Lower Roddlesworth was constructed.

The upstream tunnels were each full of silt. At Lower Roddlesworth the old scour pipe extended from the inlet down the length of the upstream tunnel, supported on brick walls at 3 m centres, and through the old plug to the inlet valves, whereas at Rake Brook the scour pipe inlet was immediately upstream of the old plug. The silt from the Lower Roddlesworth tunnel was therefore excavated by hand, the brick walls and C.I. pipe being simultaneously broken out, whereas at Rake Brook the silt was pumped out of the tunnel using a 50 mm dia. jetting pump.

Once the silt had been cleared access to the old plugs was possible. In both cases they consisted of solid brickwork surrounding the scour pipe and bonded to the rock by clay. Precast tunnel rings were erected in both plugs as excavation proceeded. The Lower Roddlesworth tunnel was 2.40 m dia. and the Rake Brook tunnel 2.00 m dia., the plugs being excavated for 2.29 m int. dia. rings and 1.83 m int. dia. rings respectively. The Lower Roddlesworth plug excavation was more difficult due to a bend. It was found necessary to support the clay at this bend by building 152 mm x 127 mm colliery arch ribs at 300 mm centres and by pouring the 2 m dia. concrete secondary lining at this section, immediately followed by grouting. The Lower Roddlesworth plug was excavated from the upstream end. Work commenced on 1st October 1972 but between 16th November and 12th December the work was brought to a standstill by heavy rains causing Upper Roddlesworth Reservoir to overflow, flooding the works. When the weather improved, Upper Roddlesworth was drawn down by opening its scour valve until the level was reduced to some 5 m below spillway. It was kept at this level for the remaining period of the work in the Lower Roddlesworth upstream tunnel by opening the scour valves as required. The Rake Brook plug was excavated from the downstream end.

Grouting was carried out to fill the voids to the surrounding rock following the erection of each group of pre-cast rings.

The upstream tunnels, as with the downstream tunnels, were concrete lined using ribs and laggings, at Lower Roddlesworth to 2 m dia. and at Rake Brook to 1.5 m dia. The existing 1.27 m dia. brick lined section at Rake Brook was not lined with concrete; a cracked section was strengthened by placing a 2 m length of 1118 mm dia. C.I. pipe immediately inside the brick lining, the annular space being filled with grout.

The extrados, together with the voids between the concrete lining and brick were grouted. Holes were drilled for this purpose through the brickwork, generally in groups of five, at 4.5 m cc. At Rake Brook the "take" of grout averaged 0.3 t/m length of tunnel and at Lower Roddlesworth it averaged 0.42 t/m.

LOWER RODDLESWORTH OLD VALVE SHAFT

This was lined with concrete to 1.5 m int. dia. using a 1.83 m length of 1524 mm int. dia. split steel pipe as a shutter arranged to 'collapse' for striking. The concrete was poured in 1.68 m lifts, one lift per day. The shaft was then drilled and grouted in the voids beyond the brick lining. The weight of grout used was 15.8 t.

RAKE BROOK DOWNSTREAM TUNNEL

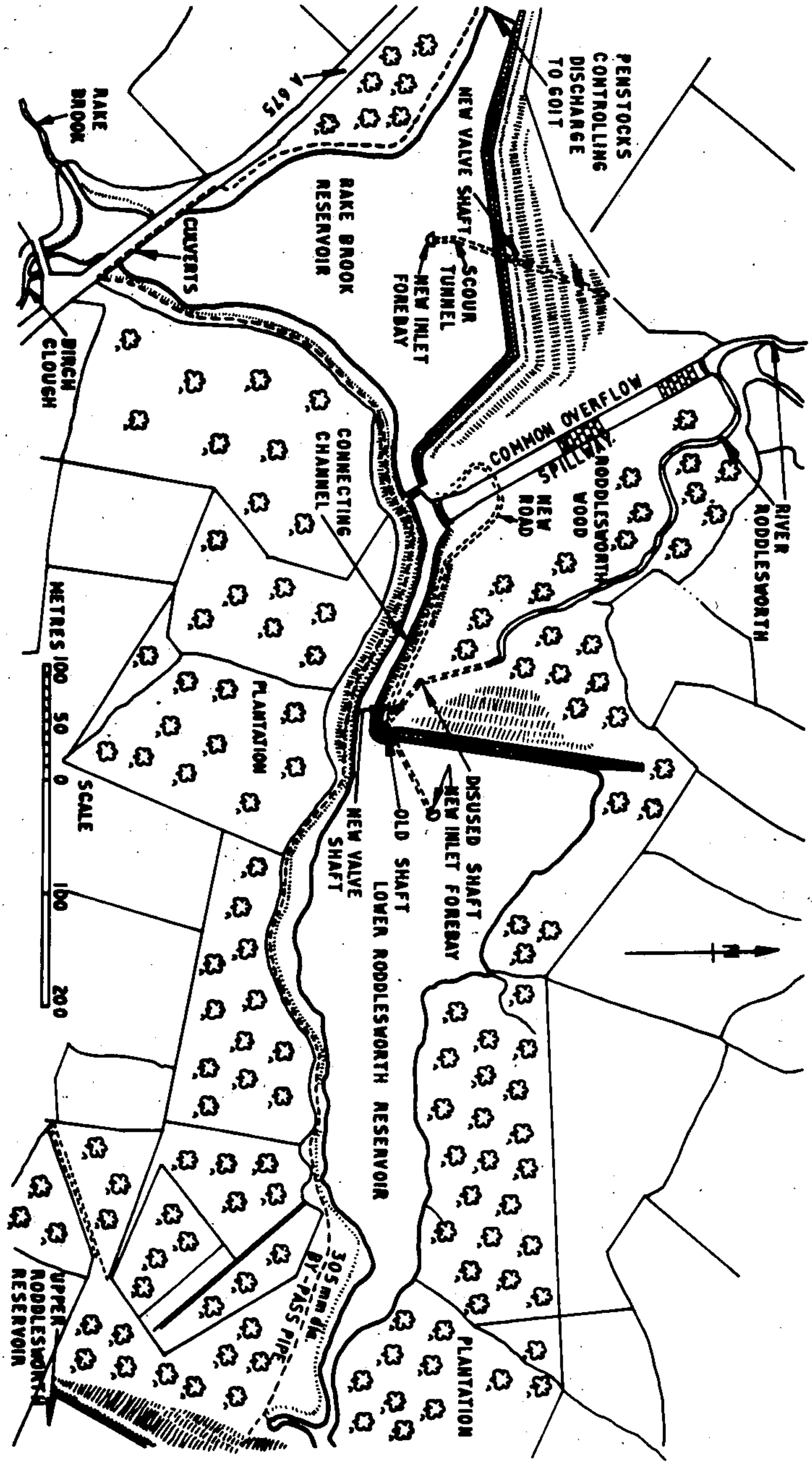
This was concrete lined to 1.5 m int. dia. and then grouted in a similar manner to Lower Roddlesworth tunnel. The grout "take' in this case amounted to 0.6 t/m.

RE-INSTATEMENT WORKS

All the shaft spoil and silt excavated from the reservoirs was tipped on two sites, one being adjacent to the Rake Brook embankment and the other alongside the new road connecting the reservoirs. These areas were subsequently contoured and grassed over and it is now difficult to detect the limits of these tips.

The bunds across the reservoir connecting channel were removed and the channel cleared of settled silt. The bunds across the spillway were also removed, together with the accumulated silt.

After the main contractor left the site on 21st March 1973, the original road contractor was re-engaged to repair, re-surface and extend the access roads at a cost of £2400.


PENSTOCKS CONTROLLING DISCHARGE TO GOIT
NEW VALVE SHAFT
RAKE BROOK
A 675
RAKE BROOK RESERVOIR
CULVERTS
SCOUR TUNNEL
NEW INLET FOREBAY
BIRCH CLOUGH
COMMON OVERFLOW
SPILLWAY
RODDLESWORTH WOOD
NEW ROAD
RIVER RODDLESWORTH
CONNECTING CHANNEL
METRES 100 50 0 100 200
SCALE
PLANTATION
DISUSED SHAFT
NEW INLET FOREBAY
OLD SHAFT
NEW VALVE SHAFT
LOWER RODDLESWORTH RESERVOIR
305 mm dia. BY-PASS PIPE
UPPER RODDLESWORTH RESERVOIR
PLANTATION

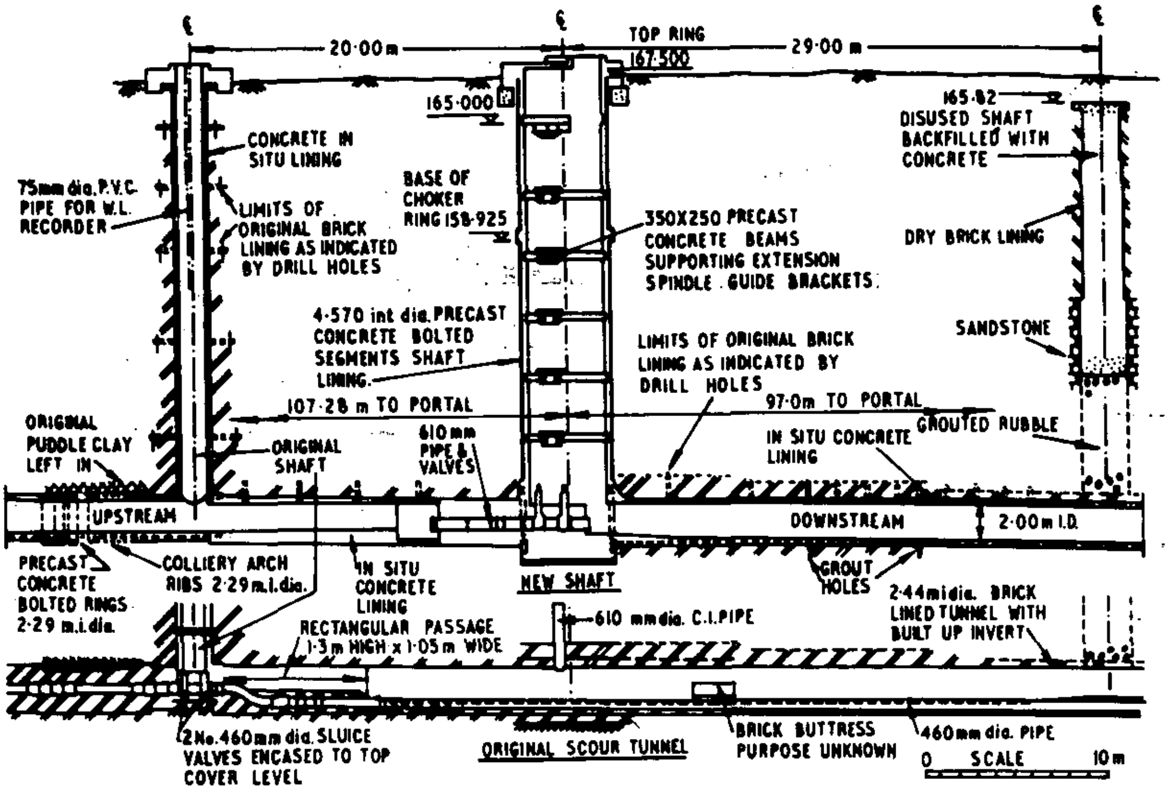


Fig. 2 Section through New Lower Roddlesworth Valve Shaft

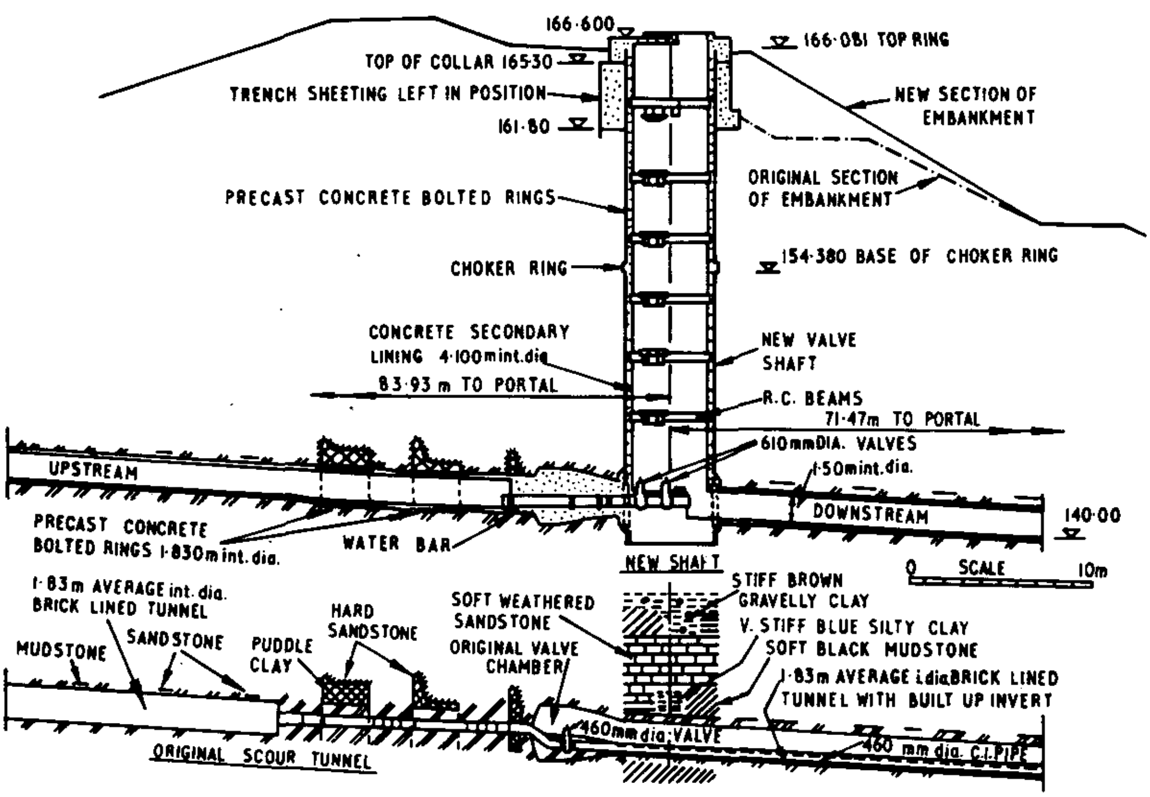
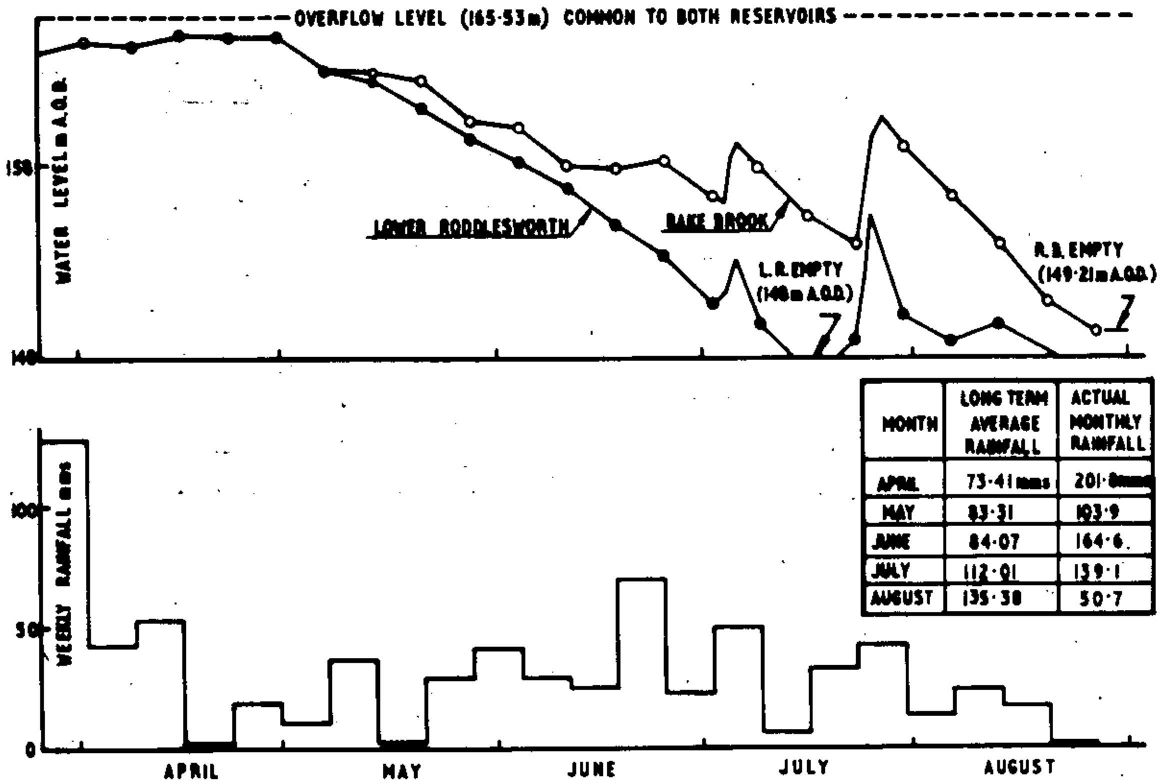


Fig. 3 Section through Rake Brook Valve Shaft



MONTH	LONG TERM AVERAGE RAINFALL	ACTUAL MONTHLY RAINFALL
APRIL	73·41 mms	201·8 mms
MAY	83·31	103·9
JUNE	84·07	164·6
JULY	112·01	139·1
AUGUST	135·38	50·7

Fig. 4 Graph showing Reservoir Draw Down and Rainfall

BUCKIEBURN RESERVOIR, STIRLINGSHIRE: FAILURE OF DOWNSTREAM SLOPE OF EMBANKMENT DAM AND SUBSEQUENT REMEDIAL WORKS

H D Osborn, CEng MICE MIStructE

ASSOCIATE

BABTIE SHAW AND MORTON

SYNOPSIS

The paper describes the failure of the downstream slope of an embankment dam at Buckieburn Reservoir and the emergency remedial measures necessary to stabilise the slope together with other improvement works.

BUCKIEBURN RESERVOIR

Buckieburn Reservoir is situated north of the Carron Valley in Stirlingshire and at the time of the failure was operated by the Mid-Scotland Water Board,and is now the responsibility of the Central Regional Council. The dam was constructed about the beginning of the century and is in the form of an earth embankment with a height of 23 metres above the valley floor and having a downstream slope of 1 vertical to 2.6 horizontal and a pitched upstream slope of 1 vertical to 3 horizontal. Whilst a puddle clay core was indicated on the original drawings this was not revealed in subsequent borehole investigations and the dam was found to be constructed of variable moraine fill.

SLIDE FAILURE

During the night of 1st/2nd November, 1970, a substantial slide occurred on the downstream face of the dam during a period of heavy rain and high winds. The slide area covered a distance of about 25 m up the slope from the downstream headwall of the culvert through the dam and over a length of about 22 m. An inspection indicated that the slide did not appear to be deep seated, but with cracks in the turfed surface above the slide it could be progressive. The slide appeared to have been initiated by excessive surface water flowing down the slope together with water from wave action overtopping the parapet wall at the crest of the dam combined with the flow of surface water in the valley formed by the intersection of the embankment with the natural hillside at the north end. A plan of the dam and slide area is shown in Figure 1. Figure 2 is a photograph of the slide area.

Immediate measures were taken to lower the level of the reservoir and this was assisted by diverting the flow of two aqueducts which supplied the reservoir from two extended catchment areas. Supply was maintained from the reservoir during the carrying out of remedial works but was not allowed to rise above a level of 3 m below the overflow sill level. This allowed water to be supplied to another reservoir from a high level draw-off.

EMERGENCY REMEDIAL WORKS

The area of the slide being in a saturated and unstable condition, a method of carrying out remedial works had to be devised which would avoid any further disturbance of the embankment. The method adopted was to provide a substantial rock fill toe wall at the bottom of the downstream slope and thereafter form a berm of borrow material, working progressively from the rock toe wall to encompass the slide area and the remainder of the embankment. An approach was made to Messrs. Shellabear Price Ltd, Civil Engineering Contractors specialising in earthworks who were operating in the area,and they agreed to carry out the remedial works. This firm moved with expedition and the first plant was on site three days after the slide occurred.

Work started immediately on the construction of the rock toe wall and in order to found this on reasonable strata a layer of 2.5 m of peat had to be removed concurrently with the placing of rock fill. The existing culvert was extended through the toe wall by constructing a temporary culvert in steel colliery arches covered with steel sheeting.

Borrow pits were opened up on the north side of the valley to provide sand and gravel fill. This material was found to be variable, containing clay and silt, and proved difficult to work during wet weather. Before the borrow material was placed against the existing embankment slope the topsoil was stripped off in sections and a 0.6 m layer of broken stone was placed to form a drainage layer. Figure 3 shows a cross section of the dam including the remedial works.

ANALYSIS OF DAM

In order to check the stability of the dam the properties of the embankment material were ascertained from boreholes which were sunk by George Wimpey & Co. Ltd. Piezometers were installed in the boreholes when completed to determine the possible pattern of the pore pressures in the embankment material.

A slip circle analysis of the original embankment indicated that if pore pressures were ignored the Factor of Safety for deep seated slides varied from 1.7 to 1.8, but for shallower slides this was reduced to 1.5. Since failure had occurred the Factor of Safety must have decreased to below unity, which indicated that there were high pore pressures at the time and this gave an r_u value in the region of 0.30. It is of interest to note that the piezometer reading in a borehole adjoining the slip area gave an r_u value of 0.355, indicating that the area adjacent to the slip was still in a critical condition prior to the placing of the remedial works berm.

With the addition of the toe wall and berm it has been calculated that the Factor of Safety against shallower slides near the affected area has been increased from about 1.5 to 2.4. The new berm terminates 6 m below the crest of the dam and therefore this remaining slope of the embankment will be subject to a lesser Factor of Safety. From piezometer readings in boreholes in the region of the crest a minimum Factor of Safety of 1.3 was assessed. It was therefore decided to install a system of drains in this area to lower the water table and to continue taking piezometer readings at intervals to give a warning of any possible reduction in the Factor of Safety due to increased pore pressure.

FREEBOARD OF DAM

The original overflow sill level was 248.720 m and it was decided to construct a new sill at 248.410 m to increase the freeboard of the dam. The maximum wave height that could be generated was estimated at 0..610 m. As the height of the parapet wall above the crest level of 249.94 m is 0.76 m, the total freeboard for wave action allowing a depth of water over the sill of 0.610 m is 1.68 m, which was considered an adequate safety margin against overtopping by waves.

SPILLWAY AND OTHER WORKS

The lower section of the existing spillway was formed by twin 460 mm dia pipes which, from check calculations, were found to be inadequate in capacity quite apart from a tendency to be blocked by ice, etc. A new weir and spillway were constructed by F J C Lilley (Marine) Ltd together with a concrete lining to the culvert below the rock toe wall, reconstruction of the stilling pond, diversion of draw-off pipes, and pitching of the rock toe wall. The completed remedial works, spillway channel, etc;. are shown on Figure 4.

ACKNOWLEDGEMENTS

The Author wishes to express his thanks to the Central Regional Council and to Mr J T Robertson, BSc, MICE, MIWE, Director of Water and Drainage Services, Central Regional Council, for permission to present this paper. He would also express his thanks to Mr. E W Denholm, FICE, FIWE, formerly Chief Engineer, Mid-Scotland Water Board and Mr. R Fellows, CEng, MICE, MIWE, Assistant Director, Water Operations, Central Regional Council, for their assistance when the remedial works were carried out. The Author also wishes to acknowledge the assistance of Mr. D J Bell in the preparation of the diagrams.

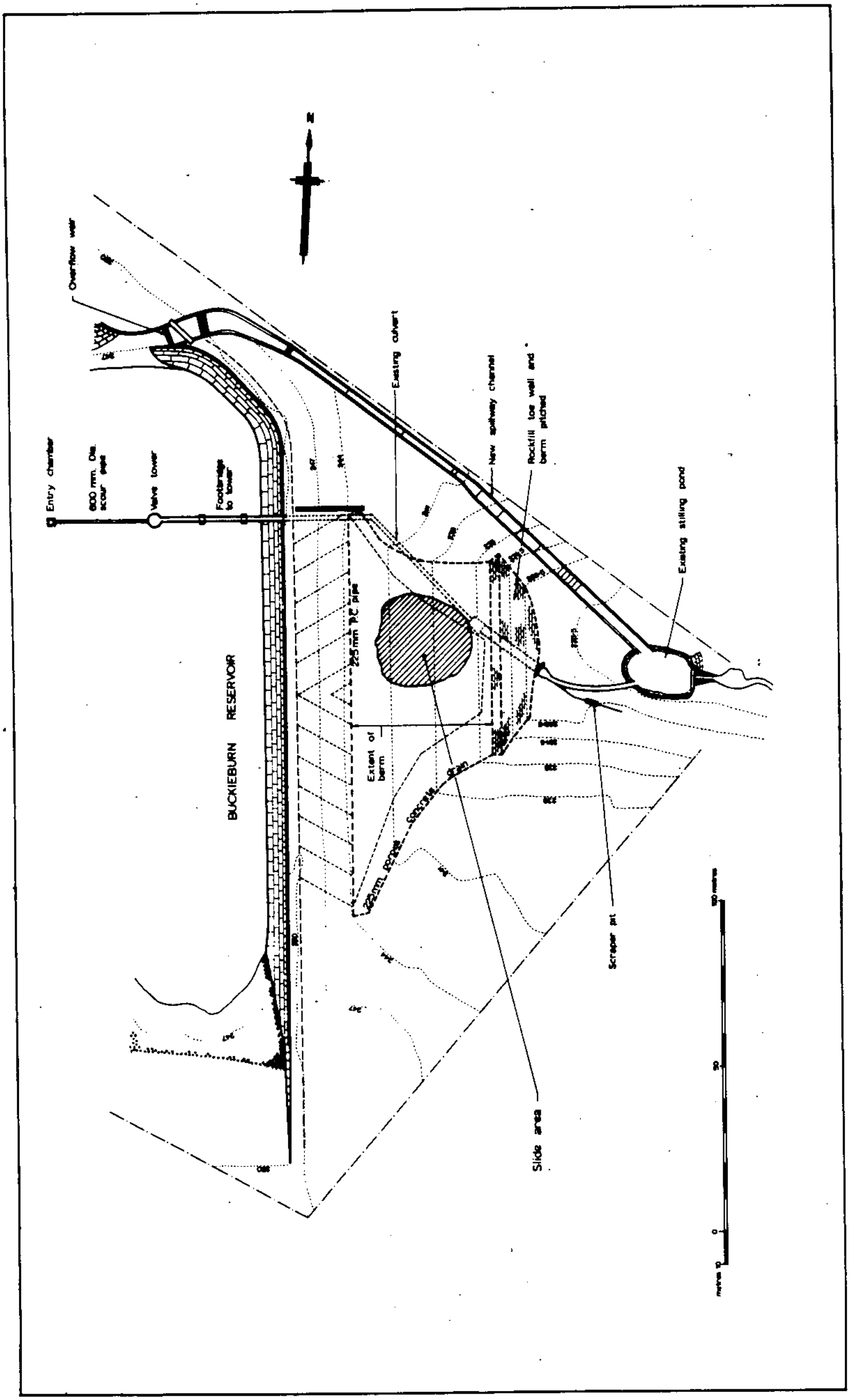


Fig 1 Plan of Dam

Fig. 2 View of dam showing slide area

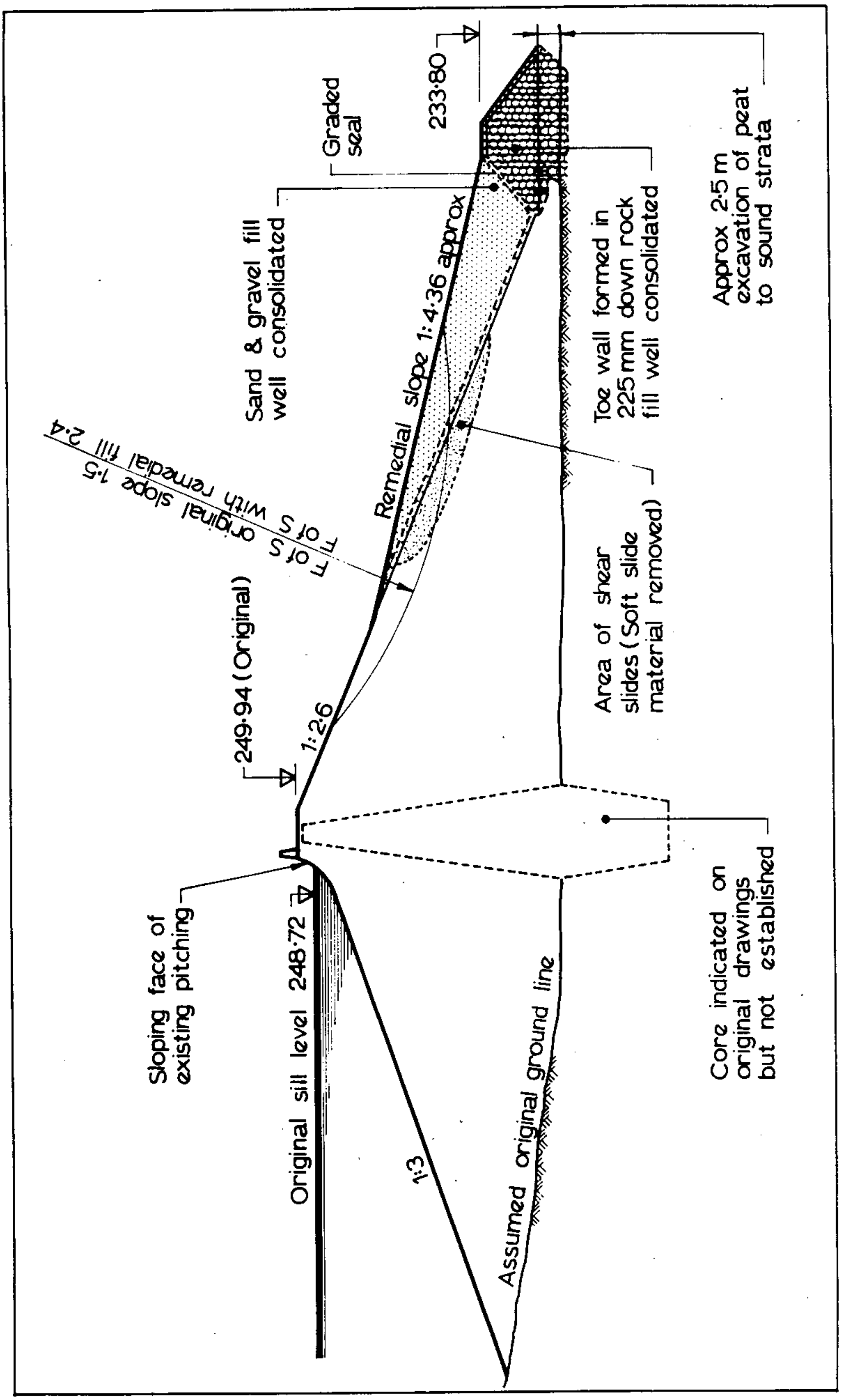


Fig 3 Cross Section Through Dam

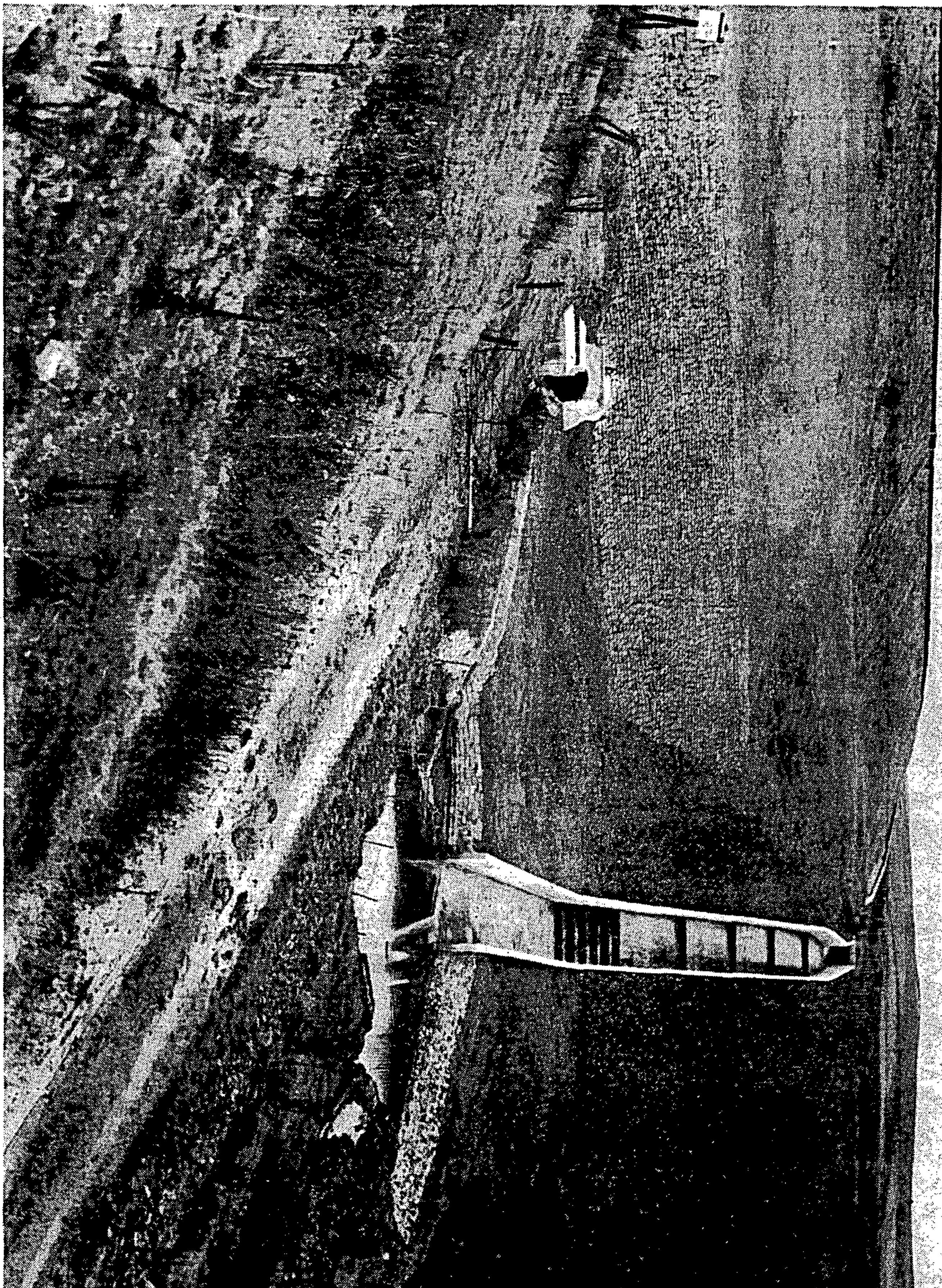


Fig.4 View of dam after completion of remedial works

DISCUSSION : TECHNICAL SESSION 5

PROBLEMS OF EMBANKMENT DAMS AND REMEDIAL MEASURES

Session Chairman : R T GERRARD BSc(Eng) ACGI CEng FICE MEIC
Senior Partner,
Binnie and Partners

General Reporter : A D M PENMAN DSc CEng FICE
Head of Dams Section, Geotechnics Division,
Building Research Establishment

CHAIRMAN: R T GERRARD :

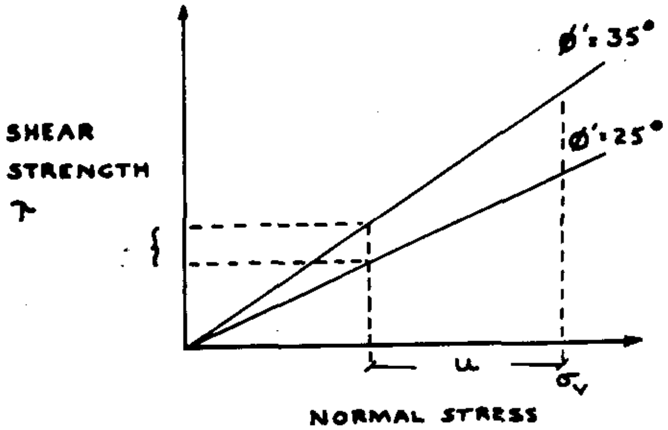
Ladies and Gentlemen, this is the fifth and final Technical Session and the theme is 'Problems of Embankment Dams and Remedial Measures'. We are extremely fortunate in having as our General Reporter Dr Arthur Penman, whom I might describe as one of this University's favourite sons, holding as he does the degrees of BSc, MSc, and latterly DSc, of the University of Newcastle upon Tyne.

He has served on the Council of the Institution of Civil Engineers and on the British National Committee on Large Dams, and he is a past Chairman of the British Geotechnical Society. He was also the Secretary of the Institution of Civil Engineers' Committee which produced the 'Report on Reservoir Safety' in 1966 and he is the author of numerous technical papers, particularly on embankment dams and related work.

REPORTER: Dr A D M PENMAN :

(S) At one time young men in the geotechnical field used to be critical of those older reservoir inspectors who were prepared to inspect embankment dams without apparent knowledge of the current condition of the fill. A ride round the reservoir by car and a walk across the embankment on a dry, warm day did not strike the geotechnical engineer as being quite good enough, but I am pleased to say that the nine papers presented to Session 5 indicate that this era is now behind us. Without exception, they advocate some form of internal examination of old dams of doubtful condition by borings and trial pits when adequate information on the condition of the fill and other records are not available to the Inspecting Engineer.

From the point of view of the stability of the bank we are concerned with the strength of the soil, and what we need to know is the position of the phreatic line, or to be more exact, the pore pressure at any point in the bank. As you know, the strength of the soil is governed by effective stress, i.e. the total stress, σ, minus the pore pressure, μ, as indicated by Fig A. Study of that diagram will make it evident that the exact value of the angle of shearing resistance, ϕ, is of much less importance than the value of the pore pressure at particular points. I suggest that a great deal of effort has been put into measuring values of ϕ' whereas it may in fact be a secondary consideration.



PENMAN, ADM : FIG. A

An assessment of conditions can readily be made once piezometers have been installed in a dam, and there is much to be said for sealing Casagrande piezometers into boreholes. The boring reveals the soil type and allows undisturbed samples to be taken, and the piezometer can be a permanent installation which will give future inspecting engineers the vital information on pore pressure conditions.

Failures are always of great interest because we all hope to learn by the experience and avoid the developments that led to it. We are therefore particularly interested in the last paper, Paper 5.9, describing a limited shallow slip in the downstream slope of a 23 m high, 70 year old earth dam. A photograph of the slip is given as Fig.2 of Paper 5.9. An effective stress analysis showed that an r_u value of 0.3 was required to reduce the factor of safety to unity, but a piezometer placed in the adjoining slope measured an r_u value of 0.355, suggesting a rather dangerous condition. The remedial measure was to construct a stone toe and place fill on the lower slope to flatten it, as indicated by Fig. 3 of the paper.

Three of the papers, Papers 5.3, 5.5 and 5.7, discuss or mention the ailments of Withens Clough Dam. It is a 25 m high earthfill dam 81 years old, and is illustrated in the figures of Paper 5.5. Eighteen years ago 16 boreholes were made and six piezometers installed; then five years ago a further 33 piezometers were installed in the downstream shoulder and five in the upstream shoulder. Tests on samples showed $\phi' = 30^{\circ}$ to 35°, and the boreholes revealed some interesting irregularities, including a 2 m thick peat layer. The high position of the phreatic surface in the downstream shoulder showed the poor condition of the core, confirmed by undisturbed samples and leakages associated with the masonry culvert that passed through the dam. The remedial measures, which included making a new core by slurry trench construction, were skillfully carried out without taking the reservoir out of service. It is a model example of what can be done for an old dam, and is succinctly described in Paper 5.5.

A nearby dam, Warmwithens, failed four years ago. Warmwithens Dam was the uppermost of three dams above the town of Oswaldtwistle, and the released water washed out an A-class road. It all but triggered a cascade failure, which would have been very serious for the town. This failure has already been mentioned by Mr Moffat in Session 1, and I hope we will have a description of the incident from him during this Discussion.

Incidentally, it is stated in Paper 5.2 that 'a high proportion of dam failures has occurred through the abutments rather than the embankments'. Could I ask the authors to substantiate this please? Their check list could usefully have 'measure position of phreatic line' added under the heading of 'embankments'.

A fascinating description of the near failure of Cowlyd Dam is given in Paper 5.6. This 14 m high dam was built in 1921, with a concrete core. The dawn of New Year's Day 1925 found waves overtopping the dam, which caused a failure of the downstream shoulder and exposed the core wall. Figure 1 of Paper 4.6 shows the position of the dam in relation to Coedty Dam, the failure of which in November 1925 was instrumental in producing the 1930 Reservoirs (Safety Provisions) Act. Looking at the relative sizes of the reservoirs, the Dolgarrog disaster would have been many times more severe if Cowlyd Dam had failed rather than Coedty. The citizens of Dolgarrog should have been extremely grateful to the team of men who turned out in the storm on New Year's Day and backfilled Cowlyd embankment sufficiently to prevent collapse of the core wall and total failure.

The description of the construction of Cowlyd quoted in Paper 5.6 is also fascinating. The fill was compacted by dropping it from a height. Three cubic metre capacity waggons were raised by cranes and then tipped as shown in Fig. 5 of the paper. It sounds a bit like a forerunner of dynamic compaction, and not very effective for producing a dense fill !

A general picture of the conditions of dams in Northern Ireland is given by Paper 5.4, and it is interesting to see the use of peat as a leak-sealer to back-up a clay core. I was also interested to read about the construction of the Silent Valley deep cut-off. Professor Cassie, in my undergraduate days in this University, used to cite this example as a dire warning to his students of the consequences of mistaking boulders for bed-rock in a site investigation.

Paper 5.7 draws attention to the damage that can be caused my mining subsidence and this is an aspect of old dams that should be aired in discussion. Types of remedial work described include the slurry trench used to form a new core at Balderhead. When first I saw Fig. 1 of that paper, which Mr Little also used in a paper to the Conference on Diaphragm Walls held at the Institution of Civil Engineers, I thought the shading a little alarming. It makes it appear as though the central part of the rolled clay core had been washed out and replaced by stones, rather than that a few fissures originally caused by hydraulic fracture had been enlarged by internal erosion, causing some leakage and depressions from local settlement.

The first paper, Paper 5.1 gives a most interesting record of eighteenth century ornamental dams and concludes with a valuable list of items for discussion. These include :

1 Resistance of old embankments to overtopping;

2 The effect of trees growing on the embankment;

3 The effectiveness of the upstream fill in preventing further erosion;

4 Design of the original sluice gates and culverts.

To these four suggestions by Mr Kennard I would add :

5 What should be done about mining subsidence?

6 A description of the failure of Warmwithens Dam;

7 What are we going to do about records to ensure that a complete picture of the behaviour of each dam is built up and preserved, despite the difficulty that the life of the dam spans over the lives of several generations of engineers?

P COOLEY (Thames Water Authority) :

Ⓢ Leakages have been observed for several years from some of the Thames Water Authority's older reservoirs in the Lea Valley, including Banbury Reservoir. These reservoirs, constructed over 60 years ago, have continuous earth embankments around their perimeters with puddle clay cores extending down to the London Clay which underlies the London basin.

The puddle clay is a silty alluvial clay which boreholes have shown to have become fissured throughout the full depth of the core. Staining of the fissures was taken to indicate that seepage of water was taking place through them. It was difficult to know how much of the leakage visible at the outside toe of the embankment was due to this seepage. There was evidence of drying out and cracking of the upper regions of the clay cores. When the top water levels of the reservoirs were lowered by as little as 0.5 m the leakage was in some cases dramatically reduced indicating that most of the leakage might be in these upper regions. Chances of finding local zones of leakage by means of boreholes were small, but there was some evidence that such zones existed, and boreholes were sunk and piezometer tubes installed.

Several methods of dealing with the problem were considered and a different remedy found from that described in Paper 5.5. . It was decided that a thin continuous grout screen of 50 mm thickness, constructed within the existing core wall by a proprietary process, offered the best solution. One of the smaller reservoirs, where some 450 metres length of the embankment showed leakage, was chosen for a trial section of the screen.

It was considered that the properties of the grout to be used in the screen were very important, and a series of tests was carried out in the Authority's Soil Mechanics Laboratory to develop a suitable formulation for the grout. The properties required were:

1 Low permeability;

2 Strength similar to puddle clay;

3 Durability - the grout should be resistant to chemical attack from the surrounding soil, and should not lose or gain moisture to an extent which would cause it to shrink or become more permeable;

4 It should be capable of withstanding minor ground movements;

5 Its flow properties should be suitable for pumping and injecting into the screen;

6 The components of the mixture should not segregate.

Over 60 mixes were tested in the laboratory, and the formulation which was finally specified had proportions by weight as follows :

Ordinary Portland Cement	4.2%
Pulverised Fuel Ash (PFA)	16.8
Bentonite	5.6
Kaolinite	16.8
Water	56.0
Sodium Tripolyphosphate	0.3
Methyl Acetate	0.3
	100.0%

This grout was required when mixed to have a minimum immediate shear strength of 35 kN/m^2 at seven days and a maximum permeability of 10^{-9} m/s.

A high shear bentonite mixer was used to disperse the bentonite and kaolinite in the water containing sodium tripolyphosphate. The suspension was then transferred to a low speed paddle mixer where the remaining constituents were added and then, with minimum delay, pumped by an air-driven double-acting ram pump through a 38 mm dia. pipeline to the pile forming the screen. The screen was formed by vibrating a single vertical I-section steel pile approximately 600 mm x 150 mm in section into the core wall with 150 mm overlaps. Grout was injected into the void through a 25 mm dia. tube welded to the pile and discharging at the bottom.

The grout when mixed had the consistency of thick cream but gelled rapidly if allowed to stand. It was necessary always to clear the pipeline with compressed air when delays occurred in the work to avoid blockage.

Boreholes made in the constructed screen revealed that in driving the pile the thickness of the preceding part of the screen was being reduced by the clay displaced by the pile and that the average thickness was only 30 mm instead of the required 50 mm. Modifications to the shape and thickness of the pile improved the thickness to 45 mm on subsequent sections of the screen. Fifteen hundred square metres of screen was constructed to a depth of 7.0 m, and 907 m^2 to a depth of 4.8 m in an aggregate period of six weeks of actual grouting. The maximum area of screen constructed in one day was 147 m^2.

Comparison of piezometer borehole readings before and after construction showed the screen to have been successful in eradicating the leakage. The durability of the screen will be demonstrated in the course of time.

The overall cost of the contract work was about £12.50/m^2 (1972/73) for about 2400 m^2, and would have been about £14.50 m^2 had the thickness been 50 mm as intended. The small and experimental nature of the work made it relatively expensive. It would be interesting to have a cost comparison with the plastic concrete diaphragm used at Withens Clough Dam (1972). The work there, described in Paper 5.5, was essentially less simple, and the cost of £288 000 quoted for cut-off work included for preliminary grouting to stabilise the core and the abutments.

J TATTERSFIELD (Watermeyer, Legge, Piesold and Uhlmann) :

S I would like to make a brief mention of those large and numerous structures built by large-scale mining concerns and their associated muck-shifting contractors.

In some parts of the world there are significant numbers of such dams, some of them quite substantial in their own right, and I think we would feel that many of them are a far cry from what we consider an earth embankment should be like.

I will deliberately use some intemperate language to try and put across a feeling. We have here undertakers whose resources are massive muck-shifting capacity and, in dam-building terms, an unlimited supply of over-burden material - a waste product, be it rock or soil. What will not excite much sympathetic attention in such circles are aspects such as the following : *Foundation preparation:* It is very inconvenient, and difficult, to handle plant of the type used, especially in steep valleys. I have seen entire trees of 10 m height left inside an earth dam. *Drainage:* Anything that is intricate and cannot be placed with a massive scraper of no interest. *Material selection:* What goes into the dam is what comes out of the pit - in that order. Anything different would upset traffic flow patterns. *Zoning:* Much the same sort of comment applies. *Compaction:* Nothing in excess of what the vehicles themselves will impart, and much placing is by end-tipping. *Moisture control and instrumentation:* Not practiced.

Stressing the point, we are considering organisations which make a tip double as a water-retaining dam. A rationale frequently offered for this kind of structure is that it is more massive in cross-section than anything an engineer would specify and therefore it is pretty safe. We know, however, that massiveness is not a guarantee of safety.

What should the engineer's position be in respect of such structures? Are we to stand back and disclaim any connection with them on the grounds that they do not conform with what we here would consider to be good practice? Or do we adopt the view that some engineering expertise is better than none, and some modifying influences from competent people will surely be better than none at all? I would suggest that there is merit in the latter point of view.

I would like to give an example of such a dam. The site in question is where a river, capable of delivering 400 m^3/s in the 100 year event, flows through an escarpment having a fall of 500 m. An ore-body straddles the river and a mine was opened up, with an encampment nearby. The mine management began to end-tip a dump all the way across the river, forming a dam. A spillway was cut through very soft weathered micaceous schist into a small tributory. The dam height is 43 m, retaining about 28 m of water.

The structure was built by end-tipping, with massive segregation during construction. Essentially its upstream and downstream slopes are at the natural angle of repose - approximately 1 to 1.4.

It was built of carbonaceous shale or mudstone, some fragments of which readily broke down on exposure to water or to air. After placement, being carbonaceous it burnt, leaving behind a mixture of ash, which seems to be the nearest thing to a frictionless, cohesionless material that there is, and a sort of sintered slag looking like expanded sponge. Tension cracks existed on the downstream face of the crest, and it was undercut by its own spillway. On the positive side the dam had a crest thickness of about 60 m. It had a crest height of 15 m above the spillway. In spite of everything I have said about the material it was extraordinarily impervious. The dam had stood for six years at the time of our first involvement. Lastly, in a very real sense and not only in the imagination of the undertaker, it had been built essentially for nothing.

I do not have time to illustrate how the remedial works were tackled, I wish only to illustrate the general point that I am making. We engineers, in some environments and some circumstances, have a very real practical problem with some very real and very big and very poor dams. In our profession generally we are naturally interested in strive for technical perfection, sophistication, the latest methods and so on. I hope I have said enough to indicate, however, that in some circumstances the engineer is still called upon to paint with a very broad brush indeed.

Dr J A CHARLES (Building Research Establishment) :

(S) In assessing the performance of an embankment dam the magnitude and rate of deformations occurring after the completion of the dam are obviously of considerable importance. Tests on a scaled-down model of the rockfill from Scammonden Dam, carried out in a 600 mm dia. oedometer under a uniaxial vertical stress of about 0.7 MN/m^2, show a roughly linear relationship between log-time and strain over a period up to 1000 days. The sample rock used was in this instance a carboniferous sandstone. The implication of this linearity is that in a rockfill structure the rate of creep deformation under constant load will rapidly decrease.

Field measurements of rockfill deformation have been made at Scammonden and Llyn Brianne dams. Both dams have wet clay cores, i.e. clay cores placed wet of optimum moisture content, and shoulders of rockfill which were placed in thin layers and heavily compacted. Llyn Brianne has a maximum height of 90 m, and Scammonden of 70 m, and we measured deformations on those two structures using the horizontal plate-gauge system. Plastic pipes were placed in the fill with annular steel plates at 15 m intervals along the pipes which acted as markers and whose movements could be detected and measured. Instrumentation was largely to measure movements during construction, but of course we have continued to take measurements subsequent to completion.

Scammonden was completed in 1969 and Llyn Brianne in 1971. Since completion we have continued to measure movement, via our horizontal plate gauges, over a period which includes the period of first impounding of the reservoirs, so that the movements that have occurred in the rockfill might possibly have three causes. The first would be the movements that would occur anyway due to creep under constant stress conditions in the rockfill. The second factor is that the clay cores will be consolidating with dissipation of pore pressures, and that will obviously influence the rockfill immediately adjacent to the core. As a third cause one might expect some effect on the downstream rockfill due to stresses from reservoir impounding.

Considering the vertical strains, which can be readily calculated from the movements of our plate gauges, we observed that the pattern of strains for the two dams after completion is very similar to the patterns measured during construction, but of a much smaller magnitude. The maximum post-construction strains of about 0.4% are about one tenth of what was measured during the actual construction. The main factor seems to be that the magnitude of the post-construction strain depends primarily on the depth of overburden and the greater the stress the greater the creep rate. Obviously those vertical strains are predominantly due to creep under constant stress conditions in the rockfill. The rate of creep is dependent on the vertical stress, which ties in with what we have found in laboratory tests, and obviously this means that the higher the dam the greater the creep, which will increase very rapidly, not merely linearly with height but to a greater power of the height of the dam.

Horizontal strains since the end of construction are very small and again of a similar pattern to what was measured during construction. At Scammonden, insofar as there were any horizontal strains after the end of construction they represented extension, i.e. horizontal spreading of the downstream rockfill. At Llyn Brianne, where the clay core had a much bigger effect and was exerting considerable lateral thrust, some small positive horizontal strains exist in this region. If reservoir impounding was to have very much effect on these strains one would have expected to detect movements on the downstream boundary of the clay core with the rockfill. At Scammonden, during the actual time of impounding no measurable downstream movements occurred on this line. At Llyn Brianne nothing much occurred on the downstream boundary of the core until the reservoir water level was at about 80% of full height, and at that stage some small downstream movements were measured.

(Editors Note : Amplification of Dr Charles' contribution and illustrations may be found in the following references :

Penman, A D M and Charles, J A (1972) *Effect of the position of the core on the behaviour of two rockfill dams*, BRS Current Paper CP 18/72

Penman, A D M and Charles, J A (1975) *The quality and suitability of rockfill used in dam construction*, BRS Current Paper CP 87/75

Dr. M S MONEY (University of Newcastle upon Tyne):

Ⓢ Withens Clough Dam has featured in four of the papers at this Symposium, and I would briefly illustrate the dam and the remedial works.

The investigation faced many difficulties in that, for example, pontoons and staging were required to put down boreholes on the upstream shoulder. The reservoir level was fortunately kept fairly stable during the investigations, and the contractor was not flooded out at any stage.

Visual observation of the face of the dam in its original condition showed a darker area which proved subsequently to be entirely covered with peat. It is believed that the peat was placed there in the early days of the reservoir because the undertakers owned the reservoir and not the catchment, and when they stripped the peat out of the reservoir area they had nowhere to put it except on the downstream face of the dam. The considerable thickness was removed and the original slope of the dam exposed. Also evident on examination was a small scarp on the downstream slope, almost certainly a slip scar of unknown date.

Study of the cross-section of the dam (see Paper 5.5) in relation to borehole observations indicated peat on the upstream and downstream sides of the core at exactly the same height, and reeds on the downstream face forming what is quite plausibly a horizontal layer of extremely poor fill going right through the bank at that level. Probing through the peat near one abutment showed a most irregular downstream slope to the fill proper. I think a slip had quite definitely taken place, probably underneath the peat so that nobody knew it was there - the peat was simply levelled off to produce a fair surface.

A longitudinal section of the core was built up from study of boreholes sunk during the remedial work at intervals of about 3 m, there being no adequate record drawings. Within the core one could distinguish certain zones. There were two horizontal layers where the drilling teams noted that they were drilling through what they described as boulders but were probably stones. Harder patches of clay were also encountered, and we know from the state of the four samples that there were some extremely hard and brittle patches in the core. Above the top layer the consistency of the core is quite different. It was much wetter, much softer, and whether that represented a change of borrow pit or a change of Inspector I am not sure. Plotting the Liquidity Index of the core against depth and the relationship of existing moisture content of the core to the Plastic and Liquid Limits gave some scatter of results. Below a certain level, however, where we were almost certain there is an original puddle trench, there is a fair degree of uniformity. The upper part of the core, above the so-called boulder layer, was distinctly wetter. Comparing this with the downstream shoulder, there was a tremendous variation in the relevant moisture content and in the Liquidity Index. No pattern was evident and I have suggested in Paper 5.3 that the fill was probably put in over a range of moisture contents - different parts of the borrow pit etc. - and it probably contains within it a record not only of the subsequent effects of seepage and consolidation but possibly even of the weather at which it was placed. We do know for another dam, Stocks Dam, that one could correlate patches of wet fill with the winter stillstand in construction.

I would like to put forward for discussion that we really know very little about the long-term behaviour of clay fill and what happens to it inside a bank, particularly when the seepage water has a pH of something like four, as it did at Withens Clough. We are only now beginning to understand what happens to natural clay slopes after 100 years or so, and I think we should look inside earth dams and try to understand what is happening to them over a similar period of time.

A J CRAIG (North West Water Authority) :

The remedial works carried out on the Lower Roddlesworth and Rake Brook reservoirs (Paper 5.8) differ somewhat from others which we have been hearing about at this Symposium in that they were concerned with the adequacy of drawdown capacity for emergencies. The alternative to carrying out such remedial works was obviously abandonment, a course considered as the reservoirs were two of a complex of eight, but this would have cost more than the remedial works and would have lost about 10 Ml/day of yield.

Ground conditions were monitored by piezometers prior to and during the course of the work, using Casagrande tips installed in boreholes and supplemented by driven standpipes. Thirteen were used at Lower Roddlesworth and 17 at Rake Brook. The downstream results showed very low pore pressures and many piezometers in the fill itself remained completely dry throughout the course of work. The upstream readings, as one would expect, reduced as the weather levels decreased during drawdown, displaying a lag due to the nature of the fill and compounded by deposits of silt found on the upstream pitching. One piezometer upstream of the assumed clay core at Lower Roddlesworth gave an exact correlation with drawdown level. As this bore had passed through a layer of organic material at valley floor level the inference was that a lens of this material lies beneath the upstream fill.

I mentioned silt on the pitching. Drawdown of the reservoirs exposed vast quantities of silt, as they have been in service for about 120 years. Towards the end of the drawdown it was estimated that about 40 000 m^3 had been lost down the river. Some of this collected in a privately-owned reservoir downstream, and to say we were rather unpopular would be putting it mildly. We built dams across the river to contain the silt during pumping. These were buttresses made of railway sleepers, tubular scaffolding, steel tracking and an impervious membrane of polythene sheet. They were fairly successful and they had the advantage that they did not require the attentions of the Panel Engineer.

Improvement of access to the dams was an important although comparatively inexpensive part of these works. The roads were made up to a standard suitable for heavy traffic and plant in case of any future emergency - I think the Lluest Wen incident taught us a lesson about access to reservoirs in remote areas. Finally, the making up of the roads and the grading and grassing of the spoil tips was considered important in this day and age. It gave us an opportunity to tidy up the area and improve the local environment.

A I B MOFFAT (University of Newcastle upon Tyne) :

S In the concluding section of Paper 5.6 Mr Knight states that 'the past is the key to the present'. I would endorse his statement and proceed further to say that valuable lessons are to be learned from the analysis of failures or near-failures of dams. A revealing demonstration of this arises from a recent instance in this country, when the small Warmwithens embankment dam near Oswaldtwistle, Lancashire, failed in November 1970.

With the permission of the then owners I had an opportunity to visit Warmwithens about 48 hours after failure occurred, and in view of the Reporter's mention of the event a brief autopsy may be of interest. I must emphasise, however, that my remarks are based entirely upon a short period of visual examination after the event, and any opinions must be purely personal.

Warmwithens Dam is - or was - an embankment of some 10 m to 12 m maximum height dating, I suspect, from the 1860's. The small reservoir it impounded lay in series above two other small reservoirs, Cocker Cobbs and Jackhouse, the latter lying just to the South of Oswaldtwistle.

The first indication of escape of water was detected about 07.30 hrs on 24 November, the outflow rising to a maximum some two hours later. By 13.30 hrs the dam was completely breached to foundation level, the 115 t c m of water retained in the reservoir discharging into Cocker Cobbs reservoir and on into Jackhouse reservoir which, in turn, spilled into a small river. It was fortunate that breaching was slow and progressive. The lower reservoirs were thus able to cope with the inflow with only very minor damage and were in no real danger, but the possibility of the cascade failure of three old dams in series - the last a not uncommon circumstance in this country - was clearly demonstrated.

Examination of the Warmwithens embankment showed it to have apparently been of homogeneous section and to be constructed from fairly heterogenous clay soils. The breach had a width of some 20 m at crest level and was situated over the line taken by the drawoff works. Precast concrete tunnel segments were lying in the debris downstream of the washout and part of the tunnel they had formed was exposed near the upstream toe. Slightly further upstream and just inside the empty reservoir basin a trickle of water could be seen disappearing into a small rectangular enclosure formed with sheet piles. On the site of the washed-out masonry headwall at the downstream end of the drawoff works a similar flow of water, say 50 l/s, was emerging as a spring.

From the evidence quoted, notably the precast segments, it can be surmised that the drawoff works had been reconstructed comparatively recently to incorporate a new drawoff tunnel driven in heading. For reasons which you will understand it would be inappropriate for me to make any comment on the failure mechanism beyond saying that seepage through, or along the perimeter of, the abandoned cast iron drawoff pipe, or possibly even along the perimeter of the later precast tunnel, clearly played a major part. Suffice to say that the Warmwithens incident underlines those problems inherent in the fact that so many of our old embankments contain a 'time-bomb' in the form of pipework buried directly in the fill.

Turning to a related theme, Dr Penman referred to Mr Kennard's mention in Paper 5.1 of the question of the resistance of clay fill to overtopping. A unique opportunity to conduct a full-scale investigation into this question using an old dam, arose last year but, for various reasons, could not be taken up. A brief description of the circumstances may be of interest.

Dunford Dam, an embankment 24 m high dating from 1858, is now submerged in the newly commissioned Winscar Reservoir. Plans for the completion of Winscar and the abandonment of Dunford called for the latter to be permanently breached by dragline at a late stage. The operation was to be carried out with Dunford reservoir at a low level and the newly impounding Winscar reservoir at the same level.

In the course of a BNCOLD visit to Winscar in 1974 it was noted that the circumstances were such as to permit a unique experiment in 'overtopping' and erosion. The plan, devised in collaboration with Mr J D Humphreys, Panel Engineer for Winscar, involved cutting a 10 m wide shallow notch in the crest of Dunford and damming the notch with sandbags placed on a steel net sling. With the Winscar impounding level about mid-height up the Dunford downstream face and the latter allowed to fill to about 200 mm above the invert of the notch, a derrick would remove the sling carrying the sandbags to permit overtopping. Erosion rate would have been studied in relation to grid markers by time-lapse photography.

We were completely satisfied that with erosion taking place into the 'cushion' formed by the Winscar impoundment no damage would occur to the latter dam and that turbidity induced by the washout would be short-lived. In the event, however, little time was available for detailed evaluation of the project by the Yorkshire Water Authority and they felt that the time schedule for putting Winscar on supply was too tight to accept the turbidity risk. The plan was therefore not put into effect.

I would conclude by making the point that opportunities to conduct experimental work of this sort are all too rare. The value of the information they could provide is potentially very high, however, and I would suggest that we ought to be on the alert for opportunities to conduct such field research at low cost as may arise in the course of construction/reconstruction work.

A J LEACH (South Staffordshire Water Company) :

Ⓢ Wave action on dam walls was already mentioned today and I would briefly like to outline, with the aid of a film, such an occurrence at Blithfield Reservoir in 1962.

Blithfield is an earthfill dam, protected on the water face by concrete panels and a precast concrete wave wall to some 2.5 m above TWL. In February of 1962 a severe storm blowing across the reservoir created a wave action of such intensity that it overtopped the dam and carried across the access road on to the downstream face of the dam. Saturation took place, causing a slip movement on the downstream face. Further erosion took place after movement of the precast concrete wave wall and under the concrete panels.

During the storm, which lasted about six hours, one of our engineers was present with a cine-camera, and he managed to take a film of the storm at its most intense which I shall now show.

Editors Note:

Mr Leach then showed and 'talked through' a short 16 mm film of the Blithfield storm, illustrating the spray being swept over the wavewall in sheets and on to the downstream face of the embankment. The conditions can be gauged from the fact that in some of the film sequences the crest of the dam was completely hidden by driven spray. The film concluded with shots of the immediate remedial action taken to deal with the damage. Mr Leach's comments on the latter continue below.)

Remedial action was taken the following morning, the concrete panels being completely undermined in one section. This area was completely refilled and the concrete slabs put back in position. As a result of the overtopping we then decided to put a mass concrete wave wall on top of the existing coping. The downstream face, although not severely damaged, also required immediate remedial action and filling of eroded and slipped sections with sandbags and ballast, and it was then re-soiled and seeded. The film does show the significance of wave action on dam walls, and it is noteworthy that the precast concrete units were completely lifted out by the wave action - one was lost completely and has never been seen.

H D OSBORN (Babtie, Shaw and Morton) :

Ⓢ Referring to Buckieburn Dam and my Paper 5.9, there are three points I would like to draw attention to.

The first is that there was a natural shoulder on the north side of the dam which tended to drain in towards the dam and probably contributed to the saturation of the slide area. Another point is that on the parapet there is a gap in the parapet wall. This was the access gateway to the valve tower, and after the slide had occurred and we inspected the reservoir there was evidence of waves having passed through that gap. There was, too, evidence of very heavy spray also having overtopped the dam. The final point is that the spillway at that time was an open spillway which continued into two 450 mm dia. fireclay pipes to the stilling pool. A scar was evident on the bank between the open part of the spillway and the stilling pool.

This was evidence of some overloading of the spillway, and in fact there was also experience of difficulties with ice blocking the pipes below the spillway.

Turning to the slide area, when this was subsequently excavated agricultural drains were found completely blocked up. Quite a system of agricultural drains existed in the downstream face of the dam, and they were all completely useless.

The first thing we did was to construct a rockfill toe-wall.. In order to continue the compensation water culvert, which also contained a 250 mm dia. drawoff pipe, through this toe-wall we adopted colliery arches with steel plates. Some 2.5 m of rockfill lay below this level at this stage, and the compensation water was temporarily flowing through the rockfill. The soft material and the top soil in the area of the slide was removed and a blanket layer of rubble fill placed before the main fill was laid.

On completion of the initial remedial works a second contract was placed to complete other works, which included the pitching to the toe-wall, a concrete lining to the culvert, extension of the draw off pipe, and renovation of the existing stilling pool and the spillway.

During earlier discussion, Session 4, Mr Hamilton referred to the question of overtopping of reservoirs by wave action, and I would like to make one or two remarks on this which also relate to the Blithfield film we have seen.

In the case of Buckieburn the upstream parapet wall had a concave face. This form of concave face, often seen in sea-defence works, turns the wave back on itself. The fact is, however, that the wind is coming in the same direction as the waves on a reservoir. This type of concave face therefore has the effect of transferring the wave from a horizontal to a vertical travel, and consequently one gets the wind carrying the wave over the crest. I would suggest that on embankment dams a concave type of wall should not be used. It is preferable to have a slope with riprap or armouring at the top in order that the wave energy is absorbed, and extending sufficiently far to ensure that the run-up of the wave does not come anywhere near the crest.

As a concluding point, the total cost of the remedial works at Buckieburn was £79,000 at 1971 prices. The initial remedial works were carried out on a daywork basis, the spillway and other works were on contract.

M F KENNARD (Rofe, Kennard and Lapworth) :

S The Reporter took up from my Paper 5.1 several points which I suggested merited further investigation. I did not know he was going to suggest that I discussed them as well, but I would take him up on the first one, the resistance of old embankments to overtopping. My point on this is that a lot of dams can withstand a certain amount of overtopping, and I think this is relevant also to the discussion in Session 4, where the inference was made that if a design flood is exceeded failure must occur immediately. I would suggest this is not always the case, although there could well be such cases, but I do not want my remarks to be taken to mean that I am saying that there is no problem even if all dams were overtopped. I am not saying that.

I would also say that we should not take immediate action where we find overtopping is possible, and certainly immediate action should not be taken in an irresponsible or hasty manner. There could well be cases where the safety of the structure could be affected by actions taken to increase the free-board to prevent overtopping. Hasty remedial action could thus lead to a more risky situation, and I would like to briefly mention four cases where such trouble could occur.

Adding a wave wall or extra fill on top of a dam could lower the factor of safety locally and lead to local slipping of the top of the dam. This is referred to in papers by Mr Little and myself, and in this connection I would like to mention that the Blithfield case could be one where adding extra weight to the wave wall could effect the stability of a small section of the upstream slope.

Another case where hasty action could lead to trouble is where a wave wall is built and could divert run-off, which previously ran into the reservoir, to the downstream side and into a mitre between the fill and the dam where there is no drainage. Erosion could occur, and I know of a case where this happened in North Devon last year.

Removing trees because people, quite rightly in some cases, do not like trees growing out of banks, could, if one removes the trees quickly and without full consideration, also alter the drainage pattern of the fill. Warmwithens showed a homogeneous bank clay fill, so with trees growing on such a bank that fill may be drained. If the trees are removed or felled there may be increasing pore pressures in the downstream shoulde

Similarly, when rapid drawdown takes place in order to carry out some hasty remedial measures there could well be local slipping.

In a recent American publication - 'Lessons from Dam Incidents, 1966 to 1972' - 120 cases were described. Five involved overtopping. In one case the dam was overtopped for two or three days during construction.

Considerable erosion took place but the embankment did not breach. In another case there was 1.6 m flowing over the lowest part of a fill which was under construction. It flowed over for several hours and it was found that the core material was highly resistant - some fill was lost, but the dam did not breach. An old dam was overtopped by 0.5 m for several hours, and again considerable erosion occurred without failure. In yet another case an old dam was overtopped by 0.4 m for at least 40 minutes. There was washout of a local breach in one of the abutments, but the dam did not fail. In another instance there was overtopping during construction, this time with considerable loss of fill and failure.

The Oros Dam in Brazil failed by overtopping during construction. To try to avert the failure about 5 m of uncompacted fill was placed on that dam over three days. The next day it was overtopped and seriousl breached, but not until 0.8m of water was flowing over the uncompacted fill. A lot of Panel 1 Engineers have had experience of seeing dams which have obviously been overtopped in their past.

There was a storm in North West London about four weeks ago when the papers reported 150 mm of rain falling in three hours - I have not been able to check these figures. If it was such a storm it falls on the line of a quarter of the maximum world rainfall, which is generally considered to be the maximum for this country. Within the area affected by that storm lies Hampstead Heath, where there is a lake at Kenwood House, and also the five Highgate Ponds which were built, probably well over 200 years ago, for water supply to that part of London. These are low banks up to about 5m or 6m high and every one of these banks was overtopped in the storm, but they are still standing. I went to have a look some two days later, and although there had been some damage to footpaths and some local erosion, not one of the footpaths was closed. Nobody seemed very concerned; in fact there were fishermen sitting on an area which had actually slipped! This does show that with excessive storms of short duration old banks can withstand a certain amount of overtopping, and with this evidence I do not consider that the theoretical inability of an embankment to cope with a design flood without overtopping is necessarily to say that failure will occur.

D M HAMILTON (Crouch and Hogg) :

With reference to Buckieburn and Mr Osborn's contribution, I would mention steps taken to deal with West Corrie Dam in Stirlingshire, which manifested something of the same trouble, and I think possibly for the same reasons, about twelve months after the Buckieburn incident.

At about the time of the Buckieburn incident there was a certain amount of 'rumpling' evident on the vegetation and the turf on the downstream side of West Corrie, at about one third of the length from one end, on the outer side of a curve in plan. In late October and early November 1971 it was seen that this was continuing and developing, and we had boreholes put down. Some of the material could not be sampled properly at all, and our observations really amounted to the fact that the embankment was saturated and the material was in a very unstable condition. What was done was to advance three trenche spaced about 15 m apart, in towards the core wall - incidentally the site slopes away downstream. We advanced these up on the basic natural clay 'bottom', installed 150 mm dia. pipes, and filled the trenches with 'no fines' concrete which had the merit, I think, of giving some strength to the bank, and at the same time providing an adequate drain to the saturated fill material.

N HOYLE (North West Water Authority) :

In Paper 5.8 the authors make passing reference, on page 6, to the reduction in the crest levels of Lower Roddlesworth Dam during the drawdown. This I find most intriguing. Engineers have regarded settlemen of completed earth dams as largely a function of time, but the Lower Roddlesworth observations suggest that, apart from being a firm structure an earth dam is an amorphous structure, the shape of which can change as a function of the water level. It is a pity that the levelling was not followed through during the re-impounding of the reservoir, but I would suggest to those engaged in research that this is a subject which is worthy of further investigation, i.e. the behaviour of a dam in relation to water level.

A second and quite separate point. In the early Sessions of this Symposium there has been a pre-occupation with the possible hazard to dams of overtopping caused by flood inflow. In paper 5.2 the authors say that in the course of inspections, and I quote, 'particular attention should be given to any culverts or pipes through the dam whether these are for spill or drawoff, as these are a potential source of leakage and eventual washout'. Having seen a very recent slide of one failure, Warmwithens, that seems to be an understatement. In fact two papers, relating to Withens Clough and to Lower Roddlesworth, describe corrective works in respect of insecure draw-off arrangements. There are many dams where the draw-off consists of a pipe laid within the body of the dam, usually at the bottom, and often with no up-stream valve. Even if there has been such a valve it is usually no longer operable, and I do know that about half a dozen engineers present here today view such situations with extreme concern.

A flood of excessive value may or may not occur. If it does occur some inflow will be stored in a reservoir which is not full, and if it does overtop the reservoir it may or may not cause erosion. Having the experience of Lluest Wen and Warmwithens to draw upon we know that collapse of a pipe within a dam will in due course almost certainly lead to collapse of the dam. The deterioration of cast iron is a function of time. I would refer back to Mr Moffat's time scale in Session 1. What is going to happen in the next 20 to 30 years? In my view the risk of collapse of a dam due to a broken outlet pipe is an ever present risk, whereas risk of failure due to flood damage is intermittent.

J L ADALID (Ministry of Public Works, Spain) :

S The Perdiguera embankment dam, some 90 years old, has a height of 11 m and a crest length of 200 m. The reservoir, fed by a diversion canal, impounds about 500 t.c.m. used for seasonal irrigation of some 1400 ha of vegetable crops. The reservoir is a private one operated by the farmers concerned.

On 17th April 1972 the waterman noted some leakage and slight settlement. The outlets were opened and the farmers surcharged the downstream face in an attempt to effect a repair. This resulted in a deep slip on 25th April. No populated area was at risk, but people were kept out of the possible flood plain and, in view of the very great value of the crops under irrigation, the attempt was made to retain as much as possible of the impounded water. Police with radio links were posted to safeguard road traffic and emergency repairs were undertaken as follows:

1. The outlet pipes were closed at the upstream end by divers.
2. Sand and gravel was imported some 1.5 km from a river bed to form a more stable and drained toe.
3. Downstream slopes were flattened by re-arrangement of displaced material.

Local resources were used to carry out the repairs, which cost about £3000. The opportunity was also taken to construct a pumping link from a canal 3 km away, thus effecting a general improvement to the facility. The dam has behaved satisfactorily since the repairs.

S NYLANDER (Swedish State Power Board) :

I would like to compliment the authors of Paper 5.2 on inspection of embankment dams and make some supplementary comments.

The authors indicate the need to inspect the dams when at or near TWL, and I would add that it is similarly desirable to inspect the dam when at low water levels in order to check if erosion damage is commencing. It may even be necessary to inspect underwater, using divers or frogmen.

The difficulty with the latter often lies in ensuring good communication by telephone and/or television and in ensuring that the diver imparts appropriate information to the engineer. One answer to this lies in the use of a small submersible or submarine. In France, for example, it is stipulated that large dams, mainly concrete dams, have their upstream face inspected every 10 years, and a submersible is now used for this purpose. Such a technique is very expensive, but the idea may be of some interest.

A C ALLEN (Allen, Gordon and Company) :

S In dealing with embankment dams, particularly old ones, inspecting engineers will often have come across the kind of situation where there is a depression on the downstream face of the dam. There is perhaps a hump, the kind of situation that we have seen at Buckieburn but probably more overgrown. There may be coarse grass around and so on, and the previous Report may be a one-page effort of about 100 words which gives little or no indication of history. The Panel Engineer may have no idea whether that slip occurred eight years ago or whether it may have occurred 80 years ago. What is the situation? How may they cope with this? They do not know if it is a rotational slip, they do not know if it is a surface slide, whether it is due to high phreatic levels in the dam, or due to saturation from overtopping by wave action such as we have seen this afternoon.

The essentials, I suggest, are as for concrete or masonry dams, i.e. to monitor a parameter such as movement to observe any change. We have not heard much mention of movement of embankments, and I would suggest that a useful indication may be obtained from at least nine vertical markers on a grid encompassing any doubtful area. If there is any movement some tilt on those markers will soon show it up, and they can be surveyed at intervals. This is a useful method of determining if the problem is an old slip which has stabilised or if it is something which is developing.

Coupled with this there should, of course, be measurement of leakage downstream of the dam. If I may turn to a problem example, on the night of 9th/10th March this year in the County of Angus, Scotland, there were severe gale conditions on an extremely cold night. The storm was so severe that trees downstream of a Glenogle reservoir were toppled over by the weight of ice forming on their branches combined with high winds. That particular reservoir is formed by a dam which is of composite type, and as a result of wave action a local failure took place. Waves overtopped the dam and water saturated the fill. The spillway arrangement was a suspended reinforced concrete overflow, with the embankment continued underneath the spillway, and a concrete dam beyond. A slip occurred under this area, but fortunatly the storm abated before any further collapse took place. It provides another illustration of wave action, and perhaps we should also bear in mind the combination of wave action happening on a very cold night, with ice on trees or on long grass on the downstream side of an embankment.

Turning to another dam, referred to in Session 1, which failed and sent 22500 tcm down a certain Scottish river in a fortnight without anybody knowing, that dam was quickly replaced for economic reasons by a concrete dam five metres high founded on what may be loosely called boulder clay or mountain till. I refer to it only because Mr Moffat was not successful with his plan to study the rate of erosion of clay fill in the Dunford Dam. Boulder clay in its undisturbed state has tremendous strength. We have bored through it and got cores out similar to rock cores. It is a different story if it gets saturated and pore pressures are generated in it, but in many Scottish streams - and in Northumbrian ones as well - it forms the bed of the streams where they run very rapidly, and it might be worth checking erosion of materials in that way.

Lastly a very small point that may be significant. I am not happy with this word *massive.* Could I suggest we use *rigid* to refer to concrete or masonry dams?

R FELLOWS (Central Regional Council) :

I would like to add something to the history of the dam referred to earlier by Mr Hamilton, West Corrie Dam.

Prior to the slip and prior to the 'rumpling' effect which was noticed on the embankment we had for some years observed a damp patch which was outwith the toe of the embankment. It was not clear whether seepage was coming from the reservoir or from high ground alongside the reservoir. We had tried sampling the water and analysing it to no effect,and so we considered using tracers, although there had been little success with these in the past. The tracer I chose, however, was one which the Water Research Association had used for calibrating meters, lithium chloride, and I was advised that this had the advantage of being non-toxic. It also did not disperse through the water too readily, so one could get it to stay around the upstream face of the embankment without rapidly dissipating. We introduced the tracer by the crude method of throwing a hose down the upstream face at intervals along the embankment and rapidly withdrawing the hose as we poured the tracer solution down. We did this over a period of 24 hours and sampled over 24 hours, and it proved effective as we were able to detect the lithium chloride downstream.

With respect to the effect of wind I have twice observed, on one of our reservoirs which has a fetch of about five or six km, that after a heavy frost when ice has formed on the reservoir to a thickness of 10 mm or 15 mm and has been followed by a severe gale, the ice banks against the embankment in slabs tipped and driven along the side of the embankment to a depth of something like 0.5 m. The effect is to raise the level of the overflow sill, and although the ice would eventually be washed over by any increase in the level of the reservoir one wonders what effect that increased level of the spillway might have during a high wind.

W J F RAY (Thames Water Authority) :

With reference to Paper 5.2 by Messrs Clarke and Le Masurier, it could perhaps be inferred from the second paragraph of the introduction that an Inspecting Engineer making his first inspection might be unclear as to his broad duties.

Surely one may expect that no Panel Engineer, and in all probability no Supervising Engineer, is ever likely to be appointed and required to carry out statutory inspections or supervisory duties until after accompanying, on numerous occasions, an experienced Panel Engineer on inspections of a similar nature. Nevertheless a comprehensive check list is obviously useful to an Inspecting Engineer.

In Mr Knight's paper (Paper 5.6) attention is drawn to high seepage profiles in the downstream shoulder of the Cowlyd embankment. If in such circumstances it can be established with reasonable certainty that the concrete core wall is pervious, then presumably if other remedies are unsuccessful consideration could be given to the construction of a further core upstream of the existing core - either by grouting or diaphragm walling in plastic concrete, of the type described in other papers.

Notwithstanding such difficulties as have been experienced with concrete core walls, if suitable clay is not readily available an articulated concrete core does represent a viable alternative provided care is taken, particularly at joints. My experience of an articulated concrete core incorporated in a water supply reservoir has been satisfactory. It is reasonable to infer that whilst a concrete core may leak it is inherently less likely to induce failure than a clay core of the type used at Balderhead. On the other hand effective remedial work on a concrete core is more difficult to achieve.

The overall point that seems to me to emerge is that any leakage or movement arising in an earth dam needs to be reported immediately and remedial action taken as quickly as possible.

T M HYDE (British Waterways Board) :

I would like to make a small correction to a point in Paper 5.9, where it refers to Norman Smith's 'History of Dams' and Todbrook and Coombs dams are mentioned as having been completed in 1794. That is an error which can be blamed on British Waterways Board's Estate Department who provided that information. In fact 1794 is the date of the Act under which the dams were constructed, and I think the first large dam in the country was March Haigh.

I would also like to make a brief comment on what Mr Ray has just said about a Supervising Engineer accompanying Inspecting Engineers. I have accompanied a number of Inspecting Engineers on numerous occasions, and the difference between the rigour of these inspections has been quite startling. Perhaps one ought to add the further qualification that there ought to be approved lists of Inspecting Engineers whom the Supervising Engineer would accompany!

I particularly want to speak about vandalism, which has not arisen in discussion yet. Twenty-five years ago vandals did not seem to get out into the hills very much. Things have changed rapidly - we have a reservoir in the Campsie Hills, near Glasgow, which is quite a stiff walk from the nearest road. Our staff built a very solid hut up there to carry out remedial work and in absolutely no time at all this hut, which was very securely put together and locked, was opened to the four winds and all four walls were flat. Amenity users can also be difficult. We have a reservoir - in fact it was Coombs - which had the spindles sawn off the outlet valves. We have never been quite sure whether it was the sailing people or the anglers, but we are very sure it was sawn off so that we could not drop the reservoir level any lower!

It is not only a problem of vandalism in direct relation to safety but also in relation to increased instrumentation. I just do not know how the increased instrumentation on some of our remote reservoirs is going to be sustained at all with the calibre of vandalism which we now have. Any advice that anybody has on this problem with old earth embankments would be very welcome.

J D HUMPHREYS (Mander, Raikes and Marshall) :

The Reporter nearly spoiled my afternoon by implying that as a Panel 1 Engineer in some recent inspectior I and others may not have done our job. I could not let this go unremarked, and I am quite sure that one or two other people will feel the same.

I do not care if all the authors of this afternoon's papers feel that in inspecting embankment dams one must get into the guts of them, and I do not care if the Reporter agrees. I do not believe that this is always the case. I think the Panel Engineer, in this as in all other respects, has to use his own judgement. I also think Dr Penman made the point for me by demonstrating the relatively small difference made to shear strength by different values of ϕ'. Even this would, I believe, be found by most engineers to often be enough to make the difference between an acceptable and an unacceptable condition. I believe that the older the dam the more one's interest shifts from the limit-state stress analysis towards an analysis of the deformations. I think that generalisations are always misleading and all I ask is that the Reporter concede tha I think that in general there is often as much value in studying, or making provision for study, of deformations as there is in pore pressures or any other limit-state stress analysis parameter.

I am glad that Mr Tattersfield made a contribution that reminded us of the existence of tailings dams. It was particularly timely, because this is now included in the list of Questions for the 12th ICOLD in 1976. I think it is worth noting that the biggest dam under construction in this country at the moment is in fact a tailings dam. In other words, we are not just talking about small sludge lagoons but about structure which are definitely significant. I think that it is a pity that Mr Tattersfield almost lulled us into believing that all mining engineers are totally irresponsible, even though he admitted to having tongue in cheek. I think if he visited the particular dams to which I have just referred he would see that the workmanship there stands as an object lesson to the average British contractor.

Finally, I must say that the fact that old engineers used to look for marsh plants downstream of the dam does not strike me as being a good reason for my not doing it.

REPORTER : Dr A D M PENMAN (Concluding Summary) :

It is good to hear that Mr Humphreys uses all the methods available when inspecting embankment dams, thereby combining the wisdom of the older Inspecting Engineers with all the knowledge now available in the geotechnical field. I do agree that measurements of deformations could give valuable information about performance.

Mr Tattersfield has given us a salutary warning about conditions of waste tips which can so easily become dams. Many sludge lagoons have been formed in this way, and when one considers the disastrous effects of failure, as witnessed in Chile and South Africa, it seems a pity that liquids other than water have been excluded from the new Reservoirs Act. Perhaps the fact that a lagoon is a reservoir 'capable of holding water' may yet bring them within the meaning of the Act.

The experiment of deliberately overtopping the old Dunford Dam proposed by Mr Moffat would have been of considerable interest in studying the resistance of some of our old dams to overtopping. It is unfortunate that it did not prove possible to carry out the experiment. I think that experimental overtopping is rather rare: the only example I know of is that carried out by Marsland (1966)[1] on an existing flood bank as part of his study of the 1953 breaches.

The film shown by Mr Leach gave us all the unique chance to study the effect of a 90 mph gale blowing straight down Blithfield Reservoir to the dam with no more personal inconvenience than the pleasant breeze of conditioned air. The amateur cameraman is to be congratulated on his photography and on his devotion to duty! Although the top of the wave wall was 2.5 m above TWL the curved shape, usually accepted a effective for sea defence works, gave the waves a vertical component which carried the water up into the high velocity air stream passing over the dam crest. Mr Osborn was quick to suggest that a very coarse rip-rap on the upper part of the upstream slope might prove a better way of absorbing wave energy, rather than attempting to deflect the wave back into the reservoir. Mr Kennard has given us some assurance that overtopping does not immediately cause failure and can be tolerated for quite a time in certain conditions.

Mr Hoyle asked for research on the movements of dam crests caused by fluctuations in reservoir level, and I would refer him to current work by the Geotechnics Division of the Building Research Establishment.

Perhaps our most exciting contribution came from Mr Adalid, with the second frightening experience from Spain which he has been kind enough to reveal at our Symposium. To save the water, and thereby a very expensive crop, in the face of a major slip in the downstream shoulder of the 11 m high Perdiguera Dam was an excellent achievement.

Reference

1 Marsland, A (1966) *'The design and construction of earthen flood banks'.* Ch 24 in 'River Engineering and Water Conservation Works' by R B Thorn, Butterworth.

CHAIRMAN: R T GERRARD

Thank you, Dr Penman, for reminding us of the highlights of this afternoon's discussion. I think we all appreciate the very considerable amount of work that the General Reporters do in reading all the papers and drawing to our attention the most important features.

I now have to make a few closing remarks about all the Technical Sessions. The first point which strikes me is the very shrewd choice that the Steering Group made in phrasing the topics for each Session. It seems to me that we have covered the ground extremely well, and there has been very little overlap in the discussions between one theme and another. The choice of themes evoked a very good response, and I thank all authors. We have had a lively discussion during the last two days, and a healthy divergence of views, and I think this Symposium has taken us another most useful step forward in the improvement of practice and procedures for dealing with existing dams.

I believe Mr Grøner would now like to say a few words.

C F GRØNER (President, ICOLD) :

Mr Chairman, Ladies and Gentlemen. On behalf of all the participants I would like to thank BNCOLD through Mr Gerrard, and I would also like to thank the University for this very interesting and important conference. The discussion yesterday and today shows the great interest everybody has in the different themes selected for the Symposium.

We all know what an enormous amount of work there is in organising such an event, and I have a feeling that Mr Moffat has had most of the burden. With the Steering Group and his various helpers he has done a marvellous job coordinating the small details which have to fit together in a successful symposium. I hope that you, Mr Moffat, will give all your helpers our gratitude for the good work they have done.

We also had a wonderful evening yesterday, and a lot of people preferred to take a taxi to Hall instead of using the buses provided!

Maintenance and inspection of dams is very important, and we have but a very short time to push this because out oldest dams are now very old. We heard that message today and we heard it yesterday. Operation of dams and flood estimation is also important to avoid damage and tragedy. Instrumentation, too, is vital, including the question of degree of instrumentation. On leaving a symposium like this a lot of people go home and have a lot of work to do. They forget many things, but I would hope that after this symposium we shall go home and try to further study of the themes we have discussed.
I think there will be different solutions in different countries, and I think also in different places here in Britain.

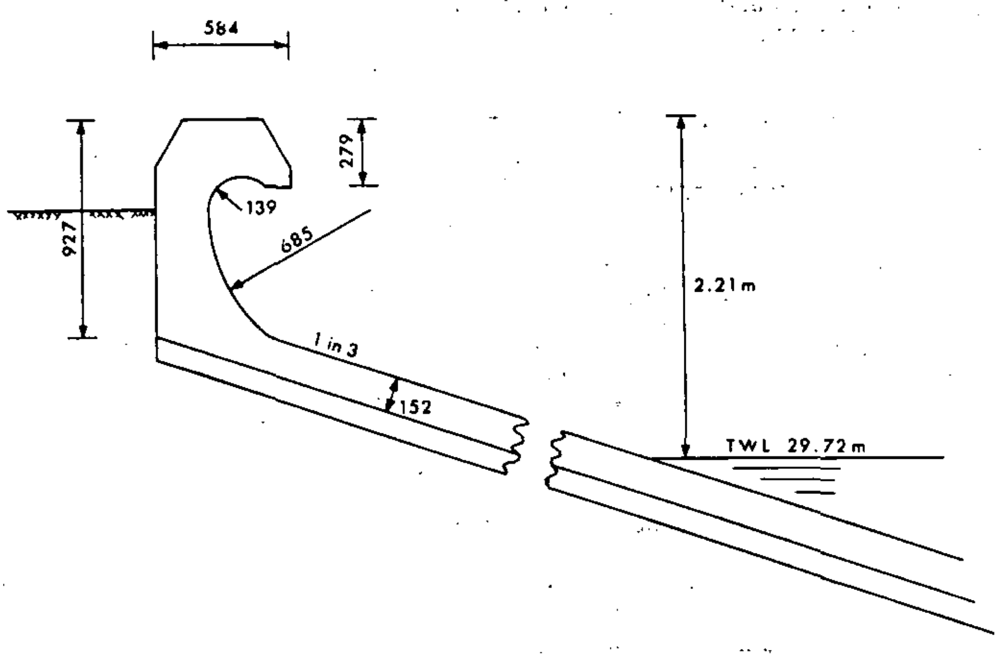
Let us conclude by thanking BNCOLD and the University of Newcastle upon Tyne.

WRITTEN CONSTRIBUTIONS

P COOLEY (Thames Water Authority) :

There has been some discussion on the erosion of embankments by wind-driven water, and it is evident that a largely vertical wall at the head of the upstream embankment slope can cast the surf up into the wind to be dropped, in the downwind eddy, on to the outer slope of the embankment. The use of rip-rap was rightly advised to absorb the energy of the waves.

There is another solution, useful where broken rock cannot be found locally and which was tested on a 1 : 12 model scale by the Metropolitan Water Board in 1946, although without wind force, under waves equivalent to 1.1 m height at full scale. It has been used effectively on the four storage reservoirs with a fetch of more than 1 km that have been built and filled since then. The wave wall is curved inward and down as shown on Fig. A so that, as in the model tests, water from breaking waves that is projected up the embankment slope is mainly cast back in a downwards direction.



WAVE WALL KING GEORGE VI RESERVOIR, STAINES

DIMENSIONS IN mm

COOLEY, P : FIG. A

C C PARKMAN (Ward, Ashcroft and Parkman) :

I am stimulated by the remarks made by Mr Hoyle to endorse the problem he has raised, that in many of the old dams the supply pipe and/or the scour pipe pass through the embankment. As we have heard these dams are upwards of 60, 70 and 80 years old. There must be a limit to the life of these pipes, and the danger of erosion from a leaking pipe must be recognised. The amount of testing to check the condition of the pipe is limited, and until such pipes are taken out of these old earth embankments there will be a serious threat to their stability. Reconstruction in many cases would mean emptying the reservoir, which, in view of the necessity to maintain supply, is often not practical.

A temporary solution might be plugging the pipes with concrete, but this would only be temporary since the danger of leakage around the pipe itself in these old dams will remain very real. I tend to agree with Mr Hoyle that in many cases this condition in old dams imposes a greater threat to safety than the occasiona high-intensity flood.

J TATTERSFIELD (Watermeyer, Legge, Piesold and Uhlmann) :

I do of course accept Mr Humphreys' point that some well engineered dams are built on mining properties. My remarks from the floor were confined to those instances where this is not the case.

However, both Mr Humphreys and Dr Penmann clearly assumed that I had been talking about tailings dams, and this is not the case. I referred to dams of earth or rockfill which are sometimes massively oversized to compensate for the lack of care and engineering expertise which goes into their construction. Most of my remarks will not be comprehensible in the context of hydraulic fill construction.

D J KNIGHT (Sir Alexander Gibb and Partners):

Mr Ray, commenting on the high downstream shell water levels for the left bank section of Cowlyd Dam (Paper 5.6), suggested consideration of a further core installed upstream of the existing core wall, failing other remedies. It is doubtful, however, whether such an expedient would be cheaper than adding free-draining fill to the downstream slope. Moreover, as the seepage at that section is thought to arise from the several sources described in the paper, a further core wall might still not be effective in reducing downstream water levels.

The Reporter drew attention to the near failure of 1925 and to the unconventional manner of fill placement. However, both these aspects are in the past; their relevance now is as a spur to continued diligence in assessing, and where necessary acting upon, any significant warning signals which might arise from monitored behaviour of the dam in the future.

M Le MASURIER and C L CLARKE (Sir William Halcrow and Partners) :

Because of the lively discussion during Session 5 we were not afforded an opportunity to reply either to the Reporter's critique or to other comments, and we wish to take this opportunity to do so.

Our paper (Paper 5.2) stood out in the Report by the dearth of reference to it apart from the implication that we, the Authors, came from the old school who were sometimes reputed not to take much care over their inspections of reservoirs. This, being the view of a research scientist, may be understandable, since our paper made little reference to piezometers, triaxials and other tools of the geotechnical engineer. We make no apologies for this specific omission which resulted from our aim to keep the paper simple and brief yet pointing to the principal aspects to be covered in connection with an inspection. The section on 'Usual Remedies' was unfortunately omitted from the text of the pre-print of the paper but was circulated to the Reporter and to delegates at the Symposium; this referred to the need, sometimes, to have further investigations carried out (surveys, pits, borings, soil tests etc.)

(Editors Note: The section referred to is included in Paper 5.2 of these Proceedings.)

We do not follow the belief that every dam in the country must be thoroughly investigated by the use of extensive instrumentation. It is the duty of the Inspecting Engineer to diagnose whether such a thorough check-up is indeed necessary, and if he is in doubt then to call for it. If he were to insist on a full investigation contractors would have a thousand or two more contracts on their plates.

Out of some 45 earth dam embankments with whose Statutory Inspections one or other of the writers has been involved during the past four or five years, further investigation of the embankments have been considered advisable in 10 cases, generally involving borings and the installation of piezometers. We consider that to have carried out detailed soils investigations at the other 35 would have been a waste of the owner's money.

Mr Ray felt that our paper would be of little help to new Inspecting Engineers since they would undoubtedly accompany experienced engineers on inspections before they carry out one of their own. This seems sound and might well be a necessary qualification for the appointment to a Panel. Alternatively, if an engineer on appointment had not already made such 'dry runs', he could be advised to do so before accepting an appointment to a reservoir.

Mr Nylander rightly pointed out that underwater inspections can be valuable. Their application to small embankment dams is, however, *usually limited to the draw-off culverts.*

J W SEDDON (Severn-Trent Water Authority) :

At a period immediatley following the last war, the reservoir keeper responsible for the maintenance of the catchment areas and embankments of three impounding reservoirs in the Irwell valley with earth embankments reported having seen that day a large hole on the surface of Holden Wood embankment. The reservoir had a tunnel cut into the rock of the valley side and running at an angle to the dam on its downstream side. Compensation water only was drawn from the reservoir,via one or two cast iron pipes running down the tunnel and into a stilling pool provided with a measuring weir. The tunnel had some 1.2 m depth of broken stone concealing the drawoff pipes, possibly a war-time precaution.

A quick survey of the embankment showed the reported hole to be about 2 m square by 1.2 m deep and to be vertically over the tunnel. On inspection of the tunnel it was found that a curved masonry stop-wall across the tunnel at the downstream side of the puddle core had many joints leaking water and one or more masonry blocks missing. Attempts were made to plug up the leakages with quick-setting grouting materials, but all proved unsuccessful. A final repair was done by building a new concrete stop-wall downstream of the masonry, then finally grouting the interspace and the surrounding tunnel rock.

No drawings were available at the time, but it was assumed that the puddle core was protected by masonry at the upstream side of the tunnel also. Seepage had presumably carried part of the clay core into the tunnel through the defective wall and this had passed unnoticed due to the stone filling over the drawoff pipes.

As far as I am aware the repair still stands without any further work having been required. I would, however, comment that in the absence of frequent inspections by an observant reservoir keeper a total failure might have occurred.

R M ARAH (Binnie and Partners) :

Mr Cooley mentioned the use of an ETF type of diaphragm to lower the phreatic surface in one of the Thames Water Authority's Lea Valley dams. We had used this type of cutoff around the Offord Intake basin in 1963, where it had proved effective, and the cost was likely to be about a third of the cost of the slurry-trench diaphragm alternative for Withens Clough. However, it could not reach the maximum depth of some 40 m below crest level, it would not allow for cutting through the old culvert or into the underlying gritstones, and its reduced thickness would leave it more vulnerable to damage from movements or from the extremely aggressive water. The more expensive process was therefore adopted. It is interesting that the ETF system at Offord used six contiguous piles, and the operation consisted of drawing the rear pile whilst injecting grout, then re-driving it as the leading pile. Could the use of a single pile (as described by Mr Cooley) have led to the problems of loss of thickness?

The description of slots cut through the turf behind joints in a wave wall and the most educational film showing an emulsion of spray being driven in a fast-moving layer down the reservoir and over an invisible dam crest reminded me of a reservoir keeper's account of having to creep across his dam in the lee of the wave wall, the rest of the crest being unuseable because of spray being driven 'like shot-blasting'. It seems that wave walls perform two quite distinct functions: they turn back solid water, but they also deflect high-speed spray away from the crest, and the second function may be just as important as the first.

ALPHABETICAL LIST OF PARTICIPANTS

SYMBOL KEY

SC1	=	Session Chairman, Technical Session 1
GR2	=	General Reporter, Technical Session 2
A1.3 JA2.4	=	Author/Joint Author, Paper 1.3/2.4
D3(2)	=	Contribution to Discussion, Technical Session 3 (figure in parentheses denotes number of contributions, if applicable)
IS	=	Introductory Session
IA	=	Introductory Address
C	=	Formal Closing

A	ACKER, F A S	'Water Power and Dam Construction'	
	ADALID, J L	Ministry of Public Works, Spain	D1,3,5
	ADEWOLE, E O	National Electric Power Authority, Nigeria	D2
	ALI, Dr K H M	University of Liverpool	D2,4
	ALLEN, A C	Allen, Gordon and Company	GR3/A3.6/D3,5
	ARAH, R M	Binnie and Partners	A5.5/D1,2,3,4,5
	ASH, R V	South West Water Authority	D2
B	BAKER, H W	James Williamson and Partners	
	BASS, K T	Rofe, Kennard and Lapworth	A4.1/D1,4
	BELL, P D	Central Water Planning Unit	
	BERRY, D W	Howard Humphreys and Sons	JA4.3/D3,4(2)
	BINNIE, C J A	Binnie and Partners	
	BOURNE, M S	Soil Instruments Ltd	
	BRADLEY, J D	Severn Trent Water Authority	
	BRIDLE, R C	T and C Hawksley	D2
	BRODERICK, B W	Central Electricity Generating Board	
	BUCHANAN, F C	Tayside Regional Council	
	BUCKLEY, J S	Soil Mechanics Ltd	
	BUSHELL, E	British Aluminium Company Ltd	
	BUTLER, Dr J E	Portsmouth Polytechnic	D3
C	CAMPBELL, A D H	W A Fairhurst and Partners	
	CARLYLE, W J	Binnie and Partners	
	CASADO, J L	Ministry of Public Works, Spain	
	CHAPMAN, E J K	James Williamson and Partners	SC3
	CHARLES, Dr J A	Building Research Station	D5
	CLARKE, C L	Sir William Halcrow and Partners	JA5.2/D4(2),5
	CLARKE, R M	Jersey New Waterworks Company Ltd	JA3.3/D3
	CLOUDSLEY, A D	Strathclyde Regional Council	
	COATS, D J	Babtie, Shaw and Morton	
	COLE, R G	T and C Hawksley	JA3.3/D3
	CONDY, W A	Department of the Environment, Northern Ireland	
	COOLEY, P	Thames Water Authority	A2.1/D1,5(2)
	COOMBES, L H	Engineering and Resources Consultants	JA3.3/D3
	COOPER, G A	Ferguson and McIlveen	
	COXON, R E	Engineering and Power Development Consultants Ltd	D1,2(2)
	CRAIG, A J	North West Water Authority	JA5.8/D5
	CROUCAMP, W S	Department of Water Affairs, South Africa	
	CURTIS, G R	North of Scotland Hydro-Electric Board	A2.5/D2(2)
D	DARGIE, R M	Sir William Halcrow and Partners	
	DAVIES, B L	South Yorkshire County Council	D1
	DAVIES, G L	Southern Water Authority	
	DAWSON, F	Newcastle and Gateshead Water Company	
	DAY, R T	Ward, Ashcroft and Parkman	
	DICKERSON, L H	formerly North of Scotland Hydro-Electric Board	SC2
	DUNCANSON, J K	South West Water Authority	

E	ELDRIDGE, J G	Binnie and Partners	D4(2)
	ELLIS, L E	Department of the Environment	A1.1/D1
	ELVERY, R H	University College, University of London	
	EVANS, D E	Binnie and Partners	
	EVANS, J D	South West Water Authority	D2,3
	FEATHERSTONE, M J	Welsh National Water Development Authority	D4
F	FELLOWS, R	Central Regional Council	D5
	FIRTH, A	British Waterways Board	
	FITZGERALD, R D	Waterhouse and Partners	A3.4/D3
	FITZPAYNE, D S O	Fife Regional Council	
	FLEMING, J H	Sir M MacDonald and Partners	
	FONG, T Y	National Electricity Board, States of Malaya	
	FORD, S E H	Binnie and Partners	D2
	FOX, R H	Department of the Enviroment, Northern Ireland	
	FRASER, D D	Babtie, Shaw and Morton	
G	GERRARD, R T	Binnie and Partners (Chairman, BNCOLD)	SC5/IS/C
	GIBSON, J N	Severn-Trent Water Authority	
	GIUDICI, Dr S	Hydro-Electric Power Commission, Tasmania	D2
	GRANT, J B	R H Cuthbertson and Partners	
	GRIFFITHS, F N	Howard Humphreys and Sons	JA4.3/D4
	GROBBELAAR, C	Department of Water Affairs, South Africa	D1,3
	GRØNER, C F	Christian Grøner A/S, Norway (President, ICOLD)	IA/D2,3/C
H	HAMILTON, D M	Crouch and Hogg	D4,5
	HARRISON, R L	North West Water Authority	
	HASELDINE, J M	J Taylor and Sons	
	HAYWARD, D	'New Civil Engineer'	
	HOLMAN, J F	South West Water Authority	
	HOYLE, N	North West Water Authority	D5
	HUDSPITH, J W	South Yorkshire County Council	
	HUMPHREYS, J D	Mander, Raikes and Marshall	D1,2,5
	HYDE, T M	British Waterways Board	D1,5
J	JARVIS, R M	North of Scotland Hydro-Electric Board	A4.2
	*JOHNSON, F G	North of Scotland Hydro-Electric Board	A1.3,JA2.3/D1,2,4
	JOHNSTON, T A	Babtie, Shaw and Morton	A2.6/D2,3
K	KELWAY, Dr P S	Northumbrian Water Authority	D4(2)
	*KENNARD, M F	Rofe, Kennard and Lapworth	A5.1/D1,3,5
	KITCHING, B W	Allen, Gordon and Company	A2.7/D2,4
	KNIGHT, D J	Sir Alexander Gibb abd Partners	A5.6/D1,5
L	LANDER, J H	Sir Alexander Gibb and Partners	
	LAUVLAND, J	Norwegian Geotechnical Institute	
	LAW, F M	Binnie and Partners	GR4
	LEACH, A J	The South Staffordshire Waterworks Company	D5
	LEE, R M	Yorkshire Water Authority	
	Le MASURIER, M	Sir William Halcrow and Partners	JA5.2/D5.
	LONG, J J	Binnie and Partners	
	LORANGE, F	Oslo Electricity Works, Norway	

M	McKENNA, J M	John M McKenna	
	MacKICHAN, R W A	Mander, Raikes and Marshall	
	McLEAN, A	Strathclyde Regional Council	
	MANSELL-MOULLIN, M	Binnie and Partners	D4
	MARRIOTT, M	Soil Mechanics Ltd	JA3.2
	*MARSH, C B	University of Newcastle upon Tyne	
	MASSEY, J E	G H Hill and Sons	
	MILLER, D	Central Regional Council	
	MILLER, J B	Health and Safety Executive - Mines and Quarries Inspectorate	D1
	MILNE, G A	Crouch and Hogg	
	MILNE, J S	North of Scotland Hydro-Electric Board	
	*MOFFAT, A I B	University of Newcastle upon Tyne	A1.4,3.1/D1(2),2,3(2),4,5.
	MONEY, Dr M S	University of Newcastle upon Tyne	A5.3/D5
	MOTTRAM, Mrs L M	'Water Power and Dam Construction'	
	MOUSSAVI, M H	Ministry of Energy, Iran	
	MUNRO, J G	Scottish Development Department	
	MURPHY, A M	Electricity Supply Board, Eire	
N	*NOVAK, Prof P	University of Newcastle upon Tyne	JA4.6/D4
	NUTT, K D	Irrigation and Water Supply Commission, Queensland, Australia	
	NYLANDER, S	Swedish State Power Board	D1,5
O	OSBORN, H D	Babtie, Shaw and Morton	A5.9/D5
	OMOUMI, A R	Ministry of Energy, Iran	
P	PARKMAN, C C	Ward, Ashcroft and Parkman	D1,4,5
	PARKMAN, H C	Ward, Ashcroft and Parkman	A2.8/D2
	PATON, Sir Angus	Sir Alexander Gibb and Partners	SC1
	PATON, J	Babtie, Shaw and Morton	SC4
	PENMAN, Dr A D M	Building Research Station	GR5
	PEPPER, R A	Sunderland and South Shields Water Company	
	PHILLIPS, R V C	Binnie and Partners	
	POSKITT, F F	Ferguson and McIlveen	A5.4/D2,4
	PYKE, P D	Department of Water Affairs, South Africa	
Q	QUINN, M	Electricity Supply Board, Eire	
R	RAY, W J F	Thames Water Authority	D5
	RAYBOULD, H K	British Waterways Board	
	REILLY, M	Engineering and Power Development Consultants Ltd.	
	RENNIE, W J H	Bush and Rennie Associates Ltd	D3
	REYNOLDS, G	North of Scotland Hydro-Electric Board	A4.4/D4
	RICHARDSON, J K	Thames Water Authority	
	ROBERTS, P D	North West Water Authority	
	ROBINSON, R D	G H Hill and Sons	D1
	ROCKE, G	Babtie, Shaw and Morton	
	ROFE, B H	Rofe, Kennard and Lapworth	A2.9/D2,4
	ROSEVEARE, J C A	Freeman, Fox and Partners	
	ROSSINGTON, D T	North West Water Authority	
	ROWBOTTOM, M J	British Aluminium Company Ltd	
	RUFFLE, N J	Northumbrian Water Authority	
S	SANKEY, K A	North West Water Authority	JA2.4/D2(2)
	SAYER, K N	Fife Regional Council	
	SEDDON, J W	Severn-Trent Water Authority	D2,5
	SEVERN, Prof R T	University of Bristol	
	SHARP, R G	Severn-Trent Water Authority	D2,4
	SPEED, H D M	Newcastle and Gatehead Water Company	
	SPENCE, W S	Lothian Regional Council	
	SUTCLIFFE, Dr J V	Institute of Hydrology	JA4.7/D4(2)

T	TATTERSFIELD, J	Watermeyer, Legge, Piesold and Uhlmann	D5(2)
	TAYLOR, E H	Sir M MacDonald and Partners	
	THOMAS, I	Welsh National Water Development Authority	
	THOMPSON, J C	Ferguson and McIlveen	
V	VÅGE, T	Norwegian Geotechnical Institute	
W	WATSON, W A	James Williamson and Partners	
	WEBSTER, J A	Strathclyde Regional Council	
	WEST, M J H	Binnie and Partners	A4.5
	WHITE, S F	Department of the Environment	GR1
	WILLCOCK, Dr R M	University of Birmingham	
	WILLIAMS, J D	Howard Humphreys and Sons	
	WILLIAMSON, J P	Lothian Regional Council	GR2
	WILLIS, H B	United States Army, Corps of Engineers	IS/D1
	WILSON, E B	Sir William Halcrow and Partners	D3
	WINDER, A J H	T and C Hawksley	
	WINTERBOTTOM, B N	Lehane, MacKenzie and Shand Ltd	
	WRIGHT, Dr D E	D Balfour and Sons	D4(2)
	WYLIE, R H F	Promon Engenharia SA, Brasil	
Y	YOUNG, A R	R H Cuthbertson and Partners	
Z	ZOHOUR, K	Ministry of Energy, Iran	