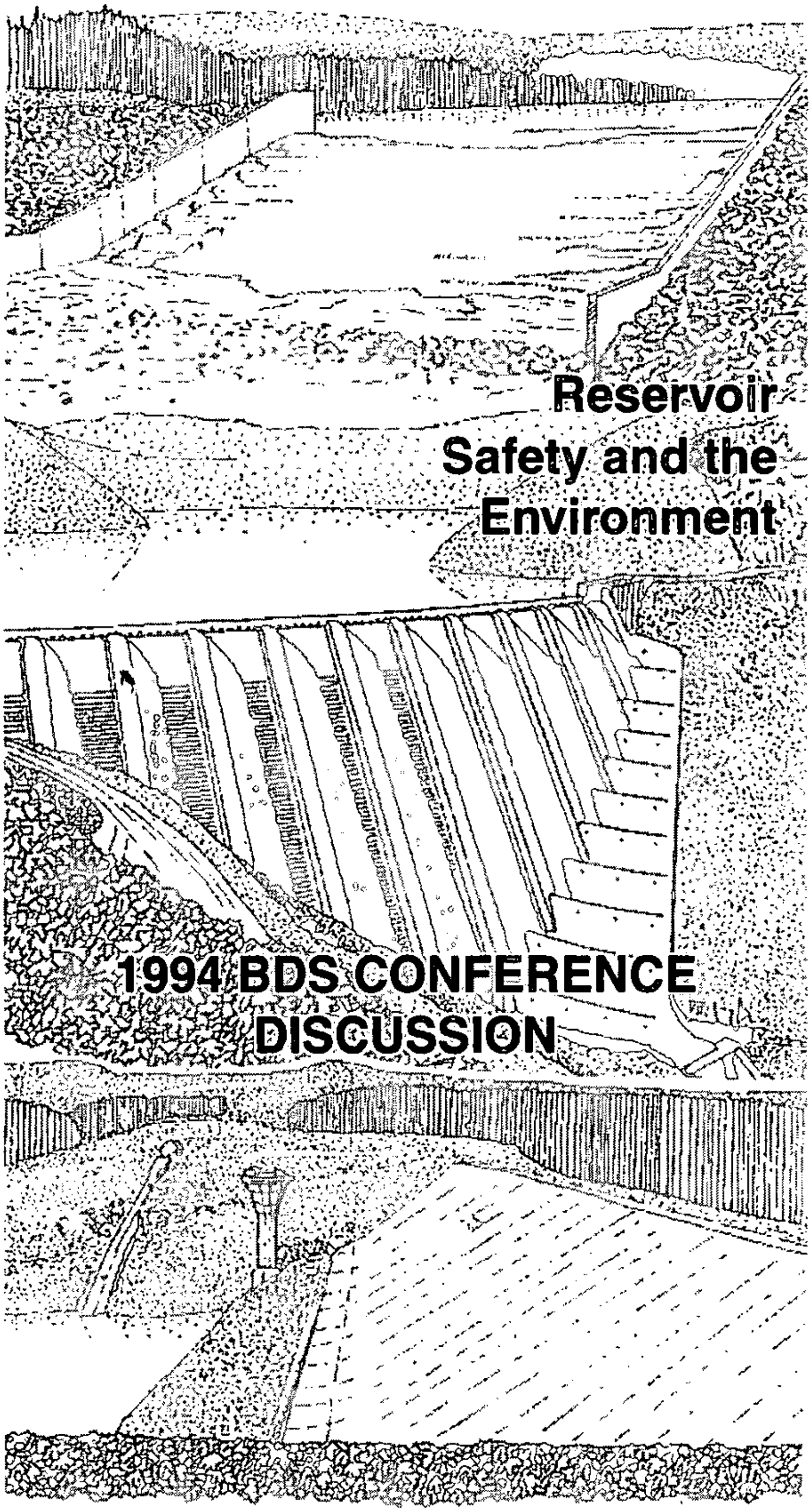
SUPPLEMENTARY ISSUE

DAMS & RESERVOIRS



Reservoir Safety and the Environment

1994 BDS CONFERENCE DISCUSSION

SAFETY OF EMBANKMENT DAMS

ENVIRONMENTAL ASPECTS OF RESERVOIR CONSTRUCTION AND MANAGEMENT

SAFETY OF CONCRETE AND MASONRY DAMS

RESERVOIRS AND NATURAL HAZARDS

BDS

THE BRITISH DAM SOCIETY

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PREFACE

This volume contains the complete record of discussions from the Eighth Conference of the British Dam Society entitled 'Reservoir Safety and the Environment', which was held at the University of Exeter, 14-17 September 1994.

The thirty papers which were discussed were published as a separate book published by Thomas Telford Services Ltd, and available from the bookshop at The Institution of Civil Engineers, Great George Street, London SW1P 3AA.

EIGHTH BRITISH DAM SOCIETY CONFERENCE
UNIVERSITY OF EXETER 14-17 SEPTEMBER 1994
RESERVOIR SAFETY AND THE ENVIRONMENT

The Society's Eighth Conference was held at the University of Exeter, which provided excellent facilities for the technical seminars and the informal discussions with followed. Thirty-one papers were presented covering a wide range of topics under the general conference theme of "Reservoir Safety and the Environment". Individual Sessions addressed the safety of embankment, concrete and masonry dams, reservoirs and natural hazards, and environmental aspects of reservoir construction and management. The high technical content of the subject matter engendered lively debate from the 181 delegates both within and outside the Conference Hall, and authors and discussion contributors are to be congratulated on the high standard of presentation achieved.

The keynote Geoffrey Binnie lecture was given on the subject of "Four Decades of Development of British Embankment Dams" by Mr Michael Kennard. This provided a fascinating insight into the development of embankment dam engineering in the United Kingdom over a forty year period and the information presented will form an essential knowledge store for members of the profession both now and in the future.

South West Water provided the opportunity for the technical visit to Wimbleball Dam, a 63 m high buttress structure completed in 1979. The technical knowledge of the guides, the engineering interest in the dam, the excellent weather all combined to make this a memorable visit.

Over 20 accompanying persons attended the event, and their programme included visits to the Royal Horticultural Society's Rosemoor Gardens, and to Dartington, as well as an open-top bus tour of Exeter.

The success of the Conference was the combined result of the activities of many individuals, not least the session chairmen, authors and presenters of papers, and all who contributed to the discussions, to whom I express my thanks. Special thanks also to my colleagues on the Conference Organising Committee whose help and advice in arranging and running the Conference was invaluable.

Alex Macdonald
Chairman, Conference Organising Committee

BRITISH DAM SOCIETY - GEOFFREY BINNIE LECTURE 1994

presented by M F Kennard BSc CEng FICE FIWEM at Exeter on 15 September 1994

"FOUR DECADES OF DEVELOPMENT OF BRITISH EMBANKMENT DAMS"

1. INTRODUCTION

I consider it an honour and a privilege to be invited to present this Geoffrey Binnie Lecture to the British Dam Society.

Having been an active member of BNCOLD, and now BDS, since its formation in 1965, and also having known Geoffrey Binnie quite well, especially as a member of the ICE Working Party, that under his chairmanship, led to the 1978 Engineering Guide on Floods and Reservoirs, I was pleased to accept this invitation to give the lecture named in his honour. Another reason for pleasure is that I was personally responsible for putting forward the proposal for a biennial Lecture to the BNCOLD Committee in 1983.

With this background, and my own professional interest, I have chosen to speak on the topic of British Embankment Dams, especially with clay cores and clay foundations, over the last four decades. This period effectively covers the period since World War II, and the period from the change of basically empirical design methods to fully geotechnically designed dams today, and it also covers my own career in which I have had some involvement in a number of British embankment dams.

These four decades cover a busy period of dam projects in Great Britain when 137 large dams forming impounding and pumped storage reservoirs were completed according to the ICOLD World Register of Large Dams - that is over 15m in height. Of these dams completed since 1953, 68 were concrete and 69 were embankment designs, and of these about 54 had clay cores or in some cases clay fills or clay foundations.

British geological conditions, resulting from the ice age, have led to the clay strata and have made such conditions more prevalent in England, Wales and Scotland than in many other countries and has led to the geotechnically led designs and associated theoretical and practical research that where properly applied has led to many satisfactory and economical designs.

This period can be considered as a modern golden age of British dam history, and with arguably one major exception, the period is also a major success story.

In May 1976 Geoffrey Binnie gave a lecture to the Newcomen Society on "The Evolution of British Dams" (Binnie 1976).

In this lecture he said:

> "From the beginning of this century up to the end of the Second World War, the design and construction of earth dams followed traditional lines based on more than a century of recorded practical experience.
>
> Following a slip during construction of Chingford Reservoir in 1938, Professor Terzaghi was called in to advise and after the war Terzaghi's concepts were welcomed with enthusiasm by the post war generation of British Engineers. Whilst experience and judgement still remain very important factors in the investigation and design of dams, the empirical methods of the past have now been superseded by methods based on analysis and scientific logic."

I wish to show how these modern methods have been applied to a number of later dams, and compare the changes from the traditional lines referred to by Geoffrey Binnie, that have occurred during this period.

These decades have seen the tremendous growth in this country of geotechnical engineering, including soil mechanics, rock mechanics and engineering geology. The first university Professor of Soil Mechanics was Dr Skempton at Imperial College in 1955. Now there are over 20 geotechnical professors in the country. It is now inconceivable that an earth dam project would be designed and constructed without experienced post graduate engineers on the staffs of the site investigation contractor, the laboratory, the consulting engineer and the civil engineering contractor.

In the 1930s and 1940s there had been considerable advances in earth dam design in USA due to the Bureau of Reclamation, Corps of Engineers, Professor Terzaghi and others, and in 1945, Creager, Justin and Hinds published "Engineering for Dams", which found its way (or should have done) into many British design offices. This book set out eight "Requirements for the Safety of Earth Dams" which stated that an earth dam should be so designed that:

(1) There is no danger of overtopping (ie sufficient spillway capacity and sufficient freeboard).

(2) The seepage line is well within the downstream face.

(3) The upstream face slope is safe against sudden drawdown.

(4) The upstream and downstream slope is flat enough that, with the materials utilised in the embankment, they will be stable and show a satisfactory factor of safety by recognised methods of analysis.

(5) The upstream and downstream slopes of the earth dam are flat enough that the shear stress induced in the foundation is enough less than the shear strength of the material in the foundation to ensure a suitable factor of safety.

(6) There is no opportunity for the free passage of water from the upstream to the downstream face.

(7) Water which passes through and under the dam when it reaches the discharge surface has a pressure and velocity so small that it is incapable of moving the material of which the dam or its foundation is composed.

(8) The upstream face is properly protected against wave action and the downstream face is protected against the action of rain.

The authors of this classic text book added that:

> "The principal purpose of stating the above criteria of design is to furnish a check list which the engineer can consult to help him make sure that he has considered all of the pertinent factors in the design of his dam. An earth dam designed to meet these criteria will prove permanently safe provided proper attention is given to the details of construction."

In addition to the eight criteria, and the reference to being "permanently safe" I would add two further requirements, based on developments in dam engineering in the period:

(9) After design, the construction control, instrumentation data and analyses have to ensure that performance is within design limits, or improvement measures are taken.

(10) After construction, the supervision and maintenance have to ensure continuing satisfactory performance with improvement measures if found to be necessary.

To keep the lecture to a reasonable length I will discuss the general topics of:

Embankment cross-sections and slope stability
Drawdown
Seepage and cut-offs
and Construction Control and Instrumentation

This will, to a lesser or greater extent, cover most of the above criteria.

In discussing these factors, without going in to soil mechanics detail, I will attempt to show in general terms how traditional British designs usually coped adequately with these criteria, and the developments and changes that have occurred in more recent years. The British designs discussed will generally be restricted to clay fills and foundations.

2. EMBANKMENT CROSS-SECTIONS AND SLOPE STABILITY

Professor Skempton at the 1989 ICE Clay Barriers for Embankment Dams Conference, gave an excellent keynote address on the "Historical developments of British embankment dams to 1960". He stated that an account of research and case records of the past thirty years (i.e. 1960-1990) would amount to a state-of-the-art review of modern practice, far exceeding the limits of his historical survey, although he did mention some of the post 1960 developments.

(Skempton 1989).

My Lecture therefore overlaps with Professor Skempton's survey, but it is not intended to follow on from his survey, or even to cover the same form of historical record. It is only complementary in that part of the same period is referred to, but it is presented so as to mainly cover my interests and experience and in the context of being a practising consulting and panel engineer in this period.

Figure 1 is a typical generalised cross-section of successful Victorian British embankment dams, especially in the Pennines.

By about 1880 design principles, developed from satisfactory performance of a large number of dams, had evolved leading to safe and reasonably economic large earth dams. Professor Skempton summarised the geotechnical aspects of this general design as:

(1) The puddle core to have a thickness of not less than one-third of the water head, a top width of not less than 2m and batters not steeper than 1 in 12.
(2) Select fine ground fill on both sides of the core, preferably with a zone of clay 2 - 3m wide immediately adjacent to the core.
(3) Coarse fill placed in outer shoulders.
(4) Upstream face not steeper than 2 to 1, with berms in higher dams.
(5) A drainage mattress under downstream bank.
(6) Any "slippery or compressible" material in the ground to be removed before construction of the bank.

It can be noted that this design does not involve clay fill shoulders or a clay foundation.

If the basic principles of a good foundation were followed, that is fine material against the core and coarser material in the outer shoulders with a drainage mattress under the downstream bank, then the design was very satisfactory, and covered Creager, Justin and Hinds' principles of:

(2) Seepage line is well within the downstream face.
(3) Upstream face is safe against sudden drawdown.
(4&5) Slopes are flat enough for properties of fill materials and foundation.
(6) No free passage of water from upstream face to downstream face.

Where the typical cross-section was found to be unsatisfactory from slope stability aspects - especially if it failed during construction or on first filling - was where the principles were not adhered to, or where certain foundation or fill conditions were not as expected by the Engineer.

Since the Second World War - or say the last 40 years - which coincides with my career, there have been tremendous and continually developing changes in British earth dams. The early dams in this period were a continuation of the pre-war empirical designs where deep cut-off trenches,
grouting, puddle clay cores, selected fill and often 1 in 3 and 1 in 2½ slopes were usually the norm.

In some cases, such as Sutton Bingham (1955) the slopes were flattened due to foundation and fill conditions as analysed by total stress analyses.
(Walters & Walton 1957)

Later dams often had no cut-off trenches, but had rolled clay cores, geotechnically designed cross-sections, extensive instrumentation and more construction control and analysis.

The changes were gradual and usually based on development from one dam to another, such that later designs bore little similarity in detail to the designs at the beginning of the period, eg Usk and Sutton Bingham.

I shall make reference, and use as examples, many of the more than 50 large earth dams with clay cores that have been completed in England, Wales and Scotland from 1953 to 1992, drawing from my own experience and others, but using information and data from the extensive list of published papers, including those of ICE, ICOLD and BNCOLD and now BDS.

In 1965, James Banks, first Chairman of BNCOLD, in his ICE Presidential address stated that

> "scientific contribution of soil mechanics to civil engineering, not least in relation to the safety of dams, can hardly be over-stated. Without this technique, the great earth and rock fill dams which have been constructed in recent years could not have been undertaken." *(Banks, 1966)*

Many dams have formed stepping stones along this development in this period. These have included Chingford, Muirhead, Usk, Chew Stoke, Selset, Backwater, Derwent, Llyn Celyn, Balderhead, Cow Green, Empingham, Kielder and the two Carsingtons. There have of course been others as well.

At Chingford, in 1937, a slip occurred when a 85 m length of the bank failed on a slip surface passing through the core and a layer of soft clay 90 cm thick beneath the outer slope. The soil mechanics group of the Building Research Station, under Dr Cooling immediately began an investigation. The investigation, sampling and testing, and analysis were the first application of soil mechanics to embankment dams in Great Britain. The Contractor, consulted Professor Terzaghi and he produced a radical re-design involving wide berms on both slopes, with keys of gravel fill taken down through existing bank material and alluvial clay to the underlying gravel stratum, to obtain a minimum factor of safety of 1.5.
(Cooling & Golder, 1942)

The more rapid construction at Chingford, using large tractor earth moving equipment, was a contributory cause as slow consolidation of the clay foundation could not occur.

Similarly at Muirhead, in Scotland, where boulder clay fill, and not free draining shoulder fill, was used with 1 in 3 and 1 in 2½ slopes, and rapid construction methods were used, very noticeable deformations developed in both slopes, with tension cracks about 6 m from the centreline of the puddle clay core, on each side. Subsequent investigations, and analyses based on a shear strength determined by back analysis knowing the factor of safety to be 1.0 led to a redesign. In order to get a factor of safety of 1.25 the bank had to be kept at the present height, with a berm added on the upstream side and material removed from the upper part of the downstream side. Knockendon dam under construction at the same time, was investigated and the cross-section redesigned by adding a large upstream toe berm. At this dam, pore pressures were recorded in the boulder clay fill for the first time in Great Britain, when in putting down boreholes in the fill, the foreman noted that the water level in two boreholes rose above bank level when the tubes were left projecting overnight. To measure these high pore pressures, standpipe piezometers were installed early in 1944. *(Banks, 1948 & 1952)*

At the 33 m high Usk Dam, in S. Wales, in 1950 a lens of silt was found in the glacial drift foundation. As the silt was sealed by boulder clay above and below, no consolidation of the silt could occur when loaded by the embankment and Binnie, Deacon & Gourley decided that it was economic to introduce a row of sand drains as an alternative to removing the silt and the ground above. The drains proved so effective that scarcely any excess pressures developed in the silt during construction. *(Sheppard & Aylen, 1955)*

To observe the performance of the sand drains and also the boulder clay fill, the engineers asked the Building Research Station to install piezometers to monitor the pore pressures. As at Knockendon, pore water was found to rise up standpipes above the level of fill showing high pore pressures in the boulder clay fill. Results led to Dr Skempton and Dr Bishop being consulted and analyses showed that building could not safely continue without a modification in design, and that the most expeditious method of achieving adequate stability was to introduce drainage blankets before adding the second, and then the third, season's fill. *(Skempton 1989).*

At the 13 m high Chew Stoke Dam, in 1951, tests showed that as the soft clay in the foundation was underlain by Keuper Marl, its consolidation would be restricted, and rather than remove the clay, which would add about 30% to the volume of the embankment, it was decided to install a system of 500 sand drains, which led to the embankment being successfully completed in 1954, with an outer slope of 1 : 2½. *(Skempton & Bishop, 1955).*

Work at Usk led Dr Skempton and Dr Bishop to realise that drainage blankets in clay fills as well as sand drains in soft clay foundations, could provide

> "a valuable feature in the hands of an engineer designing an earth dam under conditions where clay had to be used and where there was substantial rainfall. What had at first been considered an expedient to overcome a difficult situation was now showing itself as a feature which could with great advantage be incorporated in earth dams at the outset of design".
>
> *(Skempton, 1957)*

The 36 m high Selset Dam, where construction commenced in 1955 became a classic example of this. The design using boulder clay fill, but still with a puddle clay core, incorporated drainage blankets in the shoulder fill from the beginning.

Additional foundation site investigation, early in the contract at Selset, showed the extent and depth of the soft clay foundation and extensive sand drains were immediately designed. *(Kennard & Kennard; Bishop & Vaughan, 1962).*

The importance of pore pressures to the stability of the embankment, in an effective stress analysis, is shown in Figure 2 from the Selset design. For a pore pressure ratio of 0.55 in the fill, the FOS for a 1 in 4 slope is only 1. If the pore pressure ratio is reduced, by the use of drainage blankets, so that r_u = 0.3 for example, the FOS increases to 1.5. *(Bishop & Vaughan, 1962).*

This example is based on circular slip surfaces in the shoulder, but with drainage blankets, the critical slip surface would be non-circular and through the layer of higher pore pressure between horizontal drainage blankets. The factor of safety may be only 1.06 for the most critical non-circular surface, compared with 1.24 for a circular surface, although the actual surfaces are pretty similar.
The dissipation of pore pressure in boulder clay fill in the wet climate at Selset, between both horizontal and vertical drainage blankets was observed by arrays of hydraulic piezometers. Although both layouts were effective, horizontal blankets were subsequently used due to less obstruction during fill placing. Following this, dam drainage blankets in clay shoulders have subsequently been incorporated in over 20 British embankment dams.

The specification for Selset, in 1955, specifically laid down for the layers to slope outwards to shed water and to retain less water in the fill, and in addition stated that face shovels, and not scrapers, were to be used for obtaining clay fill from borrow pits so that less moisture would be in the fill when dug and transported, and therefore lead to lower pore pressures.

The long-term performance of clays used in clay embankments, especially of weathered London Clay, Oxford Clay, and Weald Clay, all in Southern England was not so well known and documented as clays in Northern England. As these materials were used in larger embankments a certain conservatism was required, especially with regard to spacing of drainage blankets and long-term softening of the clay.

At the 27 m high Diddington Dam, now Grafham Water, (completed in 1964) the initial design had a provisional spacing of the drainage blankets at 3.2 m centres. After consultation with Professor Bishop and Binnie & Partners a more conservative approach to the problems of long-term softening was adopted, with the blankets mainly at 1.6 m centres. The analyses included the stability of the upstream slope against shallow draw-down slips under the extreme condition of rapid drawdown with the shoulder material fully softened.
(Hammond & Winder, 1967).

There is probably evidence, however, that blanket spacing in some dams of modest height constructed in the drier parts of the country has been unduly conservative, but the engineer does not always have the advantage of hindsight in making design decisions.

The need to consider critical surfaces, and also the need to review the design after the initial design, is shown by 58 m high Llyn Celyn Dam (completed in 1965) where computer analyses with circular slip surfaces were initially carried out, leading to a conventional 1 in 3 upstream slope. Non-circular failure surfaces were subsequently investigated, particularly in connection with foundation failure. Re-assessment of the stability for the upstream foundation was made when, shortly after construction had started, investigations of other dams founded on similar clay cast doubt on the long-term strength of the upstream foundation. As a result, the original uniform upstream 1 in 3 slope was replaced by a toe term extending 64 m upstream, a 6 m wide berm 12 m higher and a 1 in 5 slope in between. *(Crann, 1967).*

At the 36 m high Derwent Dam, completed in 1966, a combination of geological and geotechnical conditions led subsequently to a design incorporating sand drains in the foundation, a base drainage blanket, an upstream open cut clay cutoff, a base clay blanket, drainage blankets in the fill, flat slopes and relief drains. (Rowe, 1970). The constructed cross-section, on which Professor Rowe advised, involved extensive changes from the original design, including changing from a central puddle core design as considerable information became available during construction of the cut off and foundation. The changes at Derwent probably represent the end of many aspects of the early British empirical designs to a cross-section based on the best principles of geotechnical expertise applied to a very difficult site.

At Empingham, now Rutland Water, completed in 1975, a 40 m high bank on Lias Clay also illustrates how sophisticated geotechnical investigation, testing, analyses, and constructional control and modification enabled a difficult site to be utilised. Pumped storage schemes such as Empingham, Draycote, and many others, have led to larger reservoirs and higher dams than if sites had only been used to store water from their own catchments. This introduces problems of *scale not previously encountered. The dam foundation is Upper Lias Clay* extensively sheared and brecciated by valley bulging and periglacial disturbance. The weak foundation was the controlling factor in design and made a wide cross-section with extensive berms necessary. Because determination of the reliable strength of foundation was difficult, it was confirmed by a trial bank placed early in the construction which was incorporated into the embankment cross-section. The flat cross-section makes a simple cross-section illustration difficult. The dam was designed by T & C Hawksley, and Dr Vaughan.

(Bridle, Vaughan & Jones, 1985).

At the 52 m high Kielder Dam, completed in 1982, with glacial till or boulder clay, foundation and fill, a cross section with slopes similar to the early standard design was used, but with an upstream base clay layer as a seepage control measure. This made the critical slip surface pass through the centre of the 6 m thick base clay layer which was to the same specification as the clay core. No limiting rate of construction was specified, and the results of on-going stability analyses at two day intervals showed a drop in FOS from 2.5 to 1.4 in only one week. Fill placing had to be suspended a few weeks before the end of 1979 earth moving season. If fill placing had continued, it would appear that failure could have occurred. This shows the importance of continually assessing the stability of a design based on pore pressures and comparing the design pore pressures with those measured in piezometers as the work proceeds. At Kielder during the winter shut-down, further boreholes were sunk and samples tested. The core and blanket material was found to have strength parameters which were lower than had been anticipated. Use of these lower parameters reduced the calculated factors of safety even further.

Subsequent analyses indicated that if the high rates of construction were to be maintained for the 1980 season a modest stability berm would be required at one section. During this final season close monitoring of core and blanket piezometers was maintained, and the FOS fell to 1.28 when the fill was more than 7 m below crest level. At this stage a modest toe berm was placed and this increased the FOS to 1.80 and at completion of the bank it was 1.5. At the commencement of impounding it had increased to 1.7.

(Millmore & McNicol, 1983).

The original cross-section of the 35 m high Carsington Dam in 1980, had slopes of classic proportions, and with a wide clay core with an extended boot section, and a foundation specified to be stripped of any clay weaker than 60 kN/m^2 (Figure 3). It could be anticipated that a critical slip surface would be non-circular passing through the boot of the clay core and the bottom of the upstream fill, or top of clay foundation, and that the upstream slope therefore is likely to be weaker than the downstream slope. The original published factors of safety did not show this. The reason for the extended boot section has been stated

> "to allow for any future re-grouting of the grout curtain under the dam to be done without penetrating the main bulk of the core".

The Creager, Justin and Hinds' criterion refers to a cross-section showing a satisfactory factor of safety by recognised methods of analysis.

Such conventional limit state stability analyses, using effective stresses based on pore pressure dissipation and consolidation, require several factors including:-

(a) correct and conservative shear strength parameters
(b) realistic pore pressure assumptions
(c) rigorous analysis of likely critical slip surfaces (both circular and non-circular where applicable)
(d) revision of design when further strength and performance data are obtained during construction
(e) realistic factors of safety
(f) instrumentation related to the design

In my opinion, the Carsington case did not adequately meet any of these 6 factors. This can also be deduced from Mr. Coxon's official report on the failure which occurred in 1984 (Figure 4). *(Coxon, 1986)*

It had already been shown by the Contractor's Consulting Engineer 6 months prior to the failure that, based on recognised methods of analysis, and realistic shear strength parameters, the original cross-section was inadequate and "a revised design was essential". (Kennard 1983). Comparison of the original cross-section to other designs, especially with the introduction of the widened clay core, and the soft clay foundation, can suggest a low and inadequate FOS.

The critical slip surface analysed for the Contractor prior to the failure was comparable and practically identical with the surface revealed in the post-failure investigations..

Subsequently, the dam has been re-designed by Babtie, Shaw & Morton and Professor Vaughan and re-constructed very satisfactorily using similar design principles and practices as further developed on from several of the examples I have previously mentioned. *(Chalmers, Vaughan & Coats 1993)*

There is no authoritative British Code which gives guidance on factors of safety for dam embankment slope stability. Minimum factors which have been used in the design of some new embankment dams are:

Condition	Typical minimum acceptable factors of safety
End of Construction	1.3 to 1.5
Steady seepage with reservoir full	1.3 to 1.5
Rapid draw-down	1.2 to 1.3

The BRE Engineering Guide to the Safety of Embankment Dams states that this table is relevant to factors of safety calculated in limiting equilibrium analyses using effective stress parameters for deep-seated slip surfaces, but that in existing *dams other hazards may be more critical than slope failure.*

(Johnston et al, 1990)

These factors of safety are very similar to those published elsewhere and in some countries' codes of practices or regulations and have been found to be quite satisfactory for British dams, where the effective stress parameters have been correct, where the most critical slip surfaces have been analysed, and where the performance during construction has been continually reviewed as the bank has approached full height, and design as you go principles applied.

It needs to be reminded, as Professor Vaughan stated in 1989, that:

> "Embankment stability may be examined through precedent and through analysis. Analysis can never be exact, as the method adopted is always a simplification of reality, and assumptions must be made for strengths, pore pressures, etc., which can never be determined directly by tests which match reality. Thus the calibration of analysis against field observation, particularly of failure, is of great importance".

(Vaughan, Dounais & Potts, 1989)

Finally, in this section I have covered briefly only a few cases of British embankment dams in the period, with particular reference to use of clay fills on clay foundations but sufficient to illustrate that in the wide range of British clays, such high dams can now be satisfactorily designed and constructed which was not the case over 40 years ago, and as was shown by construction failures then, and the then limitations of empirical designs.

3. DRAWDOWN

This criterion of the upstream face slope being safe against sudden drawdown was met in traditional British practice by the upstream zone being of stony material and not clay.

In clay fills, pore pressures due to the reservoir water dissipate slowly during reservoir drawdown, and this can cause instability unless the drawdown is controlled or the slope has an adequate factor of safety. In the conventional or traditional design, with a stony free draining outer upstream zone, drawdown should not lead to a stability problem. The BRE Engineering Guide to the Safety of Embankment Dams refers to a maximum rate of 300 mm/day being typical for current practice with fairly permeable shoulders and an upstream slope of 1 in 3.

The importance of this design factor is that failure could take place after a very long period of time since the dam was constructed, and also that operational procedures can lead to more rapid rates of drawdown than originally anticipated, or previously experienced.

A drawdown slip occurred at Aldenham Dam about 170 years after construction, when the reservoir was drawn-down rapidly by only 2 m in order to repair the joints in the concrete facing. The fill was weathered London Clay.

Modern designs with clay fills have incorporated drainage layers or permeable zones in order to allow safely for draw-down.

Hydro-electric reservoirs may be subject to more rapid rates of drawdown than direct impounding water supply reservoirs. To provide data for the operation of the Shira Lower Reservoir in Scotland, a full scale drawdown test of 7 m in 4 days was undertaken in the late 1950s, at the 17 m high dam that has a concrete core wall and clayey moraine fill. Piezometers showed a fall in pore pressure over the period and on the basis of the measured pore pressures the factor of safety for a worst slip circle was 1.37, and if theoretical pore pressures based on $\bar{B} = 1$ were adopted the FOS reduced to 1.25. The FOS was assessed at 1.8 for normal reservoir full conditions. This shows the critical nature of the problem, as a more rapid drawdown would reduce the FOS to a much lower figure. *(Paton & Semple, 1960).*

The concern regarding drawdown is a real one when conditions are changed. In 1943, the East India Dock in London Docklands was emptied so as to change the conditions from a wet dock to a dry dock, for construction of Mulberry Harbour concrete caissons for the Allied invasion of France. Dewatering of the dock took 11 days and three days later a length of the west wall collapsed and after another 37 days a 91 m length of the south quay collapsed. The theoretical factor of safety of the dock when full of water was subsequently assessed to be 2.03, and with the dock empty, at 0.97 under drawdown conditions. *(Hawkey, 1948)*

In the Lluest Wen emergency in S. Wales in 1969, when pumps were helicoptered in to lower the reservoir quickly and a cut was made in the spillway, the water level was lowered by 9.1 m in 20 days. *(Twort, 1977)*

Pumped-storage water supply reservoirs, such as Rutland Water, Draycote, Grafham Water, and Bewl Water, can be subject to high rates of drawdown especially in the late periods of a severe drought.

At Draycote Reservoir, where six embankments surround the reservoir, the original design of the main dam, completed in 1969, was improved in 1989 to 91 by adding a zone of fill on the upstream berm - which was placed underwater, in order to improve stability in exceptional drawdown conditions.

Although not likely to lead to failure, drawdown instability may remain a potential and serious problem, on some old as well as more recent dams, if exceptional rapid drawdown occurs, when the upstream shoulder is not relatively free-draining.

4. INTERNAL AND FOUNDATION SEEPAGE

Creager, Justin and Hinds' criteria included that the seepage line is well within the downstream face, there is no opportunity for the free passage of water from the upstream to the downstream face, and that water which passes through and under the dam has a pressure and velocity that is incapable of moving fine material. These criteria recognise that a dam and its foundation are not completely watertight, but can be made safe even if not watertight.

The traditional British cross-section satisfied these criteria by a central puddle clay core, a transition zone adjacent to the core, a stony and more free-draining downstream zone and a cut-off trench below the core to acceptable tight strata.

Puddle clay cores were still being used after 1950 but the experience of clay fill, especially at Selset, and also at Walton and elsewhere, led to rolled clay becoming the general type of core, where suitable clay was available. At Selset in 1955, the engineer still preferred to specify puddle clay for the core, although over 1.5 million cu m of relatively impermeable boulder clay fill was being used for the shoulders. This was because his personal experience and knowledge was with puddle clay. Similarly at Derwent Dam, in 1960, the original design was based on a puddle clay core.

The clay shoulders at Selset, which I have already referred to, gave adequate experience and performance data for rolled clay, with modern earthmoving and compacting plant and site testing, to be used for British clay cores. The BNCOLD publication "Dams in the UK 1963-1983" shows that rolled clay cores have been used in 33 earth and rockfill embankment dams in these two decades alone.

Wide central clay cores of about 5 m top width with about 1 in 5 side slopes and a downstream filter have often performed satisfactorily, whilst Balderhead with a narrower cross-section and a vertical top section suffered from hydraulic fracture, due to arching action of the core and also a filter zone, which was too coarse for the eroded material. *(Vaughan et al, 1970)*

Part of this topic of internal and foundation seepage includes the very important subject of filters, drains, relief wells, and observations of performance, but time precludes a discussion on these matters.

Puddle clay core embankment dams were often constructed with fill compacted in layers sloping downwards towards the core as it was thought desirable to retain rainfall during construction to keep the core and adjacent clayey fill moist. This philosophy was established in for example, the Longendale Dams for Manchester by Bateman in the 1850s, and continued to include Ladybower, Muirhead, Knockendon and others designed in the 1930s. The observations of pore pressure in the clay fills of Knockendon and Usk were evidence of the high pore pressures that may have been due to retaining water in the fill.

Turning now from cores to foundation seepage control, there are many examples especially in the Pennines of the practice of deep cut-off trenches taken through permeable sandstone and shale strata disturbed by valley bulging and cambering to generally undisturbed shales.

Professor Knill has stated that

> "There is little doubt that the "Pennine cut off" design became almost indiscriminately applied to the solution of any apparently adverse set of geological conditions whether in the Middle Carboniferous of the Pennines or not. It is possibly only in the last decade (i.e. 60s to 70s) that the question of cut off design has undergone serious re-evaluation in the light of bedrock and overburden geology, groundwater conditions and reservoir operation in relation to downstream discharge requirements".
>
> *(Knill, 1974)*

To illustrate this I would like to compare a pre-war design - Ladybower, with some later cases of Backwater, Derwent, Cow Green, Kielder and Alton.

At Ladybower Dam, at 44 m high, the highest British earth dam in 1936 when construction commenced, the cut-off trench was taken down to a maximum depth of over 80 m to an acceptable sandstone or shale stratum (Figure 5).

The trench was terminated at the ends of the embankment with grouting beneath and beyond the trench, and it appears that a rather two dimensional approach was used. The grouting must have had a very limited effect.

In these, and in many other earlier dams, little or no consideration was given to groundwater conditions, relative permeability or hydraulic gradients other than to cut-off with a water-tight trench any water intercepted in the trench excavation. Little attention was given to the groundwater, or length of seepage path, and evidence of large flows to be pumped out of the cut-off trench excavations was taken as the need for the cut-off. In some cases the long seepage paths involved and the overlying natural clay blanket would have meant that the actual seepage flows would be very small.

At Selset, an extensive cut-off, consisting of a concrete-filled trench and grout curtain beneath, failed to prevent potentially dangerous uplift pressures developing in the rock downstream of the dam. A simple system of deep relief wells relieved these pressures successfully. This experience in 1960 corroborates the later statement by Professor Casagrande that a grouted cut-off cannot necessarily be relied upon to control uplift pressures downstream of it, and that relief wells are a simple, cheap and effective way of doing this. The information on high uplift pressures at Selset arose solely because, out of curiosity, several standpipe piezometers installed beneath the base of the dam during the site investigation were retained for long-term readings, partly in connection with an adjacent landslip stability analysis where relief drainage was planned. (Bishop, Kennard & Vaughan, 1963). The standpipes under the dam were capped with a twin tube hydraulic system as for the original BRE piezometer apparatus. *(Penman, 1956)*

At Backwater Dam, completed in 1969, the maximum depth to rockhead below the valley floor is 49 m. A layer of mountain till - or boulder clay - overlies a complex of sands and gravels with thin silty layers above the bedrock. The sand and gravel complex contained groundwater under considerable artesian pressure, due to the overlying natural boulder clay blanket. It was considered that the permeable strata required a cut-off but a trench could not reasonably be constructed because of the artesian pressure. So instead, alluvial grouting of a multi-row curtain was carried out into the underlying bedrock. Banks (1966) stated that "a satisfactory reservoir could not have been constructed at all on this site but for recent developments of this kind "that is alluvial grouting"". At that time, reliance on the natural clay blanket was not considered as an alternative solution, but it was the clay blanket that led to artesian conditions that precluded a traditional trench.

The I.C.E. paper on Backwater Dam gives the phreatic surfaces obtained from *standpipe piezometers at three foundation levels - at the base of the glacial till,* which shows the effect of the core; in the sand and gravel complex, which shows the lesser effect of the alluvial grouting, and in the bedrock, which shows the minimal effect on hydraulic gradient of rock grouting. *This latter aspect confirms Professor Casagrande's views on rock grouting.*
(Geddes, Rocke & Scrimgeour 1972)

About the same time, Derwent Dam was being constructed. Here, the valley is underlain by a thick sequence of glacial lake deposits of a maximum depth of 54 m. The glacial deposits included comprised alluvium, sandy clay, silty sands, till above 14 m of laminated clay, and sand and gravel (the upper aquifer), silt, varved clay, sand and gravel (the lower aquifer) overlying 10 m till and then bedrock. Again the first approach was a concrete cut-off trench to bedrock, to be constructed by temporary dewatering between two rows of deep wells, and the contract was awarded in 1960 on this basis. *(Ruffle, 1970)*

Tender documents called for an orthodox 1.9 m wide concrete cut-off founded 6 m or thereabouts into unweathered rock. In this form the cut-off would probably have been the largest in area in Britain, although not the deepest.

At the time certain views were expressed that such a deep cut-off was not necessary - including those of Professor Arthur Casagrande who was in the country at the time presenting his 1961 Rankine Lecture (Casagrande, 1961). His lecture on "Control of Seepage through Foundations and Abutment of Dams" raised many important aspects regarding cut-offs and grouting. He showed that such measures, especially grouting, had little effect in reducing seepage pressures. Reducing leakage by filling fissures and joints, where short leakage *paths exist is a separate aspect of grouting. He illustrated piezometer* observations in pervious rock underlying a 32 m high earth dam where the hydraulic gradient observations showed no indication that there was an obstruction to seepage at the location of the grout curtain. Although deep cut-offs had been used at pre-1940 dams in USA, he considered that a deep cut-off was not required at Derwent, and downstream drainage measures could be a solution for foundation seepage control. Despite this view, the deep drainage wells were put down in order to dewater the ground for trench construction, but it was found that the varved clay was continuous across the valley and the drawdowns were quite different in the different aquifers. When it was found the upper and rock aquifers were not significantly connected, an alternative design was developed having a cut-off which did not block the upper aquifer, and terminated in the boulder clay immediately overlying and in contact with the laminated clay. As sand drains were necessary for the consolidation of the varved clay, and therefore water could enter the upper aquifer, extensive seepage and drainage considerations were undertaken before a partial cut-off design with downstream relief wells was accepted. The cut-off through the upper clay was constructed by means of de-watered open cut excavation which was a very economical solution, compared with a sheet piled trench which the Contractor proposed. This was the first major open trench cut-off, rather than a timbered trench, in the UK.

The groundwater conditions on this site of a clay blanket and artesian pressures that made de-watering very difficult, with limited response to pumping, were the *same conditions that made the seepage to be a small quantity (⅜ - ¾ l/min), due* to the long seepage paths that follows from a natural clay blanket.

As an example of a site on adverse geological conditions, mention can be made of the 25 m high Alton Water, Suffolk, completed in 1976. This dam founded on alluvium and London Clay, Woolwich and Reading sand and clay beds, and highly permeable chalk. The design by Binnie & Partners involved a 250 m long clay fill blanket, and an upstream cut-off 450 mm thick sealing 3 m into the Woolwich *and Reading clay. The combined effect of the blanket and the cut-off* was expected to limit the seepage through the dam foundations to about l l/sec.
(Cox, Avgherinos & Hetherington, 1976)

An example of the change in philosophy in the development of cut-off measures is Cow Green Reservoir. The watershed area between the proposed reservoir in the River Tees valley and the adjacent Harwood Beck valley comprised several limestone strata, and within the area extensive barytes mineral workings of shafts and long adits. When the site was considered in the mid 1950s, a view was expressed that because limestone strata were involved with a lower level outlet in the adjacent valley, a considerable cut-off problem existed with the possible need for a grout curtain all around the reservoir. Consideration of the site was therefore abandoned in 1956 in favour of an alternative reservoir site. However increasing industrial demand by I.C.I. Ltd. led to a further look at the site in the mid 1960s, and Dr (now Professor Sir) John Knill re-assessed the situation and considered that there was evidence that the ground water table in the col area between the two valleys was at a sufficiently high level for there to be a natural water cut-off to the proposed reservoir. Extensive site investigation with ground water observations in standpipe piezometers proved this to be the case. (Kennard & Knill, 1968) Although the opposition argued the case in Parliamentary Committee proceedings that extensive grouting may still be required, the scheme was approved and construction began. No grouting or cut-off measures were constructed, and the reservoir has been satisfactory with no known leakage problem.

At the Empingham Reservoir, which chronologically followed Cow Green, there is a Marlstone aquifer under the dam, dipping downstream and outcropping in the reservoir. A paper records that the Marlstone case history at Empingham is important for two reasons. Firstly as a precedent for the construction of a reservoir in England on an underdraining limestone aquifer without special cut-off measures, and secondly that the actual flow is close to the value calculated before the reservoir was filled, although it is admitted that this may be a question of good luck rather than good judgement. Nevertheless, these examples of Cow Green and Empingham show the extent of development of the solving of seepage problems, without unnecessary extensive cut-offs, especially in limestone strata, which at one time were considered unsuitable for reservoirs.

(McKenna, Horswill & Smith, 1985)

The attention to cut-offs that I have given may seem out of proportion to some other aspects, but in the past the construction time and costs of cut-off works have been very significant. Little or no design, other than solid geology considerations, was actually involved and also no groundwater observations were undertaken on their efficiency and performance. The magnitude can be shown by the following percentages of the cost of cut-offs to the total cost of the dam:

Sutton Bingham	25%
Backwater	22%
Selset	18%
Derwent (original contract)	34%

5. CONSTRUCTION CONTROL AND INSTRUMENTATION

My additions to Creager, Justin and Hinds' eight original criteria included the important post design aspects of construction control, instrumentation and on-going analyses. These topics can only be mentioned briefly, but in any coverage of British earth dams they cannot be ignored. Many dams in the period could not have been constructed satisfactorily and have safe in-service performance without such control, observations and analysis.

In British conditions regarding variability of geotechnical materials, and climate, the control and supervision of construction, including instrumentation, are essential for the satisfactory performance and safety of our dams. The instrumentation of embankment dams, especially of pore pressures of clay foundations and fills, ground water levels, and internal and surface displacements has made great advances and has generally become standard practice.

Earlier construction control specifications limited clay fill specifications to matters such as layer thickness and descriptive use of rollers.

In the post war period, specifications have covered such aspects as moisture content and relative density, thickness of layers, type of roller, non use of certain excavating plant which might lead to wetter clay fill, watering of cores and fill, outward sloping surfaces, etc. Later developments include shear strength specifications for clay cores in which an acceptable range has been stated and the method has enabled the required properties of a non-cracking and plastic clay core to be readily tested. (Kennard, Lovenbury, Chartres & Hoskins, 1978) Alongside this, measures to increase the water content of clays have been developed by contractors at different sites, and with a range of different clays, to achieve the required shear strength. Examples of the range of shear strength specified, include 70 to 110 kN/m^2 at Derwent; 50 - 100 at Empingham and 60 to 140 at Kielder. Below 60 kN/m^2 there would be problems of rutting of clay with rubber tyred plant.

Time precludes a wide coverage of this topic of construction control, but over the years the considerable use of site investigations, site laboratory testing, instrumentation, checking of design parameters, plotting of results, continuing analyses of stability, based on observational techniques and post construction performance studies have all made significant advances and developments which are well covered in the literature.

Early instrumentation, in the empirical era, appears to have been very limited and restricted to post construction crest settlement and in some cases to seepage and drainage flow measurement. Open standpipes are mentioned in some cases to observe ground water conditions at the ends of cut-off trenches. There are, no doubt, some other site specific examples to observe hillside movements and leakages, and other local aspects, but no general observations of ground water conditions, efficiency of grout curtains or toe displacements.

The value of available long-term crest settlements was shown when the records of Ladybower Dam, the highest earth dam at 44 m high, were analysed in 1985, when the crest and wave wall of the dam was raised to enable a greater design flood (i.e. PMF) by allowing considerably greater flood storage above the spillweir level.

Records of 1944-1986 were extended on a log-time base to forecast the future settlements for the next 50 years and the design of the improvement works allowed for this future settlement of 180 mm in 25 years or 308 mm in 50 years. (Figure 6)

The use of instrumentation at Usk has already been mentioned, and this was soon followed by extensive pore pressure observations at many dams including Selset, where especially the performance of sand drains and drainage blankets were studied.

For example, confidence in the design and use of drainage blankets which have been used in many embankment dams, was enhanced in the observations of pore pressures at Selset Dam (1955-1960). The contract design included blankets in the boulder clay fill, but original work by Professors Skempton & Bishop could not determine if vertical blankets at 16 m centres were preferable to horizontal blankets at 4.5m centres. Hence in the first fill placing season, both were installed and instrumentation installed for observations on pore pressures.

Figure 7 shows the effect of the blankets and the spacing. It also shows the efficiency of the method of using hydraulic piezometers. Placing 9 hydraulic piezometer tips between blankets at 4.8 m centres is not easy, but the results justified the efforts involved.

The most important developments in instrumentation and observations have included the routine studies of the performance of cut-offs, grouting and other seepage control measures, internal and external displacements of cores, fills and foundations, and the observations of pore pressures with on-going stability analyses.

To have the information so as to proceed as rapidly as possible, in construction seasons as designed, but with the ability to check the stability as pore pressures increase or decrease with control of displacements as has been done on so many embankment dams, has represented one of the great benefits of soil mechanics design as applied to clay fills and clay foundations. This should make end of construction stability failures such as Hollowell, Chingford, Muirhead and Carsington a thing of the past, provided that the foundation and fill conditions are thoroughly investigated, tested and analysed adequately, and reviewed during construction.

There are many other examples in the literature of on-going stability analyses, such as at Kielder, and at Derwent where embankment construction was stopped at 2.8 m below the crest when the FOS in the clay blanket had reduced from 3.0 to 1.4 in 10 weeks as 7.6 m of fill was placed. After 3 weeks with a slight fall in pore pressures in the clay blanket the banking was resumed to completion. *(Buchanan, 1972)*

7. CONCLUDING REMARKS

Now I turn to the end of the period of 4 decades and to conclude the lecture. My remarks have attempted to cover only some of the changes from the pre-war empirical, but generally satisfactory, designs to the position today, whereon any clay foundation an embankment can be designed and constructed to provide an acceptable safe design, provided all the aspects are adequately dealt with in the design.

We have moved from the earlier traditional 3 to 1 and 2½ to 1 embankment slopes to varying slopes and zoning, and foundation treatments as designed for the particular conditions, and using state-of-the-art practices and knowledge in a design team involving a number of experienced dam engineers, advisors and more recently a design review panel as well.

In the period of review, the reservoir safety legislation has led to a higher profile with regard to reservoir safety. The main stepping stones on the way have been the ICE Committee Report of 1966, The Reservoirs Act 1975, the implementation of the Act 11 years later, and the Coxon review in the light of Carsington (Coxon 1986). Changes have included the 5 year term of appointment for inspecting engineers rather than for life, independent inspections and supervising engineers. The Coxon review led to the major advance of having review panels on subsequent major projects, as was normal practice on USA, World Bank and other schemes.

In the course of this lecture I have referred extensively to published papers and other references. I make no apologies for this, as I believe that a design engineer involved in British Dams needs to know of past experiences, including developments, performance, and failures, and the only way of doing this, especially in a period of fewer new projects, is by studying the literature. There is a very extensive collection of papers on British dams, in many different journals and conference proceedings, but there is no comprehensive bibliography. I hope that one day that such a bibliography could be published. This should include the BNCOLD and BDS conferences, of which this is the 8th conference, when about 200 papers and 7 invited lectures have already been presented.

The history of the last 4 decades of British embankment dams is much more than the few examples I have included, but I trust that I have achieved my objective in showing only some of the developments that have occurred in this period. Although there are likely to be only very few new British dams, the history of the past, and the reasons for some of the designs and changes from time to time, should not be forgotten.

James Banks, the first Chairman of BNCOLD, and also President of the ICE expressed in his ICE Presidential Address in 1965 that

> "I think it is true to say that in Great Britain people live without fear of any impending mishap from reservoirs and it is encouraging that with the application of technical knowledge and appropriate legislation greater safety of reservoirs can be assured."
>
> *(Banks, 1966)*

I trust that I have shown how this safety was achieved at some dams in recent times, including the years since his comments in 1965. I think his optimistic view was generally justified.

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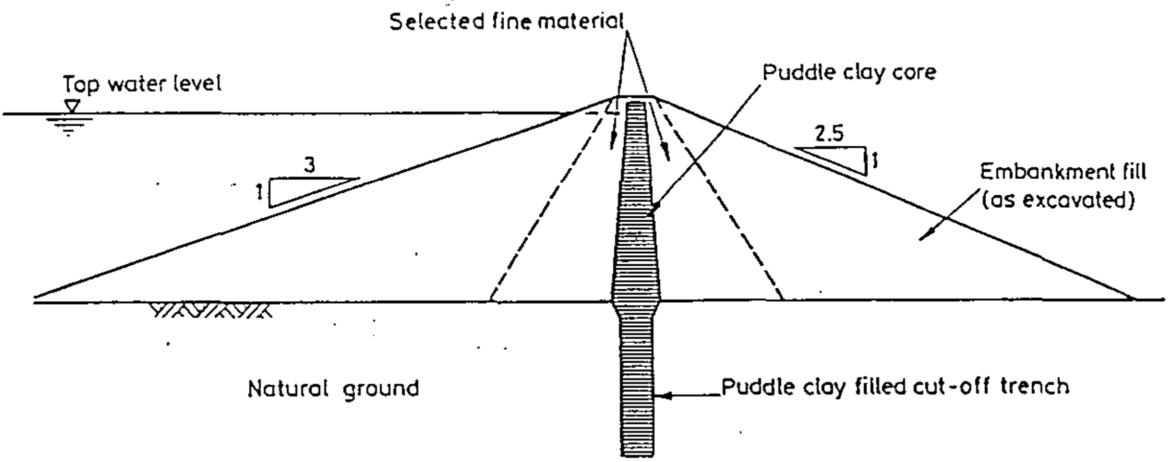
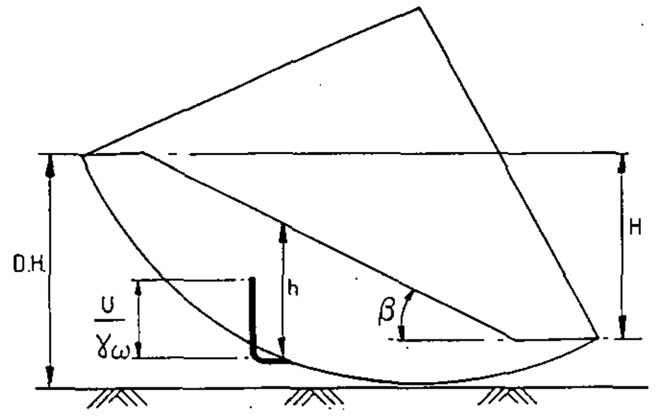
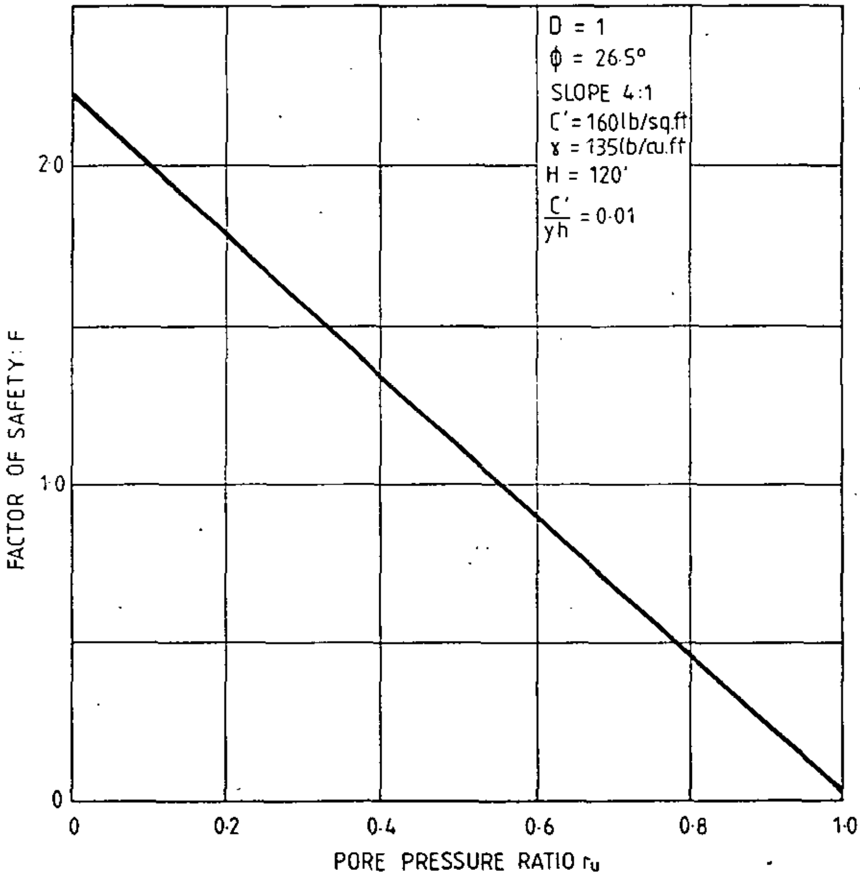
FIGURES



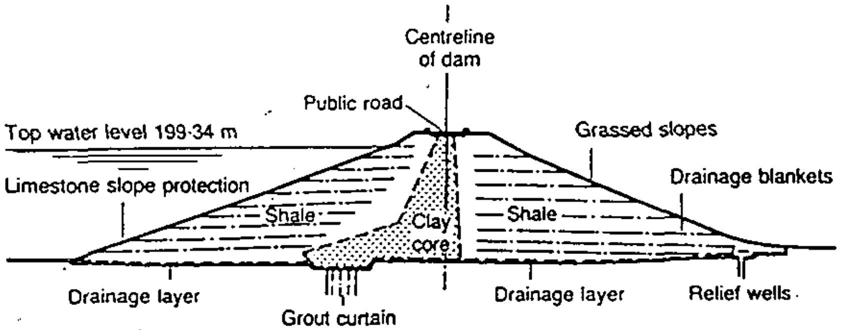
TYPICAL CROSS SECTION OF AN OLD EMBANKMENT DAM WITH CLAY CORE

(AFTER BODEN AND CHARLES,1984)

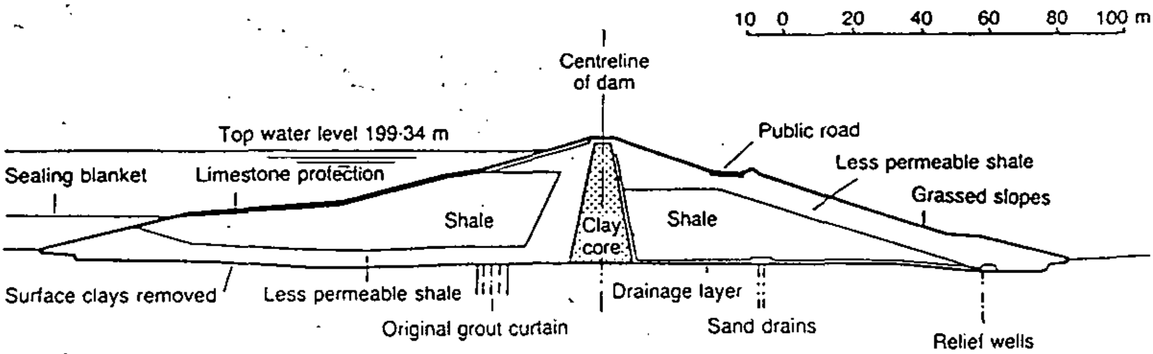




SELSET DAM
PORE PRESSURE AND STABILITY
(BISHOP AND VAUGHAN, 1962)

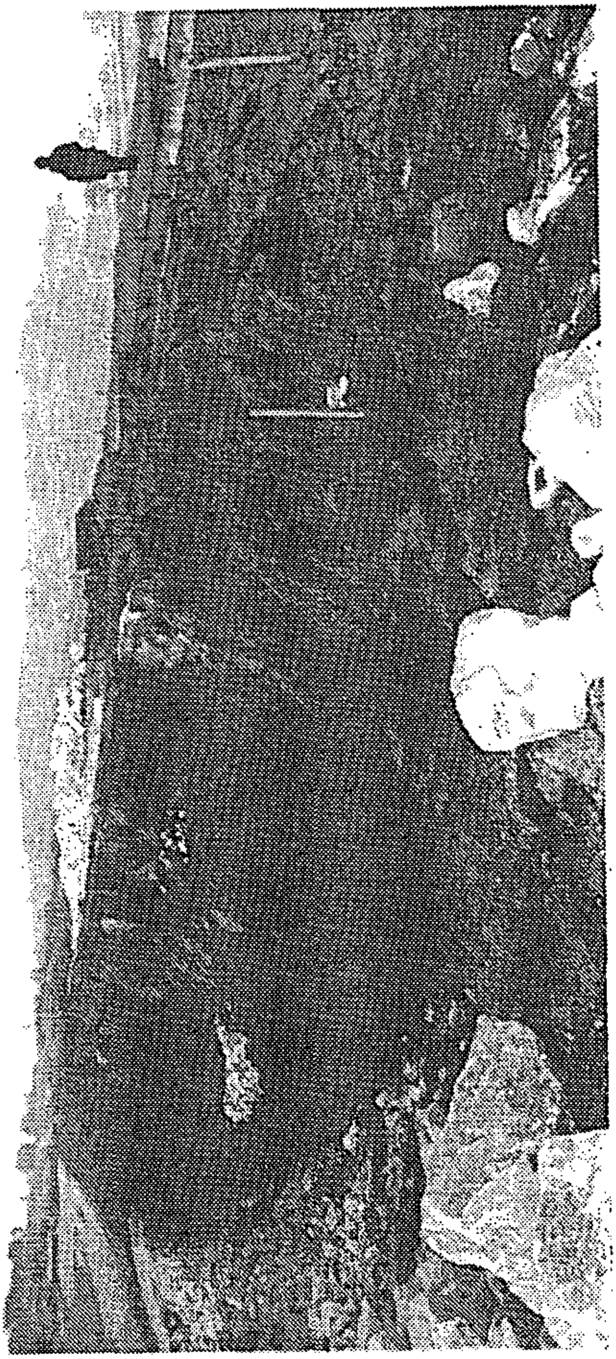


ORIGINAL DESIGN

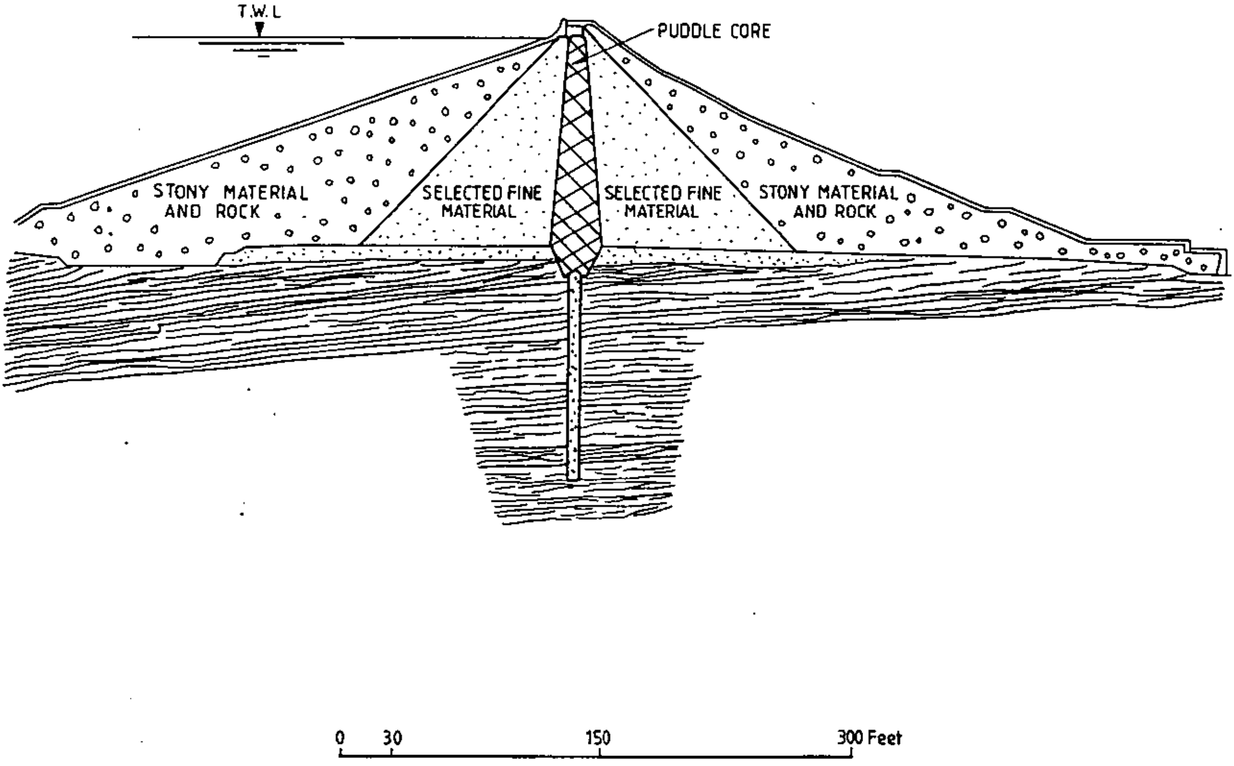


SECTION OF RECONSTRUCTED DAM

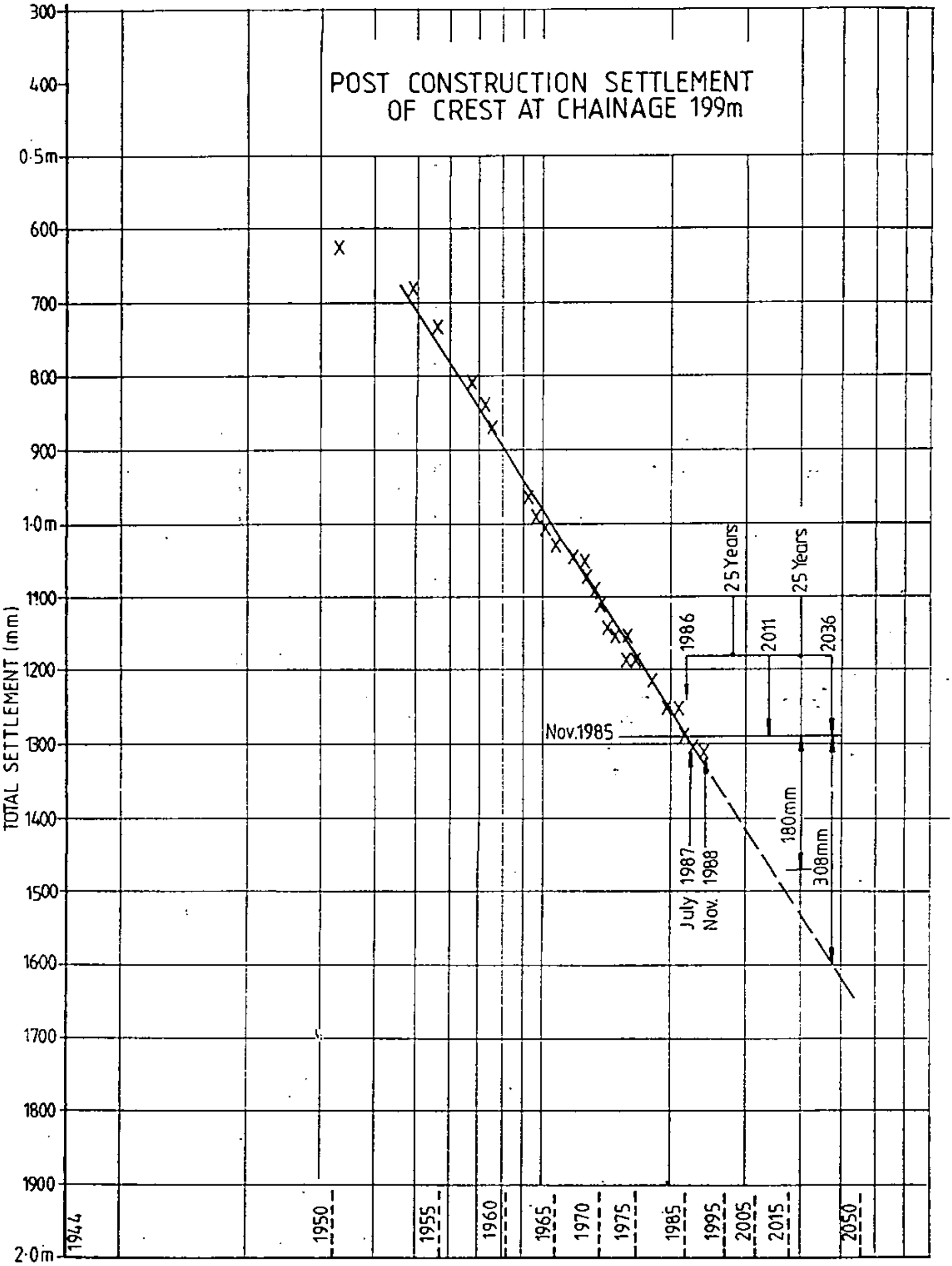
CARSINGTON DAM
(BANYARD et al,1992)



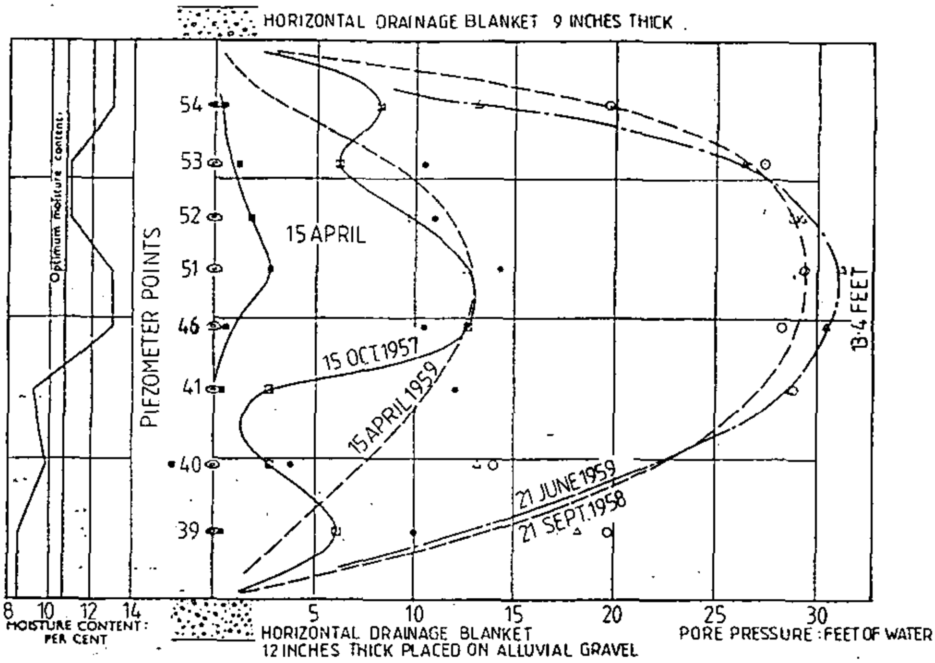
CARSINGTON DAM - SLIP FAILURE



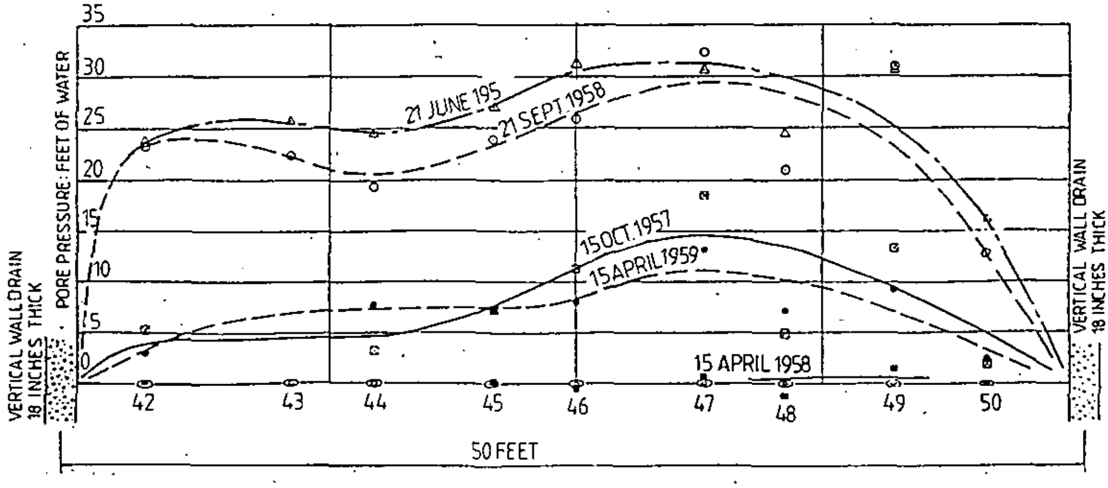
TYPICAL CUT OFF TRENCH
(LADYBOWER DAM,1936-1945)



LADYBOWER DAM SETTLEMENT
(RKL,1988)



OBSERVED DISTRIBUTION OF PORE PRESSURE VERTICALLY BETWEEN TWO HORIZONTAL DRAINAGE BLANKETS



OBSERVED DISTRIBUTION OF PORE PRESSURE HORIZONTALLY BETWEEN TWO VERTICAL SAND DRAINS

SELSET DAM
(KENNARD AND KENNARD,1962)

Address by Mr J M Cotillon, Secretary General of ICOLD

Mr Vice Chancellor,
Mr Chairman,
Distinguished Members,
Ladies and Gentlemen,

Since, as you know, English is not my mother language, and since most of you are familiar with several languages, I would like to ask you permission to deliver my speech not in English... I mean not in English-English, but in international English: the French will be in the accent only and in the turn of mind.

I have had close, distinguished and fruitful relationships for the past 16 years with some of you. To those present I would like to offer greetings and have a friendly thought for those absent. Let me name first the successive Chairmen of BNCOLD and BDS (Michael Kennard, David Coats, Roy Coxon, Ted Haws, now ICOLD Vice-President, William Carlyle, John Bowcock and Geoffrey Sims), the past Chairmen of ICOLD Committees (R G T Lane, Prof R T Severn and also Roy Coxon, David Coats, Ted Haws), and finally the present Chairmen of ICOLD Committees, Dr Arthur Penman and John Bowcock. I would also remind you that the sole British ICOLD President, John Guthrie Brown, was elected 30 years ago, in May 1964 in Edinburgh. We are still grateful to him for having been the father of the perfect Constitution and By-Laws which still govern ICOLD.

England is a land of humour and tradition. Now that I have paid my tribute to both, I feel more at east to address our hosts, the British engineers, since I understand I am in charge of the toast from the guests.

I should like to deal with three topics:

- the British Civil Engineers in the past
- the British Civil Engineers now
- what kind of help could they find within ICOLD?

<u>The British Civil Engineers in the past</u>

I must confess that England has always exerted a fascination on me: you have done so many things before all the others. For example, you started the industrial revolution 50 years before us, on the continent, and because of that, you were the main dam builder in the world in the past century, far ahead of any other country (40% of today's dams were already in service at the turn of the century and you still rank second in Europe, behind Spain, for the number of dams). Also you were the first to be confronted with environmental considerations (sporting interests, value of farmland, etc) 200 years ago, on the lower reaches of the rivers, where sites were, and still are, available for high dams.

Twenty years ago, I was walking in Victoria Street in London (between Victoria Station and Westminster Abbey) with one of your colleagues who told me that still in the 50s, all the men you might pass in this street were civil engineers, since most of the civil engineering firms had their head offices in this street. Those who have once entered the headquarters building of the Institution of Civil Engineers, in Great George Street, in

London, this temple of glory to civil engineering, can easily imagine what was your place, in the past, in this country.

The present situation

There is a general impression, outside, that now you seem to be turning your back to the world, to world affairs, where you used to be, designers and contractors, so active still some decades ago, and now concentrating only on domestic projects, on British matters, with owners and regulators playing an increasing role. This is not specific to the UK, but perhaps more visible here when comparing past and present.

This might not be my business, but I would like just to mention one fact which is significant of your absence from the scene. All of you know that our 18th Congress will be held in November this year in Durban (South Africa), from 7 to 11 November, with President Mandela delivering the inaugural speech: environment, safety, staged construction and spillway erosion will be the four topics discussed through 48 main contributions (12 minutes each) and the same number of shorter ones (3 minutes each), with 10 hours left for discussion. For the main contributions, we received no applications from the UK, and we had to invite two of you who had submitted valuable papers for this kind of oral presentations. Have you really nothing to say on these topics? Latin people, who are possibly more talkative, account for 80% of the applications. It is expected that the authors of the 10 British Papers appearing in the Proceedings will register by mid-October for the short contributions (6 minutes) which we call "Prepared Contributions". Presentation from younger engineers will be especially appreciated: in Chambéry last year, I noted that you have some good ones.

What kind of help can you find within ICOLD?

But, first, what is ICOLD?

Early this year, I was in South Africa for finalising the preparation of our Congress. I was invited to visit the Katse dam site, and I attended a formal dinner hosted by the Lesotho National Committee on Large Dams. The Vice-Chairman, Mr R Mochebelele, proposed the first toast and said, *"We were invited to apply for membership to ICOLD; we applied and were accepted; but at that time we had no idea at all of what ICOLD could be; never heard of it. The following year, we were invited to attend the Annual Meeting; we attended. An then, we made a fantastic discovery: we could freely discuss with representatives from more than 60 countries, of their problems, of our concerns. Now, we are very enthusiastic with the annual ICOLD Meetings."*

The first feature of ICOLD I suggest is being an opening to the world, to world affairs, to the world of dams.

The second feature is that, thanks to our Meetings, and more especially our Congresses, ICOLD is *de facto* an efficient tool for communication. If ICOLD was initially established to encourage advances in all the fields related to dams, in fact, things look as if ICOLD had been created by the Consulting Engineers of the West for themselves. They represent 40% of our members and are the most active on technical matters and business opportunities; other member countries are with us mainly for recognition, though more and more express the wish for a greater involvement in technical matters.

An opening to the world, an efficient tool for communication: this is possible because ICOLD is a worldwide organisation. This third feature means that 1) we make no difference between our members: they could be white, black, yellow, red and even green, it does not matter, 2) we do not use expressions such as developing countries, developed countries, transfer of technology... we are interested in technique only, 3) we have no regional organisations because of the universality of dam engineering, ie. we do not organise meetings in America for our American members, in Africa for our African members, in Asia for our member of this zone, etc. Having an American ICOLD, a European ICOLD, ie. a balkanisation of ICOLD, would be the end of ICOLD, the end of the opening to the world, of efficient communication, of our excellent reputation.

The fourth feature of ICOLD is that it offers an opportunity to keep pace with changes in dam engineering.

Those who want to be active members, can seek a seat on one of our 20 committees. Some committees are purely technical: they produce guidelines and state of the art reviews; members are specialists or come to learn by direct contact with the former. Other committees (on Safety, the Environment, Shared Rivers, Public Relations) are more a forum for discussions, for education of our members: we draw their attention and try to make them aware of new aspects in our profession. Those who cannot participate in the work of these committees or in the discussions at our Congresses can profit from our works through our Technical Bulletins, or the Proceedings of our Congresses. We are the only Organisation capable of selling Congress Proceedings to non-participants.

At any place you are, in dam engineering, you cannot ignore what is said within ICOLD and what we publish. Our capacity to make worldwide inquiries quickly, to have first-hand information, makes ICOLD a unique tool for dam engineers.

Fifthly, as you can easily imagine, running such an organisation results in many contacts between the President and the Secretary General on one side, the Chairmen of National Committees on the other. But I am extremely pleased to say that all that is done in a spirit of fairness, respect and mutual understanding. ICOLD is a club, a club for gentlemen: this is the fifth feature I would like to put forward this evening, and President Pircher and myself are especially happy about the role played in this field by BDS through its successive Chairmen, and now Dr Sims.

I could have spoken for hours about so many other problems: the cost of our meetings, cost of our publications, how changes take place within ICOLD, how to meet the various demands from our members, how to run our meetings, do we need a new name for ICOLD, eg. IDS (International Dam Society), etc. But in presenting ICOLD, I had, because of limited time, to restrict my speech this evening to five features: ICOLD is an opening to the world, an efficient tool for communication and a worldwide organisation, an opportunity to keep pace with dam engineering and a club for gentlemen.

I would add only one comment: do not believe that activity within ICOLD is for old or top people: we need, in fact, enthusiastic and imaginative members. Let me give you an example: the computerisation of the Abstracts of ICOLD Publications, mainly our Congress Proceedings, was suggested five years ago by a young Swiss engineer, still working at that time at the Lausanne University, more precisely at Ecole Polytechnique Fédérale de Lausanne. Initiatives are never discouraged though, sometimes, deeply re-orientated.

Ladies and Gentlemen, this is the end of my introduction. I have now to propose to toast from the guests. In order to help me in making this proposal, just one last short story:

Ten years ago or so, when your Foreign Minister had to chair for the first time the Council of Foreign Ministers of the EEC, he was asked, in a press interview, what he thought it would be possible to do within six months only. He replied, *"If, during this short time we already succeed in teaching them how to conduct a meeting, we will have made a significant step."*

British engineers, we have to learn from you, you have to learn also from us, we have to learn from each other. For that, and now that you are more acquainted with ICOLD, at least I hope so, may I say I sincerely wish you to join us in a more active way than you have done in the recent years, attend in greater numbers our Meetings and Congresses, and play a more active role in our Committees? I am sure it will be rewarding far beyond your greatest expectations.

Reply to the Toast to the BDS by Sir Geoffrey Holland, by Dr G.P. Sims

Mr Vice-Chancellor,
Ladies and Gentlemen,

It is an honour not given to all to reply to the toast to the Society proposed by Sir Geoffrey, and I am mindful of this as I offer him my thanks on behalf of the British Dam Society. It is our pleasant custom at our biennial conference to invite distinguished guests from the host university and from major local industry connected with our interest in dams. This year's conference is no exception, as my brief introduction will, I hope, confirm.

Sir Geoffrey Holland, from whom we have heard such a stimulating address, has had a distinguished career in education and employment, subjects close to the heart of many of us here. He spent 33 years in the Government service culminating last year in his becoming the Permanent Secretary at the Department for Education, a post he held until he became Vice-Chancellor of the University here. Earlier, as Director of the Manpower Services Commission, Sir Geoffrey played a key role in the development of youth training, of links between school and industry and the development of youth training, of links between school and industry and the development of distance learning. While Permanent Secretary at the Employment Department Group he developed and then promoted the Training and Enterprise Councils and vocational training. It is an honour for the British Dam Society to have as a guest a public servant who has made such a contribution to the development of, perhaps, our major natural resource: our people, particularly our young people. In this time of unprecedented change in our industry the way forward must surely lie in the full use of the innate skills of our colleagues, giving them fulfilling work and keeping our enterprises competitive in the world's market place.

I note that Sir Geoffrey has at least one other interest likely to be shared by a modern engineer with an interest in dams. Sir Geoffrey enjoys what he describes as 'journeying'. Our second guest, Professor Robin Turner, the Dean of the Faculty of Engineering here at Exeter University, also has a serious interest in journeying, visiting professorships in six countries. Professor Turner is a distinguished Chartered Chemical Engineer who has flourished from a first degree in Natural Sciences. His research areas include thermal diffusion in liquids, surely a matter where we have a common interest, ion exchange and reactor design. As he told us during the Opening Ceremony, Professor Turner has many links with our profession, and has indeed drawn attention to the essential similarity of approach of civil and chemical engineers.

Mr R J (Bob) Baty is the Engineering Director of SWWS Ltd responsible for the planning and implementation of the Company's massive Capital Investment Programme. As every member of the Society will know, this includes Roadford dam. We will, I am sure, all remember the three papers on this dam presented at our conference. The evident professional interest in the technical and environmental aspects of the dam is a credit to the Company. South West Water's kindness in supporting this Conference, perhaps in particular in arranging for our visit to Wimbleball, is much appreciated.

I have already introduced our Binnie lecturer, Michael Kennard. His lecture, which we have received, has dealt in a masterly way with the development of embankment dam design in this country during his generation. We are delighted to have been able to invite him to present our biennial 'state of the art' address and to welcome him to this dinner.

Johannes M Cotillon, the Secretary General of ICOLD has played a vastly influential role in the affairs of the International Commission on Large Dams. A professional engineer, he has guided the hands of at least six Presidents over a span of some 16 years since 1978 from the famous office in Boulevard Hausmann in Paris. Some may think that the Secretary has a wonderfully easy life. It is, after all, his duty to visit the location of each Executive Meeting and Congress, and this must indeed be a pleasant chore. However, the marvellously effective way he keeps such a large and potentially unwieldy organisation under control reveals the work of an imaginative and active administrator. He has also written authoratively on a variety of dam-related subjects. I myself have reason to be grateful for his excellent summary of the benefits of the Aswan High dam when required to add my voice to a recent rather public debate on the subject.

I would not be happy to let this occasion pass without a reference to some at least of the distinctions achieved by our members. An OBE was awarded to Paul Back, the last British Binnie Lecturer, who was also heard recently on Noah Richler's series of seven programmes on BBC's Radio 4 which ranged so widely on the diverse issues surrounding the construction of large dams in developing countries.

We congratulate Chris Binnie on his recent election to the Royal Academy of Engineering. He joins Messrs Back, Carlyle, Earp, Eldridge, Haws, Johnson, Newbery, Rofe, Severn and Vaughan of our distinguished colleagues who are already Fellows.

We extend a hearty welcome to members of other National Committees of ICOLD who have joined us here. In particular, Jean Jacques Fry from France who was so instrumental in organising the excellent meeting in Chambéry. Mr M B Liska, Chairman of the Slovakian National Committee, and Mr Udayasen from Thailand. We have had the benefit of three papers from Europe, although none from Germany. I mention this innocuous fact because I have been reminded of a difference in their social protocol when compared to our own by reading a speech by the Master of Trinity College, Cambridge, Sir Michael Atiyah (also a Fellow of the Academy of Engineering) at a Commemoration Feast in March 1993: he refers to a letter from a distinguished academic to Sir George Trevelyan, the father of a former master. He is asked, "*Did you ever dine at a German banquet? The speeches begin after the soup and between each speech comes a dish to be consumed. You would think it would tend to make the speeches short; but no, it tends to make the dishes cold*". Perhaps our arrangement is better.

Or it may be worse. In any event, I find this development, this drawing together of European interests under the umbrella of ICOLD, particularly welcome as it embodies a characteristic flavour to my two years as Chairman of BDS. We played an important role in organising the first European meeting at Chambéry last year. We played an active role in the OSTEMS Mission to Europe that also took place last year, with its theme of dam safety, especially through the practical aspects of operation and maintenance and legislation. With the European Union becoming ever tighter, we will, I am sure, reap the benefit of sharing the experience of European colleagues which is so different from ours in culture and need.

The art and science of dam maintenance, one of the themes of this Conference, is steadily becoming recognised as a career in its own right. It is a subject worthy of specialist academic study. And where better to do so than in this wonderfully hospitable University?

I am delighted to welcome all our guests. I, therefore, invite my fellow members of the British Dam Society to rise with me as I propose the Toast: The Guests.

SESSION 1 SAFETY OF EMBANKMENT DAMS

Chairman : TA Johnston

Technical Secretary : Dr A K Hughes

1. Remedial Works to Clay Cores of UK Embankment Dams
   Tedd, Charles, Holton

2. Internal Erosion and Stability Problems in Some Old Embankment Dams in France
   Fry, Brun, Royet

3. Examples of Problems Involving Clay Cores Affecting Dam Safety
   Penman

4. Photographic Monitoring in the Safety Assessment of Dams and Reservoirs
   Kalaugher, Grainger

5. Discontinuance/Abandonment of Killamarsh and Woodall Reservoirs Under the Reservoirs Act (1975)
   Ferguson, McFadyean

6. The Asphalt Membranes at Colliford and Roadford Reservoirs
   Evans, Wilson

SESSION 1 SAFETY OF EMBANKMENT DAMS

Contributions and responses

Mr D J Knight (Sir Alexander Gibb & Partners)
Near failure of Upper Mun dam, Thailand 1990
The examples given by Messrs Fry, Brun and Royet in their paper on internal erosion problems in some old embankment dams in France are further salutary reminders of what can happen inside homogeneous embankment dams. They state (paragraph 48, page 34 of the proceedings) 'the inability of piezometers to detect such phenomena in silt is noteworthy' and remark that 'The problems are detected visually'. In old embankment dams, or dams without good records of the design intention and as-constructed state, the detection of a problem is inevitably visual, at the time it becomes manifested in increased seepage, leakage, cracks, settlement or sinkholes.

In correspondence to *Dams & Reservoirs* October 1993, page 25, this writer summarised an example of the near catastrophic failure of a modern embankment dam in south-east Asia immediately following its first impounding. Complete failure was narrowly averted by the dedicated efforts of many people to staunch discharges at the downstream toe which peaked at 6 m^3/s. Because the incident was still under investigation at the time of the previous (seventh) conference of the BDS in 1992 it was not permissible then to inform the dam engineering community generally of it. However, a paper was subsequently published on it at an international conference on Dam Engineering, January 1993 at Jahore Bahra, Malaysia, entitled 'Causes of a near catastrophic leakage of a dam in Thailand', by N Phienwej and K Sridhar of Asian Institute of Technology (AIT), Thailand.

It was referred to in another AIT paper by N Phienwej, P Nutalaya, V Udomchoke, T Pientong and A S Balasubramaniam, entitled 'Properties of problem soils of arid northeastern Thailand', Volume 4, Proceedings 13th ICSMFE, 1994, New Delhi, pp 1535-1538. Other papers by those closely involved were also published in 1991 and 1992. The foregoing publicity, therefore, removes the need for further anonymity.

The dam was, in fact, the Mun Bon, or Upper Mun dam located in Nakhon Ratchasima province in northeastern Thailand, and completed in 1988. Its maximum height was 32 m, with a crest length of 880 m, and was of modified homogeneous cross section, i.e., it had an inclined chimney filter drain downstream of its central impervious portion. The foundations comprised deep alluvial deposits of sand, silt and clay. The dam's upstream profile was 1:3 for its upper part, flattening to 1:5 lower down, with a 1:15 berm over the central length. Downstream profiles were 1:2.5, flattening to 1:5 over the central length.

First impounding occurred on 21/10/90, when the reservoir rose rapidly to within one metre below FSL, at which the reservoir volume is 141 million m^3, following a 9.5 m rise in the preceding fortnight or so after a tropical storm. The rate of rise of the final 4.5 m exceeded one metre/day. On 22/10/90 the first of two major leakages occurred, this one being from the downstream toe of the left abutment, at a general rate of 4 to 5 m^3/s, peaking at 6 m^3/s. Transversely collinear with this discharge was a crest sinkhole, downstream slope cracking and a vortex in the reservoir over the submerged upstream berm. The next day a second leak occurred, 5m above the downstream toe, in the central section of the dam, and was again associated with crest cracking and sinkhole occurrence.

The major emergency thus generated involved the temporary evacuation of the 10,000 people living downstream who were at risk from a complete collapse of the dam. Several hundred army personnel joined the dam owner's forces to control the leakage, which was eventually fully controlled by 16/11/90. A senior dam engineer from the owner's organisation directed these efforts, which involved re-profiling the embankment crest at each of the two leakage points and using the 'on-dam' borrow material to form large weighting zones at the downstream toe and a large grouting platform on the upstream slope at the first leakage point. The vortex was covered by material pushed from the upstream side of the dam, and sand bags were also dumped into the reservoir by boat. A major grouting programme was instituted in this area from the rapidly constructed upstream platform.

It was initially proposed that the reservoir be lowered rapidly through a large trench excavated across the central part of the embankment. It was providential that the highly dispersive nature of the dam fill clay was recognised before the upstream barrier was removed, otherwise large-scale, uncontrolled erosion of the dam body would have occurred and quite possibly caused the very catastrophe the water lowering method was intended to prevent. The reservoir was in fact sufficiently lowered by a combination of discharge through the left abutment river outlet culvert and siphoning over the spillway, using 24 steel pipes of 300 and 600 mm diameter.

The main physical causes of the leakages and associated cracks and sinkholes have been attributed to piping erosion through loose foundation soils in the areas of present and old river channels, together with the ineffectiveness of drain and filter materials. However, other contributory causes have also been suspected. It is also pertinent to note that, even though the dam was subsequently totally excavated and inspected down to foundation level, the authors of the 1993 paper have stated that 'a definite conclusion about the cause of the leakage could still not be reached'. It is possible that, amongst other factors, the rapid undrained loading of the relatively rigid dam on a compressible foundation could have caused cracking and thus shortened the foundation seepage path. Construction stage causes have also been suggested.

As stated in the October 1993 *'Dams & Reservoirs'*, the dam must have come as near as it was possible to come to its limits of endurance. Thankfully, endure it did, and 'the first major dam failure of Thailand was (thus) avoided'.

Mr A D H Campbell (W A Fairhurst and Partners)
Cut-off difficulties and subsequent failure of the cut-off at Earlsburn No 2 dam
Earlsburn No 2 reservoir is situated near Stirling. It is downstream from Earlsburn No 1, described in No 1 of Vol 1 of the *Dams & Reservoirs* BDS journal as being possibly the only dam in the UK to fail completely due to an earthquake.

The dam to No 2 reservoir was constructed much later than that occurrence. Work on it started in 1902 and was considered complete in 1905. It is a traditional earth bank with a so-called puddled clay core and cut-off trench in the underlying rock. Its maximum height is about 15 m. It has the unusual feature of a long low extension over undulating ground, not considered here.

Impounding commenced in October 1905. When the depth had reached 4 m, a spring appeared on the south side of the main valley some distance downstream from the toe of the dam. The engineer for the works, the All Reservoirs Engineer of the time, had excavations carried down into a length of the cut-off trench from the crest, the clay removed and the trench examined, but no precise cause for leakage could be ascertained, and the clay was carefully put back.

Impounding recommenced and no springs appeared until the depth of water was 8 metres, when a set of springs appeared on the north valley side at a higher level than the first set and a depression appeared in the crest. At about the same time further small springs also appeared on the south valley. A short length of cut-off trench on the north side was opened up, deepened and refilled. On this side the engineer for the works concluded, on the grounds that the spring water was crystal clear, that the spring water was passing under the cut-off trench, and therefore no further action was required. The longitudinal section of the embankment is illustrated in Figure 1. On the south side of the waste weir a short length of the cut-off trench was opened up and a pit sunk through the rock to a depth of 16 m from the crest and refilled.

However, it a relatively short time a further depression appeared in the crest on the north side of the dam and the spring waters turned muddy. A section of the cut-off trench was opened up and an inspection pit sunk in the bottom. Thereafter the cut-off trench was opened up from the north valley side to about the middle of the main part of the dam, deepened and refilled.

A depression also appeared in the crest on the south side, and again a length of trench was opened up, deepened and refilled, to be followed, in due course, by further sets of opening up, deepening and refilling. The final result is illustrated in Figure 2, the dotted lines indicating the stages.

The remedial works went on from 1905 until 1911, a full six years. In all, in the northern part of the dam the cut-off trench was deepened by an average of 4 m over a length of 90 m and on the southern part by an average of 10 m over 150 m. The work was all done from the narrow crest - 4 m wide.

It was indeed fortunate that the remedial works were finally successful. The rock, referred to at the time as whin, normally a fine grained igneous rock, is, however, a vesicular lava formed by many sheet flows and of a thickness greatly exceeding the maximum depth of the cut-off trench. This type of rock has tiny cavities caused by gas bubbles which can lead to massive weathering effects, probably also has pipes formed during the cooling stage and is much fissured by fine cracks.

In July 1985, shortly after deterioration of the spillways had led to an expensive renewal and when the reservoir was at an exceptionally high level, a spring appeared at the foot of the north valley side a short distance from the mitre with the dam. The issuing water was crystal clear. The spring continued when the reservoir dropped.

Efforts to trace the spring back into the hillside disclosed that this ground was made up of the considerable amount of broken rock won in the various cut-off trench deepenings. This approach had to be abandoned and investigations from the crest of the dam resorted to.

These disclosed the cause of the leak, a short section of the deepened cut-off in the true north side of the valley where there were disturbed conditions and a cavity in the clay. Downstream was a cavity in the rock. Other cavities were found nearby. The initial investigations were stopped and a contract let to continue the investigations and at the same time carry out grouting. Over a relatively short length the rock on the downstream side of the deepened cut-off was found to have cavities and be in a highly broken condition and there were small cavities in the clay of the trench.

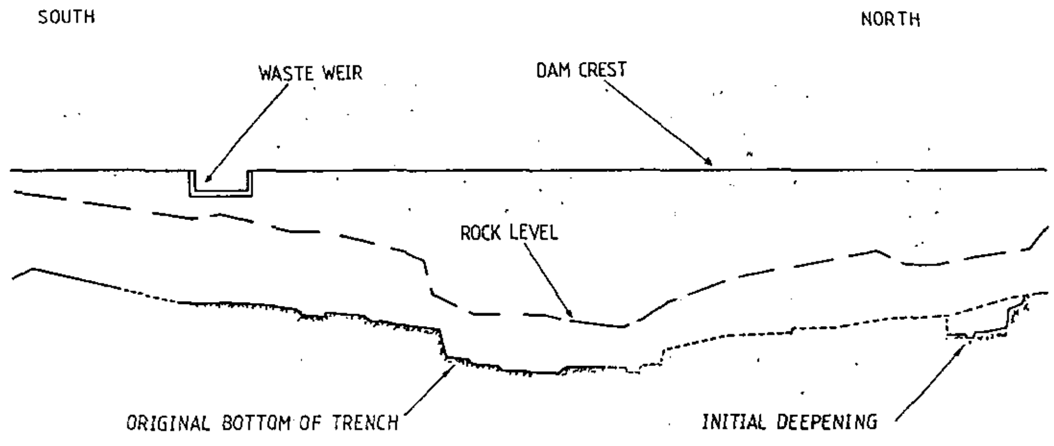


FIGURE 1

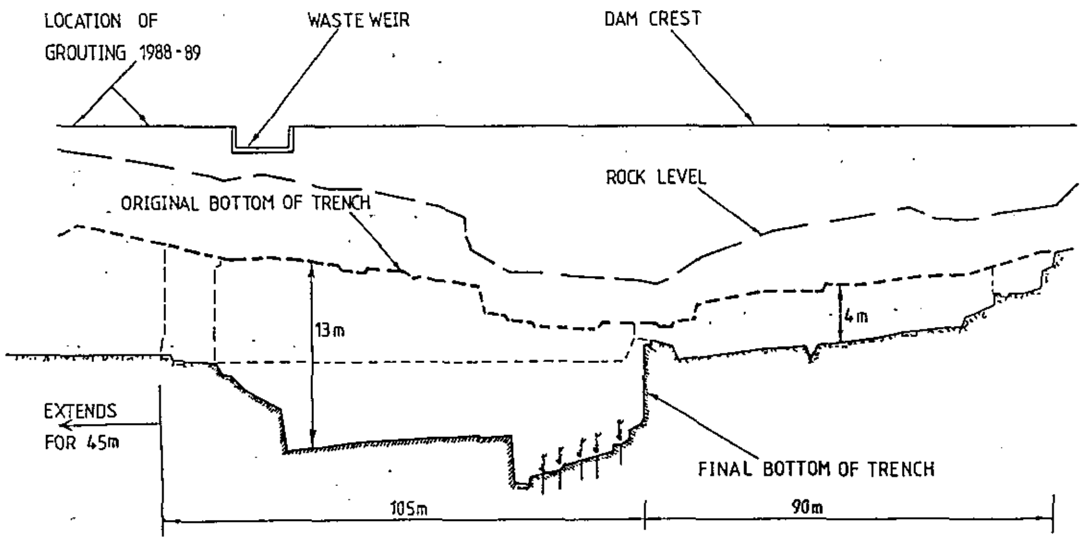


FIGURE 2

Cavity and rock fissure grouting and tube-a-manchette grouting in the trench were carried out over a length of 40 m and a further length of 30 m had single rows of *fissure grouting. The grouted depth in rock was about 20 m and about 18 m in the* trench. The materials used were 120 tonnes of sand, 492 tonnes of Portland Cement, 18 tonnes of bentonite, 12¼ tonnes of sodium silicate and 2 tonnes of High Surface Area Cement.

There have been no further leaks since the work was completed.

Dr J Brauns (Institute of Soil and Rock Mechanics, University of Karlsruhe)
Seasonal pattern of seepage through asphaltic facings
This brief comment refers to a statement in the contribution of Evans and Wilson concerning the seasonal pattern of the drainage quantities of dams with asphaltic facings. Leakages through the asphalt membrane is not ruled out by a seasonal pattern (under absence of an influence of the water level in the reservoir).

In this connection one has to keep in mind that there may be seasonal changes in the temperature of the water in the reservoir. Further, asphaltic concrete has a defined coefficient of thermal expansion/contraction as all leakages through asphaltic membranes are due to local imperfections (which in most cases can hardly be located), low water temperatures cause the asphalt membrane to contract and to open small imperfections (cracks, holes, etc). This leads to higher leakage quantities in wintertime, which is in accordance with our experiences in a number of practical cases. All in all, it can be said that there is more "life" in asphaltic material than - at a first glance - we are able to imagine.

Dr Brauns subsequently submitted a written contribution which follows.

Dr J Brauns (Institute of Soil and Rock Mechanics, University of Karlsruhe, Germany).
Blisters in an asphalt membrane - a case study
Asphalt membranes, when properly designed and constructed, are excellent means for sealing dams, particularly in cases where natural materials are not available (recent examples have been shown by Evans and Wilson in their contribution to this conference). In this paper a special problem related to this sealing technology will be dealt with in the form of a case study.

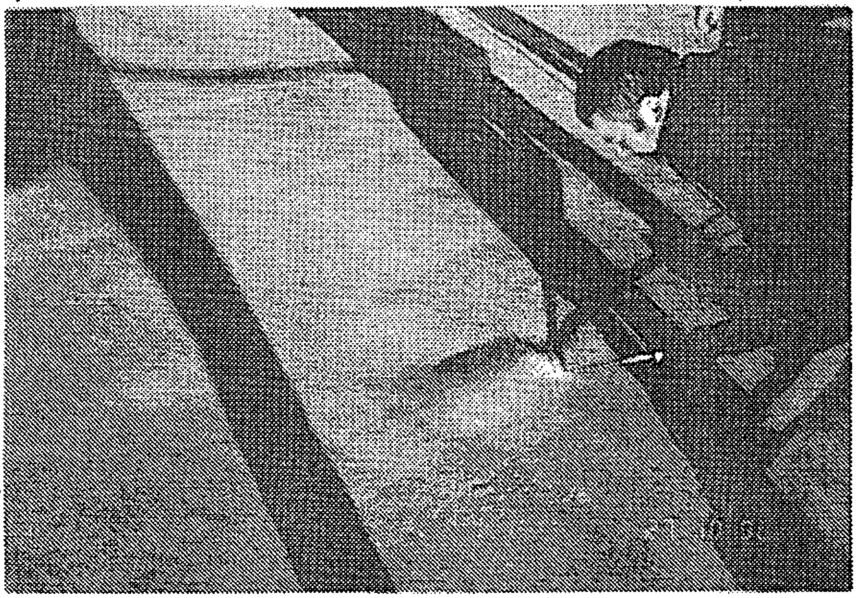


Figure 1 : Blister in an asphalt membrane

In certain cases, the owners of dams with asphalt membranes have been faced with the problem of blisters appearing on the liner surface (Figure 1), sometimes in large numbers up to several hundreds. As such blisters have a tendency to burst, they cannot be regarded as a pure disfigurement.

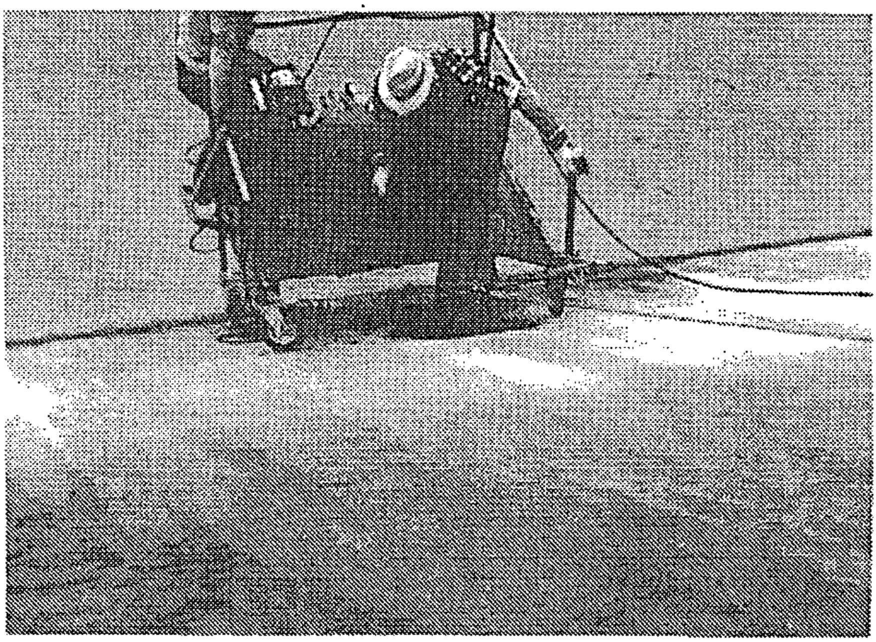


Figure 2 : Repair of a burst blister

Repairs are necessary (Figure 2), causing a lot of labour and cost. In an actual case with a continuously increasing number of such blisters, we were asked to investigate the conditions of development of these deficiencies and to try to make a prognosis as to the number of blisters to be expected in the future.

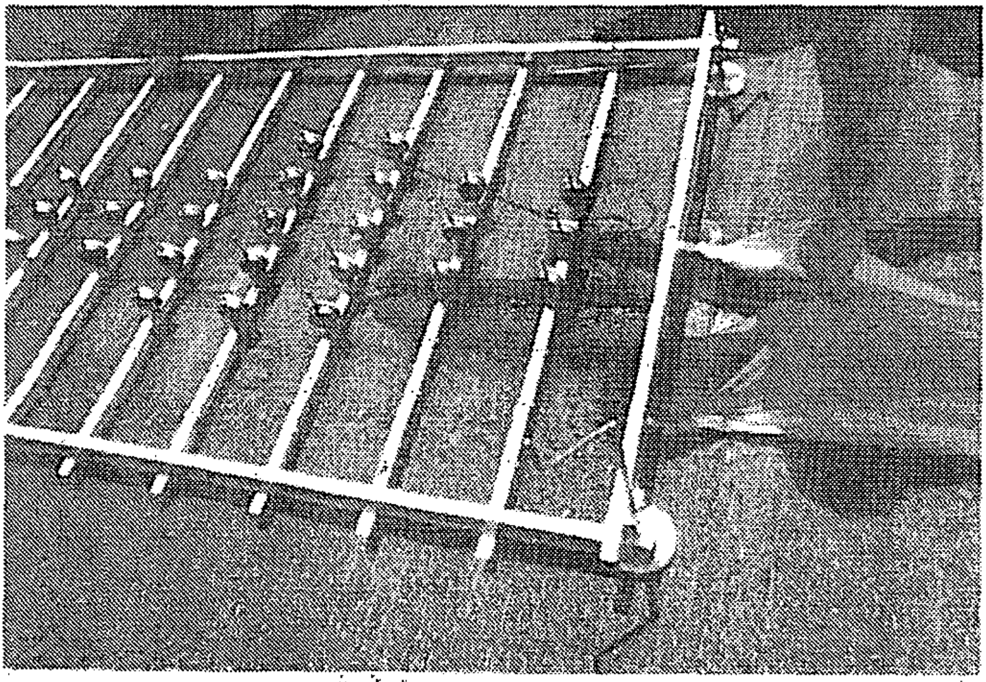


Figure 3 : Draining water from a blister

Firstly we were interested in what these blisters may contain. Thus, we made port and aeration holes and found a certain volume of water (Figure 3).

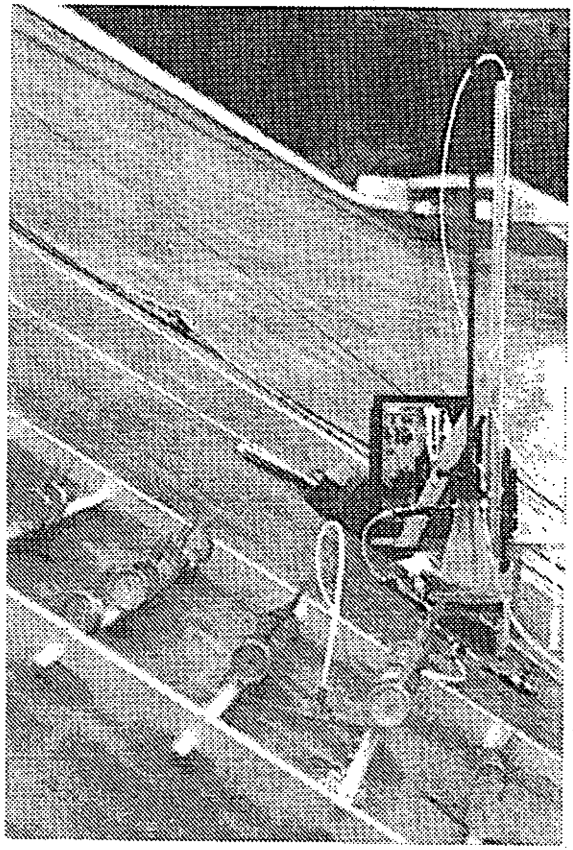


Figure 4 : Investigation of the hollow volume of a blister

Then, we developed a special procedure with a small cylinder with pressurised air connected to a blister (Figure 4). With this arrangement we were able to determine the total hollow space of individual blisters. By comparing the quantity of water (drained from a blister) with the total hollow volume, we found that the blisters were (±) filled with water.

Further with the help of a set of dial gauges (mounted on a support frame) and of applying some pressure we tried to determine the extent of a blister in area (see Figure 3).

Core samples were also taken in blister areas, and we learned that the hollow space had developed more or less at half depth in the 8cm thick asphalt sealing layer.

Large samples with blisters were taken and then brought to the laboratory (Figure 5). From these samples we could see that there were flat hollow spaces in a certain area around the blisters, but the upper and lower portions of the sealing layer were connected to each other in certain spots within that area.

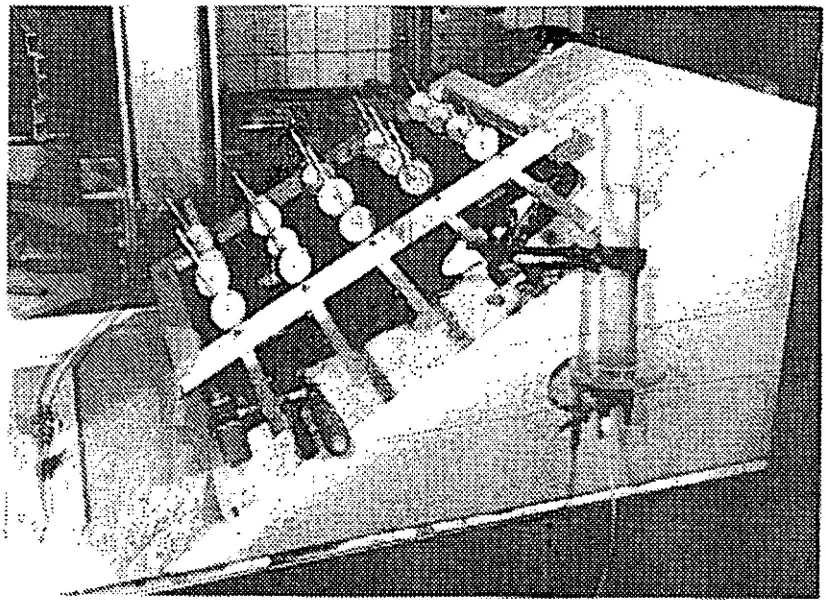


Figure 5 : Large square liner sample in the laboratory, blister in connection with communicating water reservoir

By means of tests using a water reservoir connected to the blister, the flexibility and immediate deformation of the top of the blister under very low pressure changes of a few centimetres of water head could be shown, particularly under high temperatures of nearly 60°C in some tests; these temperatures are easily exceeded in nature under sunny conditions. This confirms the general experience that asphalt membranes can be very flexible, depending on the temperature and rate of deformation.

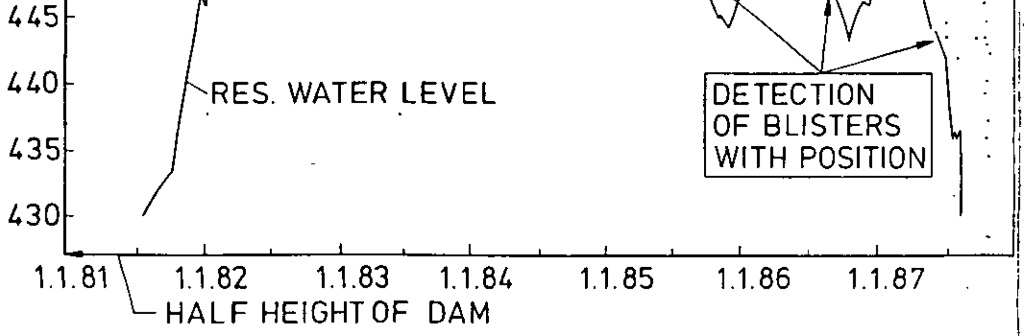


Figure 6 : Development with time of reservoir water level and of blisters in asphalt membrane

An interesting question was: where and when do blisters appear? As can be seen from the diagram in Figure 6, blisters do not appear under water. On the other hand, in the case in question, the blisters also required a certain time to develop; we concluded from some statistics that the number of frost periods plays a certain role in the development of blisters. The largest number of blisters appeared in areas where 4 to 7 cold winters affected the membrane under the absence of a water cover.

This aspect is important with regard to the number of blisters, which still may appear in the future, and also with regard to the warranty of the contractor.

One of the most important questions to be answered was: where does the water found in the blisters come from? Various reasons for blisters in asphalt liners and different hypotheses or philosophies regarding their development are discussed in the literature. In our case, we supposed that the water may have entered the asphaltic material during the process of construction, that is during placement of the hot asphalt mix, since the location of the blisters proved to be within the sealing layer. As is well known, water is used - for instance - to keep the drums of the vibrating rollers clean, or better: to prevent adhesion of the hot asphalt material to the mantle of the drums. Thus, we looked for a method to detect whether the water found in the blisters had ever been heated or boiled in the past.

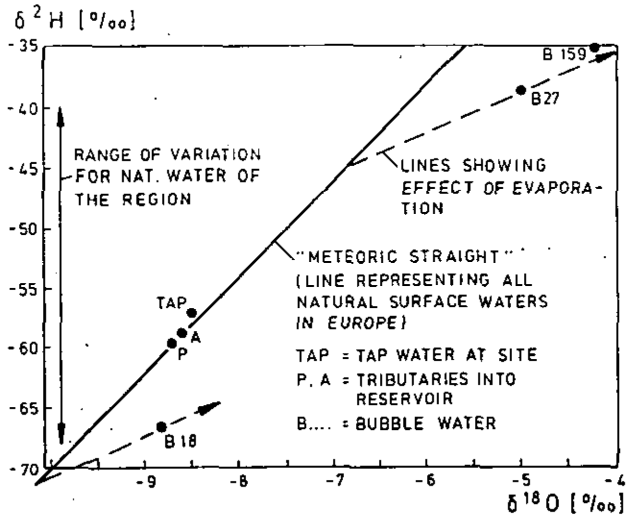


Figure 7 : δ^2H - $\delta^{18}O$ - diagram

We were glad to learn that the physics of isotopes can give this answer on the basics of the relation of Hydrogen- and Oxygen-Isotopes, δ^2H and $\delta^{18}O$ respectively. There is not enough space here to explain this in detail; but in conclusion it can be said that comparisons between the isotope contents in the water from the blisters and the contents in other water samples of the region clearly showed that the water found in the blisters had undergone a boiling process in the past (Figure 7).

From all our measurements, for the case in question, the following had to be concluded:

- the water in the sealing layer had entered the liner during construction;
- the water was contained in the liner in a ramified form over certain areas;
- most probably due to the repeated action of frost, the material contacts between the upper and lower portions (halves) of the sealing membrane and in the affected areas were weakened to causing cracking, resulting in an extended and continuous interlayer of water;
- due to its own head on the inclined slope and due to the weakness of the flexible top sheet of asphalt material, this interlayer - in a critical moment - caused bulging of the top layer forming the blister at the lower end of the areas in question.

As far as the prognosis of the number of blisters in the future was concerned, the question to be answered was: how many zones with included interlayers of water would exist? Various geophysical methods were checked as to their applicability under the prevailing conditions. Of these, only two proved to be promising:

- one was the geo-radar technique;
- the other was the infrared thermography method.

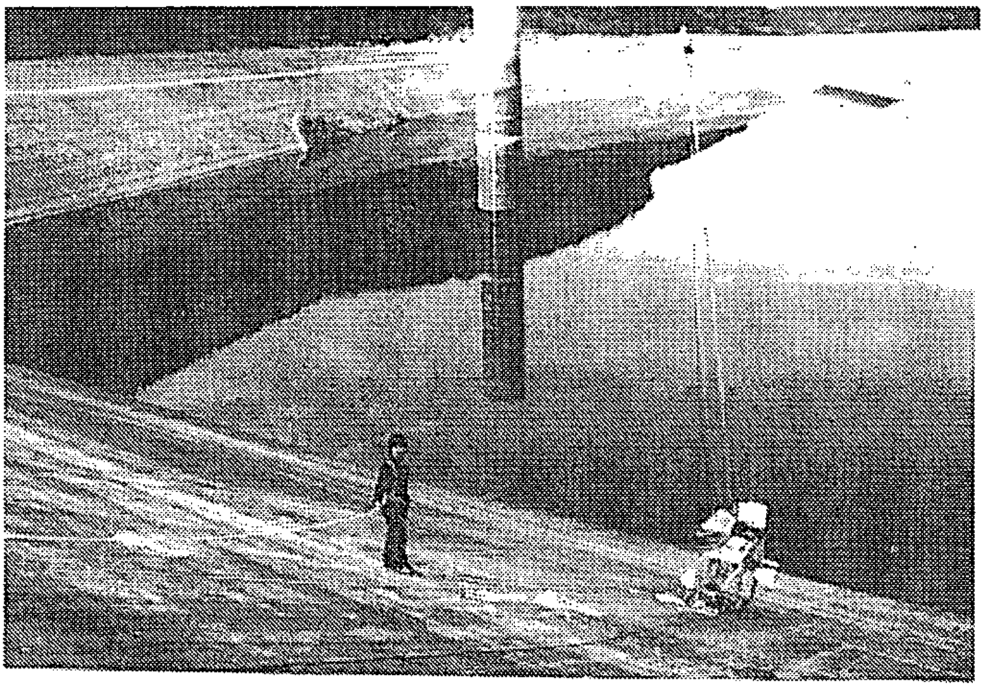
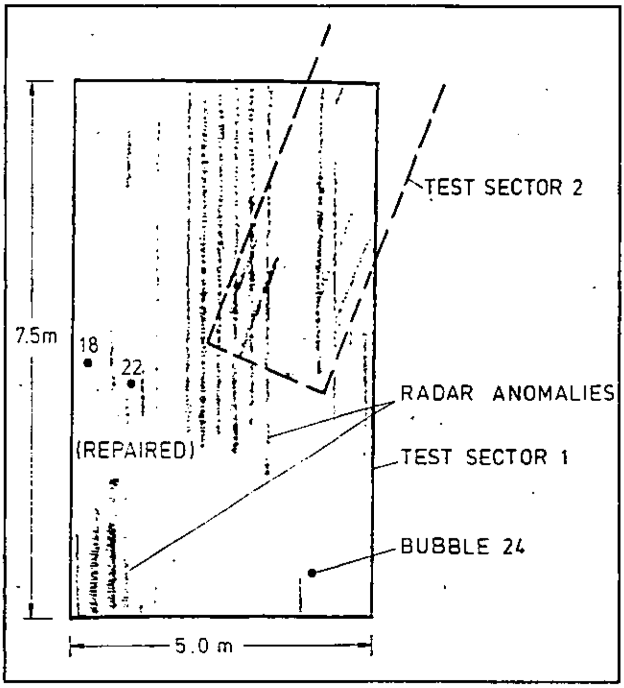


Figure 8 : Infrared thermography method applied on the membrane surface
(infrared camera in position on top of pole,
held in position with ropes to both sides)

The principal idea was to take measurements over a portion of the membrane surface and to extrapolate on that basis.

The expectations towards the applicability of the thermographical method were based on the fact that the thermal capacity of water is greater than that of an asphalt concrete by more than an order of magnitude. Thus, zones with a certain quantity of water should react with a delay to all changes in temperature. In other words: the water content should lead to anomalies in the surface temperature distribution.

We made thermographical records in a certain sector of a full day cycle of 24 hours. The method works well in areas with existing blisters. Unfortunately, in areas with thin interlayers of water, the effect of a temperature anomaly is so small and "noise effects" become more disturbing, so that a clear picture was hardly to be gained here.



Those who have been involved with geo-radar measurements know, that the interpretation of these investigations is also difficult and cumbersome. Our impression was that the measurements could be helpful in cases such as the one in question. But the owner of the project refrained from continuing these efforts in view of the expenses.

Thus, our prognosis for the number of blisters to be expected in the future had to be based solely on statistical evaluations. For normal reservoir level fluctuations, we expected some 10 to 20 new blisters in the upper zone of the membrane near the dam crest.

But, if the reservoir were to be emptied over longer periods over a number of years, so that the entire membrane were to be exposed to the atmosphere over frost periods, many more blisters - say up to 200 - should even be expected to appear. Fortunately this situation was not likely to arise.

In the case reported here, the legal proceedings led to the following agreement between the partners:

- prolongation of the warranty period by about 10 years;
- repair of the blisters by the contractor up to about the middle of that prolongation;
- after this time: repair by the contractor with payment by the owner.

In fact, blisters still continue to appear.

In conclusion, it is felt that the careless use of water to prevent the adhesion of hot asphalt mix to compaction devices is a risky technique. Special attention should also be given to the weather conditions during construction of asphalt membranes, because rainfall may probably lead to similar problems.

Concluding remark:
This case study is not reported here to provoke doubts or general concerns about the use of asphalt membranes in dam engineering. Instead, the contribution should draw the readers' attention to the fact that the construction of such structural elements calls for perfect control of all relevant procedures and for a consequent restriction of construction works to periods of adequate conditions on the site.

Mr C J Sammons (Independent Consulting Engineer)
Impermeable upstream membranes
In paragraph 4 of their paper on 'the asphalt membranes at Collingford and Roadford reservoirs', Evans and Wilson give their main reasons for selection of upstream membranes for the two dams. Could they say whether they either considered during the design or would consider now the use of a plastic membrane as an alternative to an asphaltic concrete membrane? If not, what are the major advantages they see of asphaltic concrete over plastic?

My interest comes from involvement with construction of a 3.5 km long, 21 m high embankment dam at Jibiya, in northern Nigeria. The dam was built mainly of a highly erodible, silty fine sand, probably not unlike the sandwaste at Colliford; the dam was also founded on sand. The membrane was connected into a plastic concrete diaphragm cut-off wall constructed well clear of the upstream toe of the dam. Surface protection was provided by approximately 100mm thick, 4m by 2m, cast-in-place concrete slabs connected together by a simple system of articulated joints.

In comparison with the asphalt membrane details, it is of note that the designer at Jibiya did not see a need to include a drainage layer beneath the plastic membrane. Internal drainage measures were limited to the downstream section of the dam. This may be because it would not have been practical to incorporate a gallery along the upstream toe; random collapse settlements of around 0.5m were anticipated on flooding of the foundations.

A point the designer frequently made was the ease with which any damage to a plastic membrane can be repaired. However, a concern I have is the problem of identifying the location of flaws sufficiently early to prevent harm to the body of the dam. At Jibiya initial warning of problems would come from seepage measurements at the downstream toe and from readings of an array of piezometers under the membrane. Do the authors have any comments?

Mr W J Carlyle (Independent Consultant)

Thermal movements in membranes

During the discussion on Dr Brauns' contribution to Evans' and Wilson's paper it was suggested that thermal movement of the deck might be more significant in causing cracks particularly near the perimetric joint.

Mr Carlyle referred to Marchlyn dam which at 636m above sea level was one of the highest dams in the country. The deck was exposed by a regular daily water level fluctuation which could reach 30m in level with the transfer of $6Mm^3$ of live storage in the pump/generation cycle.

The deck was exposed to severe freezing conditions in the winter and although the sacrificial surface seal coat had been renewed after 10 years of operation there had been no significant increase in seepage flow nor in settlement. Unfortunately there were no permanent thermometers installed to measure the thermal cycle: this should be considered by the owners.

Mr R Bunn (Binnie & Partners)

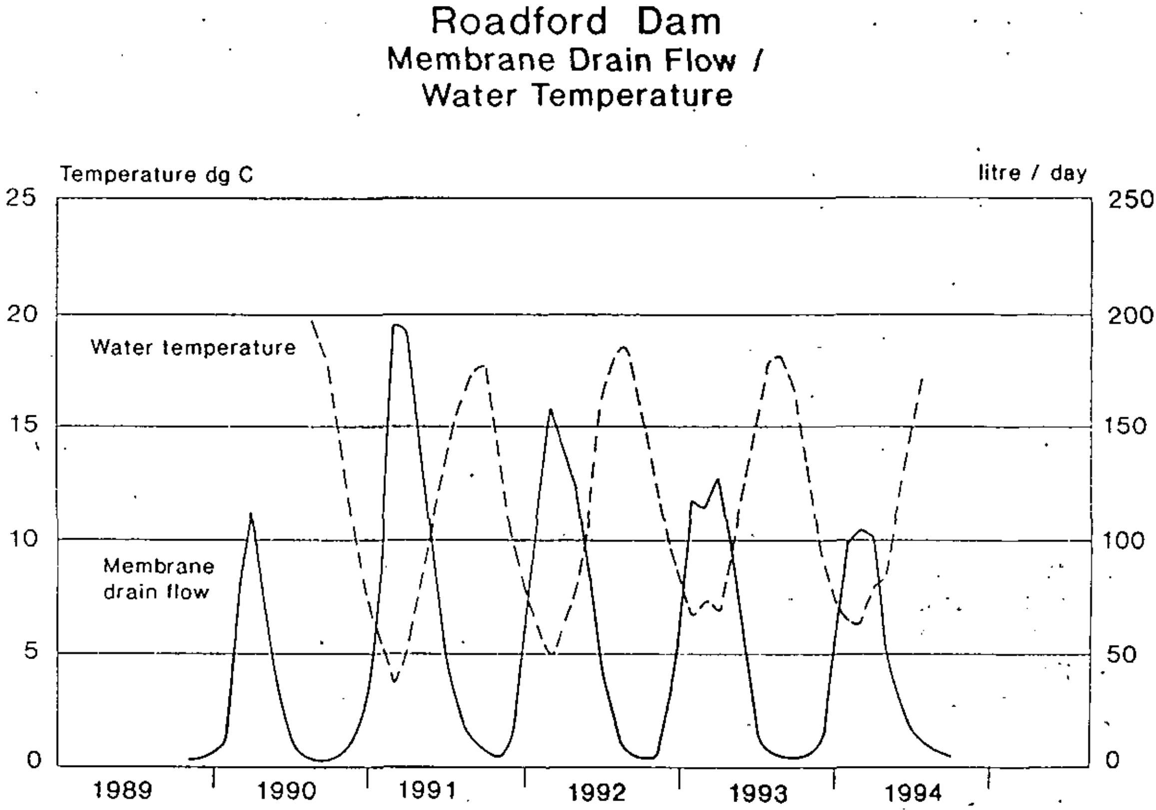
Mr Bunn informed the conference that measurements of the membrane deflection at Marchlyn Moor were made by means of a trolley-mounted inclinometer, and referred to problems with mastic seal coats. He asked the authors why a mastic seal was used at Roadford and Flintkote at Colliford.

Mr J D Evans and Mr A W Wilson in reply.

Professor Brauns has suggested that the seasonal flow from the drains under the membranes at Colliford and Roadford could be the result of local imperfections in the asphalt, responding to thermal expansion and contraction, caused by changes in water temperature. The evidence from Roadford is that both the water temperature and the temperature in the electrolevel boxes under the membrane varies between a 18.5°C in Summer and a minimum of 3.5°C in Winter; a range of 15°C. The maximum flow from the drains coincides with the minimum temperature and vice versa as is shown in the following diagram. As described in the paper, chemical analysis of the drainage water now indicates an unidentified source for the drainage flow, but in view of this latest evidence it seems probable that a proportion must result from the increased permeability of the asphalt at lower temperatures.

A factor in the choice of the membranes was the ability of the asphalt to move and adapt to settlements of the underlying structure without cracking; future designs should also take into account the effect of temperature on permeability.

Mr Bunn referred to the deflection measurements made on the membrane at Marchlyn dam by means of a trolley-mounted inclinometer. A similar system was considered for Roadford, but eventually the electrolevel system described in the paper was chosen. These are robust instruments which are simple to install and have now provided satisfactory readings for a period of over 5 years. The results continue to give confidence in the amount of settlement that occurs close to the cut-off.


Roadford Dam
Membrane Drain Flow /
Water Temperature
Temperature dg C
litre / day
25
20
15
10
5
0
250
200
150
100
50
0
Water temperature
Membrane
drain flow
1989
1990
1991
1992
1993
1994

Mr Bunn has also referred to problems with mastic seal coats that have been removed from the face of the asphalt on the membranes of some dams within a period of a year. Fortunately, similar problems have not occurred at either Colliford or Roadford. At Colliford, a hot applied mastic seal was specified, but the sub-contractor proposed a Flintkote emulsion to prevent water becoming trapped in the interstices of the asphalt. The successful application of Flintkote led to it being specified for Roadford, but the sub-contractor there preferred to use a mastic seal believing it to have a superior long-term performance.

In reply to Mr C J Sammons the use of a plastic sheet membrane was not seriously considered for the waterproofing membrane at either Colliford or Roadford dams. The authors believe that the advantages of using an asphaltic concrete membrane rather than a plastic sheet membrane are that the former is:

(i) more robust
(ii) does not require to be covered to protect against temperature effects, vandalism or accidental damage
(iii) has a reasonably long service life
(iv) is cost effective when overall costs are considered
(v) can be easily repaired, if necessary, without need to remove protective covering.

The tender design for the 5m high x 328m long subsidiary dam on the watershed of Colliford reservoir incorporated a high density polyethylene membrane as the waterproofing element. In the event, the dam was constructed with an alternative bitumen/nylon fabric membrane, Hypofors NF1000, which was proposed by the contractor. This membrane was laid on and covered by sand and soil to protect it from the underlying rockfill embankment and possible vandalism or accidental damage.

The ability to have early indication, flow monitoring and a guide to the location of any leakage through the asphaltic concrete membranes were seen as necessary features. These have been achieved by installing broken stone under-membrane drainage systems which discharge at close centres into the galleries within the toe structures. This arrangement complements the principal function of the toe galleries which is to facilitate the reinforcement of the underlying grout curtain, if that is ever found to be necessary.

Mr H Lass (South West Water)
Grids to overflow structures
I found the paper on the discontinuance/abandonment of Killamarsh and Woodall reservoirs very interesting, particularly as South West Water has a similar old embankment dam, originally constructed to serve the Bude Canal. Although not subject to mining subsidence, it is no longer in use and discontinuance may well be its future.

I note from Figure 3 that the new weir outlet to Killamarsh reservoir is fitted with a steel grille to prevent access. Many engineers have expressed concern at the potential for blockage of such grids with debris. 'Dam Safety Guidelines' draws attention to the need for unhampered operation of spillways. Indeed, I have heard the opinion that such a reservoir may still be capable of containing more than the minimum volume and not be outside the Reservoirs Act.

I would welcome the authors' comments on the resolution of conflicting requirements of safety of individual members of the public gaining access, as against safety of the public at large should the dam be overtopped.

Mr D A McFadyean replied that the reservoirs form part of a public amenity and it is intended to develop this use. Consequently, measures to ensure the safety of the public were essential and had to be as foolproof as possible. In order to achieve this and at the same time satisfy reservoir safety requirements, a multi-level approach was adopted to ensure both. Those for the overflow at Killamarsh are summarised below:

1. The grillage placed over the outlet pipe has an opening width to prevent human access and will collect only large debris. Horizontal screens were placed on top of the inclined grillage and the total open area of the screens is some 50% more than the area of the outlet pipe.

2. Should the inclined grillage become totally blocked, the horizontal screens still pass the flow from the 1:500 year return period flood.

3. In the event of the horizontal grillage becoming blocked by debris, the headwall of the overflow weir has been set about 500 mm below the crest level of the dam and water will flow down the partly backfilled original overflow channel in natural ground.

4. There is sufficient storage above the sill level of the weir to accommodate the probably maximum flood without the dam being overtopped.

5. A concrete kerb had previously been placed on the upstream edge of the crest where the embankment height exceeds about 2.5 m. The kerb is set in haunching 300 mm deep and 300 mm wide at its base.

6. At the time of discontinuance some routine maintenance and inspection requirements were recommended for the reservoir. With an active angling club using the reservoir, we are confident that natural or deliberate blockage of the grille will be noticed and action taken to remedy it.

The Reservoirs Act (1975) does not define "... hold or capable of holding..." used in Section 1. However, Part 7, Section 4 of the Prescribed Form of Record is succinct in terms of the capacities that need to be recorded, the term "top water level" is defined and this, therefore, clarifies the intent of the Act.

In summary, the decision to use screens at this site was taken in the light of the more probable and frequent likelihood of the visiting public falling and injuring themselves than the very remote possibility of the dam overtopping. The latter risk, given the particular circumstances at Killamarsh (as outlined above), was accorded the priority you see in the design. However, this is not considered to be a generally applicable case and each site should be judged on its own merits.

Mr A C Sutton (North West Water Engineering)
Update on work at Woodhead reservoir
First I would wish to add my appreciation of the work that Paul Tedd, Andrew Charles and BRE have undertaken over the years particularly with respect to Dams. We must all, without a doubt, be strident in our demands that sufficient resources are found to ensure that their valuable research work continues. Returning to the paper itself, it was with great interest that I found two reservoirs included that NWW is currently designing remedial works for, or carrying out further construction on. Both are in the Longendale Valley near Manchester. A short resumé of the current works may be of some interest.

Woodhead was, of course, an extreme example of reservoir remedial works by Bateman himself, with the building of a second embankment and the infilling of the volume between the two. This dam was recently raised in 1991, not to impound additional water but to provide sufficient additional storage that in the event of the PMF, part of the flood would be held at the upper reservoir of the cascade and allow less extensive works to be carried out on the lower reservoir. During the subsequent months, with the reservoir impounding and near top water, a flow of water was evident around the north abutment. Whether this "leakage" in whole or part has always been present, but has now been concentrated by the new spillway works, or whether the new works has in part caused it, is speculation. However, a remedial contract to expose the old Bateman concrete core (it is clay on the embankment side of the spillway) at this point and cast a wedge of concrete against it, including plugging any fissures in any exposed rock in the abutment, is nearly complete. An observation manhole has been constructed and there will be facilities to allow seepage a free flow, if this is deemed appropriate, following monitoring. Impounding will recommence shortly.

At Torside, the next reservoir in the chain, design work is at an advanced stage. Here, site investigation results have been revealing. Work has been undertaken in an attempt to determine whether the original core or Bateman's remedial upstream clay blanket is acting as the prime water barrier. To support my opening comments, the work has been undertaken with advice and support from BRE and I am sure the results will form a future paper.

Mr T A Johnston (Babtie Shaw and Morton)
Factors of safety and the funding of research
The paper on remedial works to clay cores, by Messrs Tedd, Charles and Holton, reminds us of the invaluable work done by Building Research Establishment (BRE) acting as both an information exchange and a centre of excellence in relation to old embankment dams. There must be concern among dam engineers about the change of status of BRE as part of the Government's privatisation drive. The work of BRE helps designers, owners, contractors and reservoir supervisors. Can the authors foresee how this work will be funded and organised in future?

The description of the Frontan homogeneous embankment design in France in the paper by Messrs Fry, Brun and Royet gave examples of dams which function satisfactorily without a clay core or other well defined water-barrier. The procedure described to ensure the safety of these old dams has close parallels with the current work of British engineers. The textbooks and published guidelines can help in indicating acceptable factors of safety for new structures where the materials and workmanship are well controlled but the assessment of old dams requires more judgement to be applied due to the greater uncertainty over the parameters of the fill, the foundation and the piezometric surfaces. Can the authors indicate what range of factors of safety would be considered acceptable in France under various design conditions, eg., in normal service compared with rapid drawdown.

Dr J A Charles (Building Research Establishment)
Reply to discussion contribution from Mr T A Johnston concerning funding of BRE
Mr Johnston has drawn attention to the work of the Building Research Establishment (BRE) on the safety of old embankment dams. The involvement of BRE in research on the performance of embankment dams goes back for well over 50 years. Since 1983 most of the work on dam safety carried out at BRE has been funded by the Water Directorate of the Department of the Environment (DOE) as part of their reservoir safety research programme. Dr David Coats was commissioned by DOE to carry out a review of reservoir safety research and he presented his "Assessment of Reservoir Safety Research" in 1993. His recommendations included the following:

- "Because the failure of a dam can have such disastrous consequences and because knowledge on many technical factors relating to dams is anything but complete, future research programmes on reservoir safety should be on no less a scale than at present..."
- "I do not recommend any change in direction. On-going research such as that being carried out by BRE and research already commissioned by the Department must, obviously, continue..."

Mr Johnston has questioned the impact of a change in the status of BRE on dam safety research and its funding. This is a good question but, unfortunately, it is not possible to provide an answer as no decision about a change in status of BRE has been made at the time of writing (14 December 1994). Following the Efficiency Unit Scrutiny of Public Sector Research Establishments, there was a four-month consultation period which ended on 11 November 1994. The main representative bodies in the construction industry discussed their preferred options for BRE and their views were set out in the response of the Construction Research and Innovation Strategy Panel to the invitation to submit evidence. It was stated that there was an overwhelming need for a national focus, or Centre, for research and technical activities in the construction industry, which is independent, impartial and authorative and it was recommended that no change should be made to the existing Agency status of BRE until a full plan has been developed.

Mr J-J Fry replied that in France some guidelines are published by the Ministry of Agriculture to build and assess the safety of small dams, but no such text books or guidelines are written by authorities for large dams. The general feeling is that it's not recommended to impose rigid rules because each dam is unique and that use of expert experience is a preferable way to proceed.

It is, therefore, not surprising that some old dams have different safety factors in normal service or rapid drawdown. Some dams have a Factor of Safety of 1.3 in normal *conditions and 1.0 in rapid drawdown. On the one hand if the behaviour is correct no* remedial measures are taken, on the other, to reassess stability we try to obtain 1.5 under normal service, and 1.2 with rapid drawdown. Usually the Frontard dams have good stability for both conditions, due to the fact that sandy soils are resistant (ϕ = 37 to 43°). The Lavaud Gelade dam, for instance, has a Factor of Safety = 1.58 for large circles.

In terms of cases of internal erosion, because heterogenous layers exist and some local leakage has appeared, no safety factors are computed to assess the degree of safety.

Mr Carlyle (Independent Consultant)
Instrumentation in dams
In respect of embankment dam instrumentation and experience of other engineers in:

(a) blockage of settlement gauge/slope indicator tubes due to deformation of soft clay cores;

(b) failure rates for vibrating wire electrical piezometers.

Mr Carlyle's experience with (a) has been rather poor because of the severe deflection of the first few lengths above the base under the influence of heavy plant. He accepted that the modern probes and their readout equipment were a great improvement but he doubted if the information obtained on slope change was worth the disruption in the fill placement. Settlement and consolidation can be monitored in other ways. With regard to (b) Mr Carlyle wanted to have experience of others on failure rates particularly from the risk of lightning strikes.

Dr A D M Penman (Geotechnical Engineering Consultant)
Instrumentation in dams
In reply to Mr Carlyle's question about settlement measurements in soft clay cores where vertical access tubes such as those of the USBR Cross-Arm Gauge deformed, preventing accurate measurements from being made, I prefer always to avoid vertical objects in fill being placed, because of the trouble they cause to placing machines, the damage that occurs to them (particularly during night shifts) and the distortions of the type that Mr Carlyle describes. I have always found it much better to use horizontal access tubes, for overflow and other types of settlement gauges, strain gauges and of course, piezometers. We have taken small bore tubes for settlement gauges into and through clay cores without any deleterious effects. The larger access tubes for horizontal plate gauges (see Penman et al 1991) have been taken into but not through clay cores to record the horizontal movements and settlements at the edges of the filter and the downstream part of the core.

Mr J M McKenna (Consulting Geotechnical Engineer)
The stability of Yonki dam, Papua New Guinea
I was surprised that Dr Penman has included Yonki dam in his paper linking it with two dams which had major problems. In my opinion, the stability of Yonki dam was never at risk. However, the object of the following discussion is to show how carefully instrumentation readings need to be interpreted.

In October 1990, when the embankment was 5 m from full height, filling was stopped and the stability of the Yonki dam examined in great detail. This was done because of the possibility that two inclinometers were indicating the onset of failure at the same time as three electrical piezometers went out of action. Fig 1 shows the three inclinometers in the upstream shoulder of the dam. Inclinometer I9, in the middle of the upstream shoulder, had become blocked at foundation level some two and a half months earlier in July. On October 1, the soil blocking the tube was washed away and the horizontal deflection measured. This appeared to show that in the interval, there had been significant horizontal movement, about 20 mm, at foundation level. A few days earlier, inclinometers I10 and I10A had become blocked at El 1240, that is, 25 m below the level of the fill at that time. As shown on Penman's Figure 5, the initial opinion on site was that these events were indicating the possibility of a slip developing.

During the study of the stability of Yonki dam, it became clear that the large movement in I9 was due to an 1.5 m error in the level of the readings. Once this had been founded and corrected, the movements of I9 did not cause concern. In fact, as shown in Fig 1, the maximum movement of I9, between the time the casing had reached its final design height and the end of construction, was only about 50 mm.

The blockages in I10 and I10A were only partial, as the settlement probe could still go all the way down to the bottom of the tubes, but the inclinometer torpedo could not get lower than El 1240. This had happened twice before in I10 at El 1229 and 1220. The details are given in Table 1 below. When I10 first blocked, another inclinometer I10A was installed alongside.

Inclo	Blockage No	Fill Elevation when Installed	Fill Elevation when Blocked	Elevation of Block	Height of Fill above Block (m)
I10	1	1210	1238	1220	18
	2		1254	1229	25
	3		1265	1240	25
I10A	1	1247	1265	1240	25

What is of interest is that these blockages were caused, as was thought at the time, and subsequently confirmed by a borehole camera, by a reduction in the internal diameter of the inclinometer tubing at the telescopic joints. The inclinometer tubing was installed in 1.5 m lengths with 100 m compression joints.

At the time filling was stopped, the calculated Factor of Safety was 1.3. This had risen to 1.5 by the time filling restarted in December, due to the drop in the pore water pressures. At the end of construction, it was calculated to be 1.4.

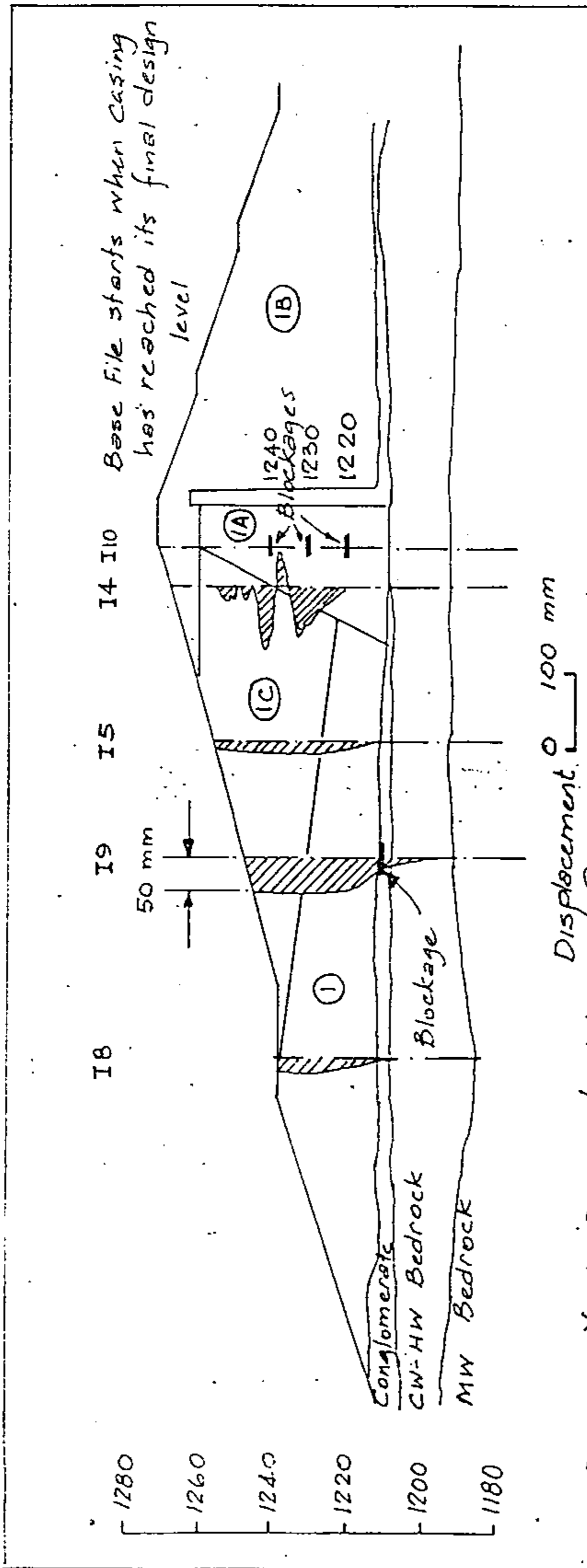


Fig 1. Yonki Dam - Inclinometer Readings - Upstream Shoulder

Mr C J Sammons (Independent Consulting Engineer)
Assessment of instrument readings
Mr McKenna, in his contribution on that part of Dr Penman's paper relating to Yonki dam in Papua New Guinea, has pointed out the need for careful assessment of instrument readings before reaching conclusions. Normal consolidation settlement had, in the case discussed, taken inclinometer tubing beyond its operating range and led some to an over pessimistic evaluation of the situation.

At an earlier stage of construction at Yonki, excavation for the outlet conduit reactivated a deep-seated, historical landslide over the downstream left abutment of the dam. The valley side was cut back to stabilise the movement before excavation for the conduit could be continued.

Standpipe piezometers were installed in and around the area of the slide to monitor the potential rise in groundwater level at the onset of the wet season. A time dependent recovery in pore-water pressure, ie. swelling in clay horizons would also be anticipated following the undrained removal of load during excavation.

Readings were taken daily and plotted up regularly. The impression gained was that rainfall and swelling was having little significant effect on piezometric pressure and work was continued with some confidence. However, on examination, it was found that many of the readings were unreliable or had been misinterpreted. Several of the piezometers were not in the strata proposed. Others had not been properly repaired following construction damage. False and, in this case, over optimistic conclusions had been reached.

A 5 mm displacement in an inclinometer at the level of the slide shear surface signalled renewed movement. Work on the conduit excavation was immediately stopped and further material removed from the top of the slide mass.

SESSION 2 ENVIRONMENTAL ASPECTS OF RESERVOIR CONSTRUCTION AND MANAGEMENT

Chairman : ET Hawes

Technical Secretary : A MacDonald

1. Environmental Assessment of Reservoir Schemes
   Elder

2. Roadford Reservoir: Enhanced Flow, Fisheries and Hydroelectric Power Generation
   Sambrook, Gilkes

3. Roadford Lake - Leisure Development
   Stacey

4. Environmental Implications of Constructing a Concrete Reservoir Within a 100 Year Old Dam
   Elder, Fletcher, Rice

5. Maximising the Ecological Benefit of a New Small Reservoir
   Rofe, Hoskins, Rice

6. Reservoir Development in Semi-Arid Countries - A Human Ecology Perspective
   Findlay

7. Engineering Approach to the Environmental Impacts Alleviation at Pak Mun Barrage
   Udayasen

SESSION 2 ENVIRONMENTAL ASPECTS OF RESERVOIR CONSTRUCTION AND MANAGEMENT

Contributions and Responses

Mr F G Johnson

Roadford Reservoir fisheries improvements

In the Paper on Roadford Reservoir on enhanced flows, fisheries and hydro-electric generation, the Authors state that "Spawning and recruitment of salmon in the River Wolf has improved, in particular more consistent density levels have been recorded each year". Could the Authors quantify the improvements achieved by relating to the counts in the years before construction of the Scheme began, during construction and from commissioning to the present time. How do these improvements compare with counts in other rivers in the area not affected by construction of the scheme? Have counts of returning adult salmon and sea trout been recorded? If so, could comparisons, as requested above, be also given.

Mr J C A Binnie (WS Atkins Consultants)

Timescale for formulation of environmental management proposals

It was heartening to see the appreciable environmental mitigation measures at several reservoirs, but in particular the post construction environmental developments at Roadford, described by Miss Stacey.

I am currently involved in the planning of potential reservoir sites in East Anglia. An important decision is at what stage to announce the scheme, at what stage to show the mitigation measures and at what stage and how to prepare the environmental management measures. It was important that as much as possible, the local community accepted the scheme and "owned" the environmental management. In the author's view, how could this best be done?

Mr T A Johnston (Babtie Group)

Roadford Reservoir economic benefits and water quality

It is standard practice for capital projects, such as Roadford, to be subject to rigorous cost-benefit analysis. This was certainly the case during the design of the turbines and the selection of the original control system by South West Water and Babtie Electrical & Mechanical. Mr Sambrook and Miss Stacey have described valuable environmental enhancements which have been made since the reservoir was impounded. e.g. to the operating regime for the turbines to encourage an increase in the fish population and leisure developments to cater for the increasing visitor numbers. Have these enhancements been subject to cost-benefit analysis in the same way as the engineering works? On the river Tay in Scotland, salmon fishing is such a valuable asset that each salmon caught has been calculated to have a capital value of £50,000 to the regional economy! The work done by South West Water must have increased the value of fishing on the Tamar. The leisure developments at the reservoir are now part of the tourist industry.

Has South West Water benefited financially from its investment in leisure and recreation or is it regarded as part of a "good neighbour" public relations policy?

On a technical point, can Mr Sambrook give more information on the water quality discharged from the reservoir. There are often unusual features during the early years after inundation. South West Water have taken particular care to monitor discharges from the reservoir and drainage water from the dam. Is it possible to say whether the pre-construction concerns of the fishing community on water quality have now been dealt with satisfactorily?

Messrs Sambrook and Gilkes responded that the aim of the paper was to raise an awareness amongst engineers to the potential of flow control from reservoirs to benefit the environment, fisheries and users of the river downstream of the dam. With an audience dominated by engineers and not biologists, the emphasis of the Paper was on the engineering aspects of the scheme, with an introduction to the concepts and opportunities available through an understanding of the "instream flow requirements" of salmonids. Hence the Authors considered it inappropriate to present exhaustive arrays of environmental data whilst being restrained by the rules imposed by the editorial panel.

The current Enhanced Flow Programme (EFP) has been developed and will continue to evolve based on extensive fisheries and environmental monitoring. The fisheries investigations on the River Tamar began in 1985 and have involved radio telemetry studies on adult salmon, routine electrofishing surveys for juvenile salmonids, redd counts, trapping for salmon and trout (upstream and downstream movements), catch statistics analysis and more recently the use of a resistivity fish counter. The EFP is in the early years of development and will continue to be adapted in order to achieve an optimum balance between the operation of the reservoir and the environmental gains. The benefits of the programme to a the wild salmonid stocks are monitored annually and are reliant on electrofishing surveys and redd counts. The electrofishing surveys are undertaken at 23 established sites which have been sampled on an annual basis since 1984. The selected sites are located on impacted and controlled river reaches in the Lyd sub-catchment. Quantifying the improvements achieved in the early years of the EFP relies on a detailed knowledge of the fisheries database and extensive analysis. The presentation of such data together with other environmental information will form the basis of a subsequent paper which will be targeted specially at fisheries management. The need to present all the appropriate data in this response is unnecessary but the conclusions are worth stating ie the potential benefits of the EFP are recognised and the implementation has resulted in improvements to the salmon stocks. Salmon recruitment and fry production in the River Wolf, post impoundment, are less variable and comparable to the pre-construction years.

This has been achieved even though the river length and wetted area available to the salmonids in the River Wolf has been reduced by 50%. To date a positive approach has been taken to the management of regulation releases.

Without such an initiative it is likely that there would have been a dramatic impact on the wild fish stocks. Specific problems have been identified for the trout stocks which have resulted in modifications to the patterns and timing of the releases. Future monitoring will hopefully record improvements in the trout stocks. In addition, consideration is being given to the promotion and implementation of in-river engineering along the River Wolf. This would result in the establishment of the "ideal" spawning and nursery areas for the young fish.

Counts of returning adult salmon and sea trout are recorded in the statutory returns and private rod records. In recent years a new fish counter has been installed at the head-of-tide. The historic trap sites have been discontinued and as a result a continuous and consistent database is not available. Attributing any improvements in the total adult runs directly to the contributions made by the EFP is considered to be unachievable. Historically, the River Wolf has contributed an average 1% of the total juvenile salmon production in the Tamar catchment. Dramatic improvement in salmon smolt output would be required to prove conclusively any significant increases in adult returns. The response of the adult stocks must be considered in relation to natural variations, since there are numerous variables which influence survival and return rates to the coastal waters. In addition, there are many factors which will influence the in-river behaviour and homing precision of the salmon to the River Wolf. With an appreciation of these issues and a knowledge of the logistical problems of attempting to achieve and prove the benefits via the adults, a decision was taken to concentrate the monitoring of improvements on the juvenile salmonid stocks.

At the Public Inquiry in 1978, undertakings were given to outside interests in order to resolve numerous fisheries issues associated with the reservoir and its operation. Final permission was given to proceed with the reservoir in the Autumn 1984 and in subsequent years these commitments have been and continue to be honoured. Cost effective management of the environmental programmes has been an integral part of the overall engineering project, which has ensured that the undertakings are honoured, statutory needs achieved and internal policies incorporated.

Working within the constraints of these "legal" confines, the investigations proceeded on the Tamar. From the outset the financial value of this river, in terms of salmon fishing, was appreciated. However, throughout the studies priority was always given to the fish and fish stocks, concentrating specifically on the fisheries and conservation value of the species. Mr Johnston quoted a capital value of £50,000 for each salmon on the River Tay, Scotland. The capitalised value of salmon in the South West region has never been as high as that recorded in Scotland. Currently the capital value of salmon in the region would be approximately £4,000-£6,000 per fish.

With a clear focus and priority on the fish and fish stocks there was the need to maintain and, where possible, to improve the salmon and trout stocks of the Tamar.

Therefore the Public Inquiry obligations continue to be honoured and this has been the priority driver when compared to the need for full costs benefit analysis. However, in recent years an opportunity has been taken to examine the practicalities of long term mitigation cost reductions through the implementation of the Enhanced Flow Programme rather than continuing with artificial stocking of fish which does not give significant benefit to the biota and environment overall. The approach adopted of giving priority to the fish means that the fishing also benefits, even though indirectly.

Miss Stacey confirmed that South West Water has statutory obligations to provide for conservation, access and recreation on its land and water. Undertakings given at the Roadford Public Inquiry restricted over-exploitation of the site. Our policy on Roadford, as at other reservoir recreation sites, is that formally organised activities must be financially self-supporting. Informal recreation and access tend to be subsidised to some extent. In the case of Roadford, this investment could be justified in that the reservoir is a major focal point for tourists in the region. Prudent investment in tourism enables the water utility to increase its customer base, vitally important in an area where the resident population is virtually static.

South West Water works closely with other agencies and tourism providers and through its promotion and development of major sites across the region is regarded as a major player in the region's tourist industry.

The impoundment of Roadford Reservoir and its operation has resulted in a significant improvement in water quality downstream and does not cause non-compliance with use-related environmental quality standards. In the years prior to construction water quality in the river Wolf was generally consistent with Class 1B of the former NWC system of Water Quality Classification. This status has been maintained and exceeded in the years following construction. The River Wolf and Roadford Reservoir are extensively monitored to ensure compliance with the appropriate EC Directives, including the Freshwater Fish Directive. These Directives and other statutory needs justify the need to release the best water quality available to the river, and to control the water quality in the reservoir. Control of water quality is achieved through regular monitoring of reservoir, inflow streams and receiving waters together with the effective use of the multiple draw-off levels and the destratification equipment. Continuous water quality monitoring instrumentation are located on the draw-off tower and on the river immediately downstream of the dam.

Reservoir promotion is a difficult and stressful process, not only for the local people but the promoters also, therefore consultation at the earliest possible opportunity is essential. At South West Water the policy is to inform people of firm alternative sites as soon as possible.

There will follow a period of uncertainty - house prices and developers for example could well claim their investments "blighted" and local people will have genuine concern regarding the disruption and change to their lives that is being imposed.

The promoters will find a whole range of environmental issues arising at each of the site options. The cost and time of undertaking this work brings pressure to move quickly to confirm the favoured option. Once chosen, pressure groups will raise a whole range of different issues, some valid, others not, that need to be answered in the course of promotion and document preparation. It is likely that this will result in a new range of environmental monitoring programmes not originally considered . Also it is likely that to gain the support of local pressure groups the promoter will need to modify the original promotion and mitigation measures perceived appropriate at the outset in order for the proposal to proceed. Effective management of the consultation phase, including periods of iteration, is essential to progress and ensure a successful outcome to any proposal.

The promoter has to view any environmental monitoring programme as the "tool" to prove the original public statement on environmental impact as correct. Objectors will view the monitoring programme differently, using the data to argue that damage/change has occurred and requiring operation of the built asset to be limited. The environmental issues do not therefore disappear once promotion has succeeded and construction has been completed.

Moving to site-specific recreation and conservation issues, associated with dam promotions from the experience gained at Roadford and the earlier Colliford and Wimbleball dams, South West Water's policy will continue to be to promote consultation at the earliest possible stage and to bring together special interest groups and local community representatives in order to arrive at a consensus on acceptable levels of activity. Most importantly, a strategy for future development needs to be agreed and made public so that development can take place preferably in a phased way, to an agreed plan and with no surprises.

Post-implementation it is important to keep contact with local community and special interest groups so that they continue to "own" the initiatives.

Mr D J Knight (Sir Alexander Gibb & Partners)
Visual impact of development at Monkswood Reservoir
The paper by Messrs Elder, Fletcher and Rice on the environmental implications of constructing a concrete reservoir within a 100 year old dam shows the tank layout on Fig.1, page 76 of the Proceedings. Was any consideration given to breaking the straight planar face of the long eastern elevation of the spring holding tank for downstream effect, where it is in visual juxtaposition with the reservoir keeper's cottage?

The latter, being there first, and of mellowed traditional appearance, is a standing visual benchmark for the new construction, which is basically a large, plain wall, albeit faced with natural local stone, but topped with a conventional security fence (Fig.2, page 79). Paragraphs 9 and 10 refer, page 77.

Dr A D M Penman

Seepage at Monkswood Reservoir

Over 40 years ago I was involved in an investigation at Monkswood Reservoir into some leakage that was occurring at that time. Can the Authors please comment on the current situation at the dam in respect of seepage flows or remedial works carried out.

Messrs Elder, Fletcher and Rice replied that various alternative shapes for the holding tank were considered including a stepped alignment for the eastern elevation. The final design was selected for a number of reasons including simplicity of construction and hence cost. The straight face will be broken by the climbing shrubs which have now been planted. The steel balustrade on the top of the tank was selected to match similar installations on traditional properties in the St Catherine's valley.

Evidence of seepage was noted in September 1931 when part of the downstream slope of the dam subsided. Some grouting was carried out and appeared successful at the time. Subsequently seepage increased again through the 1930s and early 1940s until 1945 when a length of light sheet piling was driven into the core from the crest. The Building Research Station then investigated continuing seepage and in 1946 produced a comprehensive report concluding that permeable material was present in parts of the core (it is now thought that hydraulic fracture had occurred).

High piezometric levels and seepage have continued whenever the reservoir is within 1.5 m of top water level. In recent years high flows have been noted into the brick-lined outlet tunnel and grouting from within the tunnel was carried out immediately downstream of the core in 1990. Careful monitoring of the dam continues.

Mr R Y Gibson (Gibson (Civil Engineering) Ltd)

Cut-offs in permeable river bed deposits

Mr Findlay's paper gives a well balanced account of the conflicting concerns associated with reservoirs in semi-arid regions. Indeed the wide variety of problems can surely be matched only by an equivalent variety of ingenious local solutions.

In para 51 the principle of constructing cut-offs in permeable river bed deposits would seem to contain much merit.

Diligent use of seismic surveying would help to locate suitable sites and a "steps and stairs" sequence of construction up a suitable catchment would certainly slow down the drop in water table, thereby extending the soil moisture balance over a greater area, without interfering with the sediment transport.

Can the Author expand on the economics and applicability of this principle and indicate whether any such study has been embarked upon ?

Mr A P Fraser (Binnie & Partners)

Kalabagh Dam and Ghazi Barotha Hydropower Project

Anthony Fraser, referring to Mr Findlay's paper, wished to pick up on two points resulting from Binnie & Partners' experience in North Pakistan.

The first point was the importance of sediment handling to minimise environmental impacts. At the proposed 2400 MW multi-purpose Kalabagh Dam, where annual sediment load could exceed 300 million tons a year, the design incorporated a low level spillway to allow flushing of the reservoir at the commencement of the wet season. This allowed reduction of possible long term environmental impacts, including backwater and downstream effects. Sediment flushing in such semi-arid climates with seasonal rainfall could be of great value.

The second point was the importance of the involvement of the owner's and consultant's environmental team in a project from conception to post-implementation. In the on-going Ghazi Barotha Hydropower Project the critical decisions were the location and alignment of the three main elements; the barrage across the Indus, the 1,600 m^3/s power channel and the 1450 MW power station. The consultant's environmental specialists worked as an integral part of the project team, from the beginning of the feasibility study, to ensure that all stages decisions on location/alignment were environmentally optimum. The consultants were aided by the owner/World Bank establishing a panel of experts for environmental as well as technical matters. The interaction between the project team and all stake holders resulted in an environmentally and technically sound project.

Mr Findlay responded by stating that the suggestion of river bed cut-offs contained in Mr Gibson's question is a worthwhile principle and that he also highlights the impact that trapping sediment can have both on an impounding structure and on conditions downstream. In the case of NWFP there are two main obstacles to cut off solutions.

a. Cost - the general remoteness of sites, depth of permeable material (often in excess of 30 m even at favourable sites) and lack of suitable skills and equipment leads to cut off solutions being uneconomic. They also require the water to be raised from below ground for use.

b. Flows - the nature of what are often only flood flows is such that the run off passes through the valleys relatively quickly. Infiltration time is therefore limited and sub-surface barriers would not allow maximum use of the limited water as well as doing nothing to mitigate the destructive power of these torrents.

The answer would seem to lie in some form of low cost detention structure, built of local material and capable of withstanding overtopping. Such a structure would need to be easily repaired and would not necessarily be impermeable. Sedimentation would inevitably occur and in time surface storage would disappear. However, if a workable system of installing drainage manifolds in the reservoir solum at the construction stage could be developed then the trapped water might still be effectively utilised under gravity.

Mr Fraser's two points relating to experience in North Pakistan refer to particularly large schemes on which his company has been involved. Such schemes bring their own problems of scale but also permit effective measures for dealing with topics such as sediment flushing.

The use of low level sluices as described is indeed of great value. However, the scale of the proposed Kalabagh Dam justifies the expense of the installation and also ensures that the resources are available to operate and maintain the necessary infrastructure. The projects described in the Paper are very small scale by comparison and would not necessarily have permanent dam operations staff. Together with initial cost this may act against provision of sediment flushing on small schemes where yield is unreliable and there is no power generation potential.

Referring to the second point, the importance of environmental input from the earliest stage of a project is well noted. The project described by the Paper introduced the concept of multi-disciplinary teams to the Small Dams Directorate of NWFP. Those teams included sociologists and soil conservationists and they were involved at scheme reconnaissance stage to give a balanced view of the worth of potential schemes. Based on these initial assessments schemes were ranked for suitability for further study at pre-feasibility and feasibility level. All disciplines were involved throughout this process to provide an integrated social, environmental, agricultural and engineering perspective that is essential to the success of such schemes.

Mr J E Smith (Babtie Group)
Forestry clearance within reservoir basins
In many places around the world, new reservoirs inundate areas of forest. This often creates a serious problem for the owner, as considerable volumes of timber need to be cleared. Alternatively, a conscious decision to leave the trees to be drowned is sometimes taken.

If the latter approach is adopted, careful consideration of the risks to both the dam in terms of spillway blockage and to possible uses of the lake, such as fishing and transport need to be assessed. If the timber is cleared, problems can arise from inadequate removal of felled timber and die back of trees at the edge of the new lake. It is noted that at Pak Mun Barrage the area upstream appears to be densely forested down to the water's edge. It is also of note that a hydro-electric power station is included in the barrage and floods are controlled by gates. It will be of interest to hear from the Author what measures were adopted to overcome the potential problem of the dense forestation that will be flooded.

Dr M R H Dunstan (Malcolm Dunstan & Associates)
Use of roller compacted concrete at Pak Mun Barrage
A particular aspect of Pak Mun might be of interest to the Conference. As Dr Udayasen has explained the dam consisted of three main parts: the powerhouse, the spillway and the left-abutment dam. The latter also consisted of three parts; a spillway pier, the closure section across the diversion channel (designed for a flow of 5900 m^3/s) and the wing wall. The left-abutment dam was the first roller compacted concrete (RCC) dam in Thailand. It was also notable for the first use of Mae Moh flyash in a large concrete structure. Mae Moh flyash emanates from a thermal power station in north-west Thailand and was considered to be unsuitable for use in concrete. Nevertheless, in spite of having to be transported some 1000 km, it was found to be economic and an RCC was designed containing 58 kg/m^3 of Portland cement and 124 kg/m^3 of Mae Moh flyash. This achieved the specified characteristic strength of 10 MPa at an age of 91 days.

The left-abutment dam was constructed in four sections:

1. A trial section at the base of the wing wall.

2. The spillway pier which consisted of RCC placed within a previously-cast reinforced concrete "box" forming the walls of the pier.

3. The remainder of the wing wall, and finally

4. The closure dam across the diversion channel.

The closure dam was 26 m high and the RCC was placed in 22 days, a vertical rate of placement of over one metre a day. The speed of construction greatly reduced the risk of the coffer dam being overtopped and allowed the Contractor the demolish the coffer dam at an early stage.

Mr P D R Bond (European Investment Bank)
Economic evaluation of environmental effects
In general, environmental impacts, both benefits and disbenefits, may need to be addressed more systematically to allow improved understanding of the issues and so result in optimal rationalised solutions. Considerable advances have been made in evaluating environmental effects in economic terms and more emphasis on this quantitative approach would clearly assist in the project conception and development stages of dam projects. During the development of this scheme were the environmental effects quantified for example in economic terms and if so how was this done?

Mr Udayasen responded to Mr Smith's question by explaining that Pak Mun is a run-of-river project and hence the majority of impounding was within the river bank. Tree clearance was not therefore a problem although some minor felling was required locally.

In reply to Mr Bond, Mr Udayasen confirmed that the World Bank considered the economics of the project to be marginal, but that the decision to change the site location had eliminated the need to relocate 4,000 families.

Mr M B Liska (Vodohospodorska Vystavba, Slovakia)
Gabcikovo Project on the Danube
There was an extended contribution by Mr M B Liska on the design, construction and environmental safety of the Gabcikovo-Nagymaros Hydro Scheme on the Danube. Further details of this project are given in "Dams & Reservoirs, October 1994", the Journal of the British Dam Society. Some discussion followed.

Mr F M Law (Institute of Hydrology)
Dam safety in Slovakia
Who in your country is responsible for the overall safety of these major works built to harness the Danube, especially given the rapid completion of the final extension of the works ? Is the design organisation separate from any inspecting body ?

Mr Liska replied that his enterprise was responsible for the safety of hydraulic structures. They undertake the necessary measurements of deformations, seepage, etc. Monitoring of other values - ground water table levels, water quality, state of vegetation, soil humidity etc is performed by the Hydrometeorologic Institute. The quality of ground water from deeper aquifers is continuously done by the organisation responsible for fresh water supply.

The designing organisation, Hydroconsult, is a separate body. Based on its designs Mr Liksa's organisation orders the construction by a contractor or technical equipment by a supplier, who prepare detailed designs. They then supervise the construction, and verify the quality before paying. The speed of work must not (and did not) influence the quality of structures.

SESSION 3 **SAFETY OF CONCRETE AND MASONRY DAMS**

Chairman : Dr GP Sims

Technical Secretary : C Beak

1. Val de la Mare Dam, Jersey: Instrumentation, Monitoring and Stability Analysis of an ASR-Damaged Dam
   Horswill, Snowden, Weeks

2. Concrete Dams, Stability and the Role of Internal Drainage - Tai Tam Tuk, a Case Study
   Gallacher, Morris, Mann, Chan

3. Pitlochry Dam - the Use of Post-tensioned Ground Anchorages to Increase Stability
   Sandilands, Cameron, Bryce

4. Introduction to the CIRIA Engineering Guide to Concrete and Masonry Structures in the UK
   Reader, Kennard

5. Recent Investigations at Wimbleball Dam, Somerset
   Phillips, Shannon

6. Presentation by South West Water on Wimbleball Dam
   Whiter

SESSION 3 SAFETY OF CONCRETE AND MASONRY DAMS

Contributions and responses

Session 3A

F G Johnson (formerly North of Scotland Hydro-Electric Board and Mott MacDonald)
Question on Post Tensioned Anchors at Pitlochry Dam
Could the Authors comment why the significant reduction from initially assumed to actually recorded uplift (Fig. 2 compared to Fig 3 of Paper). Although taken into account in relation to the stability of the Gate House was not taken full advantage of in determining the number/strength of the required tendons. What redundancy was built into the design of tendons? Is it possible to accept the complete failure of one tendon, by relying on the redundancy built into the other tendons, to avoid replacement of the tendon? Are all tendons retestable and what retesting policy is envisaged for the future?

Dr G P Sims (Balfour Beatty Projects and Engineering Ltd)
Question on Post Tensioned Anchors at Pitlochry Dam
In paragraph 8 of the Paper the Authors say that the initial assumption of a linear distribution of uplift was simplified, not taking account of the effect of the grout curtain. Figure 3 shows a revised diagram, based on the data provided by two piezometers. Figure 3 shows an effect strongly reminiscent of the way foundation drains reduce uplift. The paper does not discuss seepage or drainage in any detail and it is not easy to assess the effectiveness of the grout curtain in controlling seepage or uplift. Would the Authors please describe in more detail the mechanism by which they consider the grout curtain reduces the uplift.

Broadly, there are three ways to improve the stability of a dam such as Pitlochry : improve drainage, provide additional downward force or downstream support measures. It appears that either of the first two might have been appropriate at Pitlochry. Would the Authors please describe their reasoning leading to the selection of rock anchors to provide additional downward force rather than by improving the drainage.

Has provision been made for the anchors to be monitored and if necessary restressed?

Would the Authors confirm that their assessment of the concrete tensile stress was not made using finite element analysis? It appears that their analysis did not consider the presence of and effects of cracking in the region where the tensile stress was estimated to be 0.43 MPa. Would the Authors care to speculate on what the effect on their assessment would have been had cracking been considered. Would they also indicate their justification for an allowable tensile stress of 0.07 MPa?

R Freer *(Reed Publishing)*
Question on Post Tensioned Achors at Pitlochry Dam
The Authors said they had organised the Contract so that the work was done through the winter to minimise the inconvenience to summer visitors.

Did the Authors use this opportunity to keep a record of the time lost on this Contract by bad weather and short working days so that the client would have an estimate of any extra time and cost incurred by doing the work in winter rather than in summer?

A Macdonald (Babtie Shaw & Morton)

Glendevon Dam

The Proceedings of the Conference describe alternative means of stabilising gravity dams, namely by post-tensioning as at Pitlochry, and by the installation of pressure relief measures as at Tai Tam Tuke. Remedial measures have recently been completed to the 48 m high Upper Glendevon concrete gravity dam in Scotland, the solution adopted at that site being to use a rockfill stabilising embankment on the downstream side. Its construction is detailed in Question 68, Report 69, of the ICOLD Congress in Durban.

During the works efforts were made to reduce leakage through the existing monolith joints by reheating the bitumen plug waterstops set within the body of the dam some 2.135m from the upstream face. These plugs are set behind a 584 mm wide, No. 16 gauge copper waterstop, 450 mm from the upstream face.

The bitumen plugs were installed in 1959, about four years after completion of the dam, in an effort to reduce high seepage flows. The hole for the bitumen was formed by drilling a 244 mm diameter holed down the centre of the joint. Records indicate that the borehole was heated to 107°C prior to filling with bitumen. 25 mm diameter "go" and "return" steam pipes were installed in each hole and left in position. The bitumen was straight run, penetration *60-70, with a softening point 46°C to 55°C, supplied hot at 143°C.*

In 1993, heating trials of the bitumen were undertaken using the *in situ* steam pipes and a model 15E boiler manufactured by Fulton Boiler Works (GB) Ltd. The boiler and tanks were trailer mounted for ease of positioning between monoliths. The boiler had an output of around 240kg/hr at 100°C. Steam was circulated through the monolith seal pipes at 130°C. Refluidising of the bitumen took around two hours and the process was continued for up to twenty-four hours on each joint to allow time for the material to flow into the joint. Water depth in the reservoir during the trials was around 5.7 m below spillweir.

The depth to the top of the bitumen was recorded before and after heating. No change was noted in any of the joints, indicating no bitumen intake. There was no immediate perceptible change in seepage flows through the joints, which remained fairly low. However, with the reservoir now back up to top water level, higher flows are being recorded in some joints where heating trials were not undertaken, possibly indicating some beneficial effects of the bitumen reheating on the joints which had the highest seepage flows historically.

Crawford T Munro (EPD Consultants Ltd)

Meldon Dam - Monolith Joint Leakage Investigations

Following reference to attempts to stop monolith joint leakage at Upper Glendevon Dam, Mr C T Munro (EPD Consultants Limited) referred to recent investigations into an issue of water at a monolith joint at Meldon Dam in Devon. This 44 m high mass concrete gravity dam comprises sixteen monoliths. The joints are sealed with rubber waterstops and a "paracore" plug. The plug features heating pipes for the original installation and for future reheating if necessary.

An Inspecting Engineer's Report (Dr G P Sims of EPD Consultants Limited) recommended in the interests of safety that the cause of the leak should be investigated and appropriate remedial action taken..

Initially, monitoring data were analysed. This concluded that the adjacent monliths were stable although the piezometric level in the area immediately downstream was close to ground level. Pressure relief wells on each side of the joint had been proved clear. One discharged the greatest flow of all wells whilst the other had no discharge. The borehole and grouting logs did not indicate any peculiarity or noteworthy feature.

Chemical analysis of water samples was undertaken. The purpose of this was to identify the source of the leakage. Samples were taken of water from four sources : the reservoir, pressure relief well discharge, an unlined vertical drain in the dam, and an observation well containing groundwater. The analysis revealed that the water seeping through the monolith joint was similar in composition to the pressure relief well discharge. This suggests that the source of the seepage through the joint was from the rock below the dam foundation and did not pass the upstream waterbar.

Provision will be made to measure the discharge regularly, which issues variably between 1.5 to 3.0 metres above the path at the downstream toe. Previous examinations have failed to determine correlation of the flow with other data.

John Hopkins (South West Water)
Argal Dam - Structural Investigations
Argal Reservoir, situated near Falmouth in Cornwall, is formed by a concrete gravity dam.

It was originally constructed in 1940 when it impounded 668 Ml. It was raised in 1960/61 by 3 m to store 1350 Ml. The dam is 142 m long, 13 m high and curved in plan. The raised dam was anchored by 47 no. 200 ton capacity post tensioned rock anchors. Each anchor was formed by 102 no. 5 mm diameter high tensile steel wires grouted into solid rock beneath the dam.

In his report of an inspection in 1991, Peter Tye recommended that the performance and condition of the anchors be investigated and recommendations made. Also, that the stability of the dam be re-assessed in relation to the seismic risk.

Investigations have been carried out by South West Water in conjunction with *Rofe, Kennard & Lapworth, David Evans, Prof Littlejohn and BRE. These have included forced* vibration tests and the installation of electro-levels in two boreholes to monitor the performance of the dam. A precise survey system is also being implemented.

An evaluation is currently being carried out to investigate methods for long term stability improvements to the dam. These include replacement anchors, concrete gravity buttress or a rockfill berm. A modest raising of the dam is also being considered.

Dr Paul Tedd (Building Research Establishment)
Instrumentation of Argal Dam - 1994
The Building Research Establishment at the request of the South West Water have installed electro-levels in Argal Dam to monitor tilt of the structure in the upstream/downstream direction. Two strings of electro-levels were installed in inclinometer tubes grouted into vertical boreholes drilled through the dam into the granite foundation. Vibrating wire piezometers were placed in sandcells in the lower part of the boreholes to measure water pressures in the foundation. Electro-levels were also fixed on to brackets at a number of locations in the valve tower. Temperatures are also being monitored in the boreholes and in the valve tower.

Readings from the electro levels are being recorded daily with a battery powered logging system. Data is downloaded on to a portable PC. Satisfactory observations have been obtained from the instrumentation to date which was only installed in August 1994.

N M Sandilands, A T Cameron and W M Bryce (Scottish Hydro-Electric plc)
Answers to Questions on Post Tensioning Ground Anchors at Pitlochry

1 R Freer

As mentioned in the question, the most important factor to the client was minimising disruption to tourists during summer. At the planning stage it was accepted that if this resulted in a small premium due to winter working this would be acceptable. However, the variation between the returned Tenders leads us to believe that commercial considerations were more significant than seasonal in determining a Tender price.

As the Contract progressed it became obvious that the colder than average winter we were experiencing was having some effect on the Contractor's cost and programme, although this was not formally recorded. The most significant disruption was water and diesel freezing due to the low temperatures. This slowed the drilling operations, but was offset by increased productivity during clear spells and overtime working. As the site covered only a small area, floodlighting was provided to allow a full working shift during darkness. We have concluded that a similar Contract carried out during summer would have finished maybe a week or two earlier than the actual duration of twenty-two weeks. Also the Contractor would not have had the additional expense of floodlighting and overtime.

2 F G Johnson

Studies showed that the beneficial effect of reduced uplift only had the capability of solving the stability problem at the left-hand tower. Elsewhere, additional means were necessary. Accordingly, by taking measurements under the tower it was demonstrated that no additional means were required there. Where tendons were required the additional cost of designing for linear uplift was minimal and it was deemed sensible to do so.

Normally accepted safety factors were applied during the tendon design. These included considering a probable maximum event and utilising an allowable tendon force of 50% of ultimate capacity. No additional redundancy was intentionally included, although some exists due to tendon sizes being the nearest to, but exceeding that, required.

The most likely cause of failure of tendons after installation and testing would be corrosion. This was guarded against by the specification for the protective sheath and careful detailing of the anchor head. The possibility of one tendon failing due to long term effects while the other remain unaffected was not considered realistic, and the emphasis was placed on overall prevention of failure.

All of the tendons are re-stressable by the provision of threaded head blocks and it is envisaged that lift-off tests will be routinely performed in future years. The programme has yet to be finalised but is likely to be a sampling procedure on a three-year cycle.

3 G P Sims

The linear distribution assumed the dam was founded on homogeneous material, ie, the reduction in uplift pressure would be proportional to the distance from the upstream face. The grout curtain forms a vertical plane near the upstream face. It alters the foundation material around it by decreasing its permeability, causing a relative increase in head loss of the seepage flow through this zone. This leads to the sharp reduction in uplift pressure near the upstream face.

The options mentioned for increasing stability were considered before deciding on anchoring. It was concluded that in this situation anchors would be the most suitable as they could be installed relatively cheaply and easily, with the minimum of disturbance. The other options had the following disadvantages associated with them:

Drainage	:	On its own this would not be sufficient.
Downstream support	:	If this were of loose fill it would be at risk during overtopping. If structures such as buttresses were to be used, major civil works (excavations, falsework etc) would be required. The restricted space on the south side due to the power station and fish pass precluded this.

We can confirm that the anchors are restressable as described above.

Analyses were carried out assuming elastic behaviour. The results before anchoring showed high tensile stresses (0.43 MPa) in the upstream face, with a consequent risk of cracking. This was unacceptable so no further non-linear analysis was required. An anchor force was then chosen to almost elmininate this tension, ensuring an essentially elastic behaviour. The analysis of the case after installation therefore did not require consideration of cracking. The value of the small tension allowed was set at an empirical 10 psi (0.07 MPa). This figure has been widely used in the past and is judged to be such as would not cause cracking.

The following Table was omitted from the original Paper:

TABLE 4 - CONCRETE PROPERTIES

Property	Value
Compressive Strength	47 - 57 N/mm^2
Density	2420 kg/m^3
Indirect Tensile Strength	4.0 N/mm^2
Coefficient of Thermal Expansion	8.1 x 10^{-6}/°C

Session 3B

Haylor Lass (South West Water)
Relief Well Flows at Wimbleball Dam
In answer to a question raised about whether there was any evidence for either erosion of fine particles from fissures or reduction in durability of the grout curtain, some water samples have been taken from the relief wells over the years. No particles of clay, rock or cement have been found and the chemical composition is so closely identical to reservoir water that further sampling is not considered worthwhile.

Wimbleball is a well-instrumented dam but highlights the need for regular calibration of all instruments. There is no extant record of initial calibration for the south side drainage V-notch recorder: when checked in early 1986 it had substantial zero error. In conjunction with hearsay evidence from earlier investigations, I consider that flows since 1982 or 1983 have not been less than about 4 to 5 litres/sec. The apparent steady increase shown in Fig 2 between 1982 and 1986 may be misleading, although the flows have undeniably increased since first filling.

I do not find the high flows into Manhole 3 particularly surprising. Both the relief wells R13/5 and R13/6 were drilled inclined upstream; a plot of their position shows that their lower ends are within six or seven metres of the base of the grout curtain and this in an area of a shear zone of considerably fissured rock. The indicated rock mass permeability does not require grout curtain "windows" to explain the flows, although there may have been some removal of clay from fissures. In terms of the 'nuisance value' the flow from the south side drainage is less than 10% of the total leakage from the dam which approaches the compensation water requirement.

Bruce and George (Reference given in the Proceedings) explained the need for the additional south side grout curtain because of the increase in south side and spring flows on first filling, projected to reach 114 litres/sec at reservoir full level. They report a reduction in flows over the nine-month period of additional grouting, sufficiently marked to curtail any further grouting. *I have been unable to trace the flow records for the two years or so following this grouting* but would find them most interesting, particularly as the total south side flows have reached some 100 litres/sec at reservoir full level.

Owen P Williams (The National Grid Company plc)
Question on Recent Investigations at Wimbleball Dam
I refer to the paper by Messrs Phillips and Shannon and would like to illustrate from progress photographs aspects of the foundation problems experienced whilst on the site staff of RK&L at Wimbleball from 1974 to 1980.

Firstly, the right/north abutment where satisfactory rock quality was generally encountered at predicted and reasonable depths, as shown on Figure 1. However, bedding was steep and unexpectedly uniform to the valley floor. In conjunction with a shear zone only some 300 mm wide, much of the downstream foundation area of B2 to 6 was unstable, removed and replaced with mass concrete.

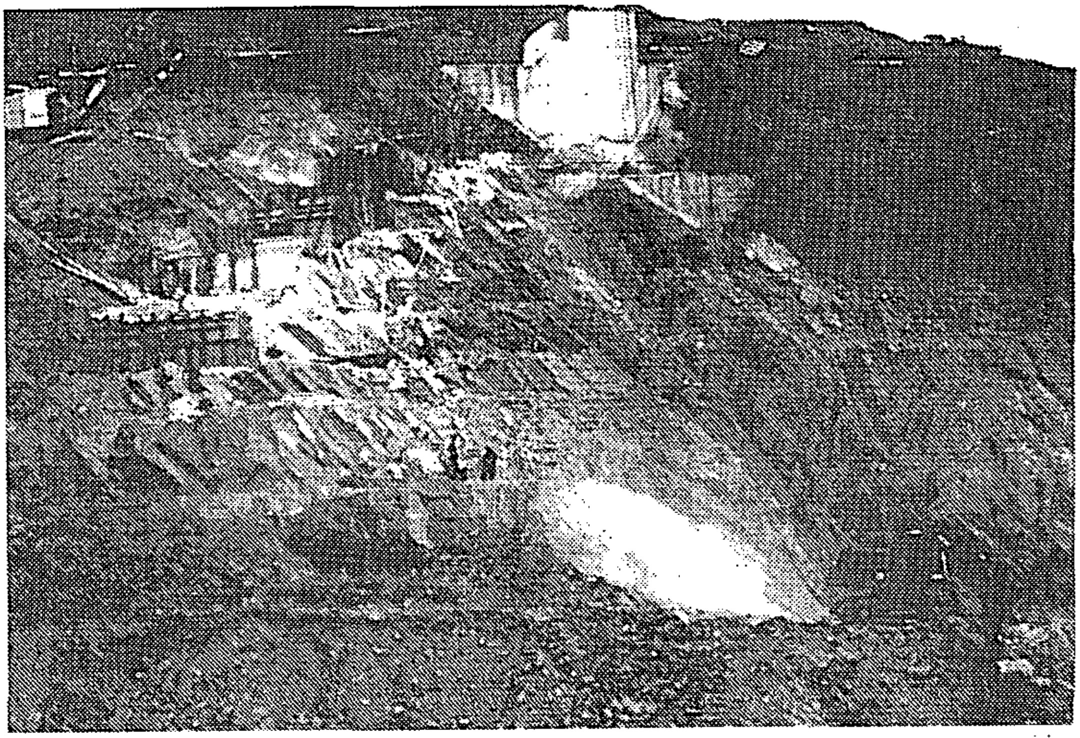


FIG 1 North Bank Excavation & Spillway Construction

Secondly, on the left/south abutment rock quality was not as good as expected nor quite as good as judged necessary at predicted depths and it did not rapidly get better with depth. Foundations of B10/11/12 were much affected by shear zones which were excavated and mass concreted. B13/14/15/16 and the abutment further south were understood to be more influenced by periglacial activity opening wide and deep fissures sub-parallel to the valley side, but which failed to overcome the component of dip into the abutment. This deeply jointed zone of otherwise relatively unweathered rock therefore failed to be eroded to the valley floor for further erosion. *Figure 2 typical fissures in the upstream foundation of B15 and 16.*

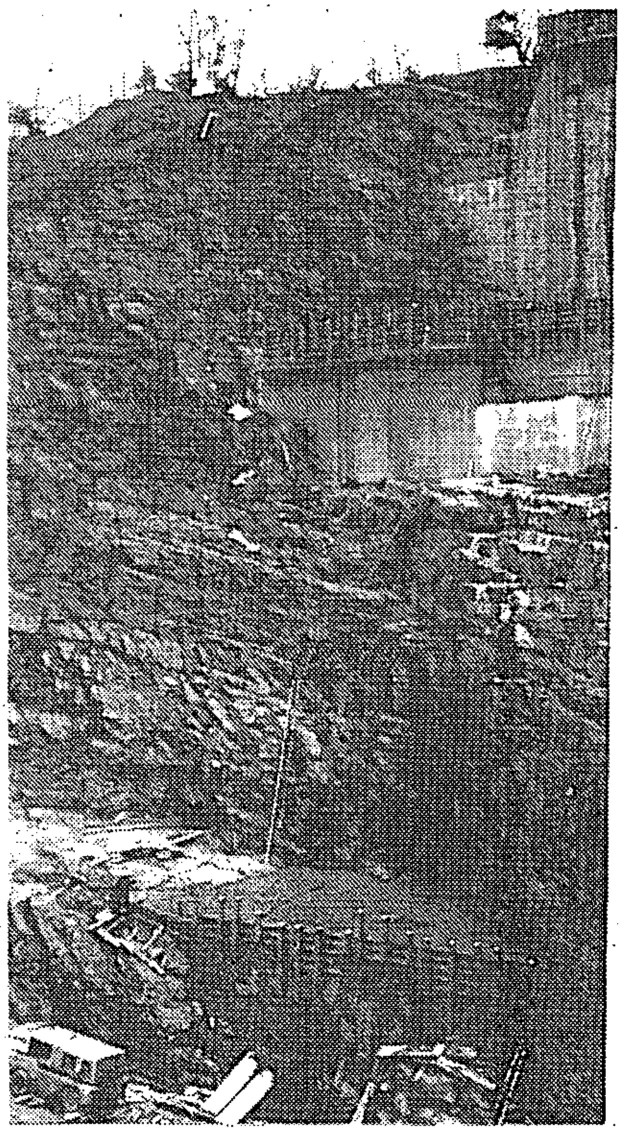


FIG 2 Buttress Nos 14 & 16, Footing Construction

With reference to paragraph 20 of the paper and because of such fissures, I do not share the author's surprise at the significant travel of curtain grout, including to the foundation relief wells, some of which are only about 10 m from the nominal curtain line.

With reference to paragraph 26 of the paper, I understood cement grouting was generally for life and so am concerned at the conclusion that reduction of grout durability, presumably by erosion or solution, is more likely to be the cause of increasing leakage in B13 foundation. I would welcome the results of any filtration and/or chemical analyses of the relief well flows which confirmed this. The flow net described for B13 would be of particular interest and its publication in addition to Figure 5 would be welcomed.

Dr G P Sims (Balfour Beatty Projects and Engineering Ltd)
Question on Recent Investigations at Wimbleball Dam
It would be interesting to know a little more about the regional geology in the area of Wimbleball Dam. Many of the joints in the foundation rock appear to be parallel, running NW; an apparently major exception is the joint running SE near buttress 13. An understanding of the regional geology might illuminate the reason for the continually increasing seepage under the south flank of the dam.

Would the Authors care to offer a comment, with the benefit of hindsight, of the appropriateness of the buttress design for a dam on this foundation?

P Phillips (Babtie Shaw & Morton) and **F Shannon** (Babtie Geotechnical)
Replies to Questions on Recent Investigations at Wimbleball Dam
In response to Mr Lass's contribution, we concur that the exact increase in Pressure Relief Well (PRW) flows at buttress No. 13 is indeterminate due to problems of instrument calibration. However, they have undoubtedly risen by at least a factor of two and present indications of flow rates show this factor to be now greater than two. Despite the proximity of PRWs to the grout curtain a reason for this increasing trend in foundation seepage must be sought and a progressively deteriorating grout curtain or section of curtain, ie, window, is a potential scenario. Another scenario would be progressive removal of clay from fissures or crush zones which were not penetrated by the grouting.

In answer to Mr O P Williams' contribution about the grouting, we comment as follows:

The significant travel referred to in paragraph 20 of the paper was commented on by Bruce and George in their paper entitled "Rock Grouting at Wimbleball Dam 1982" (reference listed in paper) where grout injected at buttress 16 location was observed in the tailbay channel some 130 m away. This considerable distance travelled was during the additional grouting to reinforce the existing grout curtain using 25% higher pressures. We would not have expected the grout to migrate more than 15-20 m from its point of injection.

Numerous papers make comment on grout durability, notably "Design of Cement Based Grouts" by G S Littlejohn 1982 where it is stated that "Cement grouts are durable under most normal conditions, but deterioration may be caused by abnormal environmental conditions, especially where there are deficiencies in grout quality, eg, low density and high permeability. The permeability of a mature cement grout is related, like its strength, to its original water content".

Deere in his paper entitled "Cement - Bentonite Grouting for Dams" reports that "A grout slurry is essentially an unstable water - cement suspension. The degree of sedimentation depends on the W : C ratio. Without the addition of bentonite a 3 : 1 (W : C) ratio mix by volume has a sedimentation of 53%, a 2 : 1 (W : C) ratio mix - 26% and a 1 : 1 mix of only 5%".

Houlsby (1982) states that a 2 : 1 starting mix by volume is normal, rapidly thickening up to 1 : 1 mix and states that a 4 : 1 mix or thinner is not a durable one. At Wimbleball Dam the grouting commenced with a 5 : 1 mix and it is estimated that up to 30% of the injected grout could be regarded as non-durable.

Mr H Lass's comments on the chemical analysis of water samples taken from relief wells may answer the query on evidence of erosion or solution of the grout within the grout curtain. He also suggests that the increases in south relief well flows may not be as great as indicated. Nevertheless there has been a significant increase in the flow rate in recent years and this trend has to be explained. Table 1 of the paper shows a lower grout consumption at buttress 13 coinciding with the increasing PRW flows. This association leads on to the conclusion set out at the end of paragraph 26 regarding the potential for a reduction in grout durability at this location.

A reply to Dr Sims' questions is given below:

Two main aspects of the structural geology in the vicinity of the Devon/Somerset border are the extensive folding to which the region was subjected by tectonic forces some 290 million years ago. These compressive forces, in a N-S direction, resulted in fold axes trending east-west with associated thrusts and crush zones. Indeed, many of the principal crush zones observed at Wimbleball Dam foundation trend approximately east-west.

Later in geological time a series of wrench faults cut across the region in a NW direction (the best known such fault being the Sticklepath Fault passing through Bovey Tracey). The recorded crush zone at the upstream end of buttress 13 is orientated in a NW direction and could have been created by a different mode to other crush zones. It has been postulated by J Knill (reference given in paper) that poorer rock conditions revealed by the seismic survey extending from about buttress 11 towards the south gravity block could indicate a major fault zone of fractured and shattered rock. This potential fault zone could extend by as much as 200 m in width and would, in part, coincide with the location of the additional grouting (undertaken from buttress 16 southwards). Further evidence of such an extensive fault zone is drawn from the depressed groundwater conditions on the south flank measured prior to construction. J Knill also reports that the examination of aerial photographs suggested that two predominant faults could be present on the south flank about 150 m apart.

The above comments are derived from reports written on the geotechnical considerations of the design with respect of Clause 12 claim deliberations after completion of the construction. As the Authors' employer, Babtie Group, was not involved in the design of the dam, which was undertaken by Rofe Kennard & Lapworth, we do not wish to comment extensively on the suitability of the design of a buttress dam at the Wimbleball site. However, we see no reason why with an adequate site investigation undertaken ahead of design work that the unforeseen ground conditions should have affected the selection of this particular type of dam. We understand that planning and aesthetic requirements also influenced the choice of a buttress gravity dam.

Dr J H Martin (Scottish Hydro-Electric plc)
Comment on R A Reader's Paper on the CIRIA Guide
I would like to support a point just made in Mr Reader's presentation of his paper on the CIRIA Guide.

The 'Engineering Guide to the Safety of Concrete and Masonry Dam Structures in the UK', like much of British dam safety activity, was both stimulated and executed by principled engineers who freely gave of their own time. That applies as much to the members of the Guide's steering committee as it does to the Contractor - Rofe, Kennard & Lapworth. It is a tribute to RKL that Michael Kennard, Chris Owens and Dick Reader did not stint on quality or manhours despite the low fixed fee and the large amount of external steering they received. This raises the question:

> Should UK dam safety be dependent upon the discretionary time of a small caucus of experienced enthusiasts?

Before concluding 'no' it is worth considering the achievements of this approach.

Firstly, the concept of indivisible and unassignable personal responsibility for Inspecting Engineers is unique to UK reservoir safety legislation and its overseas derivatives. It serves us well. I submit that such clear recognition of the need for balanced engineering judgement is a direct result of our dependence on principled, experienced enthusiasts. Highly formalised processes amongst engineers or any other professionals rarely produce such straightforward clarity or recognise uncodifiable judgement.

Secondly, dam safety in UK is by and large implemented. Missions to other advanced countries repeatedly show thorough but prescriptive systems partially implemented or limited only to a subset of reservoirs. The large number of dams inspected under the UK Reservoirs Act 1975 means that reservoir engineers cannot withdraw into narrow specialisations so common elsewhere. Thus the professional well informed generalist both feeds and needs an enthusiastic caucus to exchange experiences and to advance practice.

Thus the approach seems worth continuing but the key words 'dependent upon the discretionary time' have not been addressed.

Many reservoirs in this country are owned by public limited companies, be they water undertakings or electricity businesses like Scottish Hydro-Electric. Our task is to manage the range of hazards which our markets and our engineering installations carry. I believe that the independent audits provided by Reservoir Supervising and Inspecting Engineers are probably the most cost effective of all those we engage in our business. However in the long run you get what you pay for. The fees for Reservoir Inspection must be dependent upon discretionary time and enthusiasm, cross subsidy from other fee earning work or are vulnerable to undercutting and hedged recommendations. The end point of such vulnerabilities is litigation which does neither our profession nor reservoir safety any service.

With the Government's policy of passing more reservoir safety matters to the Institution of Civil Engineers we have the opportutnity to set UK reservoir safety practice on a sustainable footing. The time is also right to promote what we believe in to Europe, where a more prescriptive approach is preferred except apparently in France.

If we are not to rely on the energies and financial means of the few then the reservoir safety caucus will need funding. The Government is an important source, especially for generic research and international action. However, the caucus is best sustained by fixing an appropriate and uniform hourly rate for the entire Reservoir Inspection monopoly whilst vigorously upholding standards.

In addition to correctly funding the Inspections themselves, the Institution needs some funding properly to undertake its new responsibilities. In Spain the 'Colegio', equivalent to the ICE, levies a charge for its stamp on civil construction drawings. The stamp need not signify approval but legislation makes it necessary before a structure can be built. It guarantees an *income to the Colegio proportional to the level of construction activity. For each UK Reservoir* Inspection the ICE stamp could be an unavoidable levy, explicitly passed on to reservoir owners.

Part of the ICE's support to the Inspecting Engineers should be to help fund sufficient research to keep their judgements well informed ones. This will generally mean research into ageing, not new dams. Centralised funding of reservoir safety research is mainly limited to Government and CIRIA. Certain owners and consulting engineers also share their results through fora such as BDS. However, the level of research is barely adequate to address the subject. For example, the research underlying the current revision to the reservoir Flood Guide is funded at such a low level that its authority must be questioned. In view of the cost of works modifications which could result from revisions to this Guide it is important that the research basis can withstand scrutiny. The levy from an ICE stamp on Inspection Engineers' reports could contribute towards this research but could never and should not totally replace Government research funding.

SESSION 4 RESERVOIRS AND NATURAL HAZARDS

Chairman (Seismic): J Martin

Technical Secretary : Dr. JA Charles

1. The Seismic Behaviour of Gravity Dams in Areas of Low Seismicity
   Taylor, Daniell, Mir, Simic, Hinks

2. Analysis Methodologies for Rigorous and Approximate Seismic Assessment of Dams.
   Robertson, Granshoab, Hinks

3. The Seismic Behaviour and Design of Reservoir Intake Towers
   Daniell, Taylor

Chairman (Floods, waves and contigency planning) : WJ Carlyle

Technical Secretary : Dr JA Charles

4. Rapid Hazard Hazard Ranking for Large Dams
   Thompson, Clark

5. Contingency Planning for Dam Failure
   Claydon, Walker, Bulmer

6. Inundation Mapping for Dam Failure - Lessons from UK Experience
   Tarrant, Hopkins, Bartlett

7. A Practical Appraisal of the Overtopping of Embankment Dams
   Hughes, Hoskins

8. Maximum Reservoir Water Levels
   Anderson, Dwyer, Reed, Nadarajah, Tawn

9. The Construction and Performance of a Wedge Block Spillway at Brushes Clough Reservoir
   Baker, Gardiner

10. Implementing Safety Recommendations on UK Dams - A Supervising Engineer's View
    Hay

SESSION 4 RESERVOIRS AND NATURAL HAZARDS

Contributions and responses

Session 4(a) SEISMIC HAZARD

Mr J L Hinks (Sir William Halcrow & Partners)
Seismic analysis of Claerwen dam
There is much experience of earthquakes acting on concrete and masonry dams worldwide but little experience in the United Kingdom. Some damage was, however, caused to the Blackbrook Dam near Loughborough by a magnitude 5.3 event in 1957. Coping stones weighing 0.75 tonnes were dislodged and cracks appeared in the drainage gallery and on the faces of the dam. Events of magnitude 5.3 are not uncommon in the UK and it is salutary to recall that an event of comparable magnitude in an area of low seismicity in Australia caused 11 fatalities and damage estimated at over ú500 million in Newcastle in 1989.

Following publication of the BRE Guide to seismic risk to dams in the United Kingdom, Welsh Water engaged Sir William Halcrow & Partners and Bristol University to carry out a seismic study for the 56 metre high Claerwen Dam near Rhyader in Central Wales. Claerwen is a concrete gravity dam with a masonry facing on the downstream face. The site is in seismic Zone B and the dam is classified as a Category IV structure using the ICOLD method of classification adopted in the BRE Guide. The recommended Safety Evaluation Earthquake has a peak horizontal ground acceleration of 0.3g.

A number of modes of failure of the dam were considered:
1. Shearing of the dam as a result of reactivation of a clay filled fault running beneath Block 9. This fault, which is believed to be about 380 million years old was closely investigated when the dam was constructed between 1946 and 1952. An opinion was recently obtained from the British Geological Survey who believe that the fault is unlikely to provide the focus of an earthquake. The possibility of reactivation of the fault in an earthquake cannot be completely ruled out but is thought to be remote.

2. Horizontal cracking on the upstream and downstream faces of the dam. This is the mode of failure to which most attention was paid in the study. Figure 1 shows the peak tensile stresses computed using EAGD-84 for a rock modulus of 22 GPa. (The peak tensile stresses are lower for a rock modulus of 10 GPa). The actual rock modulus at Claerwen is not known but measurements of the fundamental natural frequency of the dam suggest that 22 GPa is probably a reasonable approximation. Table 1 shows the peak tensile stresses computed at various levels on the upstream face of the dam compared with the allowable stresses. The allowable stresses at each level are computed according to the recommendations of USBR Monograph 19 whereby the allowable stress is equal to the apparent tensile strength of the concrete at lift joints minus 40% of the hydrostatic water pressure. It will be seen that the minimum factor of safety against cracking is 1.07 and occurs at the heel of the dam. This was considered satisfactory.

3. Crushing on the faces of the dam. The highest compressive stress calculated on the faces of the dam was 3.6 MPa which compares with an average compressive strength of 16.5 MPa. It was concluded that crushing of concrete in the body of the dam is unlikely to take place under seismic loading.

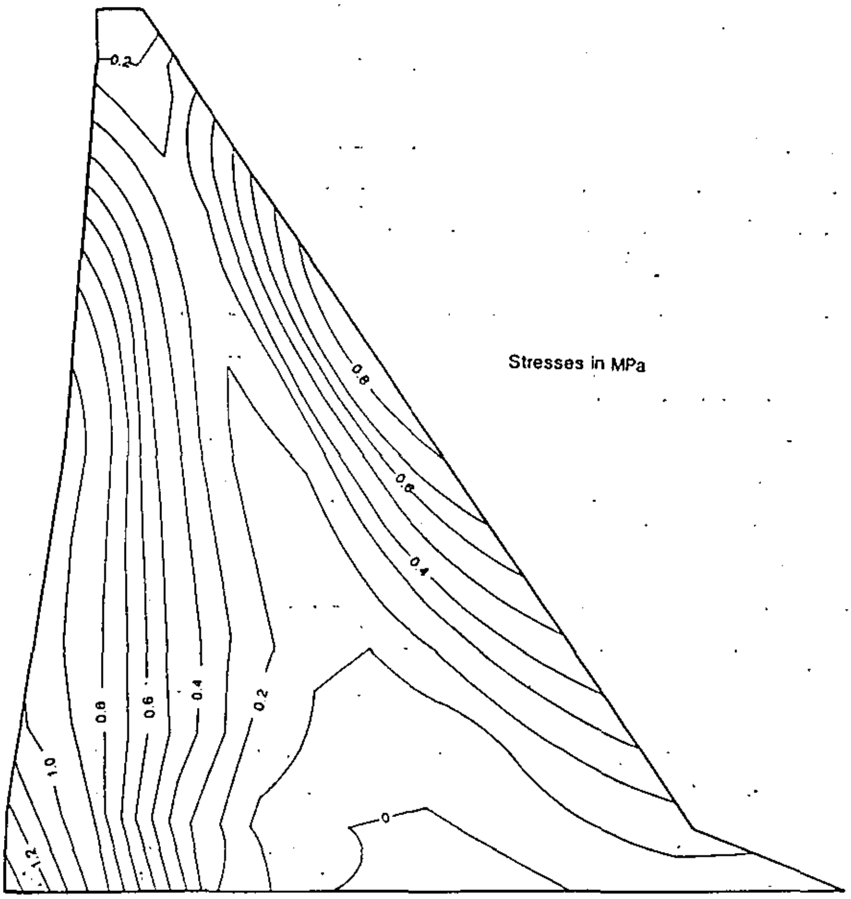


Figure 1 Peak tensile principal stresses for rock modulus of 22 GPa and peak ground acceleration of 0.3g

4. Failure by sliding. It was found that the cohesion between dam and foundation would need to be less than 3.0 MPa for there to be any danger of rupture in the Safety Evaluation Earthquake. Cohesion was expected to be at least 6.5 MPa under dynamic loading so sliding was not expected to be a problem.

5. Displacement of crest bridges, pilasters and balustrades would not affect the safety of the dam itself. It was estimated that a seismic event with a return period of about 3000 years would be needed to move the crest bridges on their supports.

6. Out of phase vibrations of monoliths. In an earthquake there would be likely to be a phase difference between the vibrations of adjacent monoliths. Relative movement between monoliths could, however, take place only if there were to be shearing of the shear keys between blocks. If this were to happen there could be tearing of the 3 mm thick copper waterstops. This should not endanger the stability of the dam but it could necessitate remedial works such as the installation of bolt-on seals on the upstream face of the dam.

Elevation mAOD	Apparent Tensile Strength at Lift Joints MPa	Deduct for Water Pressure MPa	Allowable Tension MPa	Computed Tension MPa	Factor of Safety
368.8	1.79	0.00	1.79	0.0	*
365.0	1.79	0.02	1.77	0.15	*
360.0	1.79	0.04	1.75	0.40	4.38
355.0	1.79	0.06	1.73	0.68	2.54
350.0	1.79	0.08	1.71	0.87	1.97
345.0	1.79	0.10	1.69	0.98	1.72
340.0	1.79	0.12	1.67	1.03	1.62
335.0	1.79	0.14	1.65	1.01	1.63
330.0	1.79	0.16	1.63	1.01	1.61
325.0	1.79	0.18	1.61	1.05	1.53
320.0	1.79	0.20	1.59	1.18	1.35
315.0	1.79	0.22	1.57	1.38	1.14
312.8	1.79	0.22	1.57	1.47	1.07

Table 1 Maximum tensile stresses on the upstream face calculated by EAGD-84 for the critical earthquake and a rock modulus of 22 GPa.

7. Overtopping caused by seismic seiches or by a landslide into the reservoir. A classic case of a seismic seiche in a reservoir was that at the Hebgen dam in Montana in 1959. This was, however, associated with an earthquake of magnitude 7.5 to 7.8 which is considerably larger than any earthquake expected in the UK. The slopes around Claerwen dam are fairly gentle making it unlikely that there would be any large and fast moving landslide into the reservoir such as happened at Vaiont in 1963.

8. Fracture of draw-off or scour pipes. The maximum deflection of the dam crest under seismic loading was computed to be 10 mm. The scour pipes are of steel plate lined with spun concrete and are, of course, at a very low level in the dam where deflections would be small. We did not think that they would be seriously at risk.

We are grateful to Welsh Water for the opportunity to study the response of a large UK dam to seismic loading and for giving permission for us to report our findings to this conference. The conclusion of the work was that this dam would be able to withstand the Safety Evaluation Earthquake without risk of failure. Some damage might, however, affect the crest bridges and balustrades.

Mr T A Johnston (Babtie Shaw & Morton)
In presenting the papers on analysis methodologies (Robertson et al, Proceedings pp 271-281) and seismic behaviour of gravity dams (Taylor et al, Proceedings pp 292-305) Mr Hinks showed tables of tensile strengths at different levels in concrete gravity dams. The starting point for the assessment was the apparent tensile strength in lift joints. Can the authors explain why the term "apparent" is used? Do we know that concrete has a higher tensile strength under seismic loads than under static loads? Alternatively is the adoption of the apparent tensile strength related to the method of analysis?

Mr J L Hinks replied that for a given strain above the elastic limit, the finite element calculations will predict higher stresses in the concrete than can actually occur. To compensate for this a higher permissible tensile strength may be specified for comparison against the stresses predicted from the linear elastic analysis (Raphael, 1984).

Dr Penman drew attention to the One Day Symposium on Earthquakes held on 20 May 1987 at the time of the ICOLD Executive Meeting in Beijing. The morning was devoted to concrete dams and the afternoon to embankment dams. Pierre Londe, chairman for the first session, said "concrete dams seem to be extremely resistant, judging from past experience. The world statistics yield only three cases of damage to concrete dams, but all three were not failures and the dams were safely repaired." Liam Finn, chairman of the second session, said, but did not put in writing in his published remarks, that modern well designed embankment dams were equally resistant to earthquake and none had failed so as to allow the release of the reservoir. It would seem that only dams on liquefiable foundations, or old ones built of hydraulic fill, suffered collapse due to earthquakes. He asked the authors for their opinions about these statements.

As a second point, Dr Penman asked about the artificial excitation of tailings dams. These were a type of dam extremely susceptible to earthquake damage. Would it be possible to determine strength properties from artificial excitation of the type used by Professor Severn on the Llyn Brianne embankment dam and elsewhere? The response of a tailings dam to shaking, ie build up of pore pressures etc, might give important information about the response to be expected when subjected to earthquake shock.

Mr J L Hinks replied that a number of concrete dams had suffered serious damage (eg Koyna, Sefid Rud, Hsingfengkiang). There had, however, been no complete failure caused by an earthquake of a concrete dam founded on rock. There had been many failures of earthfill dams with hundreds suffering damage [see Table 2 in the paper by Hinks and Gosschalk (1993)].

Mr D J Knight (Sir Alexander Gibb & Partners)
Effect of earthquakes on vital operational components of reservoir structures
Much work has been in progress for many years on analysing, using increasingly complex and sophisticated computer software, the seismic behaviour of the dam bodies themselves, whether concrete or embankment dams. All this is obviously welcome. It is, however, important for actual reservoir safety to appreciate the potential effects of an earthquake on all parts of a dam, especially the vital operational facilities such as spillway gates, stop beams, cranes, intake towers, valves and pipework.

Following the 7.7 Richter magnitude earthquake in the Philippines of July 1990 the writer paid two visits (August and October 1990) to inspect dams and hydraulic control structures affected by the event. Some details of the first visit were recorded in *Dams & Reservoirs*, November 1991, and included reference to:

- the collapse of stop beams for power intakes from vertical storage position at dam crest level,
- the risk of collapse of an abutment hillside threatening to destroy a spillway gate control house and to block the site access road,
- longitudinal thrust causing damage to concrete bridge beams over a service spillway,

- substantial leakage between the flange joints of vacuum relief valves in the gate chambers for power and irrigation tunnels, following considerable bending and stretching of the bolts securing the flanges, which could have led to complete failure of valves and pipework and thence uncontrolled discharge of a reservoir,
- concern about the safety of a reservoir intake tower.

Other observed features from various structures included:
- the collapse of heavy concrete roofs of valve control house structures,
- dislocation of vertical guides for lowering of control gates.

The importance of the work on the seismic behaviour and design of reservoir intake towers, as representing relatively slim, isolated control structures and described, for example, by Daniell and Taylor (Proceedings pp 236-246) is further highlighted by the above examples.

Dams are much more than embankments or concrete monoliths. Earthquakes test the parts that might be neglected when seismic safety evaluations are being conducted.

SESSION 4 RESERVOIRS AND NATURAL HAZARDS

Contributions and responses

Session 4(b) FLOODS, WAVES AND CONTINGENCY PLANNING

Mr A Robertshaw (Yorkshire Water Services)
Dam break analyses
The paper by Tarrant et al (Proceedings pp 282-291) contains a number of graphs showing the comparison of various parameters obtained from the results of 35 dam failure studies by Binnies using DAMBRK.UK. Babtie, Shaw & Morton have studied over 30 dams owned by Yorkshire Water Services and the results of these studies have been incorporated onto figures 1, 2 and 7 of the paper by Tarrant et al to both extend the number of records and compare results. These comparisons show that the relationships of maximum breach discharge with dam height and reservoir capacity are very good even though all the YWS/BSM studies are based on overtopping failures whereas the Binnie studies in the paper are a combination of overtopping and piping failures together with actual dam failures in the case of Figure 1. The length of structural damage compared with dam height also agree well as shown in Figure 7.

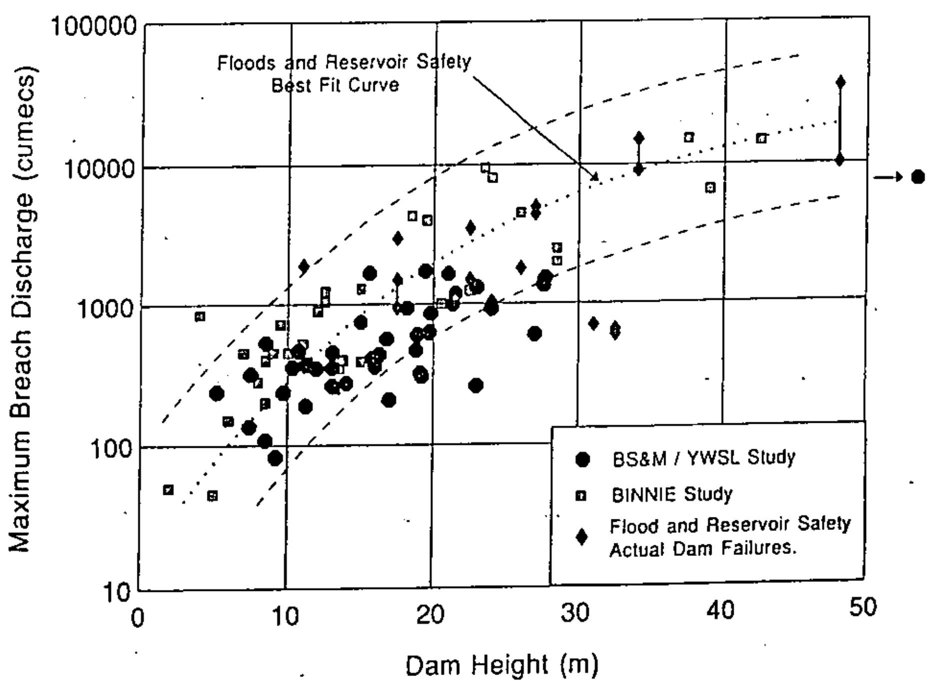


Figure 1 from Tarrant et al with additional data

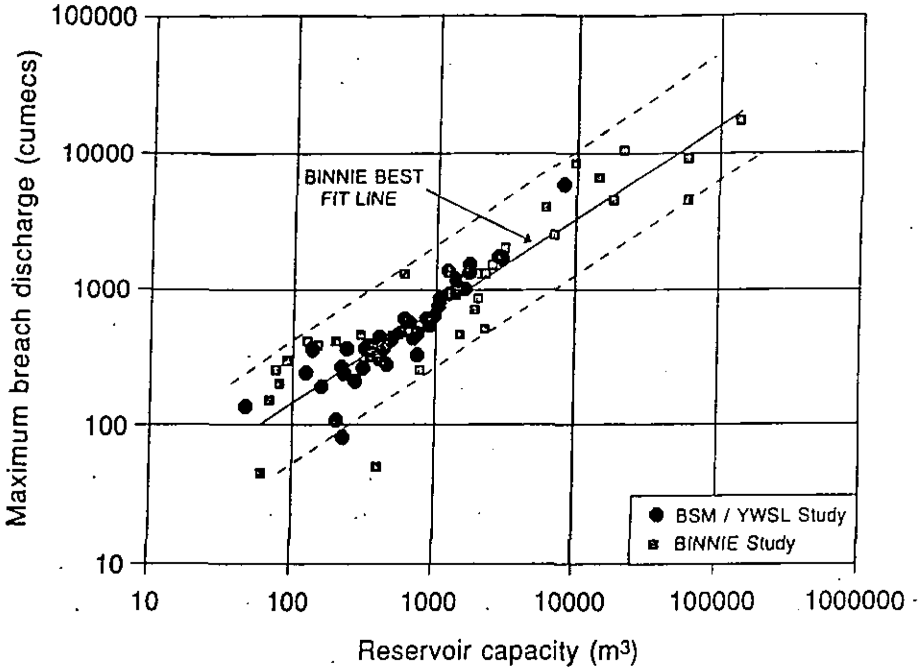


Figure 2 from Tarrant et al with additional data

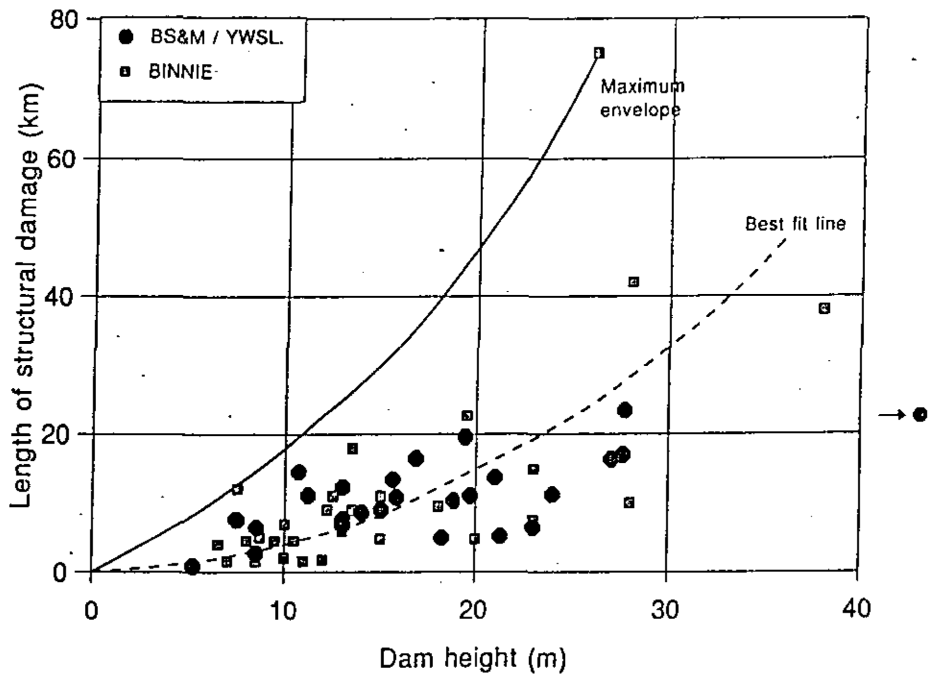


Figure 7 from Tarrant et al with additional data

Mr E M Gosschalk

Contingency planning for dam failure

Yorkshire Water has made a single minded approach to contingency planning for dam failure (Proceedings pp 224-235). It would be interesting to know, in view of the far reaching political, social and economic implications, how it is intended to publish and test the plans for all reservoirs covered by the Reservoirs Act. In their paper (Proceedings pp 306-315), Thompson and Clark mention that of 34 countries included in a recent survey, more than half require hazard mapping and emergency plans, at least for major dams. British Columbia Hydro has prepared emergency preparedness plans for its dams, with testing on a regular basis. An operational exercise reported by Nielsen (1993) involved a simulated earthquake-triggered dam breach with BC Hydro's Corporate Emergency Centre and 23 agencies participating. Nielsen reported that these exercises were well accepted by the public and agencies.

In an article in the same issue of Water Power, Professor R Lafitte, chairman of the ICOLD committee on dam safety, concluded that the concept of risk acceptance at present leads to the recommendation that dams should be designed so that the risk of death is no higher than 1 in 1 million per year person. Professor Lafitte has also stated as an axiom, that it would not be acceptable to put a monetary value on human life in calculations of damage. This thinking would lead to the adoption of extreme loading, in design of dams for large reservoirs, which would never be exceeded as far as can be foreseen. The designer would thus avoid contemplating potential failure and resulting casualties. The adoption of loading with even a low probability is an acknowledgement that greater loading and consequent failure are conceivable. The possibly expensive consequences in construction costs of requiring that dams should not fail catastrophically under maximum possible or maximum credible loading conditions, might be relaxed if emergency preparedness plans are prepared which would virtually guarantee that no lives would be lost. The corollary to this is that an emergency preparedness plan should not be essential if the dam is designed to withstand the maximum loading conditions with a factor of safety sufficiently in excess of one, to provide for margins of uncertainty in the strengths of the dam and its foundations.

It would be interesting to hear the views of the authors and others on these issues.

Mr D J Knight (Sir Alexander Gibb & Partners)

Overtopping of embankment dams

The paper by Hughes and Hoskins (Proceedings pp 260-270) is a welcome contribution to the literature on the overtopping of embankment dams. This writer was glad to hear, during the discussion session, of Dr Hughes' expectation of publishing his wealth of information on overtopping, as his reservoir of knowledge has obviously risen almost to crest level. The controlled and orderly release of it over the top of his research and other private documentation retention barrier is likely to be of considerable interest and practical use to the profession, particularly to all those involved with UK dam safety under the Reservoirs Act 1975.

The influence of local geology as affecting both the nature of embankment fill, and downstream slope and toe erosion resistance, would seem to be significant in a dam's ability to withstand overtopping, including the effects of standing waves on the upstream side of trees and bushes that may be growing on the slope. To enable each dam to "be judged on its merits with all site specific influences considered" (paragraph 30, page 269) it would be most useful if the scattered experience of the actual behaviour of UK embankment dams subjected to overtopping could be brought together for the benefit of all those responsible for their safety.

Dr D Reed (Institute of Hydrology)
Indexing reservoir hazard
Following the session two questions were submitted in writing.

- The first question was addressed to Mr G Thompson and related to the paper on inundation mapping by Tarrant et al (Proceedings pp 282-291). "Did the authors consider the use of dimensional analysis in developing a general index of reservoir hazard?"
- The second question was addressed to both Mr Thompson and Mr Claydon. "Is it not natural to link the categorisation of dams for flood safety (in Table 1 of the ICE floods guide) to an index of reservoir hazard?"

Mr G Thompson (Binnie & Partners) replied to the two questions as follows.
1. Dimensional analysis for rapid ranking
The concept of dimensional analysis was considered, particularly in relation to generalising the hazard index. Whilst it has not been highlighted in the paper, the factors in equation 5 are effectively dimensionless. Those derived in equations 6 and 7 could be made dimensionless by the inclusion of gravitational constants or relative discharges, such as mean annual flow or PMF. The final form given in equation 10 is recognised to be a generalisation of many forces and conditions. Implicitly, it must include some aspects specific to the data set used to derive it, such as population density, type of industry and type of valley, highland/lowland, incised or coastal plain. It is therefore recommended that the index is only used to differentiate between hazard of dams in broadly similar geographical and socio-economic regions. No doubt more detailed analysis could be carried out to include these factors in the index but this would require the creation of similar data sets from widely varying areas. Only then could a generalised, non-dimensional index be derived.

2. Hazard categorisation and flood safety
Regarding the linking of categorisation to hazard. This is indeed the case. Table 1 of the ICE guide does just that in terms of linking required performance to foreseen hazard. However, simplistic definition of the broad bands used in Table 1 undoubtedly result in some anomalies. From the present purely subjective assessment there may be some cases where dams with relatively low actual hazard end up in Category A but more likely, some dams with relatively high hazard are still in Category B. Using the type of hazard index derived in our paper gives a better estimate of relative hazard, but still will generate some anomalies. It is only by carrying out a proper dam break analysis for each dam that proper categorisation can be carried out. To demand this for every dam would undoubtedly cause an unwelcome financial penalty on some companies. For the present however, the pragmatic approach is to indentify those dams where the Inspecting Engineer feels that their classification is marginal and to carry out analysis as necessary. This is being carried out at present by some companies and I hope that the publication of papers on experiences of dambreak applications in the UK will give Inspecting Engineers and dam owners a better feel of potential hazard when reviewing dam categories.

Mr J Claydon (Yorkshire Water) replied to Dr Reed's second question as follows.
The categorisation of dams for flood safety in Table 1 of the ICE guide is already a simplistic hazard ranking system. I agree that it would be logical to move to an index of reservoir hazard, provided that the profession could agree the form of the categorisation. This would be preferable to raising the 25000 cubic metre volume criterion or setting a minimum height.

REFERENCES FOR SESSION 4

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