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on Large Dams (BNCOLD)

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and Scientists (IWES)

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PREFACE

This volume contains the complete discussions from the Reservoirs 1986 Conference, which was held at the Heriot-Watt University, Edinburgh, from 3rd to 6th September 1986.

The BNCOLD Lecture and the other eight papers which were discussed were published together as a separate volume, and are available from The British National Committee on Large Dams, at The Institution of Civil Engineers, Great George Street, London SW1P 3AA.

CONTENTS

Mr JOHNSON, in presenting the BNCOLD Lecture, adjusted the content to describe recent events at Mullardoch Dam which were a very good example of abnormal behaviour identified by the North of Scotland Hydro-Electric Board's surveillance strategy. Mullardoch Dam was a mass concrete gravity dam, constructed in 1951, consisting of two limbs meeting at an apex forming an angle of 140° on the upstream face with one limb 357 m long and 48 m high, and the other 370 m long and 45 m high. He described its behaviour over the past 15 years and, more interestingly, the very recent but serious excursion which occurred at the beginning of July 1986. This was being closely monitored and was under active investigation.

Measurements by collimator since 1963 indicated a progressive net downstream movement of the crest of the dam at the apex of nearly 10 mm, which became obvious around 1971. As a result of this movement, the frequency and extent of inspections and monitoring were increased. Full supervisory inspections were undertaken every 6 months, together with inspections by the Panel I Inspecting Engineer, the consulting engineer responsible for the design, and independent geological inspections. These inspections confirmed that there were no specific signs of distress in the structure or in the foundations; leakage flows had not changed significantly.

As expected, additional monitoring showed that at the mid points between the apex and the abutment of each wing, the crest was moving upstream in summer and downstream in winter in a cyclic pattern. However, at the apex, the crest was moving several millimetres downstream in summer but not fully reverting to the original position in winter, the slight difference between the readings accounting for the slow net downstream movement over the years, averaging about a millimetre per year.

Initial Report

The recent excursion occurred suddenly when the Board's site staff reported on the late afternoon of 4th July 1986 that leakage from the dam had increased from 562 litres/hour (l/h) as last measured on 25th June to 18,751 l/h. The existing cracks, which were up to 1.5 mm in width and were being monitored on the two sides of the gallery at the apex, had opened by up to 1 mm, and there was evidence that uplift pressures were also higher. In addition, a new leak was seen for the first time on the north spillway of the dam.

On receiving this report, arrangements were made for immediate inspection and survey of the dam by qualified Division staff and these were undertaken on 5th and 6th July.

Inspection of Dam

The inspection indicated that conditions were worse than had been experienced before. There was a noticeable inflow of water into the gallery, primarily through a crack about 2 mm in width on the upstream face of the bays on each side of the apex. The remainder of the bays appeared to have changed little except that one or two new cracks had appeared and existing cracks had propagated and opened. The cracks in the gallery had been first observed in 1982. There was also evidence of compression spalling of fettled concrete at the vertical bay joints. Readings of critical parameters were taken on four occasions on 4th and 5th July and were observed to be moving favourably.

Over the previous 12 months, temperatures had been low and with the wet spring, water levels were also unusually high. The mini heat wave experienced over the previous three weeks probably created high temperatures on the downstream face, which was borne out by the upstream tilt of the crest as indicated by the pendulum readings. On the basis of the inspection and assessment of parameters, it was believed that the excursion was probably due to the large temperature differential which was likely to exist between the upstream and downstream faces.

The following actions were put into immediate effect: (a) pendulum, vee notches, joint gauges, and uplift measurement pipes were arranged to be read twice daily and the results immediately analysed; (b) provided parameters continued to move favourably, the water level in the reservoir was to be reduced as quickly as practicable without wasting energy or spilling water until the level reached approximately 777 ft above ordnance datum (AOD), where it was known there was a defective open lift joint on the upstream face; (c) a further high precision survey of the dam by mekometer was commissioned, along with detailed crack surveys and stress gauge monitoring; (d) immediate contact was made with the designers of the dam, and arrangements made for the All Reservoirs (AR) Panel Engineer, Mr H.W. Baker, who had last inspected the dam in 1981, to join the detailed discussion of the situation.

The dam was inspected by Panel AR Engineer Mr E.M. Gosschalk of Sir William Halcrow & Partners, the designers, on 9th July, and a full discussion took place on 10th July.

Review of Situation

The results of monitoring the dam over the previous five days were reviewed, and showed a steady and significant improvement in the condition of the dam. Both Panel Engineers endorsed the monitoring which had been put in hand, the preliminary diagnosis of the situation, and the actions taken. The following additional monitoring was recommended:

(1) The British Geological Survey (BGS) to be approached to ascertain if any seismic activity had occurred in the period under consideration.

(2) A direct inspection of the upstream and downstream faces of the dam to be undertaken to identify and measure cracks, particularly in the vicinity of the apex.

(3) A close inspection of similar dam(s) to be carried out to see if there were any similar cracking, movement, and effects.

(4) A temperature survey of the dam, particularly the interior, to be undertaken.

Results of Subsequent Actions

The water level was steadily drawn down over a period of about six weeks, with the leakage rate falling in sympathy with level. At a level of 777 ft AOD, leakage from the north limb of the dam ceased completely, indicating that the source of that leakage was the defective joint in the upstream face of the dam.

On the afternoon of Sunday, 13th July, as readings were being taken, the waterman heard a very loud crack, and it was observed that the leakage had again increased by about 10%. This was at a time when the weather was beautiful, dry and very warm.

During July, a detailed inspection was made of the gallery of Benevean Dam, which was a mass gravity dam of the same age and design. A similar pattern of cracking was observed in the gallery, but it was minor in extent compared with that observed in the gallery of Mullardoch Dam.

The BGS reported back that there had been three insignificant seismic events in the period 30th June to 13th July, the nearest being about 20 miles from the dam. Overall, they judged that these could not have been responsible for any damage to the dam as they were only marginally greater than background noise.

On 22nd July, a statutory inspection was carried out on the Upper Shira Dam, which was a multiple arch buttress dam some 725 m long and 45 m high. A very similar pattern of cracking and width of cracks was observed to that at Mullardoch. Longitudinal cracks had been observed in this dam for the first time in 1984 and were under close scrutiny. The cracks which could be inspected on the downstream face were only hairline in thickness, whereas those in the gallery were up to 2 mm and more in width. Significant compression spalling of the heart concrete had occurred at the vertical construction joints between bays. As the water level was 8 m below the level of the gallery, it was concluded that the cause of the cracking was solely due to thermal effects and this reinforced the belief that the cause of the excursion at Mullardoch Dam was thermal in nature.

The detailed inspection of the upstream and downstream surfaces of Mullardoch Dam showed there were a number of cracks, mainly hairline, but none comparable in width with those in the gallery. The temperature survey of the concrete of the dam indicated that there were fairly significant variations of temperature of up to 6°C in the body of the dam, the coldest concrete being located in the centre of the dam in the vicinity of the gallery. Analysis of the uplift pressures revealed fairly high uplift pressures at the toe of the dam, and this was a matter of some concern.

Further Analysis and Actions

A further discussion between all parties took place on 12th August to review the latest results of monitoring, to study the additional information which had been gathered, and to consider the form of analysis and potential remedial measures which should be taken, with the following results:

(1) A number of stress gauges had been installed near the downstream face in 1983 and these had, throughout their existence, shown all stresses to be tensile, which was difficult to understand for a mass gravity dam which was designed with the object of ensuring all stresses were compressive. This aspect was being further pursued and it was proposed to install a number of strain gauge rosettes along the gallery and cross galleries to act as a check on the stress gauge readings, complemented by a number of additional joint gauges across the vertical bay joints to measure axial movement in the dam. The strain distribution might also be affected by the discontinuity of the gallery.

(2) It was agreed that the rather high uplift pressures should be relieved as soon as possible.

(3) There was some difficulty in interpreting some of the mekometer results because the survey stations might be moving very slightly geomorphologically, and this aspect was being further analysed. There were indications that the sides of the valley tended to move inwards during the summer and outwards in the winter, imposing an additional axial force on the dam. The latest survey showed that downstream movement of the dam was continuing, although not in an abnormal way, in relation to earlier recorded movements. An additional mekometer survey would be undertaken that month (September).

(4) There was clear evidence of closing up of the vertical construction joints in the dam. Whether this was due to precipitation of carbonates in the joints, valley movements, or expansion of the body concrete was uncertain. Samples were being taken from the dam and would be analysed for alkali/silicate reaction, aggregate swelling and concrete expansion under humid conditions.

(5) The survey of the upstream face would be extended to the vicinity of 777 ft AOD.

(6) It was agreed that monitoring could be reduced to once daily, subject to the parameters continuing to move favourably.

(7) The designers recommended the following analyses should be undertaken aimed at reaching an understanding of the cause of the abnormal behaviour:

(a) Simple static analysis covering dead load, uplift and hydrostatic loads, especially in the vicinity of the gallery, to provide input data for the ensuing finite element analysis.

(b) Two dimensional finite element analysis in the vertical transverse plane, in an effort to correlate thermal stressing and crack development in the vicinity of the gallery.

(c) Three dimensional finite element analysis of the whole dam to investigate longitudinal thermal effects and potential axial loads from the valley sides, swelling of concrete, etc. This study would also investigate the value of remedial actions under consideration.

(d) Formulation of a predictive equation which would attempt to explain the effects of various forces acting on the dam and its behaviour.

The task of analysing the dam was a very difficult one and the chances of producing convincing definitive results were not assured. Nevertheless, there was confidence that the analysis would demonstrate whether or not remedial works were necessary and the effects of potential remedial works.

Remedial Works

The remedial works which were under consideration were as follows:

(1) The defective concrete in the lift joint at a level of 777 ft AOD was to be cut out and made good incorporating appropriate flexible seals that month (September).

(2) A number of holes would be drilled from the downstream toe of the dam into the transverse and longitudinal rubble drains to relieve uplift pressures starting that month.

(3) The possible creation of expansion joint(s) at the apex or adjacent thereto, or adjacent to the valley walls, to accommodate axial movement of the two limbs of the dam. This expensive feature would not be committed until the results of the structural analyses were available. An expansion joint was created in Fontana Dam of the TVA to overcome cracking in the mid 1970s. This was a 146 m high gravity dam, curved in plan and 721 m long.

(4) Possible vertical pre-stressing of the upstream face of the dam to knit together the cracks which had arisen in this face.

Until the first two remedial measures had been undertaken, the water level in the dam would be kept around its present value and daily monitoring would continue. The results of the structural analyses would thereafter dictate the further control and extent of monitoring adopted.

Conclusions to Date

(1) Leakage had been shown to be the most sensitive parameter for monitoring the overall behaviour of the dam although the mekometer surveys and pendulum readings were required for determination of movement.

(2) There appeared to be growing evidence of the gradual carbonating up of the vertical construction joints of Mullardoch and other dams which was increasingly preventing the free expansion of the bays of these dams and thereby inducing axial compression forces and some compression spalling.

(3) Thermal differentials across and along massive dam structures could become critical and result in significant cracking if expansion was not properly accommodated.

(4) Significant longitudinal cracking in the galleries in a few of their longer dams had only recently been observed but appeared to be progressively increasing.

(5) Pressure relief systems could be vital to the safety of mass gravity dams and had to be carefully monitored and kept fully effective.

THE BNCOLD LECTURE

Experience with the Concrete Dams of the North of Scotland Hydro-Electric Board

by F.G. JOHNSON, MEng, MICE, MIWES
Chief Civil Engineer, North of Scotland Hydro-Electric Board

DISCUSSION OF THE BNCOLD LECTURE

Discussion

Mr H.W. BAKER (Consultant, James Williamson & Partners), in opening the discussion, said that Mr Johnson was to be commended for a late adjustment to his lecture to include a description of recent events at Mullardoch Dam and the actions taken in the light of these events. Mr Baker had been invited to give a few comments on this situation from the viewpoint of the Panel Engineer presenting the most recent (1982) Statutory Report of an inspection under the 1930 Act, and having regard to the fact that he had accepted an invitation to participate in examining the situation and deciding on future action.

His involvement with this reservoir dated from 1981, when he commenced his Statutory Inspection. It was of particular interest to him that the preceding Inspector, Mr A.C. Allen, had observed no evidence to suggest any undesirable movement of the dam, but had pointed out that due to the geometry of the dam, any expansion in the wings might lead to compression, introducing a force additional to the normal water load at the apex in a downstream direction. Additional instrumentation had been recommended to keep check on movements, and this more accurate work appeared to indicate, over the next few years, that there was a modest progressive downstream movement of the apex. This was kept under close surveillance and was debated at length by various parties, including the owner, the designer, the previous inspecting engineer, and the Institute of Geological Sciences. In his inspection he concluded that the more recent measurements indicated a reduced rate of movement of the apex downstream with practically no irreversible movement of the dam over the previous five years. He concurred that there could be a longitudinal compressive force in the wings of the dam and had recommended that steps be taken to establish the magnitude of any such force and any variation with time. This had been done but the results were disappointingly inconclusive, although they were still under investigation.

The recent incidents and the subsequent actions had been described by Mr Johnson in some detail and it had not been opportune to comment further on these. However, he considered that he was privileged to have been consulted in this incident. What had been most evident to him in assessing the problem was the rapid and responsible action of the North of Scotland Hydro-Electric Board personnel in recognizing and acting on the evidence of an untoward incident, and the ready willingness of the designer, Halcrow Water, to explore the possible causes and remedial measures which might be necessary, a task made much simpler than it otherwise might have been because of the excellent plots of the various instrument records of the dam.

Mr W.T. MEE (Sir Alexander Gibb & Partners) said that the BNCOLD lecture had drawn attention to a number of design matters which affected maintenance. He drew attention to two examples of unsatisfactory final products noted during inspections under the 1930 Act. Depending on the experience or point of view of the observer, these examples could be regarded as reflecting on design, construction materials, or construction methods inappropriate to the design concepts; or perhaps on combinations of these factors.

The first example was a typical section through an arch dam with a continuous overspill crest some 200 ft long (Fig. B.1). The upstream face was vertical to within 20 ft of the crest, which had an overhang above this, which was constructed as individual blocks separated by contraction joints which had keyed faces radial to the arch. Until these joints were concreted the blocks were quite separate, i.e., individual cantilevers.

The downstream face was reinforced above a certain level and this in itself would lead to very difficult concreting conditions. The individual cantilevers had a calculable tensile stress in the downstream face. The rate at which they were raised would need to provide time for the concrete strength to develop, stage by stage, so that it bonded adequately with the reinforcement. Inadequate concrete strength at any stage of the construction would increase the tensile stresses still further, inevitably leading to local bond failure and micro cracking.

A further aspect to consider was the principle of adopting contraction joints in an arch. If these were concreted before the concrete in the main blocks had been given adequate time to shrink, joints would continue to open up even after the contraction joints were concreted so that localized non-elastic movements could occur when the dam was loaded or unloaded. Concrete of the thickness

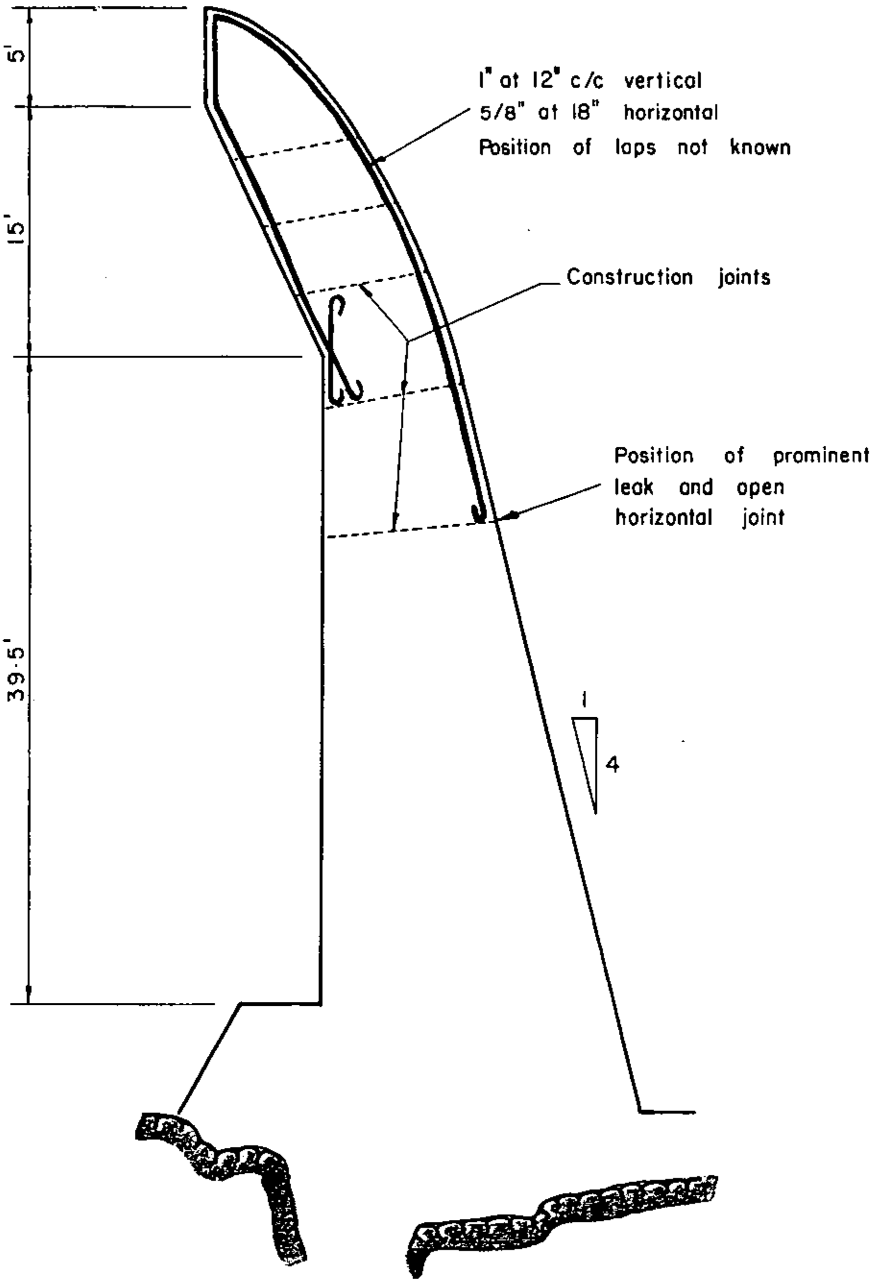

Fig. B.1. Section through an arch dam

involved in this dam would have continued shrinking for a long time after construction as it was not cooled.

Less than 20 years after the dam was completed there was extensive cracking on the downstream face, reflecting the position of the reinforcement and extensive leakage along the contraction joints. There was also a prominent horizontal joint along the downstream face immediately below the level of the reinforcement which was more than 5 mm open. The present appearance of the dam was most unsatisfactory.

A slide (not reproduced) showed a common form of closure detail, this time in a buttress (Fig. B.2).

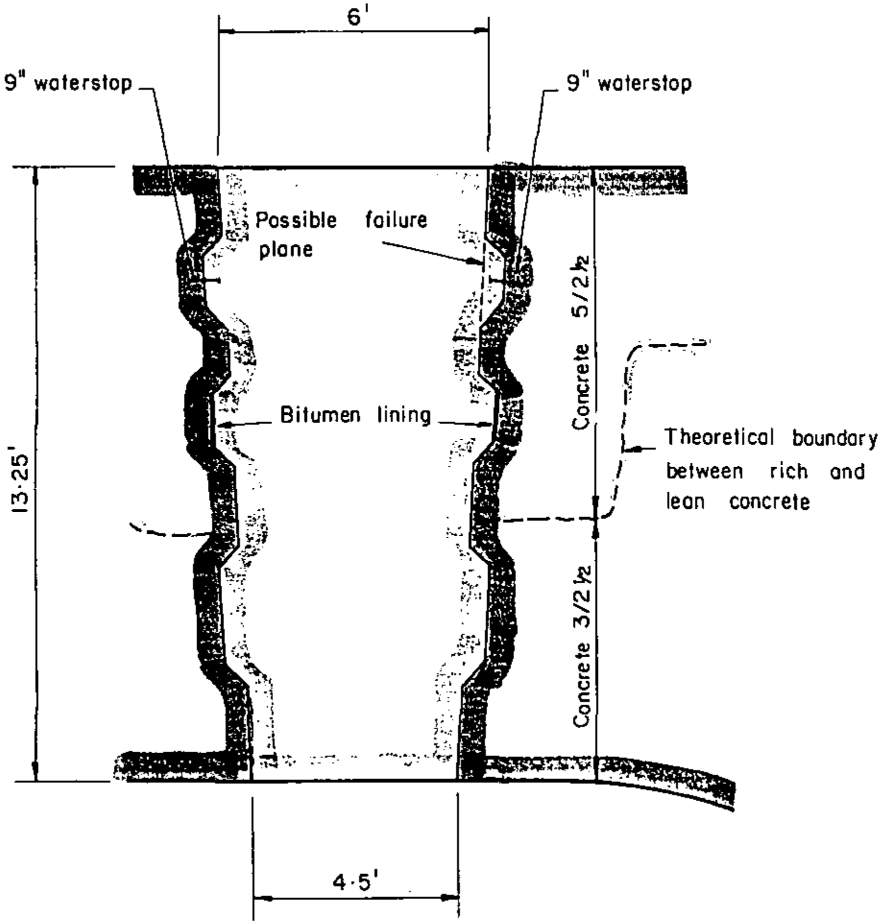

Fig. B.2. Closure detail in a buttress

The water stop was located in the keyway. When the concrete in the main buttress shrank, the wedge was prevented from moving downstream by the keys which therefore picked up the horizontal load. The key incorporating the waterstop was invariably weakened by the waterstop itself and tended to shear thereby rendering the waterstop entirely ineffective. The result was illustrated by a slide (not reproduced) showing extensive seepage discolouration along joints on the downstream face.

Mr E.B. WILSON (Sir William Halcrow & Partners) said that he ought first to establish his credentials for giving his personal view of Frank Johnson's paper. Of the approximately 50 Hydro-Electric Board concrete dams built since the Second World War, he had either certified or, in collaboration with Mr Johnson and his predecessor, Leslie Dickerson, made statutory inspections of over 50 per cent. Therefore, Mr Johnson might give him licence to criticize. He had also been intimately involved with the preliminary surveys in 1946 and 1947.

Mr Wilson said he found Table 1 of the paper especially interesting. For planning they had relied on the 2½ inch to the mile Ordnance Survey maps, isohyetals from the Meteorological Office, Captain MacLean's run-off records of several large rivers and published efficiencies from turbine/generator makers. He found astonishing the close correlation between original forecasts and average output figures over 40 years of operations. He thought some congratulations were in order for a lot of people, and especially for the Meteorological Office rainfall contours in that mountainous region.

He was impressed by the increase in the supervisory staff (by a factor of three) to cope with the requirements of the 1975 Act. These were early days and some of the procedures might be found to be not justified, for the concrete dams at any rate. Earlier speakers had emphasized the considerable discretion allowed to the inspecting and supervising engineers. There was no doubt that they would each gain confidence by watching the behaviour of a reservoir over a lengthy period. He applauded the decision to abandon the practice of changing the inspecting engineer every ten years, which surely carried the belt and braces syndrome too far. He would go further and question the wisdom or practicability of enshrining in the Act a ban on the inspecting engineer having a working connection with the construction engineer. There was much worldwide collaboration by engineers on large projects.

The weakness in concrete dams was not in the concrete, which he agreed endured for centuries. It lay rather in the holes through the dams, which had to be controlled by rusting and barnacling valves, gates and pipes. These structures had a comparatively short life. The design ought to give much more attention to ease of replacement and with minimum outages.

Mr Johnson and he had often been concerned with ensuring that there was free drainage for pressure relief and he noted that Mr

Johnson had been using very high pressure water jetting in Mullardoch Dam. He would like to hear more about the techniques and results.

This was a good paper which would be a valuable source to designers in the future, but he hoped not too far in the future, for there were good reasons for developing all available water power now. Was it not a fact that, contrary to all other types of power generation, the older the hydro scheme the cheaper the cost per unit generated? Someone in the Hydro-Electric Board should construct a league table which, in Scotland, could be something like the following (in order of rising cost):

Galloway, Lochaber, Grampian (1920s);
Sloy, Affric, Tummel (1950s);
Conon, Garry-Moriston, Shin (1960s).

The logical conclusion from this use of mainly renewable energy was that, by exploiting all that was readily available now, it would beat for economy within a few decades any source of power which relied on fossil fuels.

Mr W. LOGAN JACK (Welsh Water Authority) said that as a supervising engineer for six concrete dams, he would like to thank Mr Johnson for so much useful information in his paper. The North of Scotland Hydro-Electric Board had been very helpful to the Welsh Water Authority in giving advice on clearing pressure relief holes. They had spent four weeks this summer water jetting in two dams to clear deposits which had accumulated over the 80 years since the dams were constructed. The holes were typically only 2 to 3 inches in diameter and, having right angle bends, were not designed with maintenance in mind.

Mr Johnson's remarks on the effects of afforestation were also of special interest. At Welsh Water, their supplies suffered from loss of yield and from water quality deterioration. In addition, the rivers and streams in parts of mid Wales had been shown to have become fishless where conifers were present over significant areas. After several setbacks they had succeeded in establishing a procedure for consultation on forestry developments within their catchments. With this consultation they hoped to reduce the worst effects of coniferous afforestation but they still had to suffer significant adverse effects. It would be interesting to know if the North of Scotland Hydro-Electric Board had managed to influence the afforestation on their catchments and to what extent their interests were borne in mind by the forestry developers.

Mr G.P. SIMS (Engineering and Power Development Consultants Ltd) said he was interested to hear Mr Johnson refer to the deleterious effects of forestry in connection with the operation of the North of Scotland hydro schemes. He would be grateful if Mr Johnson could indicate in what circumstances forestry development was detrimental and in what circumstances he might expect it to be beneficial. His question arose from experience in other parts of the world where forestry development was positively encouraged as a means of protecting the catchment of a reservoir. Were there, for example, any husbandry techniques which should be avoided, or were there any which should be encouraged beyond those mentioned in the BNCOLD lecture?

Author's Reply

Mr JOHNSON, replying to the discussion, said he had particularly appreciated Mr Baker's counsel in the considerations which had taken place on the recent Mullardoch excursion. One aspect which might be worthy of mention was the fact that movement of the apex of the dam was first identified by Arthur Allen following his Statutory Inspection in 1971. At this time, EDM surveying equipment was just becoming available and advantage was taken of its greater potential to confirm the movement of the dam. High accuracy Mekometer surveys were instituted in the middle seventies and these confirmed the dam movements and in addition indicated that the sides of the valley were moving in and out seasonally by several millimetres.

Although Mr Wilson had certified or inspected over 50 per cent of the Board's dams, he had also made a major contribution to the design of many of these dams. It was therefore always a great privilege to join him on one of his inspections as he had so much to contribute and from which to learn. However, Mr Johnson did not altogether share his views on the value of not changing the inspecting engineer, or even allowing the design engineer to act as the subsequent inspecting engineer. After changing the North of Scotland Hydro-Electric Board's policy, in the very early seventies, to employing a different inspecting engineer to the design engineer for a dam, they found that a good number of important and unsatisfactory aspects were brought to light leading to improvement in the safety of several of their dams. He considered there was merit in an independent panel engineer to the designer inspecting a dam after it had been in service for a few years.

In relation to Mr Wilson's request for more information about the high pressure water jetting used to clear relief holes, he gave the following details: a submersible pump supplied water to pressure jetting plant, situated on the roadway of the dam, which operated generally at 5,000-6,000 psi. Pressurized water was delivered to the gallery through a ½ inch bore hose. A 'dump valve' was provided in the line before the jetting hose and its 5/8 inch diameter nozzle. The maximum flow was 13.5 gallons/minute and the pressure relief holes were 3 inches in diameter. As the flow was cut off as soon as water stopped discharging into the gallery, the pressure at the foot of a hole never exceeded water depth to the gallery. This method was very successful although it could not be expected to remove large solid objects.

With regard to the cost of electricity generation by hydro, this was very much affected by site conditions, age of schemes, refurbishment undertaken and particularly the accounting system adopted. If a historical cost accounting basis was employed, the early developments such as the Grampian Scheme (part of Tummel Schemes) had overall generation costs of about 0.3 p/kWh compared with the average of 0.8 p/kWh for all the Board's hydro schemes. The schemes were very attractive as they were based on a renewable and benign source of energy and represented an increasingly valuable investment which should last in perpetuity with proper maintenance and refurbishment.

The remarks and questions on afforestation from Mr Sims and Mr Logan Jack were interesting. On the Board's catchments in the Highlands, the effects of afforestation appeared to be wholly disadvantageous, for the following reasons:

(a) The yields from catchments were reduced by typically up to 20 per cent due to the evaporation and transpiration losses from mature trees.

(b) The deep ploughing which took place prior to planting resulted in higher flood run-off and greater spill along with lower drought flows which required increased discharges of water from reservoirs to meet compensation flow requirements.

(c) The ploughing cut through the vegetation/peat cover of catchments into the sub-soil beneath, and, with the cross contour ploughing, caused very rapid run-off and the transport of much gravel and sand into aqueducts and intakes. This detritus filled up the intakes which required more frequent cleaning out and greater losses of water and eroded the inverts of aqueducts leading to increased maintenance costs.

With regard to husbandry techniques which were beneficial, the Board endeavoured to ban afforestation within 25 yards of aqueducts and intakes, which reduced sand and gravel migration into these works. In addition, it was the Board's practice to meet Forestry Commission representatives every six months in each Generation Group to discuss and mutually agree future afforestation plans where they affected Board works. They also tried to meet and discuss with private forestry companies their afforestation proposals before work commenced on site; these discussions had ameliorated some of the detrimental effects of afforestation.

The Board was also directly contributing to the Balquhidder Project of the Institute of Hydrology, which was aimed at determining accurately the effects of afforestation on run-off from catchments

The above observations referred specifically to Highland catchments. On catchments abroad, where erosion was very much greater, afforestation could well be essential to stabilize soil erosion and to control rapid siltation of the reservoirs.

FIRST SESSION: MANAGEMENT OF RESERVOIRS

Session Chairman:

T.A. HARKER, President, IWES

Papers:

1. Reservoirs Act 1975: Experience so far, by S.C. AGNEW, CB, BSc, FICE, FIWES; Chief Engineer, Scottish Development Department

2. Documentation for Reservoirs and Procedures, by N.H. GIMSON, OBE, BSc, FICE, FIWES; Divisional General Manager, Western Division, North West Water Authority

3. Operational Management - Opportunities and Constraints, by R.G. SHARP, BSc, FICE, FIWES; Regional Manager, Resources and Rivers, Severn-Trent Water Authority

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DISCUSSION OF FIRST SESSION

Authors' Introductions

Mr AGNEW, in introducing his paper, summarized the extra provisions within the 1975 Act, the Act's implementation and reactions of interested parties, before describing the experiences of those concerned. He gave details of the progress made in appointing engineers to the four panels constituted under the Act: 60 appointments to the All Reservoirs (AR) Panel; 15 to the Non-impounding Reservoirs (NIR) Panel; 22 to the Service Reservoirs (SR) Panel; and, 324 to the Supervising Engineers (Sup) Panel. Registers were not yet complete, but by extrapolating information for Scotland and Wales, he estimated that the number of large raised reservoirs in Great Britain was about 2,400, including some 700 in private ownership.

Panel engineers were experiencing an increase in requests for inspections, particularly from private owners, some of whom, as a result, were deciding to take their reservoirs out of the ambit of the Act by breaching or lowering the dam. Overflow capacity was not usually a problem and accepting that rare overtopping could usually be tolerated, inspecting engineers were finding that by using their discretion they could deal quite briefly with most low risk reservoirs. The requirements for supervision in Section 12 were being organized in a variety of ways, usually with responsibility for supervising a few reservoirs being a part-time task for engineers employed by the undertaker. Inspecting engineers could give guidance on the frequency of visits, and although two visits per year looked like being the norm, this could be too frequent for some low risk reservoirs. The inspecting engineer should also give directions as to the manner and frequency of record keeping.

Public sector undertakers, as had been the case under the 1930 Act, were complying willingly with the new Act's requirements for inspections and supervision, but there had been some complaints on behalf of private owners (and conservation bodies) about the expense of complying with the Act's requirements. In commenting on the suggestion that the qualifying size should have been doubled to 50,000 m^3, Mr Agnew showed slides (not reproduced) of the damage caused by the failure of the 3.5 million gallons capacity Skelmorlie reservoir in 1925, which had resulted in the loss of five lives. By illustration of the other extreme, where very large volumes were impounded over a large area by very small structures, he suggested that the only practicable method of dealing with low risk reservoirs was for inspecting engineers to use their discretion to the full, including directions as to records and guidance on frequency of visits by a supervising engineer. Supervising engineers were required to report in writing to the undertakers not less than once a year, only on matters they had been instructed to watch in any annex to a final certificate or the latest report of an inspecting engineer. Action had been taken by the Institution of Civil Engineers and by Government to remind inspecting engineers of the need to exercise their discretion in the case of 'low risk' reservoirs.

Attention was drawn to the sections on discontinuance and abandonment (which were new), and in particular to the onus placed on an undertaker prior to abandonment. While Section 14 appeared to assume that a reservoir would be emptied and kept more or less empty, a reservoir could only be removed from the register following the issue of a certificate under Section 13(3).

Although enforcement authorities were having some problems in compiling data about privately owned reservoirs for the registers, no insurmountable problems had been reported and first reports to the Secretary of State were not due until 1st April 1987. There had been a few instances of enforcement authorities instructing inspections of 'ownerless' reservoirs, but so far no more extreme action had been required. Once the registers were complete, the receipt and checking by enforcement authorities of certificates and those reports from inspecting engineers containing recommendations in the interests of safety should be non-technical. Implementation of the Act, including appointments to panels and preparation of registers, was progressing smoothly as planned, and it appeared to the Government Departments that the Act was so far being operated effectively and responsibly by all those concerned.

Mr GIMSON introduced his paper by saying that, having interviewed a number of enforcement authorities and a representative of many private reservoir owners, he was able to bring their views to the conference to add to those of reservoir engineers.

Many people who otherwise recognized that the later Act was a considerable improvement on its predecessor nevertheless felt that parts were merely a re-hash of the 1930 Act and as such, it had missed some opportunities for further improvements and had created difficulties in the documentation. This, at the end of the day, could be said to be the fault of the legislators, but Mr Gimson felt it was a product of a rather unhappy marriage between civil engineers and civil servants. Maybe they were being too civil to each other, but the Act left matters to be interpreted which might well result in disputes which would draw in reservoir engineers who had not been accustomed to disputes of that sort before.

One example was the confusion over the word 'safety'. Frequently, the engineering view prevailed which might loosely be said to be that if a structure could fail it was unsafe. However, in many parts of the Act the implication was that safety referred to safety of persons and property, a very logical argument one might say, but the two meanings were not synonymous. This matter had already raised significant protests from many of the private reservoir undertakers and as a result they had the well-known letter from the Department of the Environment which broadly speaking said 'we know we have got this legislation passed but please don't create political difficulties for us by complying with it'.

Then they had the problem of English and the definition of a non-impounding reservoir. It was said from time to time that engineers were not very good at English (a proposition which he disputed) and that civil servants were good at English because they had a classical education. Mr Gimson would defy any classicist to define a reservoir which could not impound anything. Nevertheless, it was likely to be in an undertaker's interest to have it accepted that a reservoir was a non-impounding reservoir (even though no one knew quite what that was) because non-impounding reservoir engineers were likely to come cheaper than impounding reservoir engineers.

As a non-impounding panel man, Mr Gimson would be happy to undercut any impounding reservoir engineer! At the end of the day, however, there might be insufficient non-impounding reservoirs to justify a separate panel.

Then there were the sections in the Act on abandonment and discontinuance, which presented two puzzles for interpreters of the English language. These sections were likely to cause either confusion or hilarity, or both. Mr Gimson was interested in the process of thought which determined that a reservoir which carried a certificate saying it could not fill above the level of the surrounding ground was still required to be retained on the Register, but one which had a certificate saying it could fill up to 25,000 m^3 above the surrounding ground might be deleted from the Register. He thought that as from the engineering point of view the abandonment of a reservoir or a reduction in size presented similar problems to the construction of a reservoir, the whole thing could have been incorporated in the sections covering construction engineers' certificates.

In fact, the main problem with abandonment and discontinuance was the environmental lobby. For decades they had suffered from total opposition to the construction of any new open reservoir, but suddenly the environmental lobby had discovered that existing reservoirs were God's greatest gift to man and were opposing their abandonment. Civil engineers were verily a long-suffering breed.

The paper mentioned more than once the fact that discussions and documentation stemming from the legislation referred almost exclusively to impounding reservoirs although the legislation brought other reservoirs into the net. The influence of the Panel I or All Reservoirs engineer was understandable, but the balance did need correcting a little so that, for example, they did not omit service reservoirs altogether from parts of the documentation and elsewhere omit even the possibility that a service reservoir might be made of reinforced concrete. This was one problem created by using a piece of legislation really intended for impounding reservoirs as a catch-all for others which were quite dissimilar.

However, it was not really a catch-all because it missed out all the thousands of reservoirs at the lowest end of the scale. How did size come to be used as a parameter? If there was a rush of water onto Mr Gimson's house and garden, he would not be over-impressed by being told that it was all right because it was only 20,000 m^3. Even one tenth of that might well cause him extreme discomfort or drown him. Private reservoir owners with 26,000 m^3 in moorland country, presenting no danger to persons or property, were likely to be very cross when they found that their reservoir was controlled by the Act yet a reservoir owned by the local water authority of 24,000 m^3 had no statutory control even though it had a substantial population a few hundred yards down the road.

It might be said that the danger from such a reservoir, probably built of concrete or masonry, was minimal because it was firstly likely to show substantial leakage before failure. Nevertheless, only one disaster from such a reservoir would result in very speedy amending legislation. It seemed to Mr Gimson a pity that this Act, which was, after all, a second bite of the cherry after the 1930 Act, did not require such reservoirs to be inspected and categorized according to the dangers they presented, with the requirement that in appropriate cases the frequency of inspection was laid down. Such inspections could be very simple.

Enforcement authorities were statutorily responsible for creating the Register even though occasionally there might be little assistance from the undertaker. There were a number of difficulties in completing the Register which in many cases might take one, two or even more years to complete correctly. It seemed a pity that the Government, after waiting ten years to put the Act into effect, did not allow the authorities adequate time, with the result that theoretically, they were contravening the law. This seemed partly to be due to the bigger number of private reservoirs than had been anticipated, but examples of greater difficulties were the problems of establishing the owner, establishing the capacity and even whether it was a reservoir at all under the Act.

Establishing the capacity was difficult from several points of view, as Mr Gimson had indicated in the paper, but the treatment of silt, so frequently referred to in meetings, did seem to need special guidance from the Department of the Environment. He said this was necessary partly in order to ensure reasonable similarity of treatment across the country and partly to minimize the possibility of what could be quite bitter disputes if the reservoirs were around the statutory minimum size.

Mr Gimson made reference in the paper to computerization of the records. Before long he thought it would extend further because the rapid march of telemetry which was at last taking place would allow centralization of stability data as well as documentation information on a computer. North West Water's computer programme, designed primarily for the statutory documentation, already held useful records of structural and hydraulic significance. Graphics now allowed, for example, the examination of trends in levels of the crest, variations in the line of the crest or trends in the relationship between rainfall and drainage flow.

The next step might be to include embankment instrumentation readings. Although instrumentation was inadequate it was slowly increasing in quantity as certain reservoirs gave cause for anxiety, and readings could be transmitted by telemetry to the computer with appropriate alarm levels built in so that dangerous situations could be foreseen. One day such a system might be looked upon as a satisfactory replacement for the reservoir keepers whose passing many people regretted.

However, that was only for reservoirs of large public bodies. Private reservoirs would continue to be much less well monitored and this was just one of the contrasts that were referred to in the paper.

Finally, just as papers on engineering design and construction often seemed to be written without the benefit of operating experience, Mr Gimson thought that his paper might have been different if written three or four years later, and he hoped for the opportunity to discuss experiences again after a suitable period of time.

Mr SHARP turned, in his paper, to the operational deployment of water source reservoirs. This was the setting in which legislative and safety procedures applied. The operational deployment on the one hand, and safety procedural requirements on the other, influenced and impinged on each other. Hence the logic of considering these aspects together in the one session.

The main message of his paper was that with the comprehensive catchment based water service management and the need to exploit opportunities for efficient use, reservoirs were increasingly required to operate to give improved performance and wider benefits. They were operated as part of a catchment based resource system, in association with other sources, and with more than one (sometimes several) demands and uses to be met. These requirements led to more complex variable operation, in accordance with predetermined rules applicable over a wide range of eventualities.

There were four main aspects to this overall thesis:

(i) Integrated operation for water resource/supply enhancement gave greater efficiency and benefits (higher overall output, greater reliability, and lower operating costs).

(ii) Multi-objective use, i.e., secondary uses or objectives to the primary use, in this case of water resource enhancement or water supply. Whilst often having conflicting requirements, these secondary uses did give useful additional benefits, including greater acceptability of the reservoir and the way it was used.

(iii) Operating constraints, whether for safety reasons from natural limitations or excesses, or from man-made physical limitations of the reservoir, dam and associated infrastructure. These all gave rise to limitations on the way the reservoir was operated; the more sophisticated the mode of operation, the more such constraints might arise and become significant. Predetermined rule curves as outlined in the paper were imperative in reconciling objectives and constraints.

(iv) Significance for design and safety of variable integrated, multi-objective operation, both for new reservoirs and for refurbishment of old. The aim had to be to provide for maximizing (i) and (ii) while mitigating (iii).

These aspects were enlarged upon in the paper.

Discussion

Mr S.M. HAWES (Consulting Engineer), in opening the discussion, said he was an independent consultant who had been practising in the largest industry in the UK for over 30 years.

He would like to thank Mr Agnew for his paper, which gave interesting statistics concerning Wales and Scotland. He wished to add some statistics from England where, in the particular sphere in which his practice operated, the Act was having a very adverse effect on the very objective for which it was enacted, namely, safety.

According to the Ministry of Agriculture, Statistics Division, there were: 862 reservoirs used for irrigation in 1974, and 2,700 reservoirs used for irrigation in 1984. The Advisory Council for Agriculture and Horticulture in England and Wales produced a report called 'Water for Agriculture: Future Needs' (the Strutt Report). In this report, they considered that an estimated demand of 157 million m^3 in 1985 would increase to 350 million m^3 by the year 2000. Since most easily available sources were already taken up, storage would play a large part in conserving water for this increased demand.

Thus, on a percentage basis it was possible that there would be a demand for reservoirs to be constructed at the rate of 200 per annum for the next 15 years. Since that report in 1980, agriculture had gone through a difficult period, but it was emerging that the demand for water was still strong. Whilst it was advantageous for improving quantity, irrigation had proved itself essential for quality crop production.

Not all these reservoirs were 'large raised reservoirs', but in his practice, of the 96 reservoirs completed, 44 had been constructed under the 1930 Act, and whilst non-impounding under that Act, many were impounding under the new definition alighted upon by the Reservoirs Committee.

According to the County Council Registers for East Anglia (Bedfordshire, Cambridgeshire, Norfolk, Suffolk and Essex) there were: 20 public water supply reservoirs (excluding service reservoirs); 14 private lakes, some of which might not be within the Act; and, 45 reservoirs used for agricultural purposes. Of these 45 reservoirs: 3 were designed by Panel I members; 2 were designed by Panel II members; and, 40 were designed by Panel III members.

It would appear that the guide lines given to interviewing committees precluded any Panel III members being admitted to either of the two relevant panels under the new Act, so who was going to build the 200 or so reservoirs per year after 1991?

First, it had to be understood that selling engineering expertise to agriculture was a very personal matter. They had to get the confidence of the farmers, since it was the farmers' money they were spending, not the FAO's, not the World Bank's, not a government's or a committee's. The economics required that, at the end of the day, when all the pumping stations, underground mainlaying, and distribution equipment had been paid for, the project could pay it's way.

Secondly, the design, specification and supervision were very different from those required for the construction of a large dam, although the end result was just as safe and stable. It was no good putting up a scheme with a specification and bill of quantities weighing more than a kilogram which brought in a price ten times the economic going rate, as one large consultancy managed to do.

Now that the Ministry had reduced the grant, and carried out no supervision, farmers went straight to land drainage contractors, who employed no engineers, let alone panel members. They would use a dozer rather than a scraper, since they had no specification and no supervision, and put up a nest of jerry built reservoirs, each just small enough to avoid the Act. The failures of slurry lagoons constructed in this manner were a matter of considerable concern to the Ministry at that very moment, and the failures of chemical storage tanks might well lead to legislation requiring containment dams as with oil tanks farms.

He would like to propose that there should be a way up for young engineers who had a feel for agriculture, probably by way of colleges like Silsoe, such that the techniques of economic specification and supervision could be taught to those who had already qualified in hydrology and soil mechanics; a way whereby they could become competent members of, say, a Farm Dam Panel.

The distinction between impounding and non-impounding reservoirs should be forgotten. It was very seldom economic to build a dam across a valley, but many agricultural reservoirs had a dual function, that of intercepting small polluted watercourses whilst natural purification took place as a result of storage. In the case of frost protection, it was often necessary to guide the melt water back to the reservoir, thus, under the new definition, making it impounding. Surely any engineer who could understand soil mechanics could master the basic hydrology, especially now that there had been so much research, e.g., the Flood Studies Report, etc. How could one design a farm dam without first making sure there was enough water to fill it or it would not impede the floodway in which it might stand? All this information was part of farm dam design, and enabled the size of the spillways to be decided.

He suggested that the size limitation on such a panel should be on embankment height rather than quantity. It was seldom that a farm dam exceeded 10 m height or 250,000 m^3 capacity.

Lastly, it should be realized that the present recommendations to the Minister which effectively eliminated Panel III would reduce the scope for engineers in agriculture in the UK. Agriculture desperately needed their skills, not only in water storage but in wellpoint, borehole, pumping station and distribution design, effluent control and land drainage. How long would it be before insurance companies demanded engineers' reports before covering the risks involved in slurry, silage and chemical storage?

Therefore, please, Reservoirs Committee, think again. The present situation did a grave disservice both to agriculture and to their profession. How could the status of engineers in the UK be respected when the Reservoirs Committee deemed that 99.85 per cent of all members of the Institution of Civil Engineers could not be entrusted to build a container for more than 25,000 m^3 of water?

Dr J. PIRT (Severn-Trent Water Authority) said that in his presentation, Mr Gimson had spoken of some of the procedural uncertainties which arose in the application of the Reservoirs Act. Mr Sharp, in his introduction to his paper, had spoken of hydrological uncertainties, particularly in relation to drought. Another uncertainty which should be introduced into the discussion related to the magnitude of the design flood.

The hydrologist working in the field of reservoir safety had two major sources of information: The Institution of Civil Engineers (ICE) guide, 'Floods and Reservoir Safety', and the Flood Studies Report*. One source covered engineering philosophy and was advi-

*Flood Studies Report, 1975, NERC, London, 5 volumes.

sory, the other covered hydrological methodology and was mandatory. However, there were sufficient uncertainties in hydrological data for one to have reservations about the application of these standard techniques. For example, if the Flood Studies Report (FSR) 'no records equation' was used to estimate the mean annual flood for all gauged catchments in the Severn-Trent area, the mean error in the estimate was 45 per cent. If the regional growth curves were then used to convert the mean annual flood to a flood of moderate return period, then these errors were often compounded. Whilst the 'no records equation' was not the technique advocated within the FSR for probable maximum flood (PMF) estimation, the question had to be asked, if one technique over-estimated by as much as 50 per cent, what of the others? Figure 1.1 compared observed maximum rainfalls in England and Wales* with Flood Studies Report maximum rainfalls, and also suggested over-estimation in the Flood Studies Report techniques.

Similarly, what of the design criteria outlined in the ICE guide, 'Floods and Reservoir Safety'? In many cases, a choice was given; for example, in the case of Category B reservoirs, the greater of the 10,000 year flood and the 0.5 PMF. Was the inference here that these two events were of a similar order of magnitude? In practice they were often very different. In some cases, the two estimates could vary by as much as 100 per cent.

He did not seek to criticize the Flood Studies Report or the Engineering Guide, but he did wish to stress the uncertainties which were apparent in extreme hydrological data. They had to take these uncertainties into account, particularly if there were economic repercussions evolving from their deliberations.

Dr D.J. COATS (Babtie Shaw & Morton) made two points, as follows:

(a) An inspecting engineer's report should include any recommendation as to measures that should be taken in the interests of safety (Section 10(3)) and whether or not the report included such recommendations had to be stated in the certificate (Section 10(5)). Further, a copy of the certificate had to be sent to the enforcing authority only if the certificate included a recommendation as to measures to be taken in the interests of safety. This raised the question of what were 'measures to be taken in the interests of safety' that should be drawn to the attention of the enforcing authority. They took the view that instructions to supervising engineers on such matters as monitoring of seepage, and maintenance or good housekeeping (e.g., eradication of moles), should not be in this category, but would the removal of trees from an earth embankment dam or the freeing of a seized scour valve be measures to be taken in the interests of safety?

(b) Owners of private reservoirs which had storage capacity not greatly in excess of 25,000 m^3 were anxious to effect discontinuance to avoid the need for employing supervisory engineers, etc.,

*Wilson, E.B. 1983 'Engineering Hydrology', 3rd Ed., MacMillan Press, 309 pp.

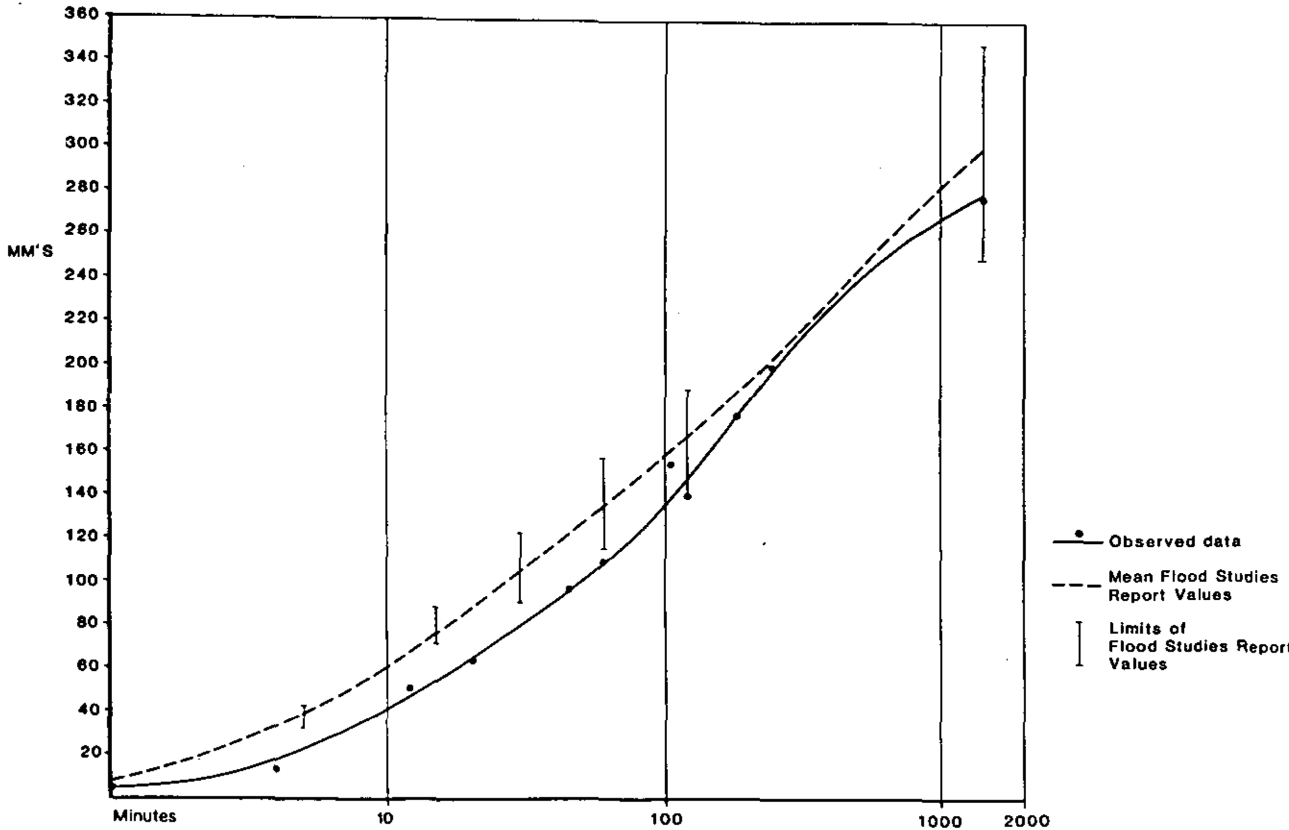

Fig. 1.1. Maximum rainfall quantity-duration relationship (England and Wales)

and, indeed, at the present time the need to register the reservoir. The engineer who advised how this could be done might be doing what the Department of Employment would wish, but if the dam was in need of repair, it came out of the Act but not out of a responsible engineer's conscience. The owners remained liable in law for the safety of persons and property below the reservoir and this should be made clear to them. Engineers might be asked to advise on necessary repairs but they might not be asked to supervise the work. Presumably engineers had then just to live with their consciences.

Mr J.D. HUMPHREYS (MRM Partnership, Bristol) sought clarification of the definition of a 'large raised reservoir'. Did the volume which the reservoir was 'designed to hold or capable of holding' include or exclude the volume above overflow level at the peak of a design flood outflow? He suggested that this should logically be included, as it formed part of the water released by a hypothetical collapse of the dam, and also represented, in general terms, one of the conditions in which failure was most likely. This was not a purely academic question, but had a material effect upon the legal status of several reservoirs and lakes with which he had been involved.

He also took friendly issue with an earlier speaker (Dr Coats) on the question of trees on the downstream shoulders of dams, which Mr Humphreys suggested probably played a useful part in removing water from the fill, and hence served as a natural underdrain, besides providing physical support.

Mr W. MACONACHIE (Scottish Development Department) explained that he had worked with the Welsh Office and the Department of the Environment on the preparation of the supporting documentation to the Act, and that he was therefore a product of the marriage between engineer and civil servant referred to by Mr Gimson. Contrary to Mr Gimson's view, however, the marriage had been a happy one.

Mr Maconachie hoped that his comments would clarify some of the points raised by Mr Gimson in his paper.

There might well be difficulties of joint ownership, but it was the undertaker who took precedence. Arguably, there could be more than one undertaker, e.g., angling, water supply, etc., but the major undertaker might well accept responsibility for the reservoir. If there was dispute or there was no undertaker, then legal advice might have to be sought. Nevertheless, the Act referred to reservoirs and not to dams, and it could be argued that, in the absence of an undertaker, owners of the solum would have equal responsibilities.

The wording 'if it is revealed by any certificate or report or is otherwise known to the authority' (Schedule 1, Statutory Instrument (SI) 1985, No. 177) had been deliberately chosen. It would surely have been unreasonable to prescribe information which the authority could not have been able to obtain or could obtain only with difficulty. In particular, the age of the dam or even its capacity might be difficult to establish.

It was said that the Departments were confused about the definition of an impounding reservoir, and that it was unusual for a meaningless term to be used in an Act and then defined outside the Act in a document with no force of law. The Act did not distinguish between impounding and non-impounding reservoirs. There was, therefore, properly, no definition in the Act. Using the powers of the Act, panels were set up which did distinguish between differing types of reservoirs. In SI 1985, No. 1086, a 'non-impounding reservoir', and therefore by implication, an impounding reservoir, was defined. The statutory instrument had the force of law.

With regard to the criticism that there was no obvious reason for the amending regulations (SI 1985, No. 548), and that they would add unwelcome expense for small undertakers, the amending regulations were legally necessary. The original SI 1985, No. 177, stated that these Ordnance Datum levels were prescribed. Unfortunately, this was not the case in the earlier statutory instrument and there was a need to make the legislation consistent. Mr Maconachie hoped that the conference would agree that not only was it desirable to have key levels above Ordnance Datum, but that it was hardly an onerous requirement.

Mr Maconachie was not sure whether Mr Gimson meant that reservoirs of reinforced concrete or prestressed concrete appeared to have been overlooked from Part 8 of the form of record (SI 1985, No. 548) or from the definition of service reservoir (SI 1985, No. 1086). In the first case, there had been a greater number of types at the drafting stage, but the number was reduced to keep things as simple as possible. The more numerous types were named. There was no question of omitting concrete dams and there was of course a box marked 'other (please specify)'. The definition of service reservoir included concrete or reinforced concrete.

Safety was not defined in the Act, but they might wish to note that the Section 16 emergency powers related to 'persons or property'.

Mr Maconachie was grateful to Mr Gimson for drawing to his attention the fact that there was no right of appeal against the recommendations of an engineer appointed by the enforcement authority under Section 16. It was doubtful that this would give rise to any significant problems.

Mr Maconachie's personal view as to whether or not silt should be included in the Act volume, was that what should be considered was what might be released in the event of a failure. If the silt was sufficiently fluid to flow then he thought that it should be included in the volume.

Mr E.T. HAWS (Rendel Palmer & Tritton) said that concerning operation of the Reservoirs Act, clearly a variety of opinions existed on several aspects of its interpretation. The letter from the Department of the Environment to panel engineers dated 26th February 1986 had been referred to in the discussion previously, and in some ways it enhanced the difficulties panel engineers faced. It drew attention to the need for discretion to keep expenditure to a scale justified by risk. It went on to refer to exceptional cases and to confirm that measures to be taken in the interests of safety had the force of law, while other measures related to good management should be listed separately, presumably without the force of law behind them. It was now pointed out that references in the Act related to the protection of persons or property and most engineers, he believed, included the dam itself under the umbrella of property.

In all these circumstances, he himself would certainly not issue any report or certificate for a dam inspection or construction which did not include all the necessary measures to ensure the integrity of the structure itself. In the absence of any indemnity from the Ministry of the Environment or the client, it would seem to him that the professional standing and, indeed, the professional indemnity insurance of a panel engineer would be at risk if a dam was certified looking only to public safety, if it then fell down without injury to persons, and if the owner then pursued the engineer under the law.

On the subject of whether naturally deposited silt was included within a reservoir volume, it might be noted that tailings dams were considered excluded from the Act. This was so even though in many cases they had supernatant water well in excess of the reservoir volume limits of the Act. In one case with which he was associated there was also impounding. In these circumstances it was difficult to conceive of silt being incorporated as part of the volume of a water reservoir.

Mr W.J. CARLYLE (Binnie & Partners) said the effect of the 1975 Act on the private owner had been the subject of debate.

Those reservoirs which came within the Act but were relatively shallow ornamental lakes presenting negligible hazard to persons or property, were a special case. Rather than argue with the enforcement authority about their registration, he had advised owners to comply with the minimum requirements of the Act and had offered a lump sum package deal to cover the inspection and report. Where the inspection and report could be completed in one day and there were no special circumstances, the fee could be less than £500. In addition to the drafting of the report, this would include the minimum of entries necessary to complete the statutory form of record.

For those old reservoirs already completed by this arrangement, there had been no records available nor drawings other than the ordnance map. Entries in the form of record had been noninstrumental survey estimates of reservoir top water level and the physical dimensions of the dam. The report included an estimate of the flood potential and the consequential effect of a flood on the spillway or on the overtopping of the dam.

In addition to the inspection, it was necessary to provide the owner with an inspecting service. He had been able to nominate qualified supervising engineers from his staff to provide this service on the basis of a minimum of one annual inspection and report. Although the report was not required unless the inspection drew attention to a matter that needed to be watched (Section 12(2) of the Act), it seemed nonetheless desirable that this minimum interval should be observed for the supervising engineer's inspection report.

Ideally, private owners should group together in a given area to try to minimize the cost of the supervising engineer's visit and report, but in any event a lump sum fee of around £300 per annum should cover this service when there were no incidents which required more frequent attention during the year.

Mr E.A.V. READING (Babtie Shaw & Morton) said that the interpretation of the definition of a large raised reservoir given in Section 1(1) of the Act had given rise to some difficulty. Section 1(1)(a) included the words 'designed to hold, or capable of holding, water above the natural level of any part of the land adjoining the reservoir ...'. The difficulty arose when the top water level was below the level of all adjoining dry ground, though above the bed level of the stream dammed. Dr Coats had on several occasions been asked to give an expert opinion, in such cases, and had always advised that 'land' meant 'dry land' and excluded the bed of the stream. If that was not a correct interpretation then many weirs on rivers must come under the Act and this was surely not the intention of the Act. The Act itself did not contain any other passages which might help in resolving this difficulty over interpretation, but there were references to 'the lowest natural ground level' (not precisely the same phraseology) in Statutory Instrument (SI) 177 and in SI 468:

(a) SI 177 C1.2 defined the maximum height of the dam as from 'the lowest natural ground level at the toe (including stream bed) ...'. Some argued that the addition of '(including stream bed)' in this definition by implication applied also to Section 1(1)(a) of the Act, but Dr Coats considered that the definition in SI 177 C1.2 was limited to definition of the height of the dam.

(b) Schedule 3 of SI 468 defined 'lowest natural ground level' as meaning 'where there is a watercourse below the dam, the lowest bed level of that watercourse'. Schedule 3, C1.8(f), referred to information to be given to the enforcement authority regarding 'capacity (in cubic metres), measured from the lowest natural ground level adjacent ...'. It could be argued that this implied use of the stream bed level for the natural level of any part of the land adjoining the reservoir in Clause 1(a) of the Act.

In a number of cases, the decision as to whether or not a reservoir came within the scope of the Act hinged on the interpretation of this definition, so an authoritative ruling would be helpful.

Mr A.G. COCKBURN (Tayside Regional Council) asked if he might make some comments as a representative of both a water undertaking and an enforcement authority. His Council had placed advertisements on two occasions in the local press asking owners of dams and reservoirs in Tayside to get in touch with the Council to confirm their ownership of reservoirs and to produce any documentation, drawings, reports or certificates or, indeed, any evidence of the capacity of any reservoirs they owned. The response to these advertisements was an overwhelming silence. Therefore, his Department had ordered 1:10,000 Ordnance Survey sheets of the Tayside area and had identified some 170 bodies of water as being worthy of examination in terms of their possible inclusion in the required Register.

About 25 water bodies had already been excluded by virtue of their size, surface areas of the order of 3,000-4,000 m^2 or below having been discounted. Of the remaining 145 or so, complete Register entries had been possible in only 37 cases, these being the reservoirs in the hands of the undertakers such as the North of Scotland Hydro-Electric Board, the Forestry Commission and other public bodies, including themselves, together with one private owner only, significantly, this being a water supply reservoir sold out of service with complete records.

In the case of the remaining 100 or so sites, responses had been made by rather fewer than half the proprietors but, in general, their replies had been helpful. Visits had been made to a number of sites and it had been possible, variously, either to satisfy the Department that the water was a natural loch, or the proprietors that they had a large raised reservoir. He would expect that perhaps a further ten sites would turn out to be natural formations and a number of others prove to be under 25,000 m^3 capacity.

It was of interest that a number of proprietors had made the point that where the reservoir was remote, and would discharge only onto the proprietor's own land, they felt that the degree of risk posed by the possible collapse of the dam should be taken more fully into account.

In general, the availability of records or drawings had been absolutely minimal, although at a number of the sites visited it was obvious that the structure had been fully designed and not merely thrown together. In connection with the dearth of records, it had been suggested to him that records of dams built in the decades around the turn of the century were probably held in Registry House, Edinburgh, but it was known that they were not fully catalogued.

It was known that a number of proprietors had engaged inspecting engineers, and well known names had been quoted, but he suspected that pressure of work might well delay completion of the appropriate reports, and thus their Register entries, for some considerable time.

Thus, he did not share the optimism of Mr Agnew who had indicated that he anticipated that Registers should be fairly complete by the time they had to be submitted to the Secretary of State for the first time on 1st April 1987. Indeed, as far as Tayside was concerned, he would consider it most unlikely that the Register could be anywhere near complete by that date.

Mr K.T. BASS (Rofe, Kennard & Lapworth) wrote that during the conference a number of doubts and uncertainties relating to interpretation of the 1975 Act and the booklet 'Floods and Reservoir Safety: An Engineering Guide' were expressed. There appeared to be hopes of changing the legislation on a number of points, e.g., to include small reservoirs with a high risk, and allow for large raised reservoirs of low risk to be removed from the control of the Act, etc.

It was suggested that these doubts caused problems for panel engineers, but was this so? The risks were not changed by the Act or the guide, so one would imagine that the advice given to a client would be the same whether these documents existed or not. Such an attitude should remove most difficulties especially as they were informed that the guide was not mandatory and they were also implored to make special cases of certain reservoirs which provided amenities only. It had to be assumed that Parliament did not expect all risks to be removed because canals were expressly excluded - a line had to be drawn somewhere. There was little doubt that they would have to live with the 1975 Act for many years.

Mr R.B. BINNIE (Crouch & Hogg) wrote, regarding discontinuance or abandonment, that under Section 13 of the Act, a procedure was prescribed for the removal of a discontinued reservoir from the register of large raised reservoirs after certification under Section 13(2). No parallel procedure was laid down for an abandoned reservoir, even when breaching rendered it incapable of storing any water above the natural level of the adjacent ground. Should abandonment by breaching be regarded as a form of discontinuance and a certificate be issued under Section 13(2) to allow the removal of the by then non-existent reservoir from the register? The last sentence of Section 14(2) seemed to imply this procedure, which would in effect convert an abandonment into a discontinuance. He asked if Mr Agnew could clarify the procedures in relation to paragraph No. 30 of his paper and to Mr Gimson's comments under this heading on page 41 of the printed papers.

Mr M.E. BRAMLEY (Construction Industry Research and Information Association) wrote that the verbal discussion on the matter of flood storage above the lowest level of any overflow sill being taken into account in the assessment of stored volume of water failed to note the many flood storage ponds which were rightly classified as large raised reservoirs even though the retained storage was normally less than 25,000 m³. Normal top water level was generally controlled by a low level overflow which deliberately restricted outflow in flood events in order to achieve flood retention. In many cases this overflow was sufficiently large for blockage to be unlikely. Engineering judgement almost invariably concluded that the maximum volume retained during an appropriate design flood was taken into account in the classification under the Act, particularly as the pond was often located (by definition) close to a conurbation.

Mr D.D. FRASER (Babtie Shaw & Morton) wrote that Mr Agnew's paper reminded them that the design floods for the categories of reservoirs set out in the Institution of Civil Engineers (ICE) Engineering Guide were not mandatory but were to be applied with discretion by panel engineers.

The spill provisions at some older Category A reservoirs could not cope with, say, a 10,000 year flood, and modification for this purpose could involve appreciable costs and/or loss of storage capacity. By comparison, discharge of, say, a 500 year flood would probably be quite feasible. This raised the question of what was an appropriate level of acceptable risk. Were they not being unduly cautious in selecting probable maximum floods (PMFs) or 10,000 year floods as their design standard and did not this represent a degree of risk far below that commonly accepted in their ordinary daily lives? For example, surely the risk of being killed or injured in a road accident had to be greater than the chance of drowning in a reservoir disaster, yet such risk was commonly accepted and indeed the growth of road traffic continued inexorably.

In these circumstances, the application of discretion by a panel engineer was not easy, and any guidance that could be given by Mr Agnew would be helpful.

Mr S.J. GOODE (British Waterways Board) wrote that in April of this year the British Waterways Board had written to the Institution of Civil Engineers seeking advice on two particular aspects of the Act that were causing them concern. In the absence of a reply from the Institution, he would be most grateful to have Mr Gimson's comments, or indeed Mr Agnew's, on the two queries that were raised. For simplicity, the main body of the letter was reproduced, as follows:

'... One matter which has been raised by a number of our supervising engineers is the question of how they should interpret Clause 20(4)(e) of the Act and I have agreed to contact you on their behalf to obtain some guidance.

The particular requirement which causes them, and ourselves, concern is that they are obliged to notify the enforcement authority of any advice that they may give which 'recommends' that we '... have the reservoir inspected under Section 10 above or to take any other action ...'.

Despite the endeavours of our operations staff, I am quite sure that there will be occasions when a supervising engineer visits a reservoir and finds that there are matters which require attention. These matters could well include the cutting of grass on the downstream face of an embankment dam, repairs to dislodged or damaged pitching, repair of valves or other maintenance works. The Board would, of course, expect him to bring these matters to their attention in his written report and advise them to take the necessary action. Strictly speaking, this falls within the category of taking 'any other action' but I doubt that the enforcement authority would be particularly grateful at being inundated with such reports.

Are the supervising engineers to use their discretion in deciding which of their recommendations are to be reported to the enforcement authority? If so, then I believe they should be given some guidelines to work to and this may best be done through yourselves.

It may be that I have missed the obvious reason for the inclusion of this phrase in Clause 20(4)(e) but I would be most grateful for your advice on the matter.

At the same time I would appreciate your comments on Clause 10(6) of the Act. Is it the intention that supervising engineers will be able to act as 'qualified civil engineers' for the purpose of this particular sub-clause? If not, then I believe that paragraph 5 of the DOE circular to panel engineers dated 26th February 1986, if applied to all reservoirs within the ambit of the Act, may leave inspecting engineers in something of a dilemma. Repairs to a wave wall, for instance, may well be considered necessary in the interests of safety, rather than desirable, but supervision and certification of such recommendations could quite effectively be carried out by supervising engineers.

There may, of course, be other, more essential, measures for which certification by a supervising engineer would be inappropriate, but it could be left to the inspecting engineer at the time of writing his statutory inspection report to decide which of these two categories his recommendations in the interests of safety fall into ...'.

As an aside, Mr Goode was sure that Mr Gimson would not be surprised to hear that there were indeed reservoirs which had been constructed without any purpose-made overflow. The Board owned such a reservoir, of the non-impounding type, which was constructed around 1840 and had an earth embankment with a maximum height in excess of 10 metres and a length of approximately 500 metres. The reservoir was filled via a side weir from the adjacent canal with the reservoir level being controlled by the difference between this inflow and the rate of abstraction from the reservoir to supply water to the nearby Water Authority treatment works.

Mr T.A. JOHNSTON (Babtie Shaw & Morton) wrote that, like all other engineers, his experience of the Act was limited. However, having undertaken five statutory inspections under the Act, he had become aware of areas of uncertainty regarding the Act's requirements.

He suggested that it would be reasonable for the Department of the Environment to issue guidance on these topics and that this would be of value to the enforcement authorities, the undertakers and the panel engineers. Procedures for issuing technical memoranda and notes for guidance were well-established in the Department, and so it should be possible to devise a format to meet the present need.

No doubt those who drafted the Act intended that its application should be uniform and so every reasonable effort should be made to provide authoritative guidance on certain terms (e.g., at present there was not unanimity on which measures should be described by the inspecting engineer as 'measures in the interests of safety') and certain procedures (e.g., the extent to which a report on a statutory inspection might deal with topics other than those listed in Regulation 4 was the subject of differing views).

Dr D.W. REED (Institute of Hydrology) wrote that during the discussion at the conference, Mr Pirt had referred to hydrological uncertainty. It was indeed true that flood estimates, made from catchment and climate characteristics alone, might be 50 per cent out, but did it follow that they should sack the hydrologist and return to pre-Flood Studies Report methods? He thought not. Surely, where major expenditure was indicated to meet, as Mr Humphreys had said 'the absurd flood that the hydrologist tells', the way forward was to seek to improve the accuracy of the flood estimate. Collection and analysis of local flood event data, and a detailed appraisal of the particular soils, were pertinent techniques (see Flood Studies Supplementary Report No. 13, 'Some suggestions for the use of local data in flood estimation').

Given the 2,500 or so large raised dams identified by Mr Agnew, one might reflect on the probability of a design flood exceedance occurring at one or other of them. If each were designed to a 10,000 year event, would one expect a design exceedance somewhere about every four years? The flaw in this conjecture was the assumption that rainfall at one site was statistically independent of rainfall at another. The distribution of reservoirs around the UK was far from uniform, with some dense clusters (notably in the Pennines). A research project, funded by the Department of the

Environment, at the Institute of Hydrology was investigating the spatial dependence of extreme rainfalls. If spatial dependence was significant, as the analysis indicated, this might explain the apparent low frequency of design exceedances at reservoired sites. A necessary corollary was that when an exceedance did occur, it was possible that several reservoirs would be affected simultaneously.

All this might be very obvious, but he felt it was worth stating against those who argued that because they had not experienced a catastrophic design exceedance in the last few decades their flood standards were necessarily over-conservative.

Mr J.E. ROSE (Anglian Water) wrote that Mr Sharp, in his presentation, used the term 'operational deployment' of reservoirs in relation to obtaining the maximum use of low cost resources. This in his experience led to pumped storage reservoirs being held for long periods at or near their top water levels, while impounding reservoirs affording gravity supplies had greater and faster water level fluctuations than might have been considered during their original design. As both sets of circumstances could be detrimental to the behaviour and safety of earth dams, he asked if Severn-Trent applied specific operational controls to individual reservoirs to mitigate these effects.

Authors' Replies

Mr AGNEW, replying to the discussion, thanked Mr Hawes for the information provided about the numbers of reservoirs built in recent years for farmers and the projection for future requirements. There had been considerable debate about the number and constitution of the panels set up under the 1975 Act, and it had been agreed that the situation would be monitored over the next few years to determine if any change in the panel structure for non-impounding reservoirs might be necessary.

In answer to Dr Coats' question on discontinuance (Section 13), Mr Agnew said that this again was an instance where inspecting engineers were required to use their judgement. Similarly, inspecting engineers were in the best position to take their own decisions as to which measures were 'in the interests of safety' when compiling their inspection reports. As to the question of an enforcement authority monitoring the work of supervising engineers, Mr Agnew pointed out that apart from supervising engineers having specific duties under the Act, the criteria for appointment to the panel included maturity and a responsible attitude.

Replying to Mr Humphreys, Mr Agnew said that it was quite clear that for the purpose of defining a 'large raised reservoir' it would be reasonable to calculate the capacity on the basis of the top water level as defined in the relevant statutory instrument. The water level used for purposes of design was a different matter. The Department of the Environment letter of 26th February 1986 simply reinforced the guidance given previously by the Institution of Civil Engineers (ICE) as to the discretion available to inspecting engineers; there would have been no need for the letter had there not been indications that this discretion was not being used. Mr Agnew said he could not comment on the criteria for appointment to the All Reservoirs Panel, but so far there was a steady flow of new applicants with appropriate experience.

Mr Agnew said that he welcomed and entirely accepted Mr Carlyle's approach, which was wholly consistent with the use of discretion by inspecting engineers.

In reply to Mr Reading's question about interpretation of the definition of a large raised reservoir in Section 1(1), Mr Agnew suggested that account had also to be taken of the wording in Section 1(2) '... where water is artificially retained to form or enlarge a lake or loch ...'. He personally agreed with the 'dry land' interpretation advocated by Dr Coats, and suggested that what was important was the volume that would escape. He also suggested that account be taken of the dictionary definition of a reservoir, namely, '... where anything is kept in store'.

In reply to the written contribution by Mr Bass, Mr Agnew commented that the Act needed to be operated in a practical way and that reliance was placed on panel engineers using their discretion. It would be counter-productive to define everything. Mr Agnew suggested that legislative change would be premature, pending operational experience of the Act. In relation to flood storage ponds, referred to by Mr Bramley, Mr Agnew suggested that what counted in assessing storage capacity for the purposes of the Act was the maximum volume retained below the main, or upper, overflow.

In replying to Mr Binnie's request for clarification of the procedure for discontinuance or abandonment, Mr Agnew said that the primary purpose of Section 14 was to require an undertaker to take precautions in advance of abandoning a large raised reservoir. The Section implied that an abandoned reservoir was one that was no longer to be used as a reservoir, i.e., it would not be used for storing water, but would be emptied and kept empty. In acting under Section 14, a qualified engineer was required to report as to the measures (if any) that ought to be taken in the interests of safety to secure that the reservoir was incapable of filling accidentally or naturally with water above the natural level of any part of the land adjoining the reservoir, or was only capable of doing so to an extent that did not constitute a risk. In the case of a non-impounding reservoir, no measures might be necessary other than the initial emptying and leaving a scour valve open. The same might apply in the case of an impounding reservoir with a very small catchment and a large diameter scour. If, however, an undertaker wished to carry out alterations to render a reservoir 'incapable of holding more than 25,000 m^3 of water ...' they had to proceed under Section 13. Sub-section (2) of Section 14 read in conjunction with Sub-section (1) of Section 13 might be held to imply that any alteration to the reservoir would be for the purpose of rendering it incapable of holding more than 25,000 m^3 of water. If a reservoir was so altered, a certificate could be given under

Section 13(2) and the reservoir removed from the register under Section 13(3). There might, however, be cases where, on grounds of economy, an inspecting engineer recommended measures under Section 14(1) that involved alterations that would render a reservoir only capable of filling to an extent that did not constitute a risk, but would still leave it capable of holding more than 25,000 m^3 of water. In such circumstances the reservoir would still be a large raised reservoir and could not therefore be 'discontinued'. Where an undertaker wished to abandon a reservoir and have it removed from the register, they could proceed entirely under Section 13 without obtaining a report under Section 14(1). Sub-sections (4) and (5) of Section 14 gave an enforcement authority power to enforce Sub-sections (1) and (2). They were additional to the emergency powers in Section 16(2) and provided a means to deal with improperly abandoned reservoirs where immediate action was not necessary.

While pointing out that a decision on whether to require provision of a scour through an existing embankment was a matter for the inspecting engineer's discretion, Mr Agnew agreed with Mr Eadie (see page 52) that an acceptable alternative could be to require the undertaker to have a plan for alternative means of lowering the reservoir in an emergency.

In answer to Mr Fraser's question about the degree of risk posed by reservoirs, Mr Agnew commented that while people voluntarily accepted the risks of crossing a road or travelling in a vehicle, the risk from a reservoir was outside an individual's own control. Also of relevance to Mr Fraser's questions were the written comments from Dr Reed on design flood exceedance. Although there were apparently no records of recent design flood exceedance on reservoired catchments, the hydrological data in Volume 1 of the Flood Studies Report would suggest that there had been a number of extreme floods in the UK this century. Because of the effect of site specific circumstances, Mr Agnew did not think it would be appropriate to offer guidance on the application of discretion by a panel engineer beyond that recommended in the ICE Engineering Guide.

In answer to Mr Haws, Mr Agnew suggested that the interpretation of the words 'protection of ... property' was a matter where, again, panel engineers had to use their discretion. Might there not be circumstances where an escape of water might give rise only to risk to property owned by the undertaker? In such circumstances would inspecting engineers not be able to protect themselves by reporting the state of the dam and advising the undertaker on any measures necessary to secure its continued existance, without necessarily making recommendations 'in the interests of safety'? On Mr Haws' point about silt, Mr Agnew agreed with the view expressed by Dr Coats and others that the test was the volume that would escape, i.e., would the silt flow or remain?

Mr Agnew would not favour Mr Johnston's suggestion for the issue of further central guidance on the particular topics he had mentioned. Schedule 2 of Statutory Instrument (SI) 1986, No. 468, used the word 'include', so other topics would not be precluded. Which measures were 'in the interests of safety' would depend on circumstances and to issue guidance would affect the use of discretion by the inspecting engineer.

Mr Mee (see page 47) and Mr Morison (see page 55) raised the question of ownership and responsibility for reservoir safety. The Act placed responsibility squarely on the 'undertakers' as defined in Section 1(4). Therefore, in the event of a dispute, Mr Agnew's personal view was that the person (or persons, if more than one) making use of the stored water, rather than the owner of the ground on which the dam was situated, was responsible for the safety of the reservoir. In the absence of any person using the stored water, Mr Agnew agreed with the decision of the Quarter Session's case cited by Mr Morison, that in this particular case the owner of the land beneath the reservoir would be responsible. Mr Morison's suggestion that a record of case law should be compiled was well worth considering, but whom should do it was very much an open question.

Mr GIMSON, in reply to discussion, said that he was grateful to Mr Maconachie for his helpful contribution resulting from his work on the supporting documentation to the Act. It was certainly right to emphasize that it was the undertaker, wherever one existed, rather than the owner, who had responsibilities under the Act.

However, Mr Maconachie's remarks did not in general alter Mr Gimson's view that there were loopholes in the legislation and/or the documentation, some of which could lead to unnecessary disputes. Although it was correct to say that the terms 'impounding' and 'non-impounding' did not appear in the Act itself, the definition in Statutory Instrument (SI) 1985, No. 1086, applied strictly only to that SI, whereas one or other of the terms appeared in more than one SI of an earlier date. Worse still, a Department explanatory circular on the documentation used a different definition.

Mr Gimson still took the view that the unquestionable importance of impounding reservoirs and the dangers they might present was sometimes allowed unreasonably to obscure what might be greater dangers from apparently lesser reservoirs. Mr Maconachie had taken up remarks made by Mr Johnson concerning the relationship between reservoir capacity and safety and also covering risk investigation. One reservoir in the country was currently the subject of a Government sponsored project, the object of which was to find a consistent method of evaluating the risk posed by reservoirs and identifying and quantifying the factors causing the risk.

Although it would no doubt be a long time before the results could be used on a large scale, Mr Gimson continued to think that the legislation should have embraced reservoirs smaller than 25,000 m^3 and that these should have a simple form of statutory inspection to make an assessment of the risk, if any, to the population below. Inspecting engineers would determine what frequency of further visits, if any, by a qualified engineer was desirable, and in particular they would send their reports to the local planning authority who might in the future be considering development downstream. The difficulties of this approach which Mr Maconachie said would have to be overcome did not appear to be sufficiently serious to permit them to ignore small reservoirs on steep, heavily-populated hillsides.

Mr Goode was concerned that the requirements of Section 20(4)(e) would result in the inundation of the enforcement authority by reports from the supervising engineers covering matters which might be thought too trivial for their attention. Mr Gimson disagreed. He did not expect the supervising engineers to need to inspect a reservoir more often than twice per year except in special cases. It seemed unlikely that most reservoirs would justify a report on every occasion and it should be no trouble for the enforcement authority to file copies away in case of need.

Mr Goode cited the cutting of grass as one of the possible low key requirements of the supervising engineer. In Mr Gimson's view, the supervising engineer should insist before undertaking the inspection that the grass was short enough to reveal all possible faults.

Mr Goode also drew attention to Section 10(6) of the Act, which required that if an inspecting engineer recommended measures to be taken in the interests of safety, those measures had to be supervised by a qualified civil engineer. He contrasted this with the Department of the Environment's circular of 26th February 1986 to all panel engineers, which had caused much comment by effectively requesting leniency in situations where safety would not be endangered.

Whereas Mr Gimson agreed that there was an inconsistency which would better have been avoided by more careful drafting in the first place, he agreed with Mr Goode that inspecting engineer's reports could and indeed should, distinguish between two sorts of recommendations in the interests of safety. The first would cover work necessary for safety in the short term, which work had to be supervised by a qualified civil engineer as a matter of reasonable urgency and certainly before the next inspection. The second sort would cover measures which inspecting engineers felt were desirable for long-term good and which were simple enough to be supervised by the supervising engineer. If, at the next inspection, such improvements had not been carried out, inspecting engineers might well be more onerous in their requirements.

Mr Gimson thanked Mr Goode for his information on a reservoir without an overflow, and he was pleased that without prior knowledge he had guessed correctly as to the probability of such an occurrence.

In reply to Mr Pirt's words of warning on the uncertainties of extreme hydrological data, a non-hydrologist such as Mr Gimson could not respond adequately. All Reservoirs Panel Engineers were usually well aware of the reservations with which these matters had to be approached. The Guide to the Flood Studies Report itself advised that records had to be collected during the planning of a new reservoir, so that sole reliance on the use of the 'no records equation' was obviated.

Mr SHARP, in reply to Mr Rose, said he was quite right when he said that in maximizing the use of low cost sources in an interlinked system there was a tendency for low cost gravity sources to be drawn on more, and far greater water level fluctuations to occur, than would arise with 'normal' use as a single source. Conversely, more expensive pump filled reservoirs, or those with costly treatment, or pumping of the water supplied, were used only in more exceptional circumstances, thus these reservoirs remained more or less full for much of the time. This latter position was also true of river regulating reservoirs, whether built as such from the outset or converted to this use. These trends were very much part of the theme of Mr Sharp's paper.

Mr Rose asked whether Severn-Trent applied specific control rules to mitigate any effects that might arise either from rapid changes in water level or from long periods of full retention of storage. The answer was that, so far, they had not included any specific controls or rules to meet these conditions. The view to date had been that rapid changes in level for operational reasons were most unlikely to exceed rapid drawdown criteria needed for safety reasons in an emergency, and long periods at or near top water level should not be a condition exceeding design provision.

Mr Rose's point was a good one, and indeed, the sort of problem that Mr Sharp's paper attempted to air.

SECOND SESSION: DESIGN ASPECTS OF DAMS

Session Chairman:

E.T. HAWS, Chairman, BNCOLD

Papers:

4. Misconceptions in the Design of Dams, by A.D.M. PENMAN, DSc, FICE, Chartered Civil Engineer; Geotechnical Engineering Consultant

5. Gates and Valves in Reservoir Low Level Outlets - Learning from Experience, by J. LEWIN, FICE, FIMechE, FIWES; Partner, Lewin, Fryer and Partners; and J.R. WHITING, BSc, MICE, MIWES; Associate, Lewin, Fryer and Partners

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DISCUSSION OF SECOND SESSION

Authors' Introductions

Mr LEWIN and Mr WHITING introduced their paper and supplemented it by dealing with air and vapour cavitation, some constructional features of butterfly valves and ring follower valves, and some aspects of hollow jet and needle valves.

Air Cavitation

Under conditions of two phase flow there was the possibility of air cavitation occurring. This was brought about by dissolved air in the water coming out of solution where there was a drop in fluid pressure. This could occur with explosive force. Although this was not as destructive as vapour cavitation, air cavitation could still cause major problems. The possibility of other gases entering the system, such as methane or hydrogen sulphide from decomposing vegetation, should not be overlooked. Since the solubility of air in water was approximately proportional to pressure, considerable quantities of dissolved air might be present at low level outlets.

Vapour Cavitation

In butterfly valves, cavitation occurred as a result of the pressure drop which resulted from flow acceleration in the restricted sections between the valve door and the conduit. Once the minimum pressure head fell below the vapour pressure, cavitation would occur on the downstream side of the valve disc. This condition arose during the emergency closure of a valve where the initial operating state with a positive back pressure changed into one of complete separation of flow. That cavitation could occur in highly sheared flow had been demonstrated. This was a reason why submerged energy dissipators of the Mica Dam type had to be approached with caution.

Butterfly Valves

Butterfly valves were in general not suitable for flow control, only as on/off devices, because of flutter of the blades and eddy shedding from the blade tips. To prevent cavitation, a back pressure was required and this was achieved by reducing the size of the terminal discharge valve compared with the butterfly valve acting as a back-up valve.

The lattice blade valve, also described as a through flow valve, had a lower head loss and a more rigid blade assembly. It was sometimes claimed that lattice blade valves were more prone to obstruction by debris. This did not appear to be the case with the larger valves.

Three different arrangements of the circumferential blade seal were employed. The one most frequently used mounted the seal on the blade with the upstream head pressurizing the seal. Alternatively, the elastomeric seal could be incorporated in the valve body or metal to metal sealing was employed. There was usually some leakage at the seals. The areas around the blade pivot axles presented the greatest problem. Some leakage rate was usually specified by valve manufacturers. Jets of water from seal leakage should not be accepted in valves which were for long periods in the closed position.

Ring Follower Valve or Gate

The ring follower valve or gate was used in similar positions and applications as a butterfly valve. It was circular with the slide section twice the depth of the opening. The upper section was blank and provided the bulkhead for shut off. The lower section was a circular opening. Its main advantage was that it provided a clear opening without slots. It could close against flow. The cost of a ring follower gate was approximately 2 to 2½ times that of a butterfly valve, and the civil engineering cost associated with the installation of a ring follower gate was also appreciably greater because the gate was over three times the height of the fluid passage. Means of flushing the low section of the body of the gate had to be provided.

Discharge Valves

The hollow jet valve, which could be described as a needle valve with the interior section terminating as a cone, was a terminal discharge and control valve. The jet was compact and there was significantly less energy dissipation. It was manufactured in similar sizes and for similar heads as the hollow cone valve. The valve operating mechanism was difficult to access for maintenance and the valve was more liable to blockage than the hollow cone valve.

The needle valve could be used for in-line regulation and terminal regulation at heads up to 200 m. The limitations previously mentioned for the hollow jet valve applied. The valve was manufactured in sizes up to 2.0 m diameter.

Hollow cone and hollow jet valves had been used submerged, as well as the sleeve valve. The latter was similar to the hollow cone valve but the sliding sleeve was internal to the valve body. In such applications the valves were no longer energy dissipating devices. Energy dissipation took place in the stilling basin and the valve acted only as a flow control device. Turbulence within the stilling basin could cause vibration of the valve.

Trashracks and screens had been excluded from the paper because they did not usually form part of a bottom outlet. Where bottom outlets were controlled by gates only, a trashrack was generally not provided, since clearing of screens was difficult to arrange at low level intakes.

When a bottom outlet took the form of a hollow cone valve with a butterfly back-up valve, the probability of flotsam getting wedged across the vanes of the hollow cone valve was greater. Smaller valves were more vulnerable than larger ones and this applied equally to butterfly valves. However, most installations did not incorporate screens.

Where hollow jet valves or needle valves were used, screening was essential.

Discussion

Mr G.A. COOPER (Ferguson & McIlveen), in opening the discussion, described remedial works that were currently in hand at Altnaheglish reservoir, where an existing mass concrete gravity dam had shown increasing uplift pressures on its foundations in recent years, and had been kept drawn down in level. The dam had been constructed by direct labour between 1930 and 1934, and the reservoir was first filled in November 1934 and officially opened in November 1935.

The reservoir was formed by a mass concrete gravity dam 110 m long at the crest and curved in plan to a radius of approximately 153 m, with the intention of developing some degree of arching against the abutments. The dam was founded on soft mica schist bedrock and aggregate from an adjacent gneiss quarry was used for concrete production on site. It was built to a height of 42 m above foundation level, making it currently the highest dam in Northern Ireland. The headworks fed from the dam had operated continuously over the last 50 years, providing a reliable gravity supply to Londonderry and the surrounding districts.

Spalling of the downstream face of the dam was first reported in 1952 and increased spalling in 1959. The dam was investigated between 1960 and 1964, and newly installed piezometers were found to indicate high uplift pressures in the dam foundations. Careful monitoring also indicated that the dam's abutments were not sufficiently rigid to provide any structural benefit from its horizontal arch profile.

Stage grouting of the entire upstream face and foundations, together with drainage improvements, were carried out, but were found to have little effect on reducing water pressures in the underlying rock. The unusually high pressures acting on the base of the dam meant that under the worst conditions, tensile stresses occurred in the upstream face, and consequently there was a risk of cracking leading to further increases in pressure and progressive failure. It was eventually decided to lower the spillweir level by 1.9 m to improve the stability of the dam, and this work was completed in 1967. The lowering of top water level (TWL) reduced the usable storage from 2,279 Ml (500 mg) to 1,760 Ml (387 mg) and this position remained unchanged until 1981.

Due to continuing concern about seepage into the inspection gallery and through the dam to the downstream face, Ferguson & McIlveen were asked by the Department of the Environment to report on the overall condition of the dam and its long-term stability.

Concrete cores were drilled, new piezometers installed, and the downstream drainage outlets refurbished, following which uplift pressures were monitored to enable the stability of the structure to be re-assessed. It was found that uplift pressures had again become excessive, and that factors of safety against both sliding and overturning were unacceptably low. In order to increase such factors to normally accepted values it was considered that the top water level needed to be reduced by a further three metres below the already lowered level, which then limited the reservoir capacity to only 1,135 Ml (250 mg), or half of the original design figure.

Following further site investigations, a detailed report on possible methods of restoring the stability of the dam with the reservoir filled to its original TWL was prepared, taking account of the up-to-date design flood standards.

Various methods were investigated as follows:

1. A new grout curtain and pressure relief system constructed from within the inspection galleries.

2. A new upstream cut-off trench continuous with a new upstream face on the dam.

3. Post tensioning from the dam crest into the bedrock foundations.

4. An additional mass concrete support cast against the downstream face.

5. An additional mass concrete weight applied to the reconstructed dam crest and incorporating an air-regulated siphon spillway.

6. A rockfill support against the downstream face, with repositioned spillweir and spillway channel.

Two further alternatives also considered, not involving improvements to Altneheglish Dam, were the construction of a new dam in an adjacent valley or the construction of a replacement dam a short distance downstream of the existing structure.

It was found that the most economical and satisfactory method was a combination of methods 1 and 6, and accordingly, designs were prepared for the following works (Fig. 2.1):

(a) An inclined grout curtain constructed from within the existing transverse gallery to supplement the original vertical grout curtain.

(b) A series of pressure relief wells in both the original (upstream) and new (downstream) transverse galleries.

(c) A rockfill berm placed against the entire downstream face of the dam to provide additional support and weight.

Associated with the above works was a new side overflow spillweir on the north abutment to discharge the probable maximum flood around the enlarged structure, the extension of the existing eduction and scour galleries under the new berm, and a bridge and access road onto the new downstream face. The dam crest would be reconstructed over the existing central spillweir section, giving a continuous profile consistent with the remainder of the dam.

Provision was being made for the installation of various instruments to monitor movements, uplift and pressures and seepage flows so that the behaviour of the dam could be checked both during and after the remedial works.

Mr G.P. SIMS (Engineering & Power Developments Consultants Ltd) said that in congratulating Messrs Lewin and Whiting on their presentation, he would like to invite them to comment further on three aspects of the design of gates and valves.

The first question referred to Mr Whiting's observation that the low level outlet on the Dartmouth Dam in Australia had not fulfilled its designers' intentions. He believed that the Dartmouth outlet was strongly influenced by the design used at Mica Dam, which he understood to have been successful in directing the jet in such a way that the collapse of cavitation cavities occurred within the body of the water and not on the surface of the conduit. The Dartmouth outlet design was developed with the help of a hydraulic model in which the cavitational behaviour was reproduced, and he would be interested to have the authors' views on the deficiencies of that design and how these might have been avoided.

His second question related to the design of the seals of gates and valves. Was there any guidance the authors could give

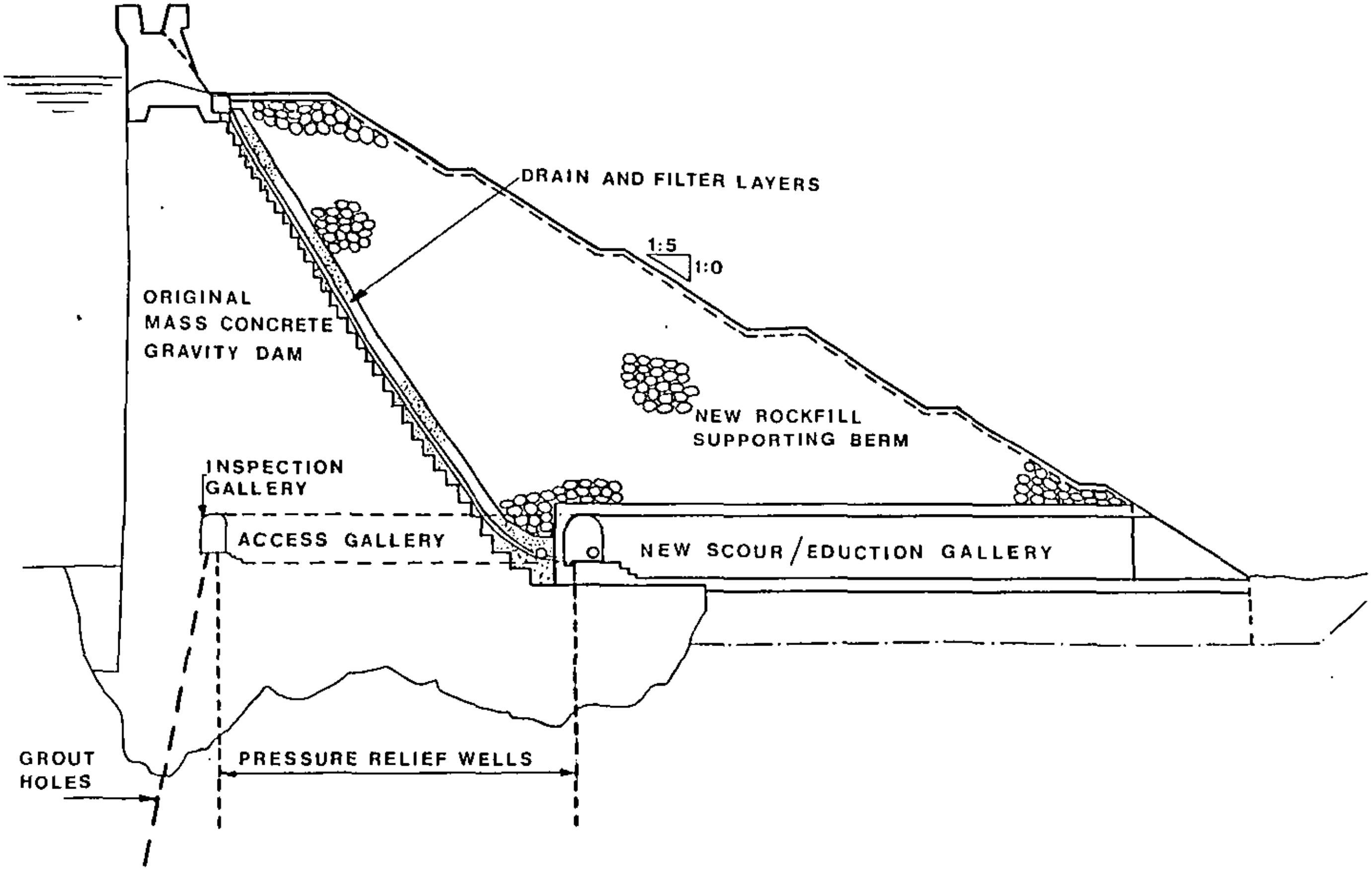

Fig. 2.1. Altnaheglish Dam

on the design of these seals to minimize the likelihood of hydraulic resonance of the water columns within the conduits controlled by such gates?

Third, would the authors like to comment on the philosophy of providing back-up systems to gated spillways to enable them to be operated in extremis. His question related principally to gates in remote locations in poor countries where doubt might exist on the reliability of a standby diesel plant, either because of poor maintenance, or because key components had been removed. Would the authors agree that all spillway gates should be designed to be opened manually in extreme conditions?

Mr W.T. MEE (Sir Alexander Gibb & Partners) said that his comments arose from references by Mr Sims to vibration.

The paper by Messrs Lewin and Whiting had dealt with aspects of gate design which could lead to gate vibrations. However, vibrations could result from a number of other causes. The 120 ton control gate at an intake which had a clear opening of 10 m x 10 m at the gate position was suspended immediately above the waterway under normal operating conditions by means of long rods. After commissioning the station it was noted that over a certain range of machine loading conditions the gate vibrated with a frequency of just under 2 Hz and a maximum amplitude of about 5 mm, the gate and its linkage acting like a spring. If allowed to continue, the vibration could have resulted in suspension rod failure.

The underside of the gate was subjected to eddies arising from the main vertical splitter and subsidiary horizontal screen supporting beams, which between them produced a wide range of frequencies for single and double eddies which included the gate vibration frequency. It was also possible to prove that the penstock downstream of the gate could have a frequency resonant with the gate frequency thereby amplifying the eddy forces acting on the underside of the gate. It should be appreciated that the range of the calculable frequencies was very large. Once the initiating mechanism had been identified, gate vibrations were eliminated completely by providing a low horizontal damping force on the very top of the gate (see Fig. 2.2).

Mr Mee also commented on Dr Penman's assertion that it would be possible to delay a decision about grouting until after a dam had been filled. Very many dams incorporated drainage as a design feature even if the need for grouting was uncertain; in many cases it would not be possible to grout after the drains had been drilled. It was a very important aspect of the construction programme that the grouting near to the drains had to be completed before the drains themselves could be drilled. A further aspect which affected particularly concrete dams was that once the reservoir was filled, the phreatic gradient caused seepage increases, adding to the problems of grout washout during injection.

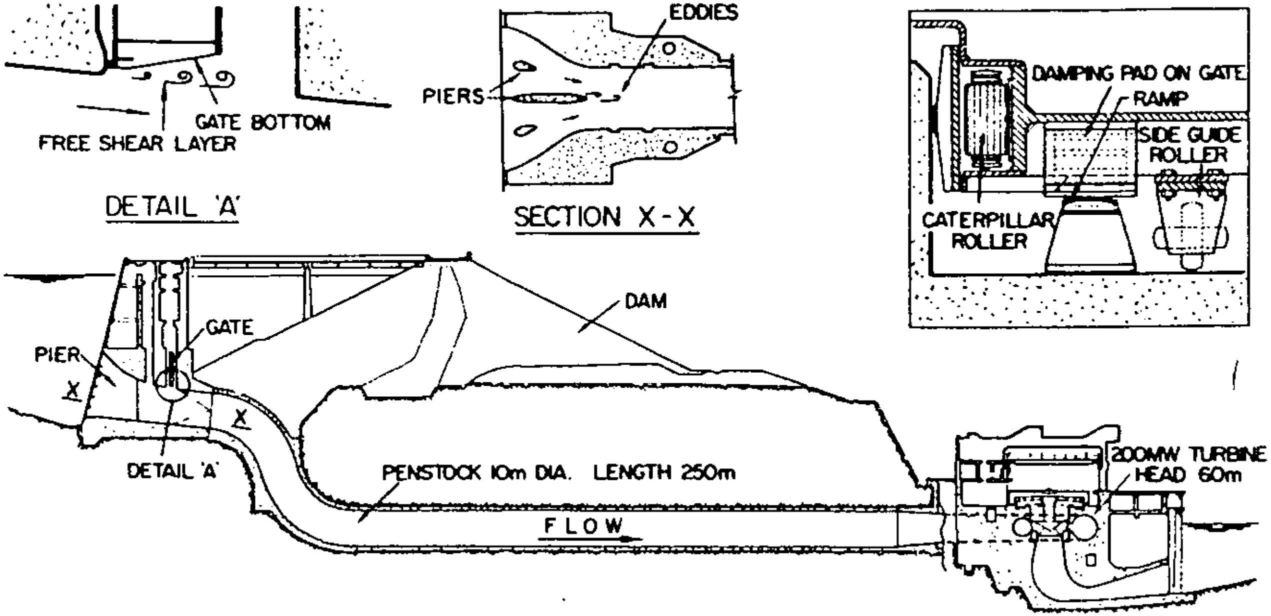

Fig. 2.2. Vertical section of intake, gate and penstock

Obviously there were occasions when Dr Penman's statement was true, but in his opinion it should not be generalized and over-simplified.

Mr R. MARTIN (Sir William Halcrow & Partners) said that the subject of Messrs Lewin and Whiting's paper was one which sometimes received detailed attention at a rather belated stage in the course of design. In drawing attention to some of the potential problem areas, the authors had performed a valuable service to reservoir designers.

He would be interested to hear any comments which the authors might make regarding the profiling of the leading edges of rope-suspended intake gates. This aspect of design had given rise to a long and continuing history of operational difficulties which could, he suspected, be avoided if certain guiding principles were more widely followed.

Authors' Replies

Dr PENMAN, in reply to the discussion, said that he was glad to hear from Mr Cooper details of the remedial measures at Altnaheglish Dam.

In order to check on the effectiveness of the rockfill support, Mr Cooper specified earth pressure cells to be installed on the face of the old dam and within the rockfill adjacent to the face. It was very difficult to measure earth pressures in rockfill and he was pleased to be able to give some slight assistance in this matter. Pneumatically operated, thin oil-filled cells manufactured by Soil Instruments Ltd were installed and, he understood, had recorded a satisfactory build up of pressure as the height of rockfill increased during construction.

Messrs LEWIN and WHITING, in reply to discussion on their paper, said that the example quoted by Mr Mee, of gate vibration at El Chocon Hydro-power Scheme*, was an illustration where a number of hydraulic conditions interacted:

1. Eddies from the intake splitters.

2. Water hammer pulsations in the penstock (caused by the turbine).

*Hardwick, J.D., Kenn, M.J. and Mee, W.T., 1979, 'Gate vibration at El Chocon Hydro-power Scheme, Argentina', 19th Congress IAHR, Karlsruhe, paper C7.

3. A free shear layer at the interface between the bottom of the gate shaft and the tunnel.

The authors suggested that a fourth factor could have been present in the form of flow re-attachment which could result from the open slot upstream of the gate and in some cases from bad shaping of the entrance to the culvert.

The paper was not intended to deal comprehensively with gate vibration but only to highlight major causes in low level outlets. Kolkman* should be consulted for the mathematical analysis of gate vibration and for the treatment of a wider range of cases.

In reply to Mr Sims, the authors' comments on the difficulties experienced in the design of the Dartmouth Dam submerged energy dissipater were based on a private communication which stressed the criticality of the three operating conditions which could be experienced: cavitation, aeration and excess aeration.

The critical conditions and the resulting operating restrictions on the outlet were mentioned by Dickson and Murley (reference No. 22 in the authors' paper).

The authors considered that these difficulties, i.e., the critical range of operating conditions, were difficult to resolve and were under the impression that this was also the conclusion of the Snowy Mountain Engineering Corporation.

The paper briefly covered the vibration of gates due to seal leakage. Authors of papers dealing with the effects of seal leakage (references 1, 2 and 8 in the authors' paper) agreed that it led to the self excitation of a gate and that it was not the cause of hydraulic resonance of the water columns within conduits controlled by gates. This was due to external excitation. The gate vibration mentioned by Mr Mee, where a resonance condition existed, was due to water hammer pulsations in the penstock. This occurred over a range of turbine load of 80-160 MW. Some of these pulsations were probably amplified and sustained over many cycles because the frequencies were close to the resonant frequency of the penstock. The origin of the pulsations in a narrow frequency band was believed to be in the disturbances shed from the central pier in the intake**.

Resonance conditions were also recorded in the paper on vibrations of the Bolarque Dam***. These were due to cavitation emanating from the turbines. In this instance it led to vibrations of the dam and not the gates.

Mr Sims had invited comment on the philosophy of providing back-up systems. The authors advocated two stages of back-up. The

*Kolkman, P.A., 1976, 'Flow-induced gate vibrations, Delft Hydraulics Laboratory, Publ. 164, July.

**Hardwick, J.D., Kenn, M.J. and Mee, W.T., 1979, 'Gate vibration at El Chocon Hydro-power Scheme, Argentina', 19th Congress IAHR, Karlsruhe, paper C7.

***Kenn, M.J., Cassel, A.C. and Grootenhuis, P., 1979, 'Vibrations of the Bolarque Dam', 19th Congress IAHR, Karlsruhe, p. 249.

first stage consisted of standby generation or other self contained power source which should never be the mobile kind. The final back-up comprised manual winding of gates despite the long time required for medium and large size gates*. Where oil hydraulic power was used to operate gates, manually operated pumps should form the last resort*. This opinion was backed by those of the authors of ICOLD Bulletin 49**.

In reply to Mr Martin, the authors said that gates had to be profiled only when they were to be operated under flow conditions. If they were rope suspended, and if the suspension was of appreciable length, there was a high risk of gate vibration under flow conditions because of the low natural period of vibration of the system. If an attempt was made to compensate for this by damping by increasing the friction of the gate reaction faces, it might not close under gravity.

If emergency closure had to be taken into account, the bottom edge had to be profiled and the gate should preferably be operated by a hydraulic ram connected to the gate by piston and rod extensions.

For a rope operated gate to meet both criteria, the design had to reconcile vulnerability to gate vibration and the possibility of getting stuck. Since some of the design factors necessary to achieve this could not be sufficiently accurately quantified, it was a difficult, if not a doubtful, exercise.

At the design stage, it should be considered whether under emergency conditions, operating limitations might be disregarded. An example of this was closure of a gate under flow conditions when it had not been designed for this purpose.

*Lewin, J., 1985, 'The control of spillway gates during floods', 2nd International Conference on the Hydraulics of Floods and Flood Control, Cambridge, September, paper 63.

**Combelles, J. and Tinland, J.M., 1984, 'Operation of hydraulic structures of dams', ICOLD, Bulletin 49.

THIRD SESSION: INSPECTION OF RESERVOIRS AND SURVEILLANCE

Session Chairman:

G.A. MILNE, Crouch & Hogg

Papers:

6. Inspections under Reservoirs Act 1975, by W.P. McLEISH, BSc, FICE, FIWES; Partner, Robert Cuthbertson & Partners

7. The Role and Training of Supervising Engineers, by N.J. RUFFLE, BSc, FICE, FIWES; Director of Operations and Works, Northumbrian Water Authority

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DISCUSSION OF THIRD SESSION

Discussion

Mr W.T. MEE (Sir Alexander Gibb & Partners) raised, by reference to a specific case, the question of ownership and responsibility of the safety of a reservoir. Although not actually challenged or tested in the courts, he had been informed that the owner of the water was responsible for reservoir safety and if this was in fact true, one might well question the justice of it. The dam in question (see Fig. 3.1) impounded a small, pleasant 'pond' which just qualified for the Reservoirs Act 1975. The whole area had extensive watercress beds, and these also occurred immediately downstream of the dam.

The original dam was completed to a level some 10 ft lower than at present over 700 years ago. The construction material was lumps of chalk which the villagers carried in reed baskets and placed in the marsh. The original dam profile was conservative and capable of some limited overtopping. The events described below then occurred apparently without any reference to the owner of the water who considered that he owned only the water and the upstream face of the dam.

1. At some time, probably in the last century, the dam was raised 10 ft to accommodate a public highway and thereby the downstream slope was considerably steepened.

2. At some time in the last century the watercress beds were widened and a drain constructed along the toe of the dam. In order to do this, the toe of the dam was removed and

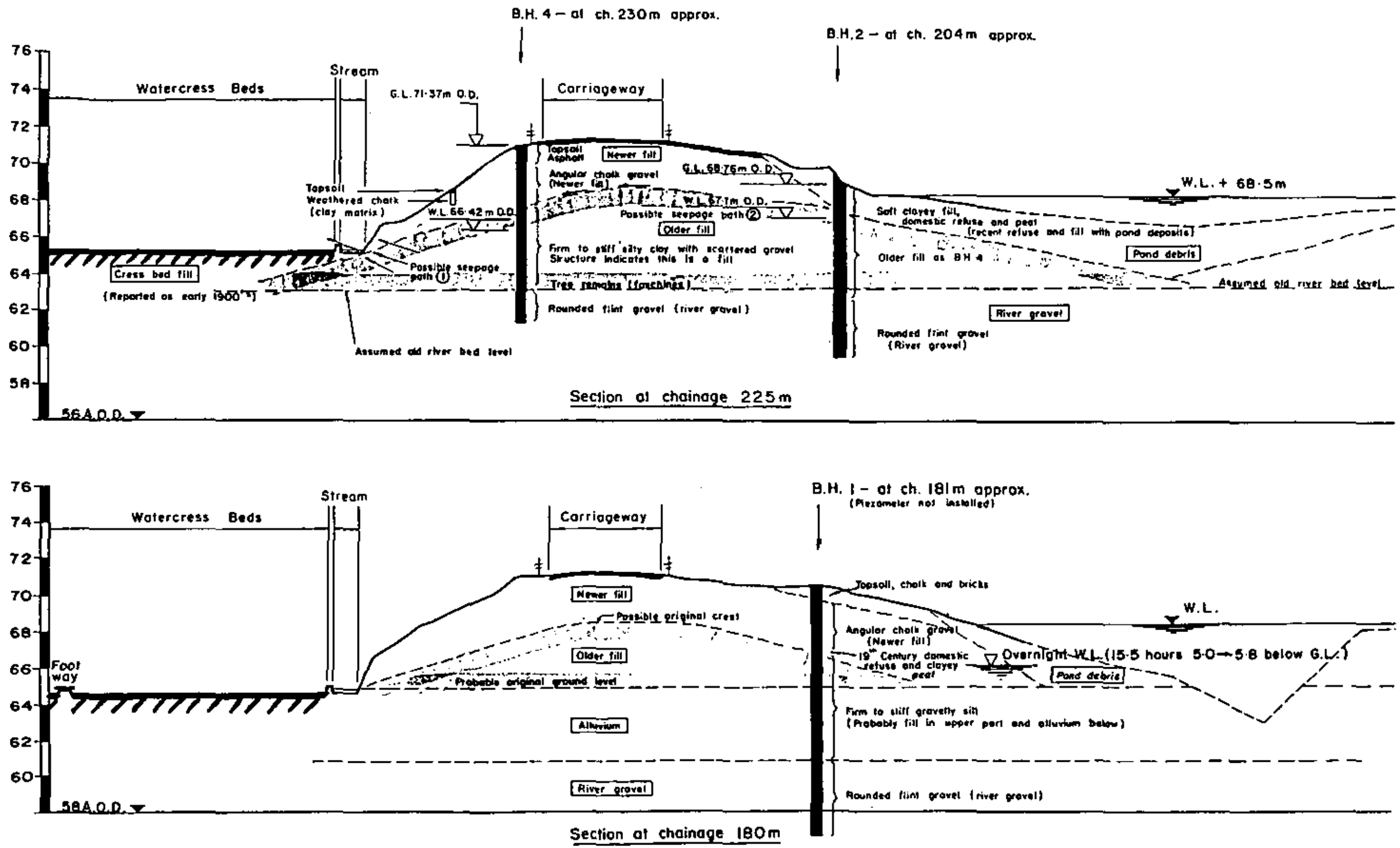

Fig. 3.1. Sections through dam

thereby the dam itself was made even steeper. This inevitably caused localized slips which were repaired, presumably by the owner of the drain. These repairs contained 8 inch pipes at about 20 ft centres which carried appreciable volumes of seepage water. The watercress beds themselves had artesian flow.

3. The downstream face was owned by the owner of the watercress beds and was covered by substantial trees. If one or more of these was blown over, slippage of the downstream face would inevitably occur.

4. Over 30 years ago, a water main was placed in the dam some 8 to 9 ft below the carriageway level. This had a diameter of 8 inches and operated under up to 300 ft head.

5. Foundations for a large TV aerial were excavated in the downstream face. The excavation was still open and had appreciable seepage flowing across the base.

6. The pond had its deepest part parallel to the bank and in one area seepage caused actual discernable eddies. However, when the owner wished to tip material along this section of the upstream face, permission was refused on the grounds of amenity.

With one owner of the water and the upstream face, a second owner of the downstream face and watercress beds, presumably a responsibility by the Highway Authority for the roadway, and a total of 5 additional owners who had properties around the spillway, which discharged through the ends of their gardens, the application of responsibility should, perhaps, be considered further. Mr Mee asked for guidance from the Chair and the delegates on this point. (See page 31 for reply from Mr Agnew.)

Mr J.W. PHILLIPS (Department of the Environment) made the following general points in response to a suggestion of imprecision in the wording of the Act:

(a) It covered all types and sizes of reservoir within its volume and other criteria, from reinforced concrete service reservoirs to amenity ponds and to large public reservoirs.

(b) It should be read as a whole together with the concomitant statutory instruments.

(c) It was a legislative framework within which the expert engineer had a very large degree of discretion on technical matters upon which it would not have been appropriate to legislate.

In Mr McLeish's paper, some doubt was expressed as to whether flood storage above top water level could bring a reservoir within the ambit of the Act. Top water level was defined in Statutory Instrument (SI) 1985 No. 177 as the lowest crest level of the overflow sill (except for those dams with gates, etc.). Mr Phillips pointed out that this was a measurable quantity; any higher water level would be based on the statistical definition of a particular flood with almost insuperable difficulties of legal interpretation. It was at the discretion of the Panel Engineer to decide what water level was used in dam stability and other calculations. There had also been some debate over the definition of the bed level of a reservoir. The interest of the Act was safety, and in Mr Phillips' view, the bed level of a reservoir would be that level below which liquid would not escape from the reservoir should there be a breach. This was a matter for the enforcement authority to determine, using any professional advice they might require.

Mr Phillips said that it was his opinion that the main differences between 'abandonment' and 'discontinuance' were that with:

(a) Abandonment (Section 14) - the dam was still in being, but the reservoir was incapable of being filled so that it constituted a risk. The reservoir remained within the Act and inspection and supervision were required.

(b) Discontinuance (Section 13) - alteration to the reservoir was involved so that it was incapable of holding more than 25,000 m^3 etc., and, following a certificate from the appropriate panel engineer, it was then removed from the ambit of the Act.

It had been suggested that there was a possible inconsistency between Section 26(2) and SI 1986 No. 468. Mr Phillips suggested that the Act made it clear when an annex to a report was required, and the Act should be followed even if there was doubt about a footnote to a regulation.

To clarify the matter of the independence of the inspecting engineer, he pointed out that the wording of Section 10(9)(b), 'nor is connected with any such engineer etc.', was in the present tense.

Regarding the prescribed form of record relating to the supervising engineer, under Section 12(1) the undertaker was required to employ a supervising engineer at all times (except when the reservoir was under the supervision of a construction engineer), and it was clear that the current supervising engineer should be on the record.

If no inspection of a reservoir had been carried out under the 1975 Act, there could be doubt about the records to be kept by the undertaker. Mr Phillips said that it was his personal opinion that undertakers should use their judgement to remain within the law on the frequency of record-keeping etc., and:

(a) carry on with the frequency of records kept under the 1930 Act, or,

(b) consult the previous inspecting engineer or another appropriately qualified panel engineer, or,

(c) consult the supervising engineer.

The content of the inspecting engineer's report was commented on. This would be largely a matter of engineering judgement and would be such as could be expected from engineers of their standing in explaining the situation to a client. However, recommendations in the interests of safety were legally enforceable and as such had to be clearly separated from any other recommendations that the engineer might wish to make.

In Mr Ruffle's paper there was a suggestion that the frequency of visits to a reservoir by the supervising engineer was a matter for agreement between the undertaker, the supervising engineer and the construction or inspecting engineer. However, the Act stated (Section 12) that at all times (except when the reservoir was under the supervision of a construction engineer) a supervising engineer should be employed by the undertaker. How the latter engineers supervised the reservoir (i.e., number of visits, etc.), apart from certain duties under the Act, was left to their discretion as expert engineers. They would doubtless wish to take into account any guidance received from the construction or inspecting engineer.

Mr Phillips commented that a number of speakers had raised the point that in their opinion 'remote' reservoirs that posed little, if any, risk should not be within the Act or should be removable from the Act by a panel engineer's certificate. He pointed out that the reservoir would need to come within the Act for an engineer statutorily to assess any risk. Conditions upstream or downstream of a reservoir could change over a period of years and thus the degree of risk that such a reservoir might pose could change. Recommendations in the interests of safety were at the discretion of the panel engineer and the Engineering Guide made it clear on page 3 that the Institution envisaged that 'there is the occasional dam that has to be treated as an exceptional case'.

Mr M.F. KENNARD (Rofe, Kennard & Lapworth) said that many of the topics presented and discussed were similar to those of the 1984 IWES Symposium on 'Water Management: A Review of Current Issues - Implementation of the Reservoirs Act 1975'. These matters included independence, safety, capacity, flood alleviation reservoirs, etc. Mr McLeish's four cases of reservoirs and whether they were 'large raised reservoirs' were also discussed at this previous meeting. These borderline cases were likely to affect only a very small number of reservoirs.

Turning to supervising engineers, it was important to remember the reasoning for this. The Department of the Environment were concerned with the reservoirs, mainly private or industrial, where no civil engineer was involved or saw the reservoir between 10-yearly inspections. Supervising engineers' duties included attending to any matters which the construction engineer or inspecting engineer had stated required to be watched, but what of the cases where no list of matters was available? What sort of matters did the two authors consider should be listed? In a recent case, as a supervising engineer (and it should be remembered that any panel engineer could be a supervising engineer), he had prepared his own list, but he was not entirely clear on what was required, or what form of report was required. What form of supervising engineer's report did the authors consider was required? The Act did not seem to lay down any requirements.

Mr H.S. EADIE (Babtie Shaw & Morton) said that Statutory Instrument (SI) 468 made it clear (in Schedule 2) that inspecting engineers should concern themselves with the effectiveness of the scour pipe and of any other means by which the water level within the reservoir might be lowered. SI 177 asked for details of the scour and drawoff works and their capacity.

It seemed reasonable to require facilities for lowering the water level to be in working order where they had already been provided, but rather onerous to insist on installing a scour through the embankment of a small reservoir which did not have one originally. In such a case it might be a good idea to require the undertaker to have a prepared plan of emergency action to lower the reservoir water level by pumping.

Nowhere was it laid down in the Act that scour or drawoff works must be provided. Was it considered that facilities such as a scour or drawoff were essential? (See page 30 for reply from Mr Agnew.)

Mr P.E. WALMSLEY (Tayside Regional Council) asked of Mr McLeish that if they agreed that a typical fishing loch would, or might, have only three basic features, namely: (i) an embankment, (ii) an overflow, and (iii) a scour valve, should the condition of any one feature be accorded greater importance that the others in the course of an inspecting or supervising engineer's examination, bearing in mind that (i) and (ii) would be accessible, whereas (iii) was unlikely to be.

Mr J.H. FLEMING (Sir M. MacDonald & Partners) said that reverting to so-called 'seminars', he drew attention to the case of flood storage reservoirs. These were empty nearly all the time and only filled temporarily when absorbing an undesirably large flood. He thought it was generally agreed that they should be included in the scope of the Act, but the wording could give cause for some doubt in interpretation. He had certified (under the 1930 Act) a temporary flood storage formed by allowing the water level of a small lake in a London park to be raised. The 'sill' of the control structure was the base of a narrow vertical slot (about 50 cm wide) and it was at the original lake level. This slot restricted flood outflow and the water level rose until it reached a level where the slot widened to a spillway about 4 m wide. Half a metre above this spillway was an emergency spillway over the grassed embankment.

Under Section 1(1) of the Act it could be argued that the 'reservoir' was not capable of 'holding' water above the bottom of the slot, and the stored volume was zero (see definition of 'top water level' contained in Statutory Instrument 1985 No. 177).

He interpreted the top water level as being the level of the 4 m wide concrete spillway. Perhaps on Mr McLeish's view, one should adopt the level of the emergency spillway.

The operation of the Act was everywhere dependent on the opinion of the construction engineer or the inspecting engineer. It would be unsatisfactory if they had to wait for test cases in the courts before general criteria for the operation of the Act could be decided. Where the body of qualified civil engineers, who were empowered to act as inspecting engineers, could itself reach agreement on such matters, a mechanism should be established by which all inspecting engineers might consult together and establish this consensus on any appropriate issues likely to arise in the working of the Act and avoid unnecessary contention about its operation.

Mr S.C. AGNEW (Scottish Development Department) pointed out that referring to the reference in paragraph 7 of Mr McLeish's paper to the meaning of safety and Section 16(1), this and the similar reference in Section 16(2) were about the use of emergency powers available to the enforcement authority. These powers, however, were permissive. Had Mr McLeish considered whether the overall effect of these sections might be the contemplation of a situation where an escape of water might not give rise to risk to persons or property?

Mr J.E. ROSE (Anglian Water Authority) asked Mr Ruffle for his views on the availability and continuity of supervising engineers. While the Act called for a supervising engineer to be employed at all times when the reservoir was not under the supervision of a contruction engineer, this would not be possible if a supervising engineer had to go overseas for a prolonged assignment.

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Mr W.J.F. RAY (G. Maunsell & Partners) commented that both authors had stated or implied that the appointment of inspecting engineers ended following the submission of their report. However, at least one major water authority had retained a panel engineer for a term period of several years to oversee a number of reservoirs during which period, in addition to carrying out inspections, he might be consulted at any time by the supervising engineer whose reports he regularly reviewed; also he would review and agree any 'measures in the interests of safety' that might need to be carried out.

It would be of interest to have the authors' views as to whether they considered such an arrangement advantageous and one that should be generally adopted.

Furthermore, the point was made in paragraph 21(b) of Mr McLeish's paper, and this was also referred to by an earlier speaker, that the inspecting engineer must not be the construction engineer or connected therewith as a partner, etc. For most major new reservoirs there was now a mass of data assembled in the course of construction which was highly relevant during subsequent performance monitoring; there was surely a case for continuing to retain the expert engineers associated therewith, as was suggested by himself during the Institution of Civil Engineers' discussion on the Victoria Dam in Sri Lanka. It would seem that while this was possible overseas, it was now precluded in the UK. Would the authors concur in the view that this provision in the 1975 Act was too rigid and indeed that implicitly it could be seen as questioning the professional integrity of the engineers concerned?

He endorsed the emphasis given by Mr McLeish in paragraph 33 of his paper to continual perceptive surveillance using the simplest available means. For very long reservoir embankments (such as the 5 km bank in London for which he had recently been responsible) it was practical to install monitoring instruments at selected cross sections, and to periodically assess movement by geodetic survey, but the daily visit and walk around the complete reservoir perimeter by the reservoir attendant was perhaps the most important routine supervision activity in the interests of safety.

Water authorities were constantly seeking to improve their efficiency, but he would be concerned if there were to be any reduction in this type of surveillance; would the authors agree?

Mr J.S. BRINDLEY (W.S. Atkins & Partners) said that the question of whether a reservoir was within the Act was not a minor one, as had been suggested. Private owners, faced with expensive costs of, in the first place, an inspection and, in the second place, remedial works which they could not afford, would do everything in their power to get their reservoir out of the Act.

A number of farmers owned lakes that posed no threat to life and it seemed unreasonable that they should be saddled with the cost of regular inspection and supervision. In particular, bunded reservoirs that stood on their own above generally flat countryside would only damage fields and crops if they burst (and the law already coped well with that) and they did not threaten life. One possible way to treat such cases would be for inspecting engineers, on their first visit to the reservoir, to be able to look at the situation and then certify that the reservoir posed no risk to life and could be excluded from the Act.

Mr A.C. MORISON (Sir William Halcrow & Partners) said that Mr Ruffle had spoken of the information which supervising engineers sent to enforcement authorities. Unlike his paper, his slide suggested that this included all matters of relevance to safety. This was not the case. The information which the supervising engineer was required to copy to the enforcement authority was limited to copies of notices to undertakers that:

(i) the reservoir was not being operated or monitored in accordance with the inspecting or construction engineer's instructions, or,

(ii) the supervising engineer recommended inspection by an inspecting engineer.

The enforcement authority would not therefore expect to hear from the supervising engineer if all was well.

The authority would also not hear from supervising engineers if their appointment had ceased. At present, the appointment of supervising engineers had to be notified to the enforcement authority, but not the end of the appointment. It would therefore be possible for an undertaker to appoint supervising engineers, notify this to the enforcement authority, and then dispense with their services. The enforcement authority would not know of this until the next statutory inspection, perhaps in ten years time.

It would seem sensible for supervising engineers to notify the completion of their appointment to the enforcement authority, unless they knew that a replacement had been appointed and notified.

Mr Mee had given an example of the problems of different owners of the reservoir and the embankment. A similar situation in 1962 led to one of the few court cases under the 1930 Act.

In this instance, a road had at some time been constructed across an old mill embankment, with flood discharge being led beneath the road in a brick culvert with a sluice at the upstream end. A piping failure along the outside of the culvert led to failure of the culvert and subsidence of the road, and this was dealt with by the Highway Authority, who then tried to recover a proportion of their costs from the private reservoir owner.

The case at Quarter Sessions ruled that the Highway Authority as owner of the crest of the embankment were not an undertaker under the 1930 Act, but that the owner of the land beneath the reservoir was, and instructed him to have a statutory inspection carried out.

Inspection of the reservoir showed a 1:1.5 downstream slope, a road verge of about 0.6 m, tilting of poles and trees on the slope and longitudinal cracking in the road surface. The inspecting engineer concluded that the embankment was well below modern standards as a road embankment and the heavy traffic (including buses) using the narrow road on the crest were a danger to the embankment, and recommended, among other points, that the road be closed in the interests of safety. The only persons at risk in the event of a failure were, in fact, the road users.

The matter was eventually resolved by ownership of the reservoir being passed to the Highway Authority, who widened the embankment.

The records of Quarter Sessions were not generally available, but included the few cases relevant to the Reservoirs Acts. It would seem a worthwhile project for cases involving the Act to be collected for reference. Perhaps this was something the Department of the Environment could undertake, as they told them that interpretation of the Acts replied on such case law. (See page 31 for reply from Mr Agnew.)

Mr J.G. ELDRIDGE (Binnie & Partners) said that there had been much discussion about the interpretation of the Act, for example, the levels to which the capacity of a reservoir should be measured, the meaning of the word 'safety', and so on. He suggested that it was more important that panel engineers, knowing the responsibilities of owners to society and knowing what they knew as qualified engineers, should say what they would do if they owned the dam they were inspecting.

It was important to be clear about the potential mode of failure. For example, a well-grassed downstream slope of a well constructed earth dam, providing it was not built of very sandy or silty material, would withstand a few hours of being overtopped without suffering any significant damage. He knew of a low earth dam which had twice been overtopped to a depth of about 30 cm during the last 50 years without ill effects.

Easy vehicle assess to dams was important, so that if trouble was imminent, emergency pumping equipment to lower the water level, and repair crews, could reach the site quickly. (See page 82 for reply from Dr Charles.)

Mr J.H. BEDGOOD (Crouch & Hogg) wrote that Mr McLeish had referred briefy in paragraphs 5 and 30 of his paper to the guide 'Floods and Reservoir Safety'. Could Mr McLeish comment on an apparent discrepancy in the Guide in Section 2.3 - Recommended Standards? Line 4 on page 8 stated 'If there is no wave wall, the dam crest has to be high enough to contain the total surcharge' (i.e., flood surcharge plus wave run-up). Line 19 onwards referred to overtopping and line 24 stated that reference in Table 1 to overtopping 'means overtopping due to excessive still-water level during the routed flood and not to wave slop'. This comment seemed to imply that wave slop and run-up could be accepted even where still-water flood surcharge equalled or was slightly in excess of dam crest level. Line 36 onwards, however, drew attention to minimum wave surcharge allowances.

Mr W.J. CARLYLE (Binnie & Partners) wrote that, regarding Mr McLeish's paper, he wished to make the following points:

1. He found that the Act and the statutory instruments were quite clear about what constituted a large raised reservoir, namely, that it should have a capacity of 25,000 m^3 (5.5 mg) or more from the lowest point of the adjoining land (which was normally the stream bed level or thalweg) to the top water level (which was the fixed crest of the conventional spillway).

2. He was quite clear about what constituted a spillway. It was not a converted outlet pipe with or without a bellmouth inlet which could be accidentally or deliberately broken.

It could be a properly designed bellmouth with a correctly sized outlet tunnel which could not either accidentally or deliberately be broken without a great deal of difficulty.

3. With regard to the requirement for inspection by an independent qualified engineer under Section 10 of the Act, he believed that the independent nature of the inspection was already showing benefit. In a number of cases in which he had been involved, the inspection had developed with the co-operation of the owner into a full re-evaluation of the design, construction and performance of the dam and the pulling together of data which had never been properly collected in the form of a design and completion report.

The international trends in this respect were for the periodic inspection to be performed by an independent inspecting engineer, attended by both the owner and the original designer. He thought this was a highly desirable arrangement which could be implemented if the owners wished it.

Mr D.F.H. PAIN (Northumbrian Water Authority) wrote concerning three points in connection with the Reservoirs Act 1975 on which he invited comment from Mr McLeish.

Firstly, he expressed the view that the panel engineer's role should ideally be concerned with engineering judgements about matters of safety and not pronouncements on legal matters arising from the need to interpret an inadequately defined act of legislation. Given that this view was commonly held and that new legislation was unlikely in the near future, surely the profession (e.g., Institution of Civil Engineers through BNCOLD?) should accept the challenge and draw up either a code of practice or working guidelines. Such action could only enhance the image of the profession which was otherwise likely to sink into disrepute through the availability of disparate legalistic interpretations.

Secondly, regarding Section 10(9) of the Act, which set out some of the criteria to ensure the independence of the inspecting engineer, did Mr McLeish agree that, although not specifically written, the spirit of this clause would seem to militate against the inspecting engineer coming from the same organization as the construction engineer even though the construction engineer might be retired or deceased?

Thirdly, he suggested that inspecting engineers should, particularly when exercising discretion regarding categorization of large raised reservoirs, be mindful of the possibility that proposals for downstream development within the potential flood plain might arise before the next inspection. In this context, it would seem that the position of both the owner and the enforcement authority in objecting to the development would be considerably strengthened by some form of warning or explanation in the inspecting engineer's report.

Mr K.J. SHAVE (Southern Water Authority) wrote that there were two points in Mr Ruffle's paper that he would comment on: firstly, the relationship with the inspecting engineer; and, secondly, the duties of the supervising engineer.

Southern Water had taken a different view from that expressed in the paper regarding the appointment of the inspecting engineer. All the Authority's dams due for inspection before 1990 under previous legislation would be inspected in 1986/7, and inspecting engineers had been duly appointed. This was a comparatively easy decision to have taken since the Authority had only eleven reservoirs which were covered by the Act and the financial and administrative demands were nothing like as heavy as for other Authorities. This decision had enabled them to begin the implementation of the Act in a positive manner, giving it full support and to be seen to be taking a lead in their area.

It was acknowledged that there had been a distinct change by introducing the formal role of an appointed supervising engineer under the Act, and involving engineers who were not employed in the operation or maintenance of dams as part of their day to day duties. The inspecting engineers had therefore been appointed not only to carry out the inspection, but also for a further three years, during which time they would receive and comment on the supervising engineer's reports and instrument readings and be advised of any changes noted between the supervising engineer's formal reports. There was therefore a facility for consultation and advice formally arranged for the supervising engineer to use, if he considered it necessary, on any matter relating to the reservoir during that three year period. The success of this type of appointment would be reviewed in due course and a decision taken as to whether to extend the appointment on the same or similar basis or to discontinue it.

He himself certainly commented on maintenance matters as a supervising engineer, and in this respect would consider it to be a disadvantage to have responsibility for the carrying out of maintenance work. He was sure that the supervising engineer should be completely independent and objective in his reporting and monitoring, and also outside the constraints of budgets or staff availability which could, under some circumstances, influence his judgement when deciding on the need for works. He accepted that it followed from this that the tactful approaches suggested in the paper were necessary and he always discussed anything that arose from his site visits with his Operations colleagues before preparing reports or memoranda. This did not alter what he intended to put on paper but had the effect generally of avoiding 'ruffling any feathers' and maintained a corporate goodwill and the respect that everyone had for the others responsible for reservoir condition and safety.

Mr T. SPENCE (North East Divers) wrote that in the past it had been apparent that many authorities had been reluctant to engage divers to carry out tasks other than inspection work. It was obviously preferable to carry out remedial work on the surface whenever possible, however, he wondered if the various bodies were aware that todays highly trained divers were capable of doing underwater the majority of tasks thought only possible on dry land. Obviously cost had been a deterring factor, but if the only other alternative meant drawing down the reservoir, surely careful consideration should be given to the use of divers.

He would be interested to hear Mr McLeish's views on past experience using divers and what possible use he forsaw for them in the future.

Authors' Replies

Mr McLEISH, in reply to the discussion, thanked the many contributors, not all of whom could be identified by name. His replies were arranged by subject, with reference to contributors being made as appropriate.

Flood Storage

The question of whether flood storage was, in law, part of the storage of what constituted a 'large raised reservoir' had been well ventilated without throwing much light on the matter. Only the courts could interpret the Act, but undertakers, particularly private owners, looked to inspecting engineers for guidance. He agreed with the spirit of a comment made by Mr Pain, that the panel engineer's role should ideally be concerned with engineering judgements about matters of safety and not pronouncements on legal matters involving interpretation of legislation. However, an inspecting engineer had to reach a conclusion involving interpretation, as Mr Fleming rightly did in the case he cited. There was confusion about the use of top water level, but the definition of it was clear. In practical terms, however, an inspecting engineer might well be justified in going along with Mr Carlyle's view that a spillway 'was not a converted inlet pipe, with or without a bellmouth inlet ...', or, one might add, any other form of grossly inadequate means of discharging flood waters. However, what was wrong with the concept of such an arrangement constituting, in effect, a flood regulating reservoir? An undertaker might even lower a bellmouth or notch a higher, wider overflow if doing so would put the volume below 25,000 m^3. Inspecting engineers should satisfy themselves that the object of the Act would be satisfied if asked to advise on such a course of action.

The Act did not, as Mr Carlyle indicated, state that a large raised reservoir was defined by the volume as measured from the bottom up to 'top water level'; Section 1(1)(b) did not specify an upper limit, except in the terms '... designed to hold, or capable of holding ...'. Nor was there any reference in the Act to a 'conventional' spillway. Mr Kennard commented that cases illustrated during presentation of the paper were borderline cases affecting only a very small number of reservoirs. This comment did not address itself to the fundamental question. As Mr Brindley pointed out 'the question of whether a reservoir was within the Act was not a minor one, as had been suggested'. No matter how few such cases there might be, owners expected their case to be considered on its merits in such a way that, consistent with meeting lawful requirements, costs were kept to a minimum.

Mr Phillips pointed out that use of top water level produced a measurable quantity with which Mr McLeish himself agreed. However, a dam was 'capable of holding' water up to the top of the dam, which was just as definite. Several speakers appeared to imply that the term 'holding' in the context of 'capable of holding' could only refer to the volume below top water level. Such an interpretation appeared nowhere in the Act or the Statutory Instruments (SIs). Surely the object of considering 'the adequacy of the margin between dam level and overflow level' (SI 1986 No. 468, Schedule 2) required consideration of 'capability of holding' in addition to sufficiency of margin by comparison with depth of flood retention above the overflow level?

He adhered to the view that whether or not top water level was adopted for purposes of registration, it was Section 1(1) of the Act that was paramount and inspecting engineers would have to reach their own conclusions as to whether the part of the dam above top water level as defined '... is designed to hold, or capable of holding, water ...'. An informal legal opinion that he had taken supported the view asserted.

Abandonment and Discontinuance

The wording of Sections 13 and 14 of the Act did create confusion in regard to risk and volume impounded and again, it would be up to inspecting engineers to make up their own minds on what the Act meant, unless, of course, an owner was prepared to take a point to the courts for clarification. The extent of the dilemma about what constituted a large raised reservoir could be visualized from reported cases of doubt as to whether motorway embankments might come within the ambit of the Act if waterways, being inadequate, resulted in water being impounded. Similar circumstances could arise with any embankment with a waterway passing through it. Such constructions might not have been designed to hold water, but often were 'capable' of doing so.

Independence of Inspecting Engineers

As Mr Phillips pointed out, the wording of Section 10(9)(b) was in the present tense; literally this meant that inspecting engineers were independent so long as their present associates were not responsible in any way as the construction engineer. Strictly read, a former partner or colleague, employee, etc., having been the construction engineer would not disqualify an individual from acting as the inspecting engineer. However, Mr Pain reasonably pointed to the spirit of the Act, which Mr McLeish would support. It would be reasonable for engineers invited to inspect, who found themselves in any doubt about their own independence, to put the matter to the owner for a decision. Mr Carlyle made a valuable point in (3) of his written contribution: an independent mind could observe with more objectivity, so producing useful advances in assessing performance. Several undertakers were already bringing together the designer and an independent inspecting engineer to deal with complex problems. This was a constructive approach.

Remote Reservoirs

Mr Phillips commented on contributions by a number of speakers dealing with 'remote' reservoirs. As it stood, however, the Act did not concern itself with location and degree of risk. It was for individual inspecting engineers to make recommendations in the interests of safety, give directions concerning matters to be recorded (11(2)), and note matters to be watched (12(2)) - or not to do so - such as, in their judgement, appropriately reflected the circumstances including the risk involved.

Duties of Supervising Engineers

Mr Kennard raised an important point about matters needing to be watched by supervising engineers in cases where no earlier directions existed. Mr McLeish considered that the list should include such monitoring as was necessary to provide data for assessment of the safety of the reservoir. Such data should be distinct from information which it might be of interest to have, but which was not essential for that purpose, e.g., water levels in the reservoir might only be useful for correlation with, say, leakages, or to indicate the largest floods which had occurred, but if they were necessary at all on a daily or weekly basis, for example, it would only be for such a period as would allow conclusions about the significance of leakage to be assessed; thereafter, such information might well be unnecessary in the interests of safety, or certainly the frequency of recording could reasonably be reduced. Matters which he considered should be watched by supervising engineers could be broadly summarized as followed:

1. Efficiency of monitoring and assessment of data collected.

2. Accuracy, significance and implications for reservoir safety of information obtained by any persons connected with surveillance (e.g., reservoir superintendents or owners might actually record data).

3. Personal observation of known, and discovery of previously unobserved, movements, deformations, settlements, leakages, and other indicators of performance.

4. Whether the supervising engineer thought another independent inspection was required at a date earlier than previously recommended by an inspecting engineer.

He considered that to 'keep the undertakers advised of its behaviour in any respect which might affect safety ...' (11(1)), a written statement by a supervising engineer which covered the above matters was all that was required. With regard to the need for another inspection, the operative words of Section 12(3) were 'if at any time he <u>thinks</u> that such an inspection is called for'.

Also on the subject of supervising engineers, Mr Carlyle pointed out that a written statement by a supervising engineer might not be required. The wording of 12(2) implied that if an appropriately qualified engineer did not note matters that needed to be watched, then a statement might not be necessary 'not less often than once a year'. This had to be taken with 12(1), 'a supervising engineer shall be employed to supervise a reservoir and keep the undertaker advised of its behaviour in any respect that might affect safety ...'. Mr McLeish thought that this required at least an annual visit and presumably most undertakers would want to have something in writing to satisfy themselves on performance; thus an annual statement would appear appropriate in any event.

<u>Enforcement Authority Emergency Powers</u>

He agreed with Mr Agnew that the overall effect of 16(1) and (2) would be that exercise of permissive powers by enforcement authorities would be directed at bringing about a situation where an escape of water would not give rise to risk to persons or property.

<u>Term Appointments of Inspecting Engineers</u>

Mr Ray referred to term appointment of inspecting engineers. Mr McLeish knew of other cases where term appointments had been made, or were contemplated. This was a constructive approach by undertakers with the objective of making available to supervising engineers the opportunity of consultation to enable a supervising engineer to fulfil his duties. Care would be needed, because each had his own statutory function and must stand on his own feet.

Regarding surveillance, far too much useless monitoring data was collected. Mr Ray agreed with the emphasis on what Mr McLeish had called 'perceptive surveillance using the simplest available means'. Subjects to be monitored and the frequency of monitoring would vary throughout the life of a dam and between times of independent inspection. It was wasteful and costly to persist unnecessarily with monitoring; potentially, term appointments of inspecting engineers would provide a constructive means of varying monitoring to achieve reservoir safety while minimizing costs. To make lawful directions of an inspecting engineer outside the report of an inspection might need careful drafting of 'recommendations made in the interests of safety' and/or directions as to 'matters which need to be watched'.

Consensus Views of Inspecting Engineers

Mr Fleming made a good point in suggesting a mechanism for consultation among inspecting engineers. Moves had already been made for informal meetings to be held.

Mr McLeish was reluctant to have more codes of practice or working guidelines as Mr Pain suggested, simply because there was enough paper already. UK reservoir safety legislation rested on the responsibility being placed on the shoulders of individual engineers, using their own experience to deal with particular cases; informal meetings to ventilate opinions, individual practice and interpretation should be a useful step towards unification of understanding. However, inspecting and supervising engineers had a statutory duty to act independently.

Downstream Developments

Mr Pain raised an important point concerning categorization of dams at the time of an inspection by comparison with the possibility of subsequent downstream developments with consequent knock-on effect on dam category. For many years Mr McLeish had reported and made recommendations in the interests of safety on the basis of dam category for the time being, but he had added an appendix to outline the implications of the occurrence of larger floods. By that means undertakers could be aware of and assess the risks they were prepared to take and have some means of assessing how to react to planning applications for downstream development.

Floods and Reservoir Safety Guide

Mr Bedgood raised an apparent discrepancy concerning recommended standards on page 8 of the Guide. There was no discrepancy, although reading through this section of the Guide was confusing. Line 4 was in a section dealing with the general standard (Table 1) which by definition must avoid overtopping of a dam. Line 19 was dealing with minimum standard, where rare overtopping was tolerable.

Overtopping did indeed mean overtopping by still water level with wave slop over and above that. One had to be careful in assessing tolerance to overtopping. He admired those who could assess a (say 1%) chance of breaching a dam. The tops of earth fill dams tended to dry out and dams were seldom inspected or even seen by anyone at exceptionally high water levels. The risk of 'accidental escapes of water' were at their greatest during exceptionally large floods and there was very little data to enable constructive assessments to be made.

Clearly, minimum wave surcharge allowance did not apply in cases of overtopping being tolerable. It should be remembered that the onus for weighing up all issues and reaching a decision rested firmly with the inspecting engineer. The big problem was that actual tolerance to overtopping might only be assessable after an event, when it might be too late.

Farm Lakes

Mr Brindley drew attention to reservoirs which if breached would cause no risk to life. Mr Carlyle pointed out that if under 12(2) there were no matters noted by the construction engineer or supervising engineer that needed to be watched by a supervising engineer, there was no need for the supervising engineer to make an annual report. Indeed, in the case of low risk reservoirs there might be no need even to make annual visits, notwithstanding that to comply with the law an owner must appoint a supervising engineer. Clearly, the opportunity to minimize costs in supervising low risk and remote reservoirs existed.

Diving Inspection

Mr Spence pointed to the use which could be made of divers in connection with maintaining reservoir safety. Inspection by divers could play an important and often vital part in ensuring reservoir safety. This was apart from the obvious use and need of divers in making repairs under water. Mr McLeish had found, however, that it was essential to have an inspecting engineer and diver working together to facilitate interpretation of factors disclosed during diving inspection; in poor lighting conditions under water and on irregular surfaces, divers could easily lose their bearings and not be able to distinguish anomalies without close guidance. It was preferable to have divers connected to the surface by telephone, so that they could be continually instructed.

Embankments, Overflows and Scours

Mr Walmsley sought an opinion about the relative importance of these elements of a reservoir. While each case had to be considered on its merits, a scour was the least important, and the other two had to be considered together. Many scours were too small to be effective and the cost of replacement might be, and often was, not justifiable. However, embankments had to be safe, with or without rare overtopping, and spillways to be adequate in the particular circumstances. If scouring arrangements were inadequate, a plan of emergency action might be essential in some cases.

In conclusion, many interesting questions had been raised and it was clear that those concerned with the operation of the Act had been forced into grappling with subtleties of meaning and interpretation. They should not lose sight, however, of the essential purpose of the Act, which was to make provision against escapes of water from large raised reservoirs by ensuring adequate standards of safety.

Mr RUFFLE, in reply to discussion, said that Mr Morison was of the opinion that the relevant slide had incorrectly suggested that the supervising engineer's advice to the undertakers on all matters concerning safety should be copied to the enforcement authority. The intention had been to paraphrase Section 20(4)(e), as indeed no doubt was also the intention of Mr Morison's (i) and (ii). The wording on the slide was 'send copy to enforcement authority of advice to undertakers requiring action', which Mr Ruffle considered was not too far wide of the mark having regard to the final six words in Section 20(4)(e)(i). Supervising engineers were required to send copies to the enforcement authority of their advice to the undertakers which 'recommends them to have the reservoir inspected under Section 10 above or to take any other action'. As he had observed in his introduction, however, such actions might be presumed to be only those concerned with safety, as that was the subject of the Act, and were not those relating to recommendations on cosmetic maintenance, e.g., painting.

Mr Morison suggested that supervising engineers should notify the enforcement authority if their appointments were ended by the undertakers. The notification of both the appointment and its termination was, however, the responsibility of the undertakers under Section 21(3) and if they failed in this respect they were liable to a fine under Section 22(2). Mr Ruffle doubted the need for duplicating the duty.

Mr Kennard asked what supervising engineers should watch in the absence of directions from the construction engineer or the inspecting engineer. The training of supervising engineers should be designed to make them aware of the matters which they should keep an eye on, and some of these were noted in the section of the paper on 'Training'. Supervising engineers should therefore be in a position to compile their own list. The form of report was discussed in the section on 'Inspections and Reports'. It was desirable that it should follow the same sequence on each occasion to ensure that nothing was missed, but Mr Ruffle did not see that the style need be other than the choice of the author.

Mr Rose asked for views on the availability and continuity of supervising engineers. The spirit, if not the letter, of supervising engineers' appointments placed them in a position where they were the first point of contact for the workers frequenting the reservoir on any out-of-the-ordinary occurrence or observation, and they should therefore be on hand. This clearly could not apply if they were overseas for prolonged periods and Mr Ruffle considered that appointments as supervising engineer were not appropriate for persons frequently abroad. If absence from the country was only occasional, perhaps it could be covered by named and notified stand-ins who would have been trained and have familiarized themselves with the reservoir concerned.

Mr Ruffle was interested in Mr Shave's account of the arrangements adopted in Southern Water which, in going somewhat further than the requirements of the Act, could only be commended.

In response to Mr Phillips, Mr Ruffle accepted that the frequency of visits by supervising engineers was strictly a matter for their discretion. They would, no doubt, in practice seek accommodation with the views of the construction or inspecting engineer, and also of the undertakers, who might be their employer!

FOURTH SESSION: INTRODUCTION TO SITE VISIT TO CARRON DAM

Session Chairman:

W.P. McLEISH, Partner, Robert Cuthbertson & Partners

Speakers:

W.P. McLEISH and D. GALLACHER, Robert Cuthbertson & Partners

DISCUSSION OF FOURTH SESSION

Discussion

Mr J. MILLER (Central Regional Council) wrote that Mr McLeish, in his introduction to the visit to Carron Valley Reservoir, made reference to just how economical, per gallon of water stored, the original dam construction had been. With due modesty he omitted to point out that not only had he continued this tradition with the remedial works contract for strengthening the dam but in fact had greatly improved upon it with the heightening. For the cost of some concrete to raise the overflow sill by half a metre and a few extra strands of wire in each prestressing cable he increased the storage in Carron Valley by 500 million gallons.

It would be interesting to know just what the cost per unit volume stored was for this additional storage and how it compared to other recently constructed reservoirs of similar capacity.

In addition, he felt he had to comment on Mr McLeish's report on the water resources of the Carron Valley and the conjunctive use of Carron Water with bulk water purchased from the Central Scotland Water Development Board.

He considered that the work was one of the finest and most useful pieces of hydrology ever produced. The savings achieved by using the draft copy (before the printed copy was received) more than paid for the cost of the work! He would recommend that all authorities should give serious consideration to having such a study undertaken where they had cheap gravity supplies augmented by expensive pumped ones. To anyone doing the work he would offer the following guidelines:

1. All records must be carefully reviewed to ensure that the catchment model was as accurate as possible. Sometimes records were not quite what they purported to be.

2. Operating rules for reservoirs must be presented in a form which was easily understood and simple to operate.

3. Operating rules should have a small 'safety net' of reserve storage.

4. Operating rules must be tested so that staff could gain absolute confidence in their use.

Speakers' Reply

Mr McLEISH said that Mr Miller had drawn attention to the value of conjunctive use studies, such as were carried out at Carron Valley Reservoir at a time when remedial works in connection with reservoir safety had in any case to be undertaken.

The general cost of £31 per million gallons (mg) in 1938 would be equivalent to approximately £680 per mg today. The cost of raising Carron Reservoir by 0.5 metres to produce an additional 500 mg was approximately £800 per mg. This remained very cheap in comparison with the cost of construction of new reservoirs. Mr McLeish had not had time to collect other costs, but the Megget Reservoir was a relatively cost effective reservoir by present day standards and the cost per mg for this was £1,926. This well illustrated the advantage of a slight raising of an existing reservoir in appropriate circumstances.

The economic advantage in this case was that so long as conjunctive support was available, the yield to supply of the Carron Valley Reservoir and associated group of other reservoirs could be increased and the annual cost of water could be reduced. The maximum increase in yield was in the order of 25 to 30% of the nominal direct supply yield, while the mean annual net benefit would pay for the cost of raising in 1½ to 2 years.

FIFTH SESSION: CASE EXAMPLES

Session Chairman:

D.J. COATS, Partner, Babtie Shaw & Morton

Paper:

8. The Significance of Problems and Remedial Works at British Earth Dams, by J.A. CHARLES, BSc(Eng), MSc(Eng), PhD, DSc(Eng), CEng, MICE; Head of Dams Section, Geotechnics Division, Building Research Establishment, Department of the Environment

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DISCUSSION OF FIFTH SESSION

Discussion

Mr J.R. CLAYDON (Yorkshire Water, Western Division), in opening the discussion, said that Dr Charles had raised several topics in his interesting paper. Mr Claydon would like to comment on the settlement index as defined on page 128 of the paper.

Table 2 of the paper showed the crest settlement of some central core dams giving settlement indices ranging from 0.004 to 0.009. A quick calculation showed that Cwmwernderi dam was settling at a rate of about 1 mm per year. He was not aware of any embankment dams that settled at such a low rate; certainly it appeared low compared to some of the old embankment dams in Yorkshire. For comparison, he had calculated the settlement indices for those dams for which he had ready access to data.

The distribution of settlement indices for this sample was shown in histogram form in Fig. 5.1. There were 39 dams in the sample. The oldest was built in 1825, the newest in 1970. Their heights ranged from 9 m to 54 m and the period of record used varied from 1 year to 7 years depending on the length of time for which accurate data were available.

Only two dams had settlement indices in the range predicted by Dr Charles. One of these was Scammonden, a rockfill dam with a rolled clay core; the other was a rockfill dam with a puddle clay core built in 1907.

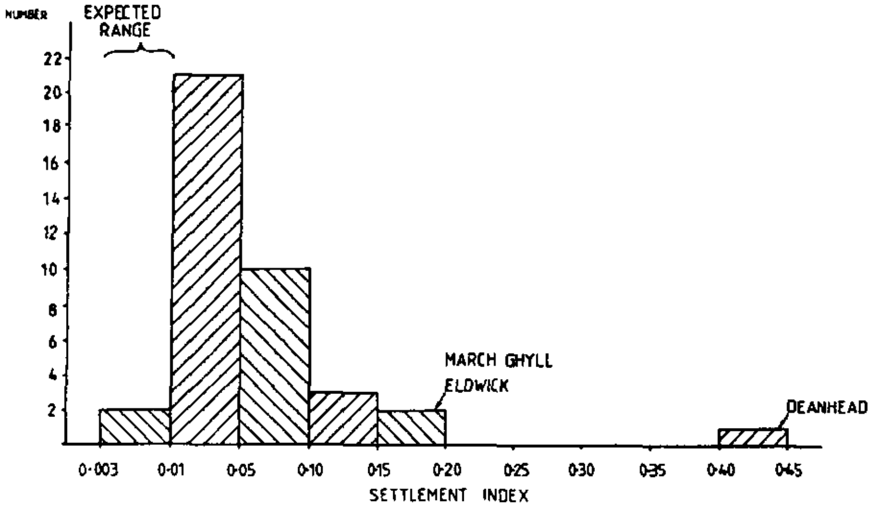


Fig. 5.1. Yorkshire Water, Western Division - crest settlement of central core dams

All the other dams had higher settlement indices, the largest group of 21 being in the range 0.01 to 0.05. There was a skewed distribution with all the remainder, except one, lying below 0.20. Commenting on the three highest settlement indices:

Eldwick, Settlement Index 0.18

This dam, built in 1860, was thought to be in reasonable condition. The settlement index was based on only 1 year of accurate levelling, with a settlement of 7 mm in that period.

March Ghyll, Settlement Index 0.20

This value applied to only one settlement pin on the crest. As this cross section was showing some other signs of stress a site investigation was instigated, the results of which were awaited.

Dean Head, Settlement Index 0.45

This was 19 metres high, built in 1840. The settlement record for the highest cross section for the period from 1977 to 1985 was shown in Fig. 5.2. This illustrated how the way in which reservoirs were operated influenced the settlement behaviour of dams. From August 1977 until August 1984 the reservoir was little used; however, in 1984, pipework connections enabled water to be taken to supply. It was drawn down by 5.9 m when the crest was levelled on 6th September 1984 (point A on Fig. 5.2). At this date the settlement rate was still linear with respect to time. The crest was not levelled again until September 1985, when 26 mm settlement was noted. In this period the reservoir had been completely emptied by 20th September 1984 (point B) and refilled again by 14th November 1984. Mr Claydon suggested that the rate of settlement was affected by the rate of drawdown, the extent of drawdown, and possibly also by the numbers of cycles of drawing down and refilling.

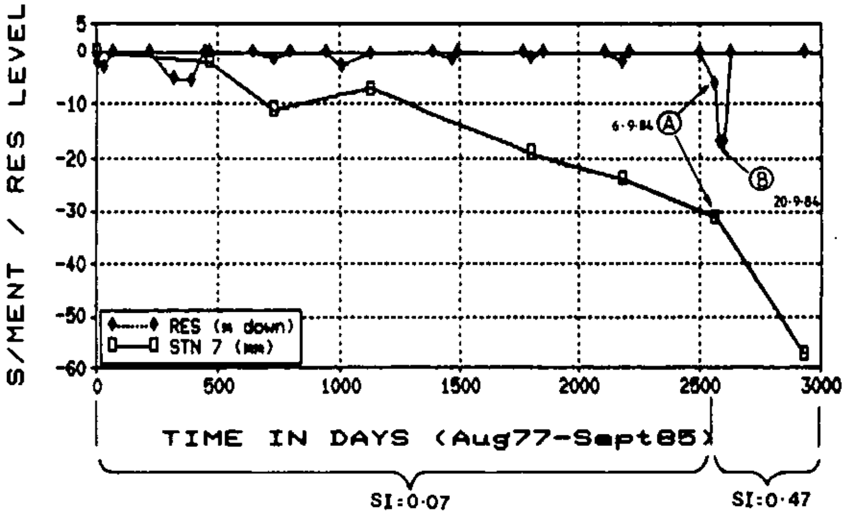


Fig. 5.2. Dean Head Dam - settlement 1977-1985

Figure 5.3 showed the settlement of four stations on the crest of Ramsden Dam, which was 22 m high and was built in 1892. These showed quite clearly that the crest not only fell on drawdown, but that it rose on refilling (points C and D). At this dam they had also recorded horizontal movement associated with reservoir draw-down, with the crest moving upstream on drawing down and moving a greater distance downstream on refilling.

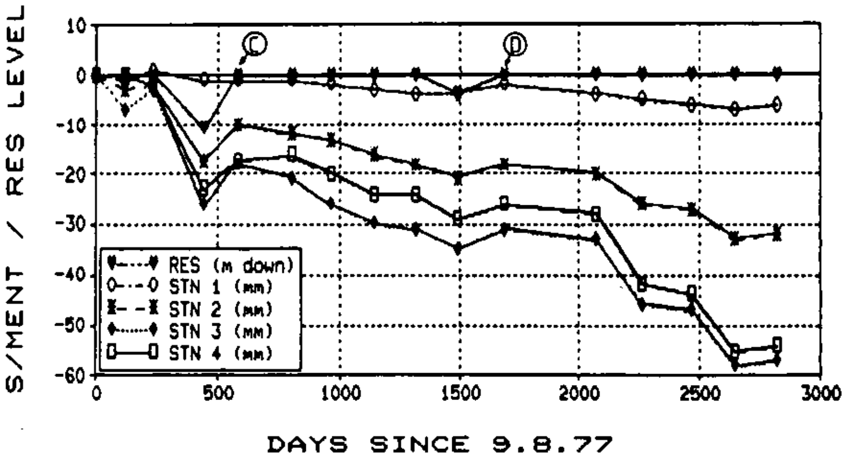


Fig. 5.3. Ramsden Dam - settlement 1977-1985

He suggested that these data showed that the settlement pattern was affected by reservoir operation and that the subject needed further investigation.

Mr P. GRAY (Strathclyde Regional Council Water Department) said that he would like to illustrate the works carried out by Strathclyde Regional Council Water Department at two earth embankment dams in the Lanark Division of Strathclyde Region.

The first of these, at Daer Reservoir, was familiar to some present at the conference from previous presentations but was repeated for the benefit of those recently empanelled as supervising engineers.

Daer Reservoir was built between 1948 and 1956. In the report of the last Statutory Inspection carried out at Daer Reservoir in 1978 under the Reservoirs (Safety Provisions) Act, 1930, the inspecting engineer identified two major areas of concern, namely, the condition of the upstream slope protection of the embankment and the ability of the structure to safely pass the design flood.

The reservoir embankment at Daer was 800 metres long with a maximum height of 41 metres. The upstream face was protected at the time of construction by a facing of in situ concrete laid out in panels 6 ft (1.828 m) square with open joints to allow for relief of any pore pressure in the embankment material. This concrete protection was of very variable quality and was found to be especially vulnerable in the area of the joints. The pounding of the face by wave action led to crumbling of the edges of the panels and continuing wave action in and out of the damaged joints soon created sizeable cavities below the slabs. Increasing damage resulted in either progressive failure across a slab or plucking of almost whole slabs from the face thereby leaving quite extensive areas of unprotected embankment shoulder material exposed to wave action. Temporary in situ concrete repairs had been carried out intermittently over the years as holes in the protection had developed but the weakened concrete, possibly with cavities below, soon suffered further damage around these patches.

The inspecting engineer in his report asked that the upstream face protection be replaced over the whole length of the embankment from the base of the wave wall at the crest to a point on the face 30 ft (9.14 m) vertically below top water level. The replacement protection was specified to be of pre-cast concrete units at least 300 mm deep and of the same areal dimensions as the existing slabs so that these could be replaced progressively across the face thereby minimizing the area of underlying embankment exposed during the construction period.

The work was carried out in two phases, initially from crest level down to 4.5 m below top water level. At the lower level of Phase I a temporary toe beam was formed by means of special precast slabs which incorporated a means of screwing into an in situ concrete key placed beneath the pre-cast units. The intention was that on completion of Phase II these special units would be unscrewed and replaced by standard units which would be capable of movement over the top of the underlying redundant key.

The upper rows of units had integral wave energy breakers on their top surface which had proved to be quite effective in destroying the energy of the incoming waves induced by such storms as had been experienced during the years which had elapsed since completion of Phase I.

The spillway channel was designed to the flood standards in force in the late 1940s/early 1950s. It was designed to contain a catastrophic flood of 192 m^3/s. Daer was a Category A reservoir in terms of the Institution of Civil Engineers' Engineering Guide, 'Floods and Reservoir Safety', the probable maximum flood being calculated as 320 m^3/s. Consequently, the spillway channel and weir basin had required to be redesigned and rebuilt, at a total cost of £2,500,000. Due to the complex hydraulics resulting from a side entry weir drowning out at high flood flows, bends in the upper section of the channel which was relatively flat, and suspected reflected waves in the flow, the required dimensions of the channel were ascertained by means of physical model study.

At the second location, Glengavel Reservoir, also in Lanark Division, the problem identified by the inspecting engineer was lack of adequate scouring capacity. The original scouring facility was provided by means of a substantial steel gate at the foot of the reservoir valve tower. The wire rope lifting gear for the gate had long since fallen into disuse and the only means of scouring was from relatively small sized 4 inch and 6 inch scour branches off the supply main.

As a result, the drop in level of the reservoir when scouring was undertaken was virtually imperceptible. Notwithstanding this slow rate of draw off, when the reservoir was drained to allow the old scour gate to be replaced, the two wing walls retaining the embankment at the outer face of the foot of the valve tower, were seen to be displaced by earth movement behind.

The inspecting engineer in his report asked that in the interests of safety the unusable scour gate be replaced by a concrete plug incorporating a 600 mm pipe and a 600 mm valve discharging at the foot of the valve tower.

Mr M.E. BRAMLEY (Construction Industry Research and Information Association) said that the aim of his brief presentation was to inform delegates of the preliminary results of field trials on the erosion resistance of reinforced grass carried out by the Construction Industry Research and Information Association (CIRIA) during the summer, 1986. The overall objective of the CIRIA research project (which had been supported by BNCOLD) was to develop guidelines and recommendations for the use of reinforced grass in waterways subject to short-duration but extreme hydraulic loading. A major application of the results would be in the design of auxiliary spillways and protection against overtopping on earth dams.

Preliminary design recommendations for steep reinforced grass waterways were produced under the first phase of the project in 1985 as CIRIA Technical Note 120. This initial phase also concluded that the lack of prototype performance data on reinforced grass systems (which were both economically and environmentally attractive) was constraining their use. The field trials installation at North West Water's abandoned Jackhouse reservoir was constructed in late 1984 to fulfil this requirement. It had been reassuring to observe the interest which the trials had generated, particularly that shown by BNCOLD members and the press at the site demonstration held in July 1986.

The field trials installation comprised ten 1 m wide trapezoidal channels constructed on the 1:2.5 upstream slope of the 10 m high dam. A water recirculation system was installed which was capable of delivering up to 1.2 m^3/s to any channel. The testing programme was scheduled so that each channel was tested on up to four different occasions, each test being carried out at a progressively higher velocity until either failure occurred or maximum discharge capacity was reached. Each test took one day, with water flowing for about 6 hours (or less time if failure occurred).

Two different types of grass reinforcement were used: perforated concrete products (cable-tied or interlocking pre-cast blocks and in situ concrete); and, geotextile products (woven fabric, grid and mats). A plain grass control channel was also constructed.

Both the concrete and geotextile reinforced channels performed well in relation to unreinforced grass. Failure (defined as uncontrolled loss of subsoil) occurred in sufficient channels to demonstrate the differences between different types of reinforcement, and to confirm that the recommendations in CIRIA Technical Note 120 were sound. The resistance of individual grass plants to being dragged out of the reinforcing system by the flow was impressive, and recovery of the grass after each test run was excellent. Observations on the performance of different systems were given, as follow below.

Failure of geotextile reinforced channels occurred at higher velocities than anticipated, typically in the range 5 to 6 m/s. Performance was clearly highly dependent on establishing a tight bond between the geotextile and the subsoil, and flow beneath the geotextile must in all circumstances be discouraged. Initiation of failure of the geotextile/grass armour layer was seen to be more velocity-dependent than time-dependent (but clearly subsoil strength and the grass plants could be affected if flow continued for a period of days as opposed to hours). Failure appeared to occur due to a combination of uplift and drag, the uplift problem becoming increasingly apparent with the less permeable geotextiles at high flow velocity (when the potential existed to mobilize high dynamic pressures).

The concrete reinforced channels (which had higher unit cost than geotextile reinforcement) performed well, and both the cable-tied and in situ systems survived the maximum velocity of about 8 m/s. The extent of flow aeration and turbulence at these velocities was impressive, and engineering judgement tended to suggest that overtopping head should be limited to ensure that substantially higher velocities did not occur. A flow velocity of 8 m/s had a velocity head in excess of 3 m; with higher velocities, the engineer faced an ever escalating problem of energy dissipation at the base of the slope. On a typical 1:2.5 slope with the observed hydraulic roughness, the discharge intensity during overtopping should thus be limited to about 1.0 $m^3/s/m$ width.

Failure of the interlocking block system (which was not underlaid by a geotextile) by undermining clearly demonstrated that concrete systems must be installed in conjunction with an underlying geotextile which formed the ultimate barrier against subsoil erosion. The function of the concrete was ultimately related to stability rather than erosion protection. The way that the interlocking block system (which was intended to be relatively flexible) bridged over the erosion gully which formed in the subsoil demonstrated that so-called flexible systems could not be relied upon to deform when localized gullying occurred.

Fortunately for CIRIA, the plain grass channel failed by formation of an erosion gully at a velocity of about 4 m/s, equivalent to an overtopping discharge intensity of about 0.2 $m^3/s/m$ width. Had this not occurred, no doubt some of the 20 sponsoring organizations would have asked for a refund! The plain grass channel also failed by stripping of the grass mat. The grass had been sown in a topsoil layer which had discouraged deep root penetration into the subsoil; clearly, attention always needed to be given to achieving integration between the armour layer and the subsoil.

In comparing the failure velocities quoted for geotextile systems and the demonstrated safe velocity for concrete systems, the non-linear relationship between velocity on the slope and overtopping discharge intensity must be appreciated. Thus, safe equivalent discharge intensities for geotextile systems might be about 0.5 $m^3/s/m$ width. The attendant risk of vandalism with geotextile systems also needed to be noted.

In conclusion, Mr Bramley pointed out that these observations were preliminary and that the Steering Group had yet to conclude their review of the results. When this was completed, CIRIA would issue a revised report which would incorporate the results of the Phase 2 activities into the existing Technical Note. In the meantime, Technical Note 120 remained the recommended design guide.

Acknowledgement was made of Salford Civil Engineering Ltd and Tarmac Cubitts Ltd who carried out the field trials; to Rofe, Kennard & Lapworth, the Institute of Terrestrial Ecology, and Hydraulics Research Ltd who had provided services in both phases of the work, and to the funders, particularly the Department of the Environment under their Reservoir Safety research programme and North West Water, for their extremely supportive attitude to this collaborative project.

Mr G.R. POWLEDGE (United States Bureau of Reclamation) said that he first wanted to express his thanks for the very hospitable reception that BNCOLD had given to him as a visiting delegate from the United States. He should certainly like to reciprocate this at some future date.

He wanted to describe the work* which the Bureau of Reclamation was undertaking to facilitate the safe overtopping of some of

*Powledge, G.R. and Dodge, R.A., 1985, 'Overtopping of Small Dams - An Alternative for Dam Safety', ASCE Hydraulics Division Conference, Orlando, Florida.

their small dams in extreme flood events. The need for this stemmed from the same deficiency in capacity of overflow works which had been described by several earlier speakers. The Bureau was particularly interested in the work on reinforcement of grass spillways currently being carried out by CIRIA; the principal reason for his visit was in fact to arrange for the exchange of information on their two programmes.

It was commonly assumed by US engineers that when an embankment dam was overtopped, erosion on the downstream slope and toe of the dam would lead to dam failure. Consequently, overtopping was not permitted in the design of an embankment dam. Of the approximately 8,500 dams inspected by the Corps of Engineers under the non-Federal Dam Safety Programme, over 3,000 dams were found to be potentially unsafe. Of these potentially unsafe dams, about 85% were unsafe due to inadequate spillway capacity or insufficient dam height which could result in dam failure due to overtopping.

The probable maximum flood (PMF) had been used by the Bureau of Reclamation as the inflow design flood (IDF) for new storage dam designs and for modification of existing dams if dam failure could cause potential loss of human life or significant property damage. Due to the larger storm data base now being considered, the probable maximum storm (PMS) and PMF magnitudes used for design of new dams and modification of existing dams had increased significantly. As a result, design of new dams and spillways or modification of existing dams for the revised and often larger PMF had become very expensive. In some instances, it might not be physically or economically feasible to accommodate large floods without overtopping. In fact, some embankment dams had been removed because of excessive costs. However, some embankments had survived moderate overtopping. Therefore, it had been decided that some existing embankment dams, especially those less than 50 ft (15.2 m) could be modified to safely permit overtopping without losing the reservoir pool.

The Bureau of Reclamation had modified a number of dams to accept overtopping, the most common means of protecting the downstream slope of earth dams from erosion being use of roller-compacted concrete. A rockfill dam was also currently being constructed to resist overtopping. This would be accomplished by virtue of a cable net over the downstream face which was tied back into the body of the fill.

Research Programme

A research effort including a model study was initiated in 1983 by the Bureau of Reclamation to gain insight into the development of cost-effective modifications to small embankment dams which would enable them to withstand overtopping. In addition, the Bureau, in assisting the National Park Service, had initiated design studies to modify two dams in the Blue Ridge Parkway, North Carolina, to allow overtopping without the loss of their reservoir pools.

Using embankment dams owned by the National Park Service as a basis, it was decided that:

1. The model should represent a 32 ft high (9.8 m) dam constructed of materials similar to those of the Blue Ridge Parkway dams.

2. The model embankment and overflow system should represent 4 ft (1.2 m) of water overtopping the crest of the dam for a period of 4 to 6 hours.

Soil Tested

The soil used for all test runs was a local clayey sand with a liquid limit (LL) of 25% and a plasticity index (PI) of 9%. The soil compaction properties were used to obtain the desired test density of 95% standard Proctor for all but one test run placement which was overcompacted to 102%.

The top of each finished layer was scarified before placing new test embankment and providing scour protective treatments.

A 6:1 slope was used for early tests but this was later increased to 4:1 because of the high stability experienced with earlier tests. A 6:1 slope was the erosion stability breakpoint. Noncohesive material on this slope had about 75% of the critical tractive shear resistance of that of the same material on a flat bed. As slope increased, erosion resistance decreased rapidly; as slope decreased, erosion resistance increased slowly. Noncohesive material on a 4:1 slope had about 60% erosion resistance relative to flat bed flow resistance.

Model Scaling

Froude scaling could be applied only to the flow near the top of the dam where friction did not dominate. Length scaling of 1:15 was selected to make the 2.12 ft high (646 mm) model dam represent a typical National Park Service embankment dam about 32 ft high (9.8 m). Equations for frictional flow and noncohesive sediment transport did not apply for shallow flow, steep and rapidly accelerating flow, nor for chutes and pool flow. Also, no adequate governing equation for cohesive soil erosion existed. Without these equations, no reliable sediment transport time nor velocity scaling relationships could be determined. This was not only a problem of using a small scale model but it was also a hindrance in making predictions from experience with one full size dam to another full size dam. To estimate sediment transport time without equations, modellers generally compared a nonrandom prototype event of significant sediment transport quantity and time duration with model performance. The random nature of soil particles and construction techniques made test replications desirable. Thus, these model results were considered qualitative, more probably indicating which treatment of those tested worked better rather than determining how much better.

Model Operation

A total of nine tests were conducted. For most test arrangements, the unit discharge represented was 40 $ft^3/s/ft$ (3.7 $m^3/s/m$). Test Run 7 arrangement was replicated for Test Run 8 but operated at a unit discharge representing 87 $ft^3/s/ft$ (8.1 $m^3/s/m$). Test

Run 1 lasted only 17 minutes because erosion was considered to be excessive and more consideration needed to be given to the boundary effects of the model and the smoothness of the hard cap at the crest. Therefore, 17 minutes (about 1 hour prototype time) was used as a common model test run time interval for the remaining tests to compare test arrangement erosion. Test Runs 4, 6, 7, and 8 were also operated for an additional hour which would be representative of 5 hours of overtopping which was expected for the National Park Service dams being considered for modification.

Results

Table 5.I contained a brief summary of model embankments tested and results of each test run. The following results and implications were noted during the studies, tempered with experience from other sediment model studies and conversations with people with experience of overtopping cases.

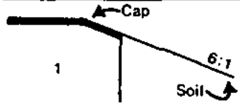
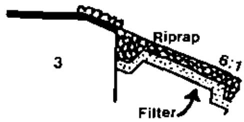
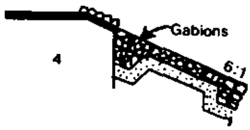
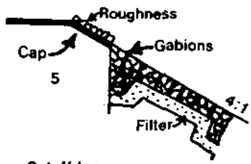
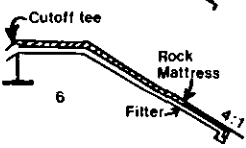
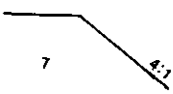
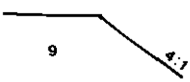
During Test Run 1 with a 6:1 slope, the flow rapidly transformed into a chutes and pool mode. This type of flow was initiated by shallow bank surface waviness or by jets and vortices caused by flow around and over isolated projecting rocks in the fill material. Brush and trees would cause local areas of increased scour. Scour holes tended to lift flow resulting in intermediate areas of less scour between areas of increased jet scour. Road pavements, curbs, parapets, and bulging or sagging of top faces of gabion compartments could initiate and affect the location of scour.

The erosion data for the 5-hour test runs were used to see if the rough later flow had a greater or lesser transport rate than the earlier smoother flow. For these tests the sediment transport rate for the last 4 hours averaged about a quarter of the rate for the first hour. Thus, it appeared that the transition from chutes and pool mode flow reduced erosion relative to earlier smooth flow erosion. For some unexplained reason, no erosion was detected for the last hour of Test Run 8 using the techniques for measuring volume.

During Test Run 1, the smooth vertical walls of the model appeared to have exaggerated scour causing deep holes in the embankment near the flume walls and the vertical curtain wall of the hard protective crest cap. Reservoir side and bottom roughness and form roughness caused eddies that were lifted by the upstream slope of the dam and stretched out parallel to the bed. These bed parallel vortices caused local areas of relatively more intense scour. Crest end treatments that contracted the flow sideways also caused vortices that intensified scour.

When flow over a fixed bed made a transition to over a soil bed or vice versa, scour occurred. This was caused by the boundary layer being forced to adjust to a change in boundary rugosity. Going from the hard crest cap to soil was an example of this type of scour. For Test Run 2, pea gravel was epoxyed to the downstream sloping part of the hard crest cap simulating 3 to 6 inch (76-152 mm) cobble roughness to increase depth and dampen vortex action. This increase of roughness resulted in significantly less, and more uniform, scour than for Test Run 1. This effect also occurred for flow making the transition from over gabions to over soil.

TABLE 5.I. Summary of Results of Overtopping Flow Model

Schmatic sketch and run numbers	Unit discharge, ft³/s/ft (m³/s/m)	Time, hrs	Erosion of available volume of material, %	Least depth of soil on 2.5:1 slope, ft (m); Location from crest, ft (m)	Deepest scour hole, ft (m); Location from crest, ft (m)
 1	40 (3.7)	1	16	-8(-2.4) 15(4.6)	10(3.0) 15(4.6)
 2	40 (3.7)	1	7	0(0.0) 10(3.0)	9(2.7) 106(32.3)
 3	40 (3.7)	1	13	-6(-1.8) 10(3.0)	8(2.4) 10(3.0)
 4	40 (3.7)	1	2	9(2.7) 41(12.5)	2(0.6) 84(25.6)
		5	5	8(2.4) 41(12.5)	4(1.2) 95(29.0)
 5	40 (3.7)	1	12	-1(-0.3) 53(16.2)	3(0.9) 59(18.0)
 6	40 (3.7)	1	4	4(1.2) 59(18.0)	6(1.8) 104(31.7)
		5	8	2(0.6) 84(25.6)	10(3.0) 84(25.6)
 7	40 (3.7)	1	9	-0(-0.0) 0(0.0)	5(1.5) 77(23.5)
		5	14	-2(-0.6) 0(0.0)	9(2.7) 59(18.0)
 8	87 (8.1)	1	6	-0(-0.0) 0(0.0)	4(1.2) toe
		5	6	-1(-0.3) 0(0.0)	6(1.8) toe
 9	87 (8.1)	1	13	-1(-0.3) 0(0.0)	3(0.9) toe

Despite the previously discussed modelling limitations, velocity, depth, and discharge were scaled by Froude's law for comparison. The scaled velocities of all test runs rapidly reached more than 30 ft/s (9.1 m/s). Riprap design methods did not account for the combination of high velocity, steep downslopes, and shallow flow. During Test Run 3, model riprap representing 6 to 24 inch diameter (152-610 mm) became fluidized and eroded out immediately. Rock contained in mesh compartments was tested next. Gabions and mattress pods modelled for Test Runs 4, 5, and 6 represented about 4 inch (102 mm) mesh compartments filled with angular rock up to 12 inch (305 mm) maximum diameter on a filter bed. During these tests there was no indication of a threat of them being dislodged by the overtopping flow. However, manufacturers of gabions had no backup data for velocities greater than about 24 ft/s (7.3 m/s). On the steeper slope, 4:1, the main erosion was just near the downstream end of the protection. Whereas for the 6:1 slope the main scour hole was 45 ft (13.7 m) downstream. Comparing Test Runs 4 and 5 indicated that the effect of increasing the down slope from 6:1 to 4:1 increased the scour volume about 5 times. The maximum depth of scour was about two times greater. A minor part of the differences could be attributed to sagging and bulging of gabion compartment tops.

Model test results of Test Runs 8 and 9 indicated that increasing standard Proctor compaction from 95 to 102% resulted in half the erosion with similar protective treatments tested.

Comparing the test results of Test Runs 7 and 9 showed that increasing unit discharge from 40 to 87 $ft^3/s/ft$ (3.7 to 8.1 $m^3/s/m$) resulted in about 40% increase in erosion.

Despite exercising care during construction and inspection, there were always low areas along the crest. Low areas could occur after construction because of crest traffic, nonuniform settling, and lack of maintenance. The mode of erosion could depend on the shape of hydrographs during overtopping. For example, with slow rising or low constant flow the erosion might start and remain at one low point causing gulley type erosion. If flow rose fast enough, the erosion could act more like sheet flow erosion despite a low area.

A well co-ordinated inter-agency team approach was necessary to fund and make positive progress in solving a problem of this magnitude and expense. The results of these studies clearly illustrated that erosion during overtopping flow was a multivariable and multi-disciplinary problem. Random aspects such as the many variables, lack of true soil homogeneity, different soil classifications, and hydrograph variations presented a strong case for more repetition of model tests and uniform documentation of failures in the field.

Much long term effort was needed to develop adequate governing equations for rate of sediment transport on steep slopes. In reality these equations were needed not only for scaling sediment time and velocity and mathematical modelling, but for making more rational inferences from one prototype experience to another and to new design cases.

Mr H.R. SAMUEL (Consulting Engineer) wrote that Lower Lliw Dam, originally constructed in the 1860s, underwent a major repair in 1879 after serious seepage had appeared near the base of the downstream slope. A shaft was put down through the core and drains were constructed to carry flow from a spring in the core foundation to a rubble drain which had been incorporated in the original construction. It was recorded that the puddle clay core had been seriously eroded and replaced with sandy materials from upstream.

Seepage appeared on the downstream slope again in 1882 and the dam was never considered safe thereafter and was kept drawn down.

In 1976/78 the dam was demolished and reconstructed on the same alignment. The 1879 remedial works were examined. The puddle clay was found to be in good condition both inside the shaft and elsewhere and no evidence was found that the core had been punctured a second time, although there had been considerable settlement of the crest centred on the shaft. The rubble drains, where identified during the demolition, were crushed and blocked.

When the old dam had been removed, springs were found in the downstream foundation. These springs flowed during the winter even when the adjacent core contact was being dewatered during injection of the grout curtain and core contact preparation for the new dam.

Flows from the blanket drain under the downstream shoulder of the new dam monitored while the reservoir was drawn down during construction and in the first two years after refilling were as shown in Table 5.II.

TABLE 5.II. Flows from the Blanket Drain

	Reservoir empty	Reservoir full
Minimum flow in summer dry weather, l/min	Nil	6
Minimum flow in winter dry weather, l/min	20	25
Maximum flow* (winter storm), l/m	100	270

*The maximum flows include some surface water

These figures illustrated how the majority of winter flow measured at the downstream toe originated in the downstream foundation and abutments. It was possible that the seepage that broke through the downstream slope in 1882 did not come through the core, but was caused by blockage of the rubble drains.

Author's Reply

Dr CHARLES said that a large proportion of the contributions to the discussion on his paper had been related to slope protection for embankment dams. Messrs Eldridge (see page 56), Bramley and Powledge had referred to the erosion resistance of downstream slopes during overtopping. Mr Eldridge had stated that a well-grassed downstream slope of a well constructed earth dam would withstand a few hours of being overtopped without suffering significant damage providing that it was not built of very sandy or silty material. He had given an example of a low earth dam which had twice been overtopped to a depth of about 30 cm during the last 50 years without ill effects. Similarly, Kennard (1975; for reference list see page 84) gave examples of low embankments 5 to 6 m high that had withstood overtopping. Penman (1972) had drawn attention to the fact that when Warmwithens dam was breached in 1970 and the full reservoir emptied into Cocker Cobbs and Jackhouse reservoirs, the embankment dam at Cocker Cobbs was overtopped but did not fail. The Institution of Civil Engineers' report (ICE, 1978) suggested that earth embankments with a level crest and a surface roadway might well resist some overtopping successfully. A Construction Industry Research and Information Association report (CIRIA, 1976) had quoted velocities that suitably chosen grass could withstand for various periods. The CIRIA report also pointed out that a badly maintained embankment with surface irregularities, shrubs and bare patches would have a much reduced resistance to erosion. An interesting warning was given by Sowers (1961) that although in a few instances earth dams of clay had withstood overtopping by a 0.3 m or 0.6 m depth of flow for several hours without failure, a large proportion of the few dams supposedly designed to withstand overtopping, and which had been protected against erosion, had been carried away. Details had been given of 10 overtopping embankments in China, but these had been protected by masonry, concrete or bituminous concrete (Shen Chonggang, 1985).

The subject was one of considerable practical importance. Revised estimates of floods might necessitate increased overflow capacity at existing dams. Large expense could be incurred in providing new or enlarged spillways. If some overtopping could be permitted during extreme events then much expense might be saved. With this background the work that had been done by CIRIA in the UK and the Bureau of Reclamation in the USA on the protection of downstream slopes was very welcome. Mr Bramley had presented a summary of the preliminary results from the CIRIA field trials on the erosion resistance of reinforced grass carried out during summer 1986 in 1 m wide channels on a 1:2.5 slope at the abandoned Jackhouse reservoir. Both concrete products and geotextiles were used. Clearly, this major field experiment would be referred to by engineers for many years to come. Mr Powledge described the work carried out by the Bureau of Reclamation in the laboratory on a model scale with 1:4 and 1:6 slopes 0.6 m high. A clayey sand was used. The effect of rip rap, gabions and a rock mattress was investigated.

Upstream slope protection was also an important subject and was referred to in the contributions of Messrs Bramley, Dunster (see page 90) and Gray. Mr Gray described the damage due to wave action sustained by the upstream facing of in situ concrete panels

1.8 m square with open joints at the Daer dam. The dam was completed in 1956 and damage had been so severe as to necessitate complete replacement of the upstream face protection. The storm force winds in January 1984 caused serious damage to precast concrete blockwork at Kielder dam (Rocke, 1985) completed in 1980 and to rip rap protection at Megget dam completed in 1981. The current investigations of Hydraulics Research into both wave prediction in reservoirs (which had included wind and wave measurements at Megget reservoir) and protection of embankment dam faces against wave attack should provide valuable information. Mr Bramley pointed out the significantly different mechanisms of behaviour exhibited by permeable and impermeable protective layers. Mr Dunster considered that the failures of wave protection facings at Kielder and other embankment dams was due to the use of a relatively impervious facing on top of a permeable drainage layer. He recommended the use of a type of permeable armour block placed on a permeable drainage layer.

A settlement index for puddle clay core dams was introduced in Dr Charles' paper and three contributions referred to this. Dr Pugh (see page 86) correctly pointed out the problems of using a logarithmic parameter based on the coefficient of secondary compression. However, it should be noted that the settlement index had been introduced to distinguish between those cases where long term crest settlement might be explained by secondary compression and those cases where it clearly could not. Thus, the significance of the secondary compression did not lie in its precise magnitude but rather in its order of magnitude. Mr Claydon supplied data on the crest settlement of 39 dams in West Yorkshire in the form of a settlement index histogram. The information on the effect of reservoir drawdown and refilling was obviously of great importance. There was a need for much more detailed information about the settlement of dams which were subjected to major drawdown in order to isolate this effect from the general settlement pattern. Mr Gallacher (see page 85) presented information relating to the investigation of March Ghyll dam. The publication of such information about old puddle clay core dams was extremely valuable.

Mr Samuel provided information about seepage flows associated with the reconstructed Lower Lliw dam (the dam was rebuilt in 1978). It was instructive to compare these flows with the leakage measured in the early years of the original Lower Lliw dam built in 1867 and quoted by Binnie (1981). It was seen from Table 5.III that the flows measured at the new dam were comparable in magnitude with those that occurred initially at the old dam but much smaller than those recorded in 1873 when serious problems became evident.

Mr Samuel's data showed a strong correlation between seepage flow and rainfall. His comment that most of the water flow originated in the downstream foundation and abutments was particularly pertinent. The high pore water pressures that caused instability in the downstream slope of Lambieletham dam were found to be due to seepage from the abutment rather than leakage through the core.

Mr Milne (see page 86) asked for further comments on this. It seemed likely that many cases of downstream slope instability were caused by water seeping around or under cores or flowing in from the abutments rather than water coming through leaking clay cores.

TABLE 5.III. Seepage at Lower Lliw with Reservoir Full

	Flow, l/s
Old dam (1867-1872) after heavy rainfall	1.4 - 2.9
Old dam (1873) turbid water	26
New dam (1978-1980) dry weather, minimum flows	0.1 - 0.43
New dam (1978-1980) winter storm, maximum flow	4.5

References

Binnie, G.M., 1981, 'Early Victorian Water Engineers', Thomas Telford, London, England.

Construction Industry Research and Information Association, 1976, 'A Guide to the Use of Grass in Hydraulic Engineering Practice'. Technical Note No. 71.

Institution of Civil Engineers, 1978, 'Floods and Reservoir Safety: An Engineering Guide'. London, England.

Kennard, M.F., 1975, BNCOLD Newcastle University Symposium, D5/9-10.

Penman, A.D.M., 1972, JIWE 26, 147-148.

Rocke, G., 1985, 15th ICOLD 5, 559-562, Lausanne, Switzerland.

Shen Chonggang, 1985, 15th ICOLD 5, 524-529, Lausanne, Switzerland.

Sowers, G.F., 1961, 'Earth dam failures', Earth and Rockfill Dams Symposium, Roorkee, India, 196-216.

SIXTH SESSION: PANEL DISCUSSION

Session Chairman:

R.E. COXON, Chairman, Engineering and Power Development Consultants Ltd

Panel (the authors of all papers):

F.G. JOHNSON
S.C. AGNEW
N.H. GIMSON
R.G. SHARP
A.D.M. PENMAN
J. LEWIN
J.R. WHITING
W.P. McLEISH
N.J. RUFFLE
J.A. CHARLES

PANEL DISCUSSION

Discussion

Mr D. GALLACHER (Robert H. Cuthbertson & Partners) said that 16 settlement monitoring stations were established along the top of the embankment to March Ghyll reservoir near Ilkley during a statutory inspection in November 1984. The embankment had settled up to about 1.04 m between its construction in 1904/06 and 1970, and in 1971 the settlement was made up by raising the wave wall and embankment crest. The maximum height of the embankment was about 20 m.

Levelling in 1984 indicated continuing settlement of the embankment at a rate of about 4.5 mm per year since the raising works. During the first 12 month period from November 1984 after establishment of the monitoring stations there was a general settlement of about 5 mm over the full length of the embankment, but one levelling station on the main embankment apparently settled about 24 mm. Further levelling checks confirmed that settlement was continuing with the maximum rate at the point of previously measured maximum settlement.

The settlement indices (SI) for the average and maximum settlements were 0.065 and 0.26, which were both well above the settlement index figure of 0.02 which Dr Charles suggested should merit further investigation. The average settlement index of 0.065 was just below the settlement index stated (0.08) for the nearby Walshaw Dam lower dam, where investigation showed softening of the core below original ground level.

There were other anomalous features on the embankment local to the area of maximum settlement which it was considered might be related. A recommendation was made that a site investigation of the puddle clay core and shoulder fill should be carried out using continuous sampling techniques in the puddle clay core. The site work had been completed but laboratory testing had not. The site vane test results for the continuous undisturbed samples from the borehole in the area of maximum settlement were shown in Fig. 6.1, and also the moisture contents, and plastic and liquid limits. The puddle clay plotted above the A-line on the plasticity chart, and above the original ground level, which was at a depth of about 17 m, it varied from very soft to soft but was generally firm below this level. A particularly weak area was indicated at between 9 and 14 m depth. A spade-shaped total pressure cell was installed in an adjacent borehole in the weak area, as at Walshaw Dean lower dam, to establish the minimum horizontal pressure in the longitudinal direction of the core and its susceptibility to hydraulic fracture. However, results were not yet available.

The establishment of a sufficient number of settlement monitoring stations had enabled an accurate assessment of overall embankment settlements to be made and areas of unusual settlement to be detected. (See page 83 for reply from Dr Charles.)

Mr G.A. MILNE (Crouch & Hogg) said that it had been suggested at this and previous meetings that perhaps there was a risk that they were paying too much attention to flood assessments, important though they were, possibly because they had recent techniques available, and were paying correspondingly less attention to seepage and settlement problems which were much more difficult to assess.

In this context, could Dr Charles comment further on the Lambieletham Reservoir slip and subsequent abandonment. In particular, Dr Charles stated that no defects had been found in those parts of the clay core which were examined and that seepage from the adjacent hillside was suspected as the cause of embankment slip. (See page 83 for reply from Dr Charles.)

Dr R.S. PUGH (A.G. Weeks & Partners) said that Dr Charles had proposed the use of a 'secondary compression index' to assess whether the settlements being experienced by an earth dam should be cause for concern. Data from settlement measurements of earth dams (given by a previous speaker) illustrated that this index varied considerably for earth dams which were not apparently showing signs of distress. In particular, the problems associated with changing rates of settlement as reservoir water level was altered were highlighted and acknowledged by Dr Charles. The coefficient of secondary consolidation of natural soft deposits was also known to vary widely with, for example, the water content and plasticity of the soil. It was difficult to measure the

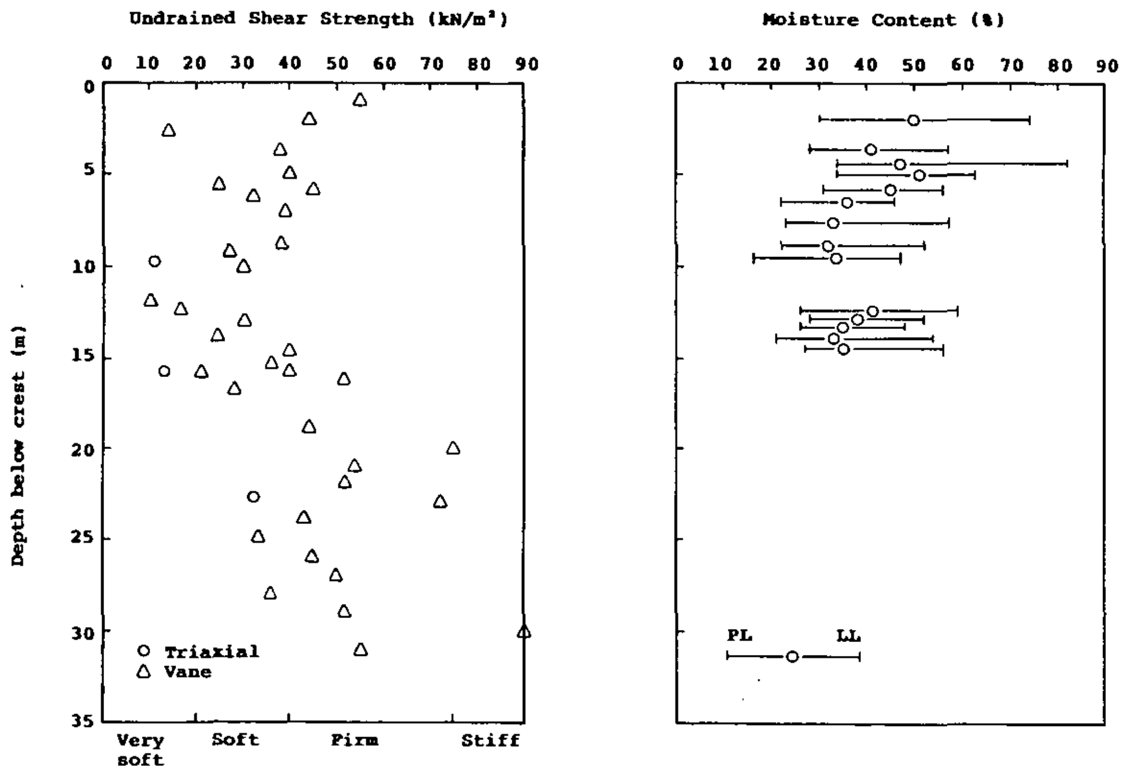


Fig. 6.1. Properties of puddle clay core at March Ghyll dam

coefficient of secondary consolidation under laboratory conditions and even more difficult to separate it from other effects in the field. Given the variety of clays used in dam construction, the wide range of water contents likely at the end of primary consolidation and the cyclic changes in water content which would occur during reservoir operation, he wondered if the index proposed by Dr Charles would in fact be sensitive enough to predict detrimental changes in embankment behaviour. (See page 83 for reply from Dr Charles.)

Regarding Dr Penman's paper, he wondered if some of Dr Penman's 'misconceptions' were quite as clear cut as the paper seemed to indicate. While accepting Dr Penman's prognosis on the need to maintain total stresses above reservoir pressure in order to avoid the possibility of hydraulic fracture, this, as pointed out by Mr Haws, need not necessarily be at the expense of high construction pore pressures. Many earth dams had been successfully constructed in, for example, the tropics, using wide cores of clays known to be erosion-resistant, placed dry of optimum with very low construction pore pressures. In many such cases, adoption of a wet core would have led to significant cost increase because of the need to add significant amounts of scarce water. As Dr Penman had indicated, earth dam engineering seemed to follow a 'fashion pendulum' from, for example, wet cores to dry cores to wet cores and so forth. They should remember that every earth dam design had its own particular technical and economic requirements. An earth dam core might be constructed in a dense fluid state with no compaction, or dry of optimum with heavy compaction depending on, among other things, climatic considerations. Likewise, despite Teton Dam, dams had been built on silty materials and on fractured rock foundations and, despite Carsington, dams had been built with upstream clay blankets. It was a misconception to think that there were hard and fast rules for embankment dam design. Thanks, however, to papers such as Dr Penman's, they had at their disposal broad guidelines, based on experience, to help them overcome the problems invariably posed by both foundation and fill materials.

Mr G. ROCKE (Babtie Shaw & Morton) said that the reference by Mr Johnson in the BNCOLD Lecture to damage to dam face materials by wave forces at Orrin and Glascarnoch Dams, Scotland, were valuable in bringing to the attention of the Conference the paucity of design knowledge which existed in relation to the mechanics of wave action in large UK reservoirs. Damage had been reported in modern large dams where severe storms had affected concrete facings at Orrin, Blithfield, Daer, and Kielder, and rip-rap at Megget and Foremark. This list was by no means comprehensive. The relationships between regionally recorded wind velocities and actual wind velocities on reservoir surfaces, wave spectra adjacent to the dam slopes and wave forces in the voids of the slope materials were relatively scarce in existing UK design manuals. Efforts were currently being made to remedy the situation with limited funds and included field studies by Hydraulics Research and theoretical and laboratory studies at the University of Strathclyde. To enable these studies to be fruitful, a wider interest was required and the Construction Industry Research and Information Association (CIRIA) would seem to be an appropriate organization to stimulate such interest leading to design guidance on economic and aesthetically acceptable reservoir slope protection methods.

Commenting on Dr Penman's paper, Mr Rocke said that the period 1960/80 saw the design and construction of earth embankment dams of considerable size in the UK. These dams, 40 to 50 m in height, involving the placement of up to 4 million m^3 of material, were built rapidly by large capacity trucks and motor scrapers. The materials of construction varied, but most were sensitive to softening by heavy rainfall. Consequently, contractors placed fill very quickly indeed in the limited dry weather period of June to September. Designers of such dams made reasonable allowances for variations in material characteristics, compaction conditions and rates of construction. To enable the sufficiency of these allowances to be assessed during construction and to permit timely corrective measures to be made, many monitoring devices were built into the dams. Valuable information on stability was derived and enabled dams to be completed safely under controlled conditions. The control adopted for each condition was preconceived and implemented in good time as at Derwent (see Mr Ruffle's contribution) and at Kielder. It thus seemed unfortunate that Dr Penman should invert the situation at these dams and offer them to the Conference as misconceptions in design. Designers surely had to have the freedom to incorporate such controls as an extension of their designs in the construction of large projects.

Mr M.E. BRAMLEY (Construction Industry Research and Information Association) said that he felt he had to add a few points to the discussion on the protection of the upstream face of a dam against wave attack, both on behalf of CIRIA and from the viewpoint of the hydraulic design engineer.

Firstly, there seemed to be confusion in some sectors about how the protection interacted with the wave. Two types of protection were common on UK dams: blockwork (which might be masonry or concrete) and rock (which might be closely packed pitching or have a relatively open structure in the surface layer, e.g., riprap).

When the wave impacted on the upstream face of the dam, it created a rapidly varying hydraulic flow field. If the protection layer was relatively impermeable, this flow field was largely external to the protection, which had to sustain impact forces and surface drag forces due to wave up-rush and down-rush. If the protection layer was relatively permeable (either by virtue of open joints in the blockwork and varying degrees of porosity in the underlayer, or due to the packing and gradation of rip-rap), then a more complex flow field was set up around the protection. This flow field, while providing the potential for improved energy dissipation and reduction in wave run-up, could give rise to substantially larger out of balance forces on the protection. These facts had been observed experimentally and in prototype performance. They were perhaps better encompassed within accepted design practice in coastal engineering than in dam engineering.

For a given unit size of block, the protection would be more stable if the joints and underlayer were relatively impermeable (some traditional forms of coastal revetment were laid closely-jointed on clay). Grouting or overlapping of joints between individual blocks was thus beneficial from the flow field viewpoint. Interconnection also improved the stability of the protection since locally high forces resulting from wave action were distributed more widely.

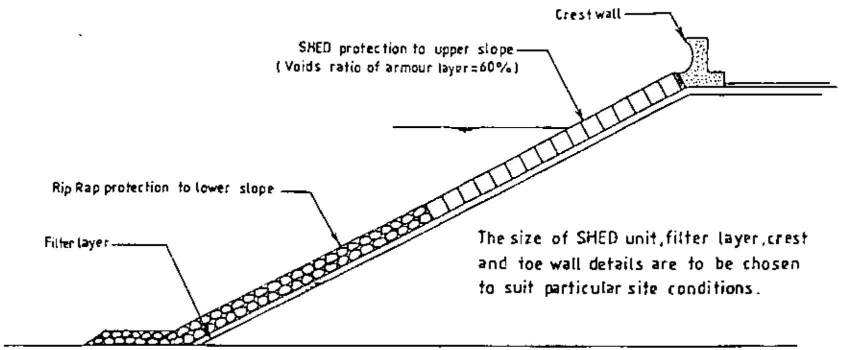
Secondly, it was important for the dam engineer to appreciate the influence of the time period over which inflow or drainage took place on whether or not the protection appeared permeable. Thus, a protection layer could be made relatively impermeable to wave action (which was essentially short period), while remaining relatively permeable from the viewpoint of allowing drainage of the upstream shoulder to take place when the reservoir was drawn down.

While research was undoubtedly needed to provide a better understanding of these complex phenomena, he believed that better use could be made of the existing empirical and qualitative understanding that engineers (and here he included dam, coastal, and inland waterway engineers) had built up. Why not, for example, consider bitumen pattern grouting to improve the stability of some of the blockwork on existing dams?

However, he had to add that UK research had been responsive in this field. CIRIA had produced a design report on rip-rap, and he was aware that Hydraulics Research were currently looking at the stability of blockwork in the laboratory under DoE Reservoir Safety research funding. CIRIA was shortly to publish a short Technical Note on coastal revetments, and had duly noted the interest in further research or information expressed by delegates at the conference.

Mr J.A. DUNSTER (Shephard, Hill & Co., Ltd) said that with regard to the failures of wave protection facings at Kielder and other embankment dams, it appeared to him that the problems had arisen due to the use of a relatively impervious facing on top of permeable drainage layers. In his opinion, the solution was to use single layer wave energy dissipators of the 'SHED' (see Fig. 6.2) or 'COB' type placed on a permeable base.

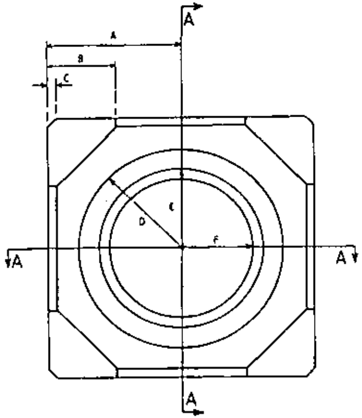
A protective layer of 'SHEDs' with edge restraints could be designed to withstand any wave conditions likely to be encountered in reservoirs. To date, they had been unable to cause failure of these units either in the laboratory or in service and they might be interested to know that further research into the design of single layer armouring was about to be carried out in the UK. (See page 82 for reply from Dr Charles.)



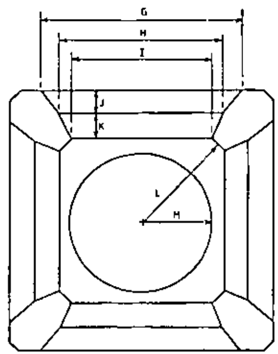
TYPICAL SECTION THROUGH UPSTREAM SHOULDER OF EARTH DAM.

SECTION PROPERTIES.

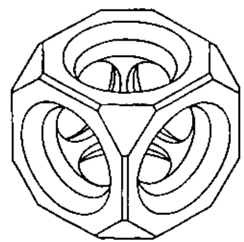
Dim.mm	1300mm SHED	650mm SHED	433mm SHED	325mm SHED
A	650	325	217	162
B	335	167	112	84
C	40	20	13	10
D	500	250	167	125
E	400	200	133	100
F	350	175	117	88
G	1000	500	333	250
H	800	400	267	200
I	700	350	233	175
J	120	60	40	30
K	120	60	40	30
L	538	269	179	135
M	350	175	117	88
Mass.kg	2000	250	74	31



SIDE ELEVATION/PLAN.



SECTIONAL ELEVATION-A A.



ISOMETRIC VIEW.

Fig. 6.2. SHED armour blocks for reservoir embankments and river revetments

Mr C.J. SAMMONS (Consulting Engineer) said that he would like to raise two points about the contact between clay cores and dam foundations.

Dr Penman suggested covering the valley floor with a thin layer of concrete to provide a smooth surface on which to found the clay core of a dam. He certainly agreed that any major hole left after cleaning the foundation should be filled with dental concrete. In many circumstances, however, particularly where weathered rock had been left in situ, might not it be better to place an initial layer of soft clay which would easily conform to any local surface irregularities in the rock rather than provide a general skim of concrete? The concrete might crack, either through drying or differential movement, producing seepage paths which could lead to erosion of the bottom of the core.

It was clearly necessary, as Dr Penman said, to fill in and remove large step shapes on the valley slopes with concrete to avoid cracking of clay cores above sharp corners. However, if the abutment had a relatively steep overall slope, say of about 30 to 40 degrees, there would still be a tendency for low horizontal pressure and transverse cracks to develop because of differential settlements within the core. It was partly to avoid this problem that the face of concrete structures, such as spillway walls, which abutted clay cores were built at angles of about 10:1 so that movements were taken up in shear. Would it be possible to effect a similar result on steep valley sides by placing a zone of softer clay next to the abutment to induce local shearing?

Mr N.J. RUFFLE (Northumbrian Water Authority) referred to Dr Penman's paper (page 63, second paragraph) where it was stated that more modern dams were equipped with filter drains in contact with the core. It so happened that neither Derwent nor Kielder, both of which featured in the paper, had core filters. For indeed, as Dr Penman went on to remark, core filters had to be cleverly designed, so cleverly, in fact, that to be effective in all circumstances they were likely to be multi-stage filters which were troublesome and costly to construct. In these two dams, drainage blankets and the ground blanket offered means of controlling core seepage.

On page 65 of his paper, Dr Penman had quoted his argument in 1979 that end of construction pore pressures should be at least equal to reservoir water pressure to ensure avoidance of hydraulic fractures. The phenomenon of hydraulic fracture only became widely recognised in this country after the trouble at Balderhead in 1968, but some years earlier the clay blanket and core at Derwent were designed to be proof against cracking during foundation consolidation by deliberately ensuring the high pore pressures shown in Fig. 5 of the paper. Would Dr Penman agree that placing the core sufficiently plastic to produce high end of construction pore pressures had merit, not only in avoiding hydraulic fracture due to relief of stress by transfer, but also to avoid the danger of cracking due to deformation?

As storm damage to blockwork on the upstream face at Kielder had been mentioned by another speaker, Mr Ruffle commented that remedial work at Kielder had just finished. The 300 mm blocks and 200 mm of the underlying 10 to 35 mm size broken stone had been removed. These had been replaced, first with concrete slabbing, and then the blocks, relaid on a mortar bed and grouted.

Mr E.T. HAWS (Rendel Palmer & Tritton) said that time had been very limited for discussion of the paper by Dr Penman on misconceptions in design of dams. In particular, he believed a considerable discussion could have developed on the recorded principle of aiming for contruction pore pressures in the central core higher than reservoir water level, to avoid hydraulic fracture. Clearly, the aim of having total stresses higher than reservoir water pressure was entirely necessary. However, the use of construction pore pressures for this purpose implied to him that they were thought to be the easiest means of monitoring to ensure that total stress exceeded reservoir pressure. However, the use of the pore pressure parameter had led, as recorded, to significant hold ups in construction while some dissipation took place to ensure stability against slip. Other actions, such as berms, were also referred to in order to maintain stability.

The progression had thus been from unsatisfactory strong brittle cores to plastic cores with pore pressures high enough to create construction difficulties. Problems did not arise in earlier days when the central plastic puddle core was narrow and construction was in any case very much slower. They now had a wider plastic core proposed with significant stability implications. In these circumstances, other solutions should clearly be seriously considered, including such configurations as a core sloping upstream. The overall stability of this arrangement with lower pore pressures might well exceed that of a central vertical core with high pore pressures. There could be no rule of thumb solution. The optimum would always be site specific. However, the fundamental requirement of total stress had to be emphasized, not that of pore pressure.

Concerning the argument over embankment dams curved in plan, it had never been settled to his satisfaction whether curving upstream or downstream was the safer. For the circumstances of reservoir full and steady seepage, a plan arrangement curved convex downstream did, in fact, produce more fill material at the downstream toe as the adjacent wedges for analysis diverged in that direction. The converse was true for convex upstream. Against this argument was the possible generation of some favourable longitudinal stresses in the convex upstream arrangement. Finally, there was the matter of a drawdown condition in which the reverse of the above arguments might apply.

In the light of the variable factors referred to above, there did seem to be a fairly plausible case for adopting the more economic arrangement of a straight axis. That left the final say with the landscape architect.

He believed the clay core issue was one of great importance and of current technical development. He would be happy, as Chairman of BNCOLD, to get a discussion on this matter included in the future BNCOLD programme.

Mr J.G. ELDRIDGE (Binnie & Partners) said that there had been suggestions that the Government should provide guidance to panel engineers as to safety standards which should be adopted. Surely the Reservoirs Act 1975 and its predecessor, the Reservoirs (Safety Provisions) Act 1930, were remarkably well conceived in that they placed responsibility for saying what was safe squarely on the professional people who specialized in dam design and construction. Government had no greater expertise than that of the people whom it employed. Unless they were more experienced, and had better judgement, than qualified engineers on the panels, seeking guidance from government was tantamount to the profession saying they wished to avoid the responsibilities which, by their own skills and training, they were best qualified to assume. This was an issue on which the profession should be prepared to stand up and be counted.

Regarding the very interesting information about Mullardoch Dam given by Mr Johnson, it was perhaps relevant to remark that Mullardoch was unique in that its centreline consisted of two limbs at an angle with the apex pointing downstream. It had to be more difficult for downstream movements at the apex caused by thermal movements to be fully recovered than if the apex pointed upstream.

In his exceptionally valuable BNCOLD Lecture, Mr Johnson gave a figure of 0.1% per year of current estimated capital cost of the North of Scotland Hydro-Electric Board's dams as representing maintenance costs. It would be very helpful to know what that figure covered. Was it the cost of maintenance and repair contracts only? An estimate of the total average operating and maintenance costs of the projects would be very useful as it would cover a large number of schemes over a long period.

Dr Penman referred to sheared clays caused by old landslips and the like. It should be remembered that sheared clays were also common where alternating clay and (for example) sandstone strata had been folded tectonically. The sheared layer typically occurred just below the base of the stronger sandstone layer. Sheared clay horizons often governed the stability of excavated slopes and dam foundations. For this reason, instrumentation of the foundations of a dam was often as important as instrumentation of the dam itself.

Mr D.M. HAMILTON (Donald Hamilton & Associates) said that Mr Eldridge had referred to Mullardoch Dam, upon which Mr Johnson had commented. Apart from the angle in plan, this was a standard gravity dam. It was his responsibility to produce the whole Affric

Scheme (design, contract documents, tenders received) so that Sir William Halcrow could report to the North of Scotland Hydro-Electric Board by 1st January 1947 from a start on 1st September 1946. It might amuse them to know that the quantities of rock were, of course, carefully worked out from the drawings and contour surveys, but the staff concerned were not pleased with him when he doubled the volumes which would be put into the Bill of Quantities; later, Edgar Wilson said that after the dams, Mullardoch and Benevem, had been completed, the Bill of Quantities proved about right.

Mr Hamilton then commented on the vagaries of the weather, and said that in the course of his acting as Panel I Engineer for dams being constructed at the outlets from Lochs Minnoch and Dungeon in Forrest Estate near Dalry, Kirkcudbrightshire, at the former, rainfall was experienced continuously from 1st to 12th January inclusive, giving a total of 56.3 inches (224.3 mm) i.e., a daily average of 4.7 inches (18.67 mm).

Regarding the construction of gravity dams, he mentioned that at Laggan Dam, Lochaber, plumbs of natural rock broken out at the quarry as big as an office desk or larger, were lowered into the concrete between pours which, projecting above finished lift surface, formed keys unlikely to provide uplift in themselves and reducing markedly the area of lift surface which might, through water ingress, cause uplift.

Mr Hamilton also commented on work at Glashan Dam, and said that as designer and Panel I Engineer for this dam, he had been surprised that new holes were thought to be necessary. The drains, as installed, were, from memory, 6 inches in diameter, and designed with:

(a) vertical holes capable of being reamered out down to the rubble drain or, if thought necessary, below that, with no fear of puncturing the watertightness of the concrete and rock upstream of that line of holes;

(b) inclined holes teeing on to each vertical hole brought to level just above downstream standing water; a special feature was each intersection of vertical and inclined holes was formed in cast steel left in place so that reamering of the inclined holes could never go beyond the line of the vertical holes.

These features he would have been happy to draw to the attention of the North of Scotland Hydro-Electric Board had he been asked; indeed, it suggested that the employer might be obliged to refer such maintenance problems to the original designer with the possible avoidance of unnecessary expenditure.

Mr R.C. BRIDLE (Watson Hawksley) wrote that regarding the brief discussion arising from Dr Penman's remarks about filter design in his paper, he hoped people were not left with a feeling that filters were an 'optional extra' in dams. On the contrary, filters were a major weapon available to dam engineers to safeguard dams against internal erosion, a major cause of dam failure.

If Teton Dam had had filters, it might have survived; the same could be said of Baldwin Hills; Roppe Dam might have survived, and the distress and remedial works, including filters, they saw in the film would have been avoided.

There used to be problems in designing filters to retain clays, because the fineness of the filters resulted in them containing fine particles which led to cohesion and a possibility of failure by cracking of the filter itself. This was largely overcome when it was realised that the particles that filters should be designed to retain were clay flocs, not the dispersed particle sizes recorded in the particle size distribution (PSD) test. If PSD tests were done using water resembling that expected in the proposed reservoir, instead of the usual sodium hexametaphosphate, as the dispersant, very different results were obtained and non-cohesive filters could be designed using conventional rules, which had been improved by the work of Vaughan and Soares*.

The days when filter protection could be omitted from dams were over, and he would urge dam designers to include filters unless it could be convincingly demonstrated that suitable filters could not be designed. There was also scope for examining the degree of protection against internal erosion provided by the zoned fill in old dams and how filters might be incorporated to improve the situation where necessary.

Mr W.S. MACONACHIE (Scottish Development Department) wrote that Mr Morison, in earlier discussion from the floor (see page 55), had mentioned the difficulty if an enforcement authority were not informed of the termination of a supervising engineer's appointment to a particular reservoir. Section 21(3) of the Act placed a duty on undertakers to inform the enforcement authority within 28 days if a person ceased to be a supervising engineer for a reservoir. Failure to provide that information without reasonable excuse was a criminal offence under Section 22(2).

In Mr Johnson's concluding remarks on the last day, he said that the raised volume of a reservoir should not be the only safety criterion, and that he recommended investigation into risks of possible reservoir failure. There were already proposed research projects into the likely flooding and damage that could be caused downstream because of reservoir failure. A number of mathematical models of such flooding existed, including the United States Weather Bureau's Program, DAMBRK. It was hoped that the program could

*Vaughan, P.R. and Soares, H.F., 1982, 'Design of filters for clay cores of dams'. Journal of the Geotechnical Division, American Society of Civil Engineers, Vol. 108, No. GT1, January, pp. 17-31.

be made more user friendly for use on a personal computer and adjusted to UK topography. It was difficult to quarrel with Mr Johnson's view that raised volume was not the only valid parameter, but if the suggestion was that reservoirs legislation should take account of the varying parameters including the downstream conditions then the following difficulties would have to be overcome:

(a) All or most reservoirs would have to be subject to some form of legislation to ensure that there was an initial assessment of risk.

(b) This would either increase the number of reservoirs coming under the legislation or again some criterion was needed to identify which might be initially excluded.

(c) An inspection by a panel engineer would presumably be needed to recommend whether ongoing reports, supervision, etc., were necessary.

(d) Although a remote reservoir might present little risk, there was a possibility that downstream development (e.g., tourism, timeshare holiday homes, etc.) would change the risk significantly. To ensure continuing safety, the downstream situation and reservoir category would need checking at intervals. Whether or not, during such checking, inspecting engineers would recommend work to the dam was a matter for their professional judgement. This was already the situation under the 1975 Act.

(e) The inclusion of additional criteria in legislation would increase the scope for disagreement about interpretation, and whether or not particular reservoirs would be subject to requirements for formal inspections, record keeping, supervision, etc.

Authors' Replies

Mr JOHNSON, in reply to discussion, said that Mr Eldridge had questioned what was covered by the figure of 0.1%/year of current capital value for maintenance of the North of Scotland Hydro-Electric Board's dams. The figure included the cost of all maintenance and repairs, whether carried out by contract or the Board's civil squads. The cost of operation and maintenance of all civil works, buildings and plant on all the Board's conventional hydro schemes amounted to just less than 0.5%/year of their current capital value or, expressed another way, just under 0.25p/kWh. It should be recognised that the Board's schemes were small/medium installations with extensive catchment diversions to make them viable and with a multiplicity of 84 dams and 57 power stations.

He referred to Mr Hamilton's contributions, and said it would be interesting to know in which year and exactly where in Kirkcudbrightshire the rainfall of 56.3 inches between 1st to 12th January was experienced.

He had been surprised that plums had not been more extensively used in heavy construction in such structures as dams in recent decades, since there were technical advantages to be gained. His understanding was that the use of plums had been abandoned due to economic reasons.

With regard to the pressure relief holes at Glashan Dam, the existence and purpose of the vertical and raking pipes were fully known to their engineers. However, access to the top of the vertical pipes was by walkway except on the spillway section (B17 to B23) where there was no access. The blockage was in B16 or B17. It was much easier to move drilling rigs to the toe of the dam, and with modern control it was possible to target on, and strike, the rubble drain with the new raking holes.

Finally, in his concluding remarks on the last day, Mr Johnson said that after 40 years of experience with the old Reservoirs (Safety Provisions) Act 1930, he was sad and disappointed that the new Reservoirs Act 1975 was still using reservoir capacity as the only parameter for determining whether a reservoir presented a risk to a community. He suggested that a major objective over coming years should be to devise a Reservoir Risk Index which would accurately reflect the key parameters which contributed to the risk presented by a reservoir.

Mr AGNEW said that he welcomed and agreed with the view expressed by Mr Eldridge that the 1975 Act was a well conceived framework for ensuring protection from an escape of water from a large raised reservoir. The discussions had shown that it was not difficult to raise a number of hypothetical problems. It was important, however, not to lose sight of the Act's basic principles. In dealing with real situations, the Act provided ample opportunities for the use of discretion by inspecting engineers and enforcement authorities in operating the Act and thus ensured the achievement of its primary objective of protecting persons and property from an escape of water from a large raised reservoir.

Dr PENMAN, in reply to discussion, said that he agreed with Mr Eldridge that instrumentation of the foundations of a dam was often as important as instrumentation of the dam itself, and he hoped that nothing he had said in his paper implied otherwise. He also agreed with Mr Eldridge's major point on responsibility. The fact that Government had no greater expertise than that of the people it employed did require emphasis. Fortunately, those government employees responsible for the administration of the Reservoirs Act were well aware of their need for advice on appointments to panels and requested it from the Institution of Civil Engineers.

Mr Haws had made a valuable observation when he said that they had progressed from unsatisfactory strong brittle cores to plastic cores with high construction pore pressures. His progression should have started before the advent of strong brittle cores, and gone back to the time, mentioned in his second observation, when the central plastic puddled clay core was narrow and construction was in any case very much slower so that problems did not arise.

Had he evidence that high construction pore pressures did not arise in these earlier puddled clay core dams? Unfortunately, Dr Penman had not realised that the use of puddled clay cores was in decline when he measured pore pressures at Usk Dam, so he took no interest in measurements in the core itself. At Selset, there was tremendous concentration on measurement of pore pressures in segments of the shoulder fill separated by the vertical and horizontal drainage layers, but the core was ignored until the dam was at about three quarters full height, when three piezometers were installed. These showed clearly that the developed pore pressures corresponded closely with the total vertical stress that could be expected, in accordance with the undrained shear strength of the clay and a silo action mechanism given by Bishop and Vaughan*. Beavan et al.** had since given several examples that showed that the increase of pore pressure, δu, gave a measure of the increase that had occurred in the total vertical stress, $\delta\sigma_v$, in a clay core.

Since Selset, he had not had an opportunity to measure construction pore pressures in a narrow, vertical puddled clay core, but the indications were that they were probably close to the vertical total stress in the core and even with relatively slow construction might well have equated with top water level at the end of construction.

The fact of the matter was that the core material had to be weak enough and of such a bulk density that adequate total stresses were developed to ensure that hydraulic fracture could not be caused by the pressure of the reservoir water. The shoulders had to withstand the horizontal thrust applied to them from such a core. If stability was endangered by this required thrust, the fault laid with the overall design of the dam, not with the core.

He was delighted to see that Mr Haws believed that the clay core issue was one of great importance and that he proposed that a discussion on this subject would be included in the future BNCOLD programme. As could be seen from Dr Penman's Rankine Lecture*** and his paper on waterproof elements for dams#, he was particularly interested in this aspect of dam design and looked forward to an opportunity to contribute to the BNCOLD discussion.

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*Bishop, A.W. and Vaughan, P.R., 1962, 'Selset Reservoir: design and performance of the embankment'. Proc. Instn Civil Engrs, Vol. 21, 305-346.

**Beavan, G.C.G., Colback, P.S.B. and Hodgson, R.L.P., 1977, 'Construction pore pressures in clay cores of dams'. Proc. 9th Int. Conf. Soil Mech. Fdn. Engng, Tokyo, Vol. 1, 391-394.

***Penman, A.D.M., 1986, '26th Rankine Lecture: On the embankment dam'. Geotechnique, Vol. 36, No. 3, 303-348.

#Penman, A.D.M., 1985, 'The waterproof elements for embankment dams'. Int. Water Power and Dam Construction, Vol. 37, No. 7, pp. 33-48 and 112.

Mr Rocke had made a succinct plea for 'design as you build', a concept ideally suited to dam construction where, as Mr Haws had pointed out, each optimum design would always be site specific.

In order to resist the lateral thrust imposed on the shoulders by a clay core, it was usually considered desirable to develop a good key between shoulder and foundation to help resist this thrust. The excessive lateral movements at Chingford Dam* were due to a soft clay layer in the foundation which was not removed under the shoulders. Excessive lateral movements at Draycote Minor were also attributed to a soft clay layer in the foundation.

Mr Sheppard, Senior Partner of Binnie & Partners, responsible for Usk Dam, was very concerned about a layer of silt discovered in the dam foundation during pre-construction excavation for the stilling basin. It was his approach to the Building Research Station which gave Dr Penman the opportunity of carrying out stability analyses and showing that if large pore pressures were prevented from developing in the silt layer, stability could be assured. Dr Penman designed a simple drainage system for the silt, later described by Sheppard and Little**, and agreed to install piezometers in the silt layer to monitor the effectiveness of his drains. This gave him the opportunity, a little later, of measuring pore pressures in the fill of the dam, an exercise which proved most worthwhile***.

In view of this, it seemed slightly perverse to purposefully include a clay layer under the upstream shoulder, as was done at Açu in Brazil and at numerous sites in this country. Undoubtedly, the designers had sound reasons for the use of the upstream clay blanket in each case, but clearly they had to be mindful of the effect it had on dam stability and make provision, as Mr Rocke had indicated, for temporary cessation of construction to give time for additional pore pressure dissipation from the blanket, or construction of stabilizing berms if found to be necessary to prevent excessive movements.

It would be wrong for anyone to suggest that there was only one solution for the design of a dam for a particular site. There were usually so many factors present at a site that there could be a wide range of designs which would provide perfectly suitable dams. The final choice was often made on economic grounds. His criticism of upstream clay blankets under upstream shoulders was based only on consideration of the lateral thrust imposed by a soft clay core. Clearly, if the blanket was strong enough it would transmit thrust to the foundation and in cases of large depths of soft soil, as under Alibey Dam in Turkey, then shoulder width had to be increased to provide the necessary resistance and discussion of an upstream blanket became irrelevant.

*Cooling, L.F. and Golder, H.Q., 1942, 'The analysis of the failure of an earth dam during construction'. J. Instn Civil Engrs, Vol. 19, No. 1, pp. 38-55.

**Sheppard, G.A.R. and Little, A.L., 1955, 'Stabilising an earth dam foundation by means of sand drains'. Trans. 5th Int. Congress on Large Dams, Paris, Vol. 1, pp. 639-646.

***Penman, A.D.M., 1978, 'Construction pore pressures in two earth dams'. Proc. Conf. on Clay Fills, Instn Civ. Engrs, 177-187.

Dr Pugh had drawn attention to the fact that skilful design had enabled many successful dams to be built from what might at one time have been regarded as unsuitable materials on unsuitable sites. A theme of Dr Penman's Rankine Lecture* was that developments in geotechnical science had empowered them to be able to build dams of almost any natural material on almost any site. An interesting example was Empingham, with its extremely flat slopes and more than ten thousand sand drains.

He joined Mr Bridle in hoping that his audience was not left with a feeling that filters were an 'optional extra'. His views on filters were perhaps best expressed in his General Report to the 14th Congress on Large Dams**, and he referred the interested reader to that report.

Mr Ruffle had pointed out that neither Derwent nor Kielder Dams had filters to protect and support their cores. Mr Bridle might have felt that the designers of these dams did regard filters as optional extras, but Mr Ruffle assured them that in these two dams, drainage blankets and the ground blanket offered means of controlling core seepage.

He agreed with Mr Ruffle that placing a clay core sufficiently plastic to produce high end of construction pore pressures had merit in avoiding hydraulic fracture. Hydraulic fracture might be encouraged by a reduction of total stress through several causes, including what was commonly referred to as arching or silo action and tensile strains produced by differential settlements. Differential settlements due to foundation consolidation could cause visible cracks on the surface of a stiff, brittle core, but to keep a crack open at depth, where erosion damage might occur, usually required reservoir water pressure in the crack. The designers of Derwent were to be congratulated for their advanced approach, as described by Rowe***, of specifying a wet and flexible core with the intention of avoiding cracking.

Mr Sammons had raised an interesting point in suggesting that a thin layer of concrete might crack and leave fissures in contact with the clay which could have an even greater detrimental effect than fissures in the bedrock. Dr Penman's concept of a concrete covering was one of sufficient thickness, plus light reinforcement, to ensure a smooth, crack-free contact for the clay. Where there were high sun temperatures and particularly in cases where the bedrock rapidly weathered after exposure, it was usual to place the concrete not too far in advance of the clay, so that it might be covered to reduce temperature rise and avoid drying. As he saw it, the purpose of the concrete was to seal over any natural fea-

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*Penman, A.D.M., 1986, '26th Rankine Lecture: On the embankment dam'. Geotechnique, Vol. 36, No. 3, pp. 303-348.

**Penman, A.D.M., 1982, 'Materials and construction methods for embankment dams and cofferdams'. General Report to Question 55. Trans. 14th International Congress on Large Dams, Rio de Janeiro, Vol. 4, pp. 1105-1228.

***Rowe, P.W., 1970, 'Derwent Dam - embankment stability and displacements'. Proceedings of the Institution of Civil Engrs, Vol. 45, pp. 423-452.

tures that might be present in the rock, whatever its state of weathering, and overcome any local irregularities over which the clay might try to span.

He thought it was usual practice to make the contact zone of the clay somewhat wetter than the rest of the core to ensure compliance with the shape of the contact surface and improve the opportunity of a more uniform contact pressure. As an example, at Chicoasen Dam (250 m high), the main body of the core was placed at about 0.8% below optimum water content, but to allow easy shear deformations at the core/abutment interfaces, a 4 m wide strip was placed 2 to 3% above optimum. Thus, he agreed entirely with Mr Sammons that a zone of softer clay next to the abutments was good practice. It might, however, be better to specify a limiting maximum undrained strength (obtained from three dimensional analyses) rather than use a water content specification.

Dr Penman concluded by saying that he would like to express his thanks to those who had contributed to the discussion and hoped that they might continue it, with the desirable aim of arriving at agreed conclusions, during the BNCOLD meeting proposed by their Chairman, Mr Haws.