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EXPERIENCE WITH THE CONCRETE DAMS OF THE NORTH OF SCOTLAND HYDRO-ELECTRIC BOARD

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Development of large scale commercial hydro generation commenced in Scotland about 60 years ago inaugurating an era of intensive dam construction extending up to the late sixties primarily based on concrete designs. The paper appraises the experience of these dams by reference to the surveillance policies developed, the operation and performance of both dams and reservoirs, the extent and nature of the maintenance undertaken and concludes by reviewing the lessons learned and how they might influence the future specification and design of dams.

INTRODUCTION

The North of Scotland Hydro-Electric Board owns 84 dams on 76 reservoirs which are registered under the Reservoirs Act. Fifty-six of these dams are listed in the World Register of Dams. There is a wide variety of types made up of 53 gravity, 9 buttress, 3 arch, 1 pre-stressed concrete, 6 earthfill, 6 rockfill and 6 combined fill and gravity dams. The highest is Sloy buttress at 56 m and the longest Mullardoch Gravity at 727 m. Sixteen dams have gates for the release of floodwater and four have syphons. The largest reservoir, Quoich has a capacity of 374 x 10^6 m^3. The wide glacial valleys of the North of Scotland with their good rock conditions and moderate overburden, coupled with the economics at the time of construction and lack of suitable clays, led to the predominance of concrete gravity and buttress designs.

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

Whilst Scotland may be said generally to have a temperate climate, conditions in the Highlands can often be severe and subject to rapid change. The winters are generally wet with snow lying at higher levels and January and February can provide a succession of severe night frosts accompanied by bright clear days bringing a daily freeze/thaw cycle which seeks out and attacks any weaknesses of design and construction. The acid waters in the reservoirs derived from the large tracts of peat on the catchment areas is another source of attack on the concrete and the high humidity provides conditions for extensive growth of moss on concrete surfaces.

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The reservoirs are generally operated on an annual cycle to meet the requirements of electrical demand. The Board have two pumped storage schemes where the reservoirs are operated on a weekly cycle - so far this more onerous duty has caused no adverse effects.

SURVEILLANCE

Development of Policies

After the Vaiont and Malpasset Dam disasters, the Board commissioned, in the early 1960s, leading consulting engineers and geologists to undertake comprehensive inspections of all their reservoirs. As a result, settlement and alignment readings were taken for most dams with heights above 23 metres. For concrete dams under construction, temperature and strain measurements were also specified along with pendulums for measuring deflections. By 1971, up to 10 years of readings had been obtained making it possible to assess the value of inspections and the results of the instrumentation. This review (1) led to the formulation of a dam surveillance policy made up of three components: field inspections, leakage measurements and monitoring by instruments. Recently, as a result of the issue of the Reservoir Guide (2) and the implementation of the Reservoirs Act, 1975, (3) the policy has been further refined and developed.

Field Inspections

Five types of field inspections are carried out:

(1) Statutory Inspections, made by Panel AR Engineers appointed under the Reservoirs Act. It has been the Board's policy for many years to appoint panel engineers who have not been responsible for either the design or construction of the dams or for previous inspections; one engineer is normally appointed for all the reservoirs in a valley and the engineer has been changed every 10 years.

(2) Up to this year, Safety & Maintenance Inspections were made by engineers of the Civil Division, based at Head Office in Edinburgh, at intervals of 2 to 5 years, the period varying according to the conditions and behaviour of the dams and their importance with respect to public, third party and Board interests. For dams with abnormal behaviour or defects more frequent inspections were instituted (eg annual or even more frequent). The civil engineers employed on these inspections were all chartered with not less than 10 years' aggregated dam experience. They were regularly changed so that fresh eyes and minds were applied to the situation to detect and evaluate critical aspects. At the same time as undertaking an Inspection for safety purposes, the engineer assessed the condition of maintenance of the reservoir works, classifying work into priorities and defining the department responsible for undertaking the work. Standard report forms and checklists were developed and are still used to ensure thorough inspection and progressing of subsequent actions.

The Reservoirs Act 1975 came fully into force on 1 April. This requires a Supervising Engineer to be appointed for each reservoir and for him to carry out at least one inspection of the reservoir each year as well as to scrutinise documentation, check on actions, recommendations, work done and to report. Detailed maintenance inspections are being integrated with these supervising inspections in exactly the same way as previously in order to define in detail dam maintenance requirements.

The large amount of paperwork now required to meet the demands of the new Act will be serviced by computer with a program which has been specially devised to minimise manual work. In Northern Hydro Group, containing some 30 dams, all reservoir water levels are about to be remotely recorded at 30 minute intervals in the Control Centre and stored with other performance date on a computer database.

Thereafter selected data will be transmitted to Head Office for feeding into the Reservoir Record Book with minimal manual intervention. It is hoped to eventually automatically record leakage and other instrument readings for computer entry into the Reservoir Record Book including possible remote indication and recording of key parameters.

(3) Inspections at 3 to 12 month intervals are made by the operating engineers who are normally mechanical or electrical engineers resident on the Schemes. These inspections are mainly connected with maintenance aspects but any important safety issues which arise are reported to the Civil Division.

(4) Inspections, testing and maintenance of all reservoir gates, valves, rakes and syphons by the operating staff. These are required to be exercised at least annually and reported in writing to the Civil Division annually.

(5) Weekly visual inspections are made by watermen as far as is practicable at the same time as recording water levels, leakage, reading pendulums, etc.

Monitoring by Instruments

The desirable objective is to install instrument stations on all important dams and to maintain them in first class order so that they can be brought into use immediately to provide an up-to-date comparison with the initial datum base. The reading of instruments, particularly where the dams are remote and situated in areas of inclement climate, can be heavy on surveyors' time, and a very strict scrutiny is carried out to select only those dams where instrumentation will be of real value. Full monitoring comprises the measurement of vertical and horizontal movement by EDM equipment, level, pendulum, rotating laser, and joint gauge as appropriate, in parallel with the reading of related parameters (eg temperature and strain).

The Board has 36 dams with one or more systems of instrumentation. For recent dams which are fully instrumented during construction, regular readings are taken over a sufficiently long period of time (eg 5 years) to gain confidence in the dam's behaviour. Thereafter, if the instrument readings are consistent and the behaviour predictable, the number of instrument readings is reduced or the period between readings increased. Typically, readings would be reduced from an annual cycle of reading instruments twice or four times a year to one complete seasonal cycle every five years or from reading all instruments to reading selected instruments. As a dam becomes older or deteriorates, the frequency of the cycle may require to be increased. For a dam in doubtful condition or with abnormal behaviour, more extensive instrumentation is installed and readings are taken more often (4). For arch dams and dams on pumped storage schemes subject to rapid variations in head more frequent cycles are carried out. Supervisory Inspections and monitoring are, in addition, undertaken after major floods, seismic activity or unusual events.

From experience of 25 years of surveillance of the Board's dams by instrumentation, certain conclusions and recommendations for future instrumentation arise. It is considered important to differentiate between two types of instrumentation viz firstly, that required to determine the condition of the dam during construction and its performance in relation to its design. This instrumentation has a limited life of say a few years, extending up to possibly a maximum of 5 years after filling. Secondly, permanent instrumentation which is required for the whole life of the dam. Only that instrumentation which is really necessary should be installed, for it has been estimated that the capitalised cost of monitoring amounts to five to ten times the initial cost of the instruments (5). Permanent instrumentation should be durable, reliable, robust and replaceable.

Resources

Until two years ago, the Reservoir Safety Section comprised a Reservoir Safety Engineer and a Surveyor together with Vacation Staff and Clerical Staff supplemented by inspections by 6 Supervising Engineers on a part-time basis, drawn from all sections of the Division, who typically carried out 4 or 5 dam inspections per year. As a result of the new Reservoirs Act, this Section has had to be increased to a Senior Engineer, 2 Chartered Engineers, a Surveyor and Vacation and Clerical Staff. It is also supplemented by 7 Supervising Engineers who look after typically 4-6 reservoirs each and are drawn from other Sections of the Division working on a part-time basis.

Results of Surveillance Policy

From the Board's experience, extending over 15 years, a well balanced combination of detailed visual inspections by well experienced engineers, leakage measurements and monitoring by instruments, coupled with flood studies and stability analyses, are a most effective means of detecting deterioration and ensuring dam safety.

The employment of panel engineers for Statutory Inspections who were not connected with the design or construction of the dam has led to fairly probing appraisals and the exposure of aspects not suspected as being unsatisfactory. In the light of the requirements of the new Act, reconsideration is now being given to the desirability of changing these engineers every 10 years. The Supervisory Inspections have broadened and tightened up the surveillance of reservoirs for safety, as well as leading to a much improved procedure for reporting and defining maintenance requirements. Contrary to past practice, these engineers will not be changed regularly as this would lose the advantage of continuity of surveillance which is a new and important principle enshrined in the 1975 Act and would also lead to considerable additional costs. Overall, the Board's experience has led to greater emphasis and resources being placed on inspections by qualified engineers with a somewhat reduced effort on monitoring by instruments. This policy has led to a more effective and thorough safety inspection of reservoirs and to more efficient use of the experienced staff available for this essential work as well as ensuring a good standard of dam and reservoir maintenance.

The situation with respect to surveillance of the Board's reservoirs is abnormal, in that the Headquarters in Edinburgh, where the Reservoir Safety Section is stationed, are some 50 miles from the nearest reservoir and some 220 miles from the furthermost reservoir in Lewis, in the Western Isles. Travelling time and expenses are therefore unusually high. To meet the requirements of the new Act, it is estimated that the surveillance, processing of data and paperwork associated with each reservoir will require, on average, one man week's work including travelling time amounting to the order of £400. This will increase the cost of reservoir surveillance by a factor of about 3. It remains open to question whether this much greater effort will result in a corresponding improvement in safety of the Board's dams, although increased surveillance of dams of uncertain design, doubtful condition or behaviour is essential and undoubtedly long overdue.

OPERATION AND PERFORMANCE OF RESERVOIRS AND DAMS

Operational Aspects

As a result of the experience gained over the past 40 years, it has been found advantageous to operate the reservoirs some 1-2 metres below spill level. Although this results in slightly less head at the turbines, there is a much reduced chance of spill and overall a much greater output from the scheme. The objective (6) is to operate them within a band between minimum duties level and maximum output level.

Over the last 20 years, it has been possible to reduce manpower needed to operate the Board's reservoirs by about 60% and the majority now have no keeper living at the reservoir. This trend is continuing and is now leading to improved auto control schemes for floodgates although these are usually backed up by stationing men on the dams during serious floods.

Performance

It is to the credit of the designers of the schemes that the run-off, as defined by generation, has largely been as estimated as will be seen from table 1. A factor which is now beginning to have a significant effect on yield is afforestation of catchments. When a catchment is ploughed in readiness for planting, run-off becomes much more flashy and droughts more severe, with attendant problems due to gravel accumulation in intakes and aqueducts. As the trees develop the yield from the catchment gradually falls and at maturity can be 15-20% down due to increased evaporation and transpiration by the trees. An example of the fall in run-off and hence scheme output due to afforestation is given in fig 1. Overall it is considered that the effect of afforestation is totally disadvantageous to hydro generation. Siltation of reservoirs does not appear to be significant.

TABLE 1 - Comparison of Estimated and Actual Output of Schemes.

Scheme	Reservoirs in Scheme	Original Estimate of Output (GWh/annum)	Actual Average Output (GWh/annum)
Shin	2	137	158
Conon	9	438	438
Affric/Beauly	6	492	487
Garry/Moriston	12	379	389
Tummel	11	635	663
Breadalbane	7	377	340
Sloy/Shira	9	220	215
Awe	6	183	187
Others	14	56	63
TOTAL	76	2917	2940

Flood Studies

The publishing of the Flood Studies Report in 1975 (7), the Engineering Guide (2) in 1978 and the new Act (3) in 1975 has had a significant impact on the standards applied in relation to the safety of dams and reservoirs. By the early 1970s some 20 years of operational experience was available for many of the Board's schemes and it was recognised that there were possibly operational deficiencies in a number of them. The opportunity was therefore taken to make an appraisal (8) of the design of the schemes and to assess flood management procedures in force. A small flood study group was set up in 1972 with the objective of developing a general method of analysing floods taking into full account the FSR. Its aims were to re-assess the magnitude of floods in the catchment areas and river basins developed by the Board; to establish safety criteria for each reservoir and river basin and to review and revise as necessary normal operating and flood management procedures. A general flood routing computer programme was developed incorporating the techniques of the FSR but specially modified to take account of conditions in the Highlands derived from actual operating experience. The results of these studies have been interesting and helpful. They have been recorded in a way which will allow an experienced hydro engineer in the future with no knowledge of the scheme to fully comprehend the design and operating philosophies adopted. The flood management reports include the original design parameters, the new design criteria, the reasoning behind the new flood management procedures, the predicted results from their adoption, the consequences of the plant and equipment failures, flood control and warning procedures, and a full bibliography of related background information and data. To date the operational impact of the flood studies has confirmed that the existing flood control procedures frequently developed from operating experience are generally satisfactory. In a good number of cases, particularly those in which long duration storms are important, the revised maximum reservoir levels were found to exceed design levels but generally the dams have been shown to be capable of accepting the higher levels. The effect of applying the Reservoir Guide and the new Reservoir Act has led to significantly more onerous criteria being applied to the reservoirs and as a result it has been found necessary to improve some of the dams and to recertify the majority of them; this work is currently in hand and will continue over the next 10 years. This is despite the fact that most of the Board's dams are modern dams designed and constructed post war to the highest standards.

The spill capacity of the large majority of Board dams has been adequate to meet the requirements of the Reservoir Guide and the new Act. Experience of operation and performance of flood gates and spillways has been generally good. Odd problems have arisen as a result of operation of syphons particularly the very large ones due to the step change in flow. On one occasion a road bridge downstream of the dam was swept away when the flood wave hit it. Serious problems can also be created where very long spillways are provided and give rise to increased discharges from a valley in which the outlet was formerly restricted. Overall, the application of the FSR and the new procedures has given increased confidence in the operating and flood management procedures and has led to improved operational efficiency and flood control.

MAINTENANCE AND FUTURE SPECIFICATION

The majority of the Board's dams are now 20-40 years old and generally they are in good condition for their age. To illustrate deterioration, it is necessary to concentrate on a very small minority of the Board's dams which are not typical. The defects which have arisen will be reviewed under a number of headings, viz concretes, joints, dam crests, pressure relief systems, wave protection, gates and valves, and steelwork. The lessons learned and how these might influence the future specification and design of dams are given at the end of each section as this gives a logical progression of thought.

Concretes

None of the defects observed to date have had any significance in relation to the structural stability of the dams. Deterioration, once started in concrete surfaces, is usually progressive in extent and in depth, is frequently difficult and expensive to repair and is often unsightly. Five main types of deterioration are observed:

I Frost action on downstream faces, horizontal surfaces and particularly thin sections.

2 Cracking of relatively thin sections in the uppermost parts of dams for a variety of reasons followed by subsequent frost action.

3 Erosion of spillways and other waterways.

4 Action of acid waters and ice on upstream faces.

5 Deterioration of downstream faces and other surfaces including the growth of moss and lichens.

Many types of concrete were used in the dams (9). Ordinary Portland cement was used in most cases but Blast Furnace Portland cement, Low Heat cement and Trieff cement were also employed along with the incorporation of fly ash. Aggregates varied with the localities of the dams and ranged from crushed granite and greywacke, to washed pit and marine beach gravels, while sands were derived from crushed rocks, pit or marine sands or mixes of these. In two dams, colloidal concrete (Colcrete) was used and in one case pre-stressed concrete.

Overall the Portland cement concretes have performed well except for one or two isolated dams where the specification for the concrete was based purely on strength with inadequate cement content. In the very hostile climate of the Highlands, it is necessary to design for durability as well as strength and the minimum cement content must be specified to accommodate this requirement. Those dams which have included fly ash have been fully satisfactory and there would appear to be advantage to specifying its incorporation in future dams provided its content does not exceed 15-20%.

The Blast Furnace Slag and Trieff cement concretes have also been satisfactory. With these concretes, no free lime is released as water flows over and through concrete and therefore any cracks or leakages do not heal as is frequently the case with an ordinary Portland cement concrete. Two dams, Loyne and Cluanie were faced with precast concrete panels and were constructed with Trieff concrete. In general, these dams have been satisfactory except where porous pockets have been left behind the panels which have filled with water and due to freeze-thaw action have cracked and spalled. It is interesting to note that Loyne dam has the highest leakage of any of the Board's dams - about half a cusec. There would appear to be no reason why these types of cement should not be used for future dams.

The impounded waters in the Highlands have pH values ranging from about 5 to 8. Over the last 30 years or so, typically 3-6 mm of the concrete of the upstream faces of the dams has been lost, particularly over the normal range of water levels due to the action of chemical solution and ice attack. These acid waters also seek out and attack any honeycombed concrete on lime rich areas (eg at the top of lifts) which can lead to deep penetration. In the past, it was customary to apply a bituminous coating to the whole of the upstream face to mitigate the attack but this is an expensive procedure and over the last 10 years it has generally been abandoned apart from those dams which have thin water retaining membranes or are very much in the public view. A good example of this is Quoich Rockfill dam which has a concrete upstream face which acts as the watertight membrane of the dam. This concrete, which is 300-400 mm in thickness, was vacuum cured and is, after 30 years, in excellent condition.

The Allt-na-Lairige pre-stressed concrete dam has performed well although the edges of the main blocks of the dam have tended to curl and there has been a certain amount of spalling at the corners of the blocks.

Loch Dubh dam constructed in 1955, is a small concrete gravity dam with a maximum height of 20 m formed with bays of both "Colcrete", using large aggregate and ordinary Portland cement concrete; this was the first time the Board had used

Colcrete for dam construction. By the late sixties, there had been a considerable loss of concrete to a depth of 50 mm in places on the downstream face especially at lift joints - see fig 2. In the light of this unsatisfactory situation pore pressures were measured and the drainage system of the dam tested with low pressure water and shown to be largely blocked. A series of 60 mm diameter holes was therefore drilled from the downstream face of the dam to intercept the rubble drain and to form new outlets. As a result of this work, the high pore pressures were reduced along with a visible reduction in the rate of deterioration of the surface of the colloidal concrete. The Board have a second Colcrete structure, viz, the spillway adjacent to Dunalastair Dam. This is also exhibiting the same spalling of the downstream face which in all probability is due to the porosity of the concrete and the freeze-thaw actions which take place in both these locations which are subject to very low temperatures. It is considered that colloidal concrete is unsuitable for exposed faces in the onerous climate of the Highlands.

On the whole, downstream faces have suffered little damage apart from the growth of lichen and moss. This is encouraged where there are air holes on the surface of the concrete. It is considered that bagging of the concrete is well worthwhile to discourage this growth. There has been a certain amount of erosion, probably 3-6 mm in depth, on a number of the spillway sections but this is to be expected.

With the large number of dams and geographical spread of their locations it would not have been surprising to find instances of alkali-silicate reaction. One location was suspected but the testing of the concrete proved negative. However, there has been the odd instance in South-west Scotland on the Galloway Scheme, in particular, the Muck Burn dam.

The main maintenance problems with concrete have arisen with thin sections such as bridges, parapets, kerbs where, due to wet mixes, low cement contents and freeze-thaw cycles, disintegration occurs. More massive sections and/or the specification of concrete with say 3% air entrainment as in Scandinavia would seem to be the best answers to this problem. Good detailing with first class facilities for shedding water and free drainage is most important.

The repair of damaged concrete is expensive and, unless very carefully carried out, can be unsightly. For surface damage up to 50 mm in thickness, a rich mix of Portland cement mortar with a styrene butadine additive is used. Above this thickness areas are mesh reinforced and pinned or steel fibre reinforced concrete is used. Fibre reinforced concrete has been used in the Board since 1968 most successfully, particularly in situations where very onerous conditions arise or there is very high erosion, eg spillways. It has also been found possible to pump the fibre reinforced concrete using a Putzmeister pump for tunnel repairs. This type of concrete is also being used for aqueduct repairs, particularly at high altitude where freeze-thaw cycles can be very severe.

Joints

Many of the Board's dams were built on the alternate block principle incorporating single contraction joints between blocks which usually had keys formed in their end faces. Water stops were installed in these joints and took the form of bitumen plugs, copper strip, rubber and PVC water stops or a combination of them. Joints in arch dams were grouted after initial contraction had taken place. Vertical water stops have not in all cases been fully effective because they were too small relative to the massive structure of the dam. Honeycombing of concrete adjacent to the stops is probably a greater source of leakage than the water stops themselves. Bitumen plugs on their own have been successfully employed on some dams but bitumen is more commonly used as a sealant between blocks in conjunction with copper strips. In some instances bitumen has extruded at the joints and clearly the specification of the material or workmanship was not wholly satisfactory. For other types of joints, there can be up to 3 water stops in the joints of some dams and the joints can still leak compared with other dams where there is just a single stop and they are tight.

Overall, it is recommended that two stops of different types (eg PVC plus bitumen plug) should be provided if bay joints are to be satisfactorily and permanently sealed. The watertightness of joints is critically dependent on first class materials and a very high standard of design/detailing and workmanship on site. It is essential to be most careful in the exact definition of construction and contraction joint planes so that joints in minor structures, such as parapet walls and bridges, line up and move with the main bay joints of the dam.

There has been a certain amount of disintegration on horizontal lift lines, particularly on the upstream face of dams due to the surface layer of concrete being lime rich. With the very acid waters, dissolution occurs and is aggravated by air wave compression and ice attack. Cold joints during construction can also give rise to this type of defect.

Dam Crests

A fair amount of concrete spalling due to frost action has occurred on roadways and parapets, in most instances due to designers overlooking basic details, particularly in respect of the shedding of water and drainage. Roadways and footways were finished in concrete, frequently without sufficient drainage falls and channels and adequate drains, resulting in the surfaces remaining moist and open to frost attack. Often the surfaces were finished smooth with much trowel work put into the finish. It has been found that these roadways have suffered extensively whereas on those roadways finished in bituminous macadam there have been no signs of deterioration. In repairing these crests, bituminious macadam and/or tar spray and chip have been used and found to be fully satisfactory. The cross fall and drainage has been corrected at the same time. Drainage from these roads should be kept well clear of the upstream face and away from piers and abutments.

It is especially important that bridge abutments and bearings must be free draining and again this can give rise to very difficult problems when the bearing disintegrates. In some bridges, flat mild steel plates were used for bearing surfaces and these have rusted due to water attack giving rise to friction on the plates. This has led to crushing and other stresses arising in the adjacent concrete and subsequent spalling. It is important to provide frictionless and non-corrodable bearings and advisable to increase the width of contraction/shrinkage joints in road and bridge slabs in order to ensure that the full temperature range can be accommodated without the adjacent concrete faces being overstressed beyond that which can be satisfactorily resisted by the concrete.

Another feature which has caused trouble is wavewalls. It is important that these wavewalls deflect water so that it cannot cause damage. A typical example of this is at Glascarnoch dam which is in a very exposed location subject to the full force of the westerlies from the Minch. It is a composite embankment and gravity dam. In two instances, when there have been very high gale force winds, waves have been generated which have run up the vertical face of the gravity section of the dam and dislodged the copings on the parapet wall up to 12 m above the water surface - see fig 3. In such instances, it is recommended that the upper section of the wall has a return curve to discharge the waves back into the reservoir.

It has been found that considerable maintenance is required to parapets and particularly handrailings and standards. Many handrailings were simply painted, and mounted on light kerb walls and this has given rise to corrosion and expansion with break up of the concrete into which the standards were fixed. If steel handrailing is used, it must be heavily galvanised and mounted on a substantial kerb. Overall, it is considered that concrete parapets are superior aesthetically, safer operationally and require much less maintenance but naturally are more expensive. An alternative would be to specify the attractive, modern aluminium handrailings and standards used for motorways where maintenance should be minimal. Again good detailing in design and close supervision in construction is all important in mounting the standards in order to avoid water and ice traps.

Pressure Relief Systems

Since the 1930s most designs of gravity dams have incorporated some form of internal pressure relief system with the objective of reducing uplift pressure on the foundations and so obtaining a reduction in the dam section. It is vital that a relief system is kept in first class condition since structural stability depends on it being fully effective otherwise tension can develop at the upstream face particularly in gravity dams at high water levels. Of the Board's 49 concrete dams, 33 have provision for the relief of uplift but only 10 have been designed and constructed so that these systems are readily maintainable. In some of the Board's larger dams, inspection galleries have been provided. Evidence of slow deterioration has come to light during regular inspections particularly over the last 10-15 years. Where it is suspected that the relief system is not satisfactory, testing by low pressure water is undertaken to identify blockages and to determine how the system might be restored to a satisfactory performance. This testing has revealed that the drains, rather than being designed and constructed as well graded runs of straight pipes, are often curved, have bends and are badly misaligned at joints. In addition, holes have been found to be blocked with constructional materials such as timber, steel and grout as well as deposits of organic matter, gravel and calcium carbonate. In order to clear these pipes, downhole drilling equipment has had to be used. A photograph of a Wombat flexible drill which can be very successful in this work is shown in fig 4. This equipment can clear blockage down holes caused by deposits of calcium carbonate and, in conjunction with pressure water jetting, gravel and timber but it cannot deal with tight bends, badly misaligned joints and blockages by concrete or steel. In some instances therefore, new holes have had to be drilled to intercept the rubble drain in order to create a satisfactory relief system.

Very recently in one of the Board's major dams, Mullardoch, where there had been serious problems with the relief system due to restricted access and sharp bends, pressure jetting equipment has been used very successfully adopting pressures at the nozzle of the order of 5,000 psi. This is a new development and on the basis of results to date, provided it is carried out by experienced contractors, the results appear to be considerably better than those achieved by other methods.

Pressure relief systems require regular maintenance such as rodding to keep the drains clear and flushing with water to remove soft deposits. The deposition of calcium carbonate is a continual problem but it has been found possible to prevent this by trickling in a flow of the naturally acidic reservoir water through the system which dissolves away any depositing calcium carbonate. This system has been installed at three of the Board's dams. The dams are rodded every 1-2 years and a test is carried out every 10 years, often resulting in the requirement to drill out drains. Drainage systems should be readily testable, roddable and maintainable with no bends and good access. Pipes should not be less than 6 inches in diameter, desirably greater. Inspection galleries have been found to be valuable in this respect provided they are large enough to accommodate drilling equipment.

Wave Protection

Several of the Board's dams have long fetches of many miles and in these cases the dams are subject to heavy wave action. Four types of wave protection have been provided.

The first is stone pitching grouted with cement mortar which initially looks very attractive. However this type is poor in service since the fines and small gravel on which the pitching is laid are easily washed out leading to undermining and ultimate collapse of the pitching and its subsequent destruction.

The second is stone pitching which has been pinned or wedged by thin slivers of stone and which is flexible and again very attractive aesthetically. Overall this type of pitching has given a good performance extending up to 50 or 60 years. Both types of stone pitching are expensive to provide and to maintain when defects arise and are therefore being superseded by single size rip rap.

The third type is pre-cast concrete blocks which are laid on a bed of fine gravel. Again experience of this type of wave protection on Orrin dam has not been good. Every few years it is necessary to true up the upstream face. The blocks are approximately 900 x 900 x 300 mm thick and some are frequently and easily dislodged and even moved out of position by wave action.

The fourth form is heavy rip-rap which provides a very much better protection which readily dissipates wave energy, settles without failure and is easily replaced if damaged. It is also the cheapest form of wave protection.

Armouring is frequently needed downstream of a dam due to spill and airborne dispersion of water and this can cause washouts. It has been found that for downstream faces and situations subject to erosion, gabion baskets and mattresses generally of plastic coated wire filled with stone and well staked to the underlying strata are very satisfactory and economic. Where it is necessary to secure gabions in position (eg on a dam face) pitch is poured into the baskets which, although flexible, locks the stones within the gabions.

Gate and Valves

There are over 150 gates of various types associated with the Board's dams. These include drum gates for flood control purposes up to 27 m long by 5 m high, radial gates up to 8.5 x 8.2 metres and vertical lift gates up to 7.6 metres square. Most of the gates were constructed when stainless steels were practically unobtainable or prohibitively expensive and when long life coatings were in their infancy.(10)

The Board's policy for testing, inspection and maintenance of gates is that gates should be tested during commissioning and every 25 years for their full duties. They are partially exercised every 3 months, fully exercised annually and it has been found generally necessary to refurbish them after 25-30 years service. Radial gates have required little maintenance apart from recoating and renewal of steel wire ropes. Drum gates require comparatively much more protection but apart from that, little maintenance has been needed except for the replacement of the associated small bore pipework. Serious problems have been encountered when major maintenance has had to be undertaken on ground sluices or scour gates where only one gate has been provided on the culvert. In some cases, slots have been provided for emergency gates or stop logs but in a few, no provision has been made and this has necessitated very difficult underwater work or the construction of cofferdams to refurbish the gates or the very expensive alternative of dewatering the reservoir.

An example of such a cofferdam is the barrage at Kinloch Rannoch which was built some 50 years ago. Considerable wear had occurred to the roller trains and the built-in sections of these gates had corroded as they were manufactured from mild steel and cast iron. It was impossible to maintain them under water and this necessitated either dewatering the reservoir or building a cofferdam. All bearing surfaces have been replaced with non-corrodable materials and where wear is anticipated replaceable components.

Another feature which may be of interest is the electrical systems for operating these large gates which in the case of flood gates have to be backed up by a guaranteed system. In many of the dams this is either a battery driven system or by standby generators. These systems frequently give trouble; a very cheap and effective alternative is to provide petrol driven flexi-drives at each of the dams which not only cover the electrical system but also are a totally independent standby for the motive power for the gates. In the ultimate, hand operation is always possible.

In the onerous climate of the Highlands it is very important to have first class housings for the winches and electrics of gates and major valves. In general, gate houses are built of concrete at most of the Board's dams. These are not attractive aesthetically and have given problems with respect to leakage through flat roofs and vandalism. In the future cavity walls with DPC's would be specified and flat roofs would be avoided wherever possible in favour of pitched roofs. To reduce vandalism windows would be either non-accessible (facing over a reservoir) or roof lights would be provided; any windows which were necessary would be glazed with unbreakable polycarbonate glass.

Steelwork

Repainting of steelwork has, over the years, constituted a significant part of the costs of maintenance of the Board's schemes, particularly gates, pipelines and the steel linings of tunnels. The original specifications for paintwork on these structures normally comprised red lead with two or three coats of bituminous paint solution. In the onerous conditions of the Highlands, with its acid waters, these paint films have a life of only 5-10 years even with the best of materials and high standards of application. Over the past one to two decades, high build epoxy pitch coatings have been increasingly adopted for these structures with excellent results. The surfaces are taken down to bright steel, Swedish Standard Sa $2\frac{1}{2}$ using silica-free abrasive and then protected with a wash coat and 3 subsequent coats of coal tar/epoxy based materials to give a minimum dry film thickness of 375 microns.

These coats have been used on many high erosion areas such as turbine spiral casings, draft tube gates, flood drum gates, scour culverts and tunnel nosings with notable success. The longest period of service is on a tunnel lining at Gaur dam which was coated in 1960 and which has subsequently remained in almost perfect condition without the need for any maintenance whatsoever.

Application conditions in relation to humidity and temperature are very critical and onerous and the materials are hazardous to apply with respect to health, toxicity and flammability. Needless to say, the cost is high - about four to five times that of a conventional paint system - but their performance has been excellent and most attractive economically when the significant reduction in outages and the consequent savings in spilled water are taken into account. The Board's policy, in the long term, is to coat all steel surfaces, wherever practicable, with these high build, long lasting coatings.

Resources

The large majority work is directed, managed and controlled by the Hydro Maintenance Section of the Division working from Edinburgh. Most of the work is undertaken on site by placing contracts on a competitive basis and is supervised by staff from the Section which comprises a staff of 9 responsible for all significant maintenance work on the Board's Hydro Schemes. Routine minor maintenance is undertaken by the staff of the Generation Groups, who are responsible for managing typically 10 to 15 dams. Each Group has a squad of 6-8 men who not only carry out routine maintenance but also are responsible for operational water duties. All maintenance work undertaken by the Division is recorded on a punch card system for future reference and this has been found to be invaluable - this data is now being placed on the computer. Likewise, checklists and standard forms are used for recording all the details of dam inspections and for classifying the priority in which the work is to be carried out. This also applies to tunnels, intakes, aqueducts and stations.

Maintenance Costs

Maintenance of the Board's concrete dams has been minimal and largely superficial. Most of the deterioration is in appurtenant structures which unfortunately is often very noticeable. Costs have been very low and have averaged less than 0.1% per year of the current estimated capital cost of the dams.

CONCLUSION

The performance of the Board's concrete dams and their reservoir yields have been very good. Deterioration of the dams over the past 50 years has been very small, superficial and not significant with respect to structural stability, safety, maintenance costs or the serviceable life of the dams. Experience indicates that ageing in these concrete dams is an extremely slow process which is likely to be measured in a timescale of a hundred or more years rather than tens of years. There has been no appreciable silting of the reservoirs.

Overall, there would therefore appear to be no limiting feature which will prevent the dams having lives of hundreds of years and continuing to be an excellent and increasing investment to the Board reflecting great credit to their designers and constructors.

ACKNOWLEDGEMENT

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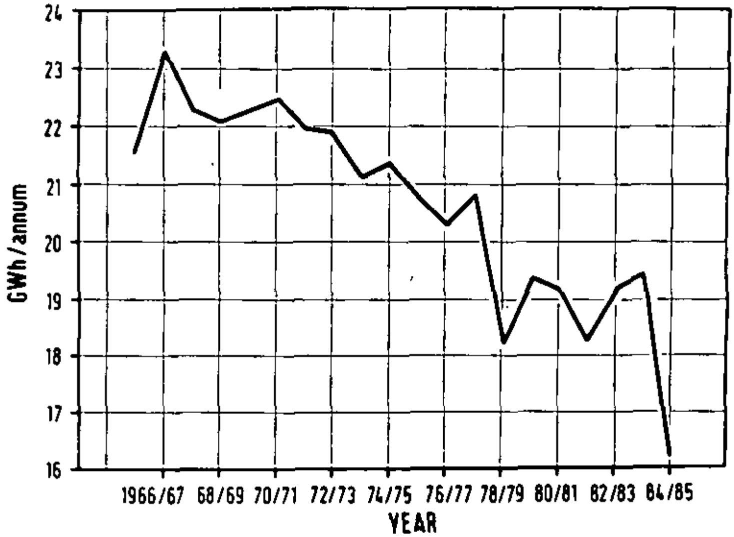

Figure 1 Output for Striven Power Station after adjustment for weather and storage

Figure 2 Downstream face of Loch Dubh Dam

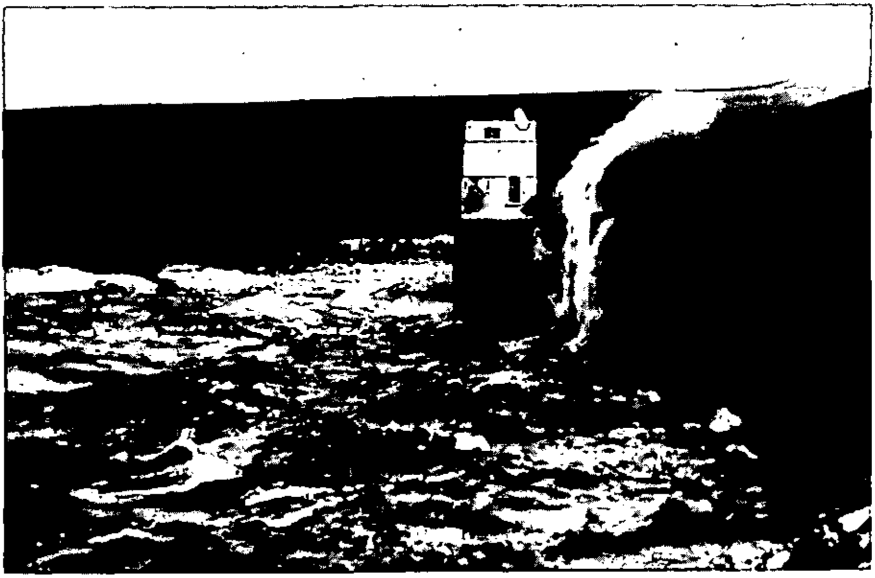

Figure 3 Waves at Glascarnoch

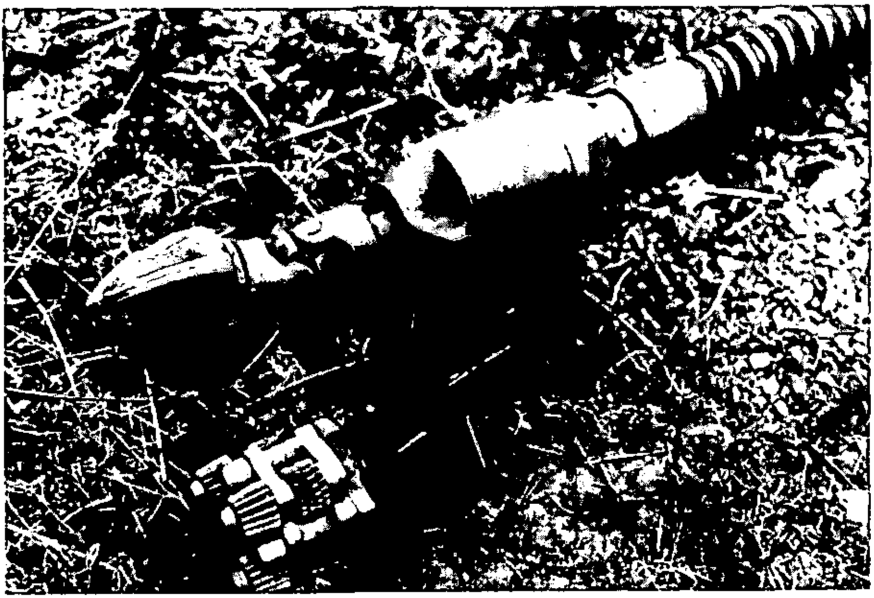

Figure 4 A Wombat flexible drill

RESERVOIRS ACT 1975 : EXPERIENCE SO FAR

S.C.Agnew*

After describing the implementation stages of the 1975 Act, progress with the appointment of engineers to the new panels and on the compilation of registers of large raised reservoirs is recorded. Experience of operation of the Act so far is dealt with from the standpoint of inspecting and supervising engineers, reservoir owners, enforcement authorities and Government departments.

INTRODUCTION

1. It might to some seem surprising that the experience so far of an Act which has been on the statute book for over 10 years justifies the presentation of a paper at this Symposium. But although enacted in 1975, the Reservoirs Act of that year came fully into operation only on 1 April this year over most of Great Britain, and will not be fully in operation in the former metropolitan counties and the Greater London area until 1 April 1987. So in practical terms operational experience of the Act at the time this paper is being written is very scanty.

2. On the other hand experience in the operation of reservoirs safety legislation in this country has quite a long history, and in principle the main provisions of the 1975 Act follow closely those in the predecessor Reservoirs (Safety Provisions) Act of 1930. While the 1930 Act has in general proved effective for over 50 years, in that there has been no major dam disaster involving loss of life, experience has shown up certain weaknesses First, with many dams in this country now well over 100 years old, it was thought that some degree of expert surveillance of dams was necessary in between the 10 year maximum intervals for statutory inspections prescribed in the 1930 Act. And secondly, the absence of provisions imposing a duty on local authorities to enforce the 1930 Act's requirements for inspection and remedial works, resulted in considerable doubts about the effectiveness of the Act in the case of many privately owned reservoirs.

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PURPOSE OF THE RESERVOIRS ACT 1975

3. The essence of the 1975 Act is to provide machinery to ensure that in Great Britain all "large raised reservoirs" (generally defined as "designed to hold, or capable of holding, more than 25,000 cubic metres of water above the natural level of any part of the land adjoining the reservoir") are not, or cannot become, a safety hazard. The Act continues the 1930 Act system of requiring not only that the design and construction of any new, or certain alterations to an existing, large raised reservoir, are carried out by, or under the supervision of, a qualified engineer, but that all large raised reservoirs shall be inspected by qualified engineers at prescribed maximum intervals. Additionally however the 1975 Act:-

(i) imposes a duty on local authorities to enforce the Act and provides them with stronger and more explicit powers to satisfy themselves that the Act is being complied with, to undertake emergency action and to carry out remedial work and recoup the costs from reservoir undertakers;

(ii) requires reservoir undertakers to appoint a named supervising engineer to keep each reservoir under supervision at all times;

(iii) makes non-compliance with the safety provisions of the Act a criminal offence unless there is reasonable excuse;

(iv) limits appointments of engineers to qualified engineer panels to 5 year terms, subject to reappointment;

(v) lays down a procedure if a reservoir is to be abandoned or discontinued.

The main provisions of the Act are summarised in Appendix A.

IMPLEMENTATION PROGRAMME AND PROGRESS TO DATE

4. Implementation of the 1975 Act was deferred by successive governments but concern from several quarters, including a recommendation from a sub-committee of the House of Lords Committee on Science and Technology in January 1983 convinced the Government of the day that implementation should not be further delayed. The Government decision had also been influenced by the outcome of a questionnaire to local authorities about the use of their discretionary powers under the 1930 Act, which revealed that in addition to little use having been made of the 1930 Act powers, there was a lack of information about the number and condition of privately owned reservoirs, whose owners in many cases had not had them inspected as required by the 1930 Act.

5. The Government decision to implement the 1975 Act was announced in the House of Lords on 8 March 1983 and the first stage of implementation took place with the making of a Commencement Order bringing into force on 30 November 1983 Sections 1, 4, 5, 29 and 30 and Schedule 1. The second Commencement Order, made on 13 February 1985, brought into force on 1 April 1985 those provisions concerned with the setting up of registers of reservoirs and with the enforcement of the Act by local authorities, including the power to take emergency action. Also included were those provisions imposing obligations on reservoir undertakers to keep records and supply information. This Commencement Order did not apply in the metropolitan counties or in Greater London. The third and final implementation stage came into effect

on 1 April 1986, except in the metropolitan counties and Greater London and brought into force all remaining provisions of the Act including the supervision requirement. At the same time, the provisions that came into force on 1 April 1985 in most of Great Britain were introduced in the metropolitan counties and Greater London. It is expected that on 1 April 1987, full implementation will be introduced in those areas.

6. In addition to the Commencement Orders, implementation has required the making of 8 other Statutory Instruments containing regulations and rules about all the matters which require to be prescribed in several sections of the Act. For the record, these are listed in Appendix B.

REACTIONS TO IMPLEMENTATION PROPOSALS

7. For some years the Institution of Civil Engineers had been strongly advocating implementation of the 1975 Act, as had the majority of civil engineers involved with reservoirs. One or two civil engineers did however express doubts about the effect of the Act on small privately-owned reservoirs and suggested that to avoid unnecessary bureaucratic control on for instance the owner of a single small reservoir, the requirements for record keeping, supervision, inspection, etc should be modified to accord with the risk. There was also some concern among older members of the existing 1930 Act Panels about their eligibility to continue to inspect reservoirs when the 1975 Act came into force.

8. When local authorities were first consulted in 1976 about implementation, they accepted in principle that the Act should be implemented, but in view of the pressure to contain public expenditure at that time, suggested that implementation be delayed for a couple of years. They were concerned not only about the cost associated with the setting up of registers and monitoring the submission of certificates of inspection etc, but also at the possibility of having to take action to make safe ownerless or abandoned reservoirs. The same concerns were repeated during the 1983/84 consultation process, but after meetings between Government Ministers and representatives of the local authority associations, it was accepted that the costs that would fall to local authorities would be relatively modest. There was no reason to suppose that enforcement authorities would have to incur substantial irrecoverable costs and it should be a rare occurrence indeed for an authority to have to take action to breach a dam. The average cost of breaching some 10 dams of various sizes over the previous few years had been about £50,000 at 1984 prices.

9. During its passage through Parliament there appeared to be general support from all sides for the Reservoirs Bill and it was not until after the Government had announced its implementation plans that organisations such as the Country Landowners' Association began to express concern about the effect of the Act on small privately-owned reservoirs, particularly those that existed primarily for recreational or amenity purposes. Although in principle most of the requirements of the 1975 Act were similar to those in the 1930 Act, it was realised that enforcement authorities would now be obliged to require all owners to comply with the requirements for inspection and owners would not be able to ignore the recommendations for repairs etc made by the inspecting engineer in the interests of safety. There would also be the cost of a supervising engineer's services. It was pointed out that many of these reservoirs do not represent an economic asset to their owners but nevertheless are of local importance as an amenity feature or a wildlife habitat. Rather than face the expense of complying with the 1975 Act, many owners could choose to lower or breach the dams of such reservoirs. The Country Landowners'

Association went on to argue that as such reservoirs rarely presented a hazard to public safety, steps should be taken by amending the legislation or otherwise to mitigate the effect of the 1975 Act on such reservoirs. The same problem had earlier been recognised by the Institution of Civil Engineers, who in September 1984 issued a supplementary note to their publication "Floods and reservoir safety : an engineering guide" drawing attention to the fact that the guide was not mandatory and that panel engineers, in consultation with owners, should use their discretion in the selection of the "design flood" bearing in mind that while the "general" standard was recommended for those existing dams unable to withstand a short period of limited overtopping, the "minimum" standard would be acceptable in many cases. It was considered by the Institution that this, coupled with the discretion exerciseable by panel engineers, should go a long way to ensuring that owners of small "low risk" reservoirs are not put to unnecessary expense, and this was the view taken by Government Ministers in responding to the concern of the Country Landowners' Association and others. The Department of the Environment has also drawn these concerns to the attention of all panel engineers and pointed out the scope for discretion that is available to inspecting engineers. The extent of any problem, if one exists, will not become clear for some little time, but meantime Ministers take the view that safety must be the paramount consideration.

10. The reaction of certain conservation bodies, including the Nature Conservancy Council, has also been to express concern at the possible effect of the breaching or permanent lowering of reservoirs that are considered environmentally important as wildlife habitats etc. While this concern is understandable, it would clearly not be acceptable to ignore a safety hazard just because a reservoir is of some environmental significance. If the owner cannot, or will not, meet the costs of maintaining the reservoir in a safe condition, there seems little alternative to those who wish to preserve it offering some financial assistance if the reservoir is to be maintained.

APPOINTMENTS TO PANELS OF QUALIFIED ENGINEERS

11. Following considerable debate and advice from the Institution of Civil Engineers, the Secretaries of State announced in December 1984 that they intended to constitute 4 panels of qualified civil engineers under Section 4(1) of the Act. Descriptions of the panels based on the Explanatory Notes to the relevant statutory instruments are-

All Reservoirs Panel (AR Panel)	:	Civil engineers who may be employed for the purposes of any section of the Act in connection with any type of reservoir, however constructed.
Non-impounding Reservoirs Panel (NIR Panel)	:	Civil engineers who may be employed for all purposes of the Act, except Section 19, in relation to non-impounding reservoirs, and who may undertake the supervisory duties for which Section 12 of the Act provides in relation to all reservoirs.
Service Reservoirs Panel (SR Panel)	:	Civil engineers who may be employed for all the purposes of the Act, except Section 19, in relation to non-impounding reservoirs that are constructed of brickwork, masonry, concrete or reinforced concrete and who may undertake the supervisory duties for which Section 12 of the Act provides in relation to all reservoirs.
Supervising Engineers' Panel (Sup Panel)	:	Civil engineers who may be employed under Section 12 of the Act, which provides for the supervision of large raised reservoirs.

12. The process of appointing engineers to the 1975 Act panels commenced with the coming into operation of SI 1984 No 1874 on 20 December 1984 and the appearance of an advertisement in New Civil Engineer inviting applications for the Supervising Engineers' Panel. At the date of writing, 378 applications have been received and 281 civil engineers appointed to the Supervising Engineers' Panel. In general the applicants have met the criteria decided upon by the Reservoirs Committee of the Institution of Civil Engineers as appropriate for their recommending appointment to the Panel. It is clear that the special course run for supervising engineers by the Water Industry Training Association has proved to be very popular and extremely valuable.

13. As the emergency powers in Section 16 of the Act came into force on 1 April 1985, it was necessary to have some engineers appointed to the AR Panel by that date. In December 1984 the Department of the Environment wrote to all members of the 1930 Act panels drawing their attention to the 1975 Act panel structure and advising them of the arrangements for appointment to the new panels. The Act does not provide for transfer from the existing to the new panels and members of the former wishing to join the new panels need to apply in the prescribed manner. For the AR panel this was laid down in SI 1985 No 175 which came into operation on 21 February 1985. Applications for appointment to Panel AR started to come in soon thereafter and a number of appointments were made by 1 April. At the date of writing 58 appointments have been made to Panel AR. This compares with a total of about 100 on the old Panel I.

14. Recruitment to the two remaining panels for non-impounding reservoirs (NIR and SR) began with the coming into operation of SI 1985 No 1086 on 7 August 1985. At the date of writing there have been 25 applications for Panel NIR and 13 appointments have been made. This compares with a membership of 37 on the old Panel II and 40 on the old Panel III. It will be realised that the new panel structure does not provide an equivalent to the old Panel III - which covered non-impounding reservoirs with a capacity of less than 50 million gallons. There had been a considerable debate about whether or not to continue to have two panels for non-impounding reservoirs other than those of brickwork, masonry, concrete, etc, but the view prevailed that a single panel would be sufficient. Nevertheless in arriving at this decision it was recognised that based on past experience of appointments to the 1930 Act panels, when in recent years very few new appointments had been made to Panel II whereas several had been made to Panel III, difficulties might arise, depending on the demand for the services of engineers qualified to inspect smaller non-impounding embankment reservoirs such as those used for agricultural purposes and the success of those experienced in such work in gaining appointment to Panel NIR. It is intended to monitor the situation for the next few years to determine if any change in the panel structure might be necessary. It can be argued however that those engineers who were members of Panel III at the date of commencement of the 1975 Act, will be able to continue to inspect non-impounding reservoirs of less than 50 million gallons capacity for up to 5 years (Section 23(2)).

15. Applications for the Service Reservoir Panel have been surprisingly slow to come in when compared with a membership of 67 on the existing Panel IV. At the date of writing 35 applications have been received and 12 appointments made.

16. With the requirement in the 1975 Act that appointments to panels are for terms of 5 years, the question arises as to whether there should be an age bar to reappointment. It has been suggested that reappointments should not be made beyond a certain age, say 70, but no hard and fast rules have been laid down. Perhaps a more important criterion is the extent to which an applicant for reappointment has been actively and personally engaged in reservoirs work over the preceding 5 years. I have little doubt however that in the knowledge that age may be a factor to be considered, panel engineers will be the best judge themselves of whether or not they should apply for reappointment.

COMPILATION OF REGISTERS

17. Prior to a start being made on the compilation of registers, the best estimate of the number of large raised reservoirs in Great Britain had been based on various sets of figures collected by the Environment Departments over a number of years. The total was thought to be about 2,000. Of these about 1,550 were owned by the water authorities and other public bodies such as district councils, the Electricity Boards and British Waterways Board. It was assumed that this figure was reasonably correct. 400 privately owned reservoirs had been listed and it was estimated that there were at least a further 50 large raised reservoirs that had not been identified or where the owners had not been traced. Since enforcement authorities have started to think about compiling registers, several have suggested that the number of privately owned reservoirs greatly exceeds the number previously assumed, though most of these are likely to be relatively small and some remote.

18. Enforcement authorities commenced the task of compiling registers on 1 April 1985 and under Section 24 of the Act undertakers were required to provide enforcement authorities with information about their large raised reservoirs by 1 January 1986. Enforcement authorities initially advertised in local papers drawing to the attention of undertakers their duty to provide information, but it is understood that in a number of cases authorities have found it necessary to draw to the attention of individuals, companies etc that they appear to own a large raised reservoir about which they were required to provide information. This has involved enforcement authorities in examining maps, making local enquiries and in some cases carrying out simple surveys (or asking the owner to do so) to determine the capacity of a reservoir.

19. While the registers are understood to be virtually complete in respect of publicly owned reservoirs, it appears that a good deal of data is still lacking about privately owned reservoirs, particularly the smaller ones that have not previously been inspected. Inspecting engineers who have been called in to assist private owners point to the practical difficulties of obtaining sufficient information about some of the older reservoirs and have suggested that enforcement authorities should not press too hard for this information as they have until April 1987 to report to the Secretary of State Nevertheless sufficient information has been obtained from the enforcement authorities in Scotland and Wales to give a good indication of the likely total number of reservoirs on the registers in these countries. Because of the greater number of enforcement authorities in England and the delayed implementation in Greater London and the metropolitan counties, it has not been practicable to obtain information about the number of reservoirs on the registers there in time for this paper. As expected the information available for Scotland at least, does show that there are more large raised reservoirs in private ownership than had previously been recorded. The total number in Scotland is likely to be about 736 as compared with earlier

estimates of 582, while the figures for Wales are 234 and 250 respectively. Of these there are currently known to be 151 privately owned reservoirs in Scotland (previous estimate 120) and 22 in Wales (previous estimate 90). It is probable, however, that most of the reservoirs yet to be registered or categorised will be privately owned. The distribution of ownership and size so far as it is known at present for Scotland and Wales is as follows:-

	Reservoirs on Register		Earlier Estimates	
	Scotland	Wales	Scotland	Wales
Regional and Islands Councils and Welsh WA	356	108	344	110
Other Public Bodies	148	43	118	50
Privately owned	151	22	120.	90
On register - Category not yet known	-	42	-	-
Estimated number still to be registered	91	26	-	-
Sub-Total	746	241	582	250
Deduction for reservoirs on more than one register	10	7	-	-
TOTAL	736	234	582	250

If a similar picture emerges for England, the total number of large-raised reservoirs in Great Britain might be about 2,400, of which some 700 would be privately owned. This compares with the earlier estimate of about 2,000 and 450 respectively.

EXPERIENCE OF INSPECTING ENGINEERS

20. Some inspecting engineers have already experienced an appreciable increase in requests for inspections of privately owned reservoirs. This of course was only to be expected. What also appears to be happening is that some inspections are leading to decisions by owners to take their reservoirs out of the ambit of the Act, either by sufficiently lowering the spillway level or breaching the embankment. (Either operation would of course require the services of a panel engineer). This is amounting to about one in six of privately owned reservoirs being inspected for the first time by one inspecting engineer. Another feature that has emerged is that some private owners are seeking competitive bids from a number of panel engineers to carry out inspections. While this may be in sympathy with the current trend to encourage competition, some panel engineers are worried that such a practice could lead to cut price inspections and a reduction in safety. Nevertheless panel engineers are having to give very full consideration to the financial circumstances of the reservoir owner in framing their recommendations. The supplementary guidance issued by

the Institution of Civil Engineers on design floods (referred to at para 9) was certainly timely and inspecting engineers appear often to be able to devise ways of satisfying the safety requirements without imposing unacceptable financial burdens on private owners. Existing overflow capacity is not usually a problem with small privately owned reservoirs and where necessary it can often be increased for modest expenditure. It has been suggested however that not all inspecting engineers are following the guidance that it is not necessary for overflow works to be able to contain the design flood within the channel walls, and that some overtopping of the walls and embankment crest can usually be permitted. I understand that inspecting engineers are generally finding that the Act and ICE Guide between them provide ample discretion to deal briefly and clearly with small reservoirs which present little risk.

21. A new requirement placed on inspecting engineers by Section 26(2) of the Act, is the provision of drawings and descriptions giving, so far as they can, information about the construction of a large raised reservoir which was completed before the commencement of the 1930 Act, but which is being inspected for the first time under the 1975 Act. Where the owner possesses drawings showing the works actually constructed, this should cause no difficulty, but if not inspecting engineers will be faced with a problem in deciding how much information it is reasonable to provide and owners in turn may be faced with having to pay for the necessary survey and preparation of drawings etc. It is not yet known if this has presented any problems.

22. While in principle the certificates to be issued under the 1975 Act differ little from those used for 1930 Act purposes, there are some additions covering for example abandonment and discontinuance of reservoirs. A major difference however is that construction and inspecting engineers are required to send copies of all certificates (and some reports) to the enforcement authority. There is little experience so far as to how this procedure will work, but it is the clear intention of the Act that the duties of the enforcement authorities in relation to certificates and reports will be essentially administrative. Their task is to ensure, by checking the receipt of reports and certificates, that inspections and recommendations as to measures to be taken in the interests of safety are carried out in good time. It is the responsibility of inspecting engineers to issue certificates, confirming that recommendations made in the interests of safety have been carried out satisfactorily, so there should be no need for enforcement authorities to make any engineering judgement on reports or certificates submitted by inspecting engineers.

EXPERIENCE OF SUPERVISING ENGINEERS

23. Information so far indicates that different undertakers are organising the requirement for supervision in a variety of ways. In some cases full time supervising engineers are being appointed and given responsibility for the supervision of a large number of reservoirs. At the other extreme some undertakers intend to allocate the duties of supervision among a number of engineers each having responsibility for a small number of reservoirs. Again some undertakers take the view that the supervising engineer should have no connection with the day to day operation of the reservoirs which he is to supervise, while others take the opposite view that supervision is best carried out by those involved in a particular reservoir's day to day operation and management. Clearly there is no single correct method. Local circumstances and organisation will largely dictate what is best for each particular undertaker. and there is no reason to suggest that any of the alternatives will not be effective. Another open question is the number of occasions per year on which a supervising engineer should carry out a formal inspection. Some

guidance on this may be given in future by construction or inspecting engineers, but again local circumstances will dictate what is necessary. It has been suggested that a minimum of 2 visits per year will be necessary but it may be that even this would be excessive for some small reservoirs presenting an insignificant safety risk in the event of failure. So far supervising engineers have experienced no problems of conflict where they are full-time employees of the undertaker and there is no reason to suppose that they will not be allowed to exercise their full authority as supervising engineers by their employers and line management. In-house supervising engineers will also of course have to be issued with letters of appointment by their employers and it has been suggested that these should specify clearly the facilities that the undertaker will provide for visits of inspection etc.

NEW FORM OF RECORD ETC

24. It has been suggested by some that the keeping of a record for each large raised reservoir as required by Section 11(1) of the Act will be unnecessarily onerous for certain reservoirs and in particular for small privately owned reservoirs in remote locations. This appears to have been based on the erroneous assumption that particulars of water level etc have to be recorded at least weekly as was required by the prescribed Form of Record for the 1930 Act. This is incorrect as under the 1975 Act the information to be included in the record is to be given in such a manner and at such intervals as the construction or inspecting engineer directs. The bound record form has been criticised in that it does not give enough space for some records and too much for others. This publication is of course a commercial enterprise by Thomas Telford Limited and although modelled on the prescribed form in SI.1985 No 177, if not suitable it need not be used by undertakers. So long as they comply with the prescribed form, they can draw up their own tabulated record.

25. It has been pointed out that the Act is not clear about the frequency and manner of keeping records of a reservoir inspected under the 1930 Act prior to a 1975 Act inspection, when the inspecting engineer will give directions on these matters under Section 11(2). It is not in doubt that from 1 April 1985 in most of Great Britain that the requirement for all large reservoirs has been to keep records in the form prescribed in SI 1987 No 177, rather than Form F. In many cases the undertaker will be content to continue recording the prescribed information in Parts 1 and 2 of the new form at weekly intervals as for Form F, but when he considers this too onerous I would have thought he could seek the advice of a panel engineer pending an inspection under the 1975 Act.

EXPERIENCE AND ATTITUDE OF RESERVOIR OWNERS

26. Water authorities and other public sector owners of reservoirs have in general conformed with the requirements of the 1930 Act, so there is no indication that they will have any difficulty, or show any relunctance, in complying with the requirements of the construction, inspection, etc provisions of the 1975 Act. They will not however be able to delay unduly in putting into effect any measures that may be recommended by inspecting engineers in the interests of safety. As to supervision, while public sector owners as a whole recognised and accepted the need for this additional requirement, it has to be said that one public sector owner at least has expressed concern at the additional cost of this requirement. However, many public sector reservoir owners, including most of the water authorities had, in preparation for implementation, already started systematically to supervise their reservoirs on a regular basis and there is every indication that they will wholeheartedly meet the supervision requirement of Section 12.

27. Apart from the concern of the Country Landowners' Association and others referred to at para 9 above, there is little experience as yet about the attitude of other private owners or as to the effect of the Act on privately owned reservoirs in general. It is clear however from what has been said above that many owners are taking action to have reservoirs inspected for the first time. It also seems likely that a number of reservoirs that are no longer required for their original purpose and which may present a safety hazard, will be permanently lowered or drained or have their dams breached. There is no hard evidence so far that the Act is bearing unfairly on private owners having regard to the over-riding requirement to ensure that any large reservoir does not present a threat to safety.

28. There has been some criticism from representatives of private owners, as well as civil engineers, of Parliament's failure to double the threshold figure of 5 million gallons as recommended by the Institution of Civil Engineers in 1966. However, it must be said that this would not have met wholly the criticism of private owners that the Act's requirements will be unnecessarily onerous for small "low risk" reservoirs. A small number holding between 25,000 and 50,000 cubic metres would have been taken out of the ambit of the Act, but at the same time there are undoubtedly some reservoirs holding less than 50,000 cubic metres that could constitute a safety risk. On the other hand there are others holding even greater volumes where because a very low dam impounds water over a large area, or because of location, the risk to persons or property from dam failure would be insignificant. In such cases the answer may be for the inspecting engineer to use his discretion to the full by recommending only very minimal requirements in the way of record keeping and in any suggestions he makes on the frequency of visits by a supervising engineer, though such frequency is of course a matter for agreement within the law between the undertaker and the supervising engineer. It should be noted that a supervising engineer is only required to give a written statement to the undertaker not less than once a year on matters he has been instructed to watch in an annex to a final certificate or report of an inspecting engineer. This whole matter is one that may need further consideration in the light of experience. In the interests of safety however, and to check that conditions have not changed, it would seem unwise to contemplate any relaxation in the 10 year maximum interval betwen formal inspections.

EXPERIENCE OF ENFORCEMENT AUTHORITIES

29. Enforcement authorities, who are required to report to the respective Secretary of State at 2 yearly intervals on the action they have taken to ensure compliance with the Act, are due to submit their first reports on 1 April 1987. Therefore at the date of writing this paper there has been little opportunity to ascertain to what extent enforcement authorities may have used, or considered using, their statutory powers. Some may have already had to use these to obtain information in compiling the registers, but it is unlikely that any authority will yet have had to use the powers in Section 8 (non-compliance with requirements as to construction or enlargement of reservoirs) or even in Section 10(7) requiring an undertaker to appoint a qualified civil engineer to carry out an inspection or to carry out a recommendation as to measures to be taken in the interests of safety. Neither is it likely that enforcement authorities will have yet considered it necessary to take action under Section 12(4) to require an undertaker to appoint a supervising engineer. So far very little information has reached me about any problems that may have arisen for authorities in dealing with "ownerless" reservoirs. I have heard however of one case, where as the result of a report by a hiker of leakage through the dam of a disused reservoir in a remote area of north Wales, and where the owner could not immediately be identified, the enforcement authority decided to

appoint a panel engineer to carry out an inspection. Although this did not call for any immediate action in the interests of safety, some expenditure will be necessary and the enforcement authority is concerned at the financial consequences of being unable to recover the costs involved, although these are likely to be relatively modest. In Scotland, the concern of persons living downstream from a reservoir, has led, on at least two occasions, to the enforcement authority calling in an inspecting engineer to report on whether or not any action was needed and whether or not the reservoirs were covered by the Act. It has been observed that at the outset different enforcement authorities were applying different standards to their requirements for information etc but these now appear to be more or less the same.

DISCONTINUANCE OR ABANDONMENT OF RESERVOIRS

30. The requirements of Sections 13 and 14 of the Act as to the measures to be taken on the discontinuance or abandonment of large raised reservoirs are new. They provide machinery to remove a reservoir from the register or to ensure that an abandoned reservoir is not, or cannot become, a safety hazard. Discontinuance involves rendering a reservoir incapable of holding more than 25,000 cubic metres and Section 13 provides for its removal from the register. Abandonment on the other hand requires a report and certificate under Section 14 from a qualified civil engineer to secure that the reservoir is incapable of filling accidentally or naturally or is only capable of doing so to an extent that does not constitute a risk. The reservoir remains on the register unless a discontinuance certificate is issued under Section 13. But the new provisions do not overcome the obstacles to abandonment or discontinuance that may have to be faced by a reservoir owner from planning authorities or other bodies with an amenity or recreational interest who would wish to see the reservoir preserved. Examples have also been cited of old water retaining structures being designated as "listed buildings", in which case statutory objection could be raised against any proposed alteration.

RELATIONSHIP WITH HEALTH AND SAFETY LEGISLATION

31. An administrative agreement has been reached with the Health and Safety Executive (HSE) on how their inspectors will in future enforce the requirements of their legislation - the Health and Safety at Work etc Act 1974 (HSW Act)- at reservoirs falling within the scope of the Reservoirs Act 1975. In the case of reservoirs falling within the scope of the 1975 Act, HSE inspectors will not attempt to cover matters relating to structural integrity. They will however remain responsible for other aspects of reservoir safety such as duties towards employees and duties towards members of the public when a reservoir is used for recreational purposes, eg sporting activities. It has also been agreed that in the event of an HSE inspector noticing during his visit, obvious features which give him cause to doubt the structural integrity of a large raised reservoir and which could cause a hazard to members of the public then the appropriate local authority will be alerted.

INITIAL VIEWS OF GOVERNMENT DEPARTMENTS

32. The impression gained so far is that implementation of the Act has progressed quite smoothly and that it is beginning to have the intended effect Thanks to the co-operation of the Institution of Civil Engineers and the hard work of members of its Reservoirs Committee (and other panel engineers invited to assist with the vetting of applications for supervising engineers) appointments to the 1975 Act panels have progressed according to programme. Departments await with interest submission of the first formal reports from

from enforcement authorities, but meantime the lack of any representations would suggest that the task of compiling registers and setting up the enforcement arrangements is going according to plan. The number of appointments to the new panels of qualified engineers will continue to be monitored and the panel system kept under review so as to ensure that the needs of the various classes of reservoir owners for inspecting and supervising engineers can be met without undue inconvenience or expense. Departments will also welcome information that will enable them to assess whether or not there are any grounds for the fears that have been expressed as to the effects that the Act will have on smaller recreational and amenity type reservoirs. In general the Departments get the impression that the provisions of the Act are sound and that it is being implemented effectively and reponsibly by all concerned - the enforcement authorities, reservoir owners and panel engineers alike.

APPENDIX A

MAIN PROVISIONS OF THE 1975 ACT

1. The provisions of the 1975 Act can be grouped conveniently under 3 headings. First the duties and powers laid on enforcement authorities, secondly the obligations laid on undertakers and thirdly the provisions affecting the appointment and responsibilities of qualified civil engineers.

2. Enforcement authorities - in England and Wales the county councils, in the Greater London and metropolitan county areas the borough and district councils, and in Scotland regional and islands councils:-

(i) are required to establish and maintain registers (which will be open to the public) of all large raised reservoirs wholly or partly in their area (Section 2);

(ii) shall ensure that other undertakers observe and comply with the Act in all respects (Section 2(3));

(iii) shall report to the respective Secretary of State at intervals (prescribed as 2 yearly) on the steps they have taken to ensure that undertakers (including themselves) observe and comply with the requirements of the Act. If the Secretary of State is not satisfied, he may order a Public Inquiry and make an order declaring the authority to be in default (Section 3);

(iv) may, in the event of an undertaker failing to comply with the requirements of the Act, act in default and recover the cost from the undertakers (Section 15);

(v) may use emergency powers to take immediate action where a reservoir is unsafe (Section 16).

3. Undertakers, who are generally the reservoir owners (but see Section 1(4) are required by the Act to:-

(i) have any new large raised reservoir designed by and constructed under the supervision of a qualified (as defined in Section 4) civil engineer (Section 6);

(ii) have any alteration to increase the capapcity of an existing large raised reservoir carried out only by a qualified civil engineer (Section 6);

(iii) ensure that their large raised reservoirs are at all times kept under supervision by a qualified supervising engineer, when not under the supervision of a construction engineer (Section 12);

(iv) have their large raised reservoirs inspected periodically by a qualified civil engineer at intervals of not more than 10 years. The first periodic inspection must be within 2 years of the date of the final certificate issued by the construction engineer. Where the inspecting engineer recommends measures in the interests of safety the undertakers must as soon as practicable, put these recommendations into effect and if satisfied a qualified engineer must provide a certificate to the effect that the necessary measures have been taken (Section 10);

(v) keep a record of water levels and depths, overflows, leakages, settlements, repairs and any other information that may be prescribed, do so in such a manner and at such intervals as may be required by the construction or inspecting engineer and provide and maintain any necessary instrumentation (Section 11);

(vi) In the event of wilful default of the Act's requirements, an undertaker (or an officer or member of a composite body) may be guilty of a criminal offence and liable to a substantial fine (Section 22).

4. In addition to providing for the appointment of panels of qualified engineers, the Act lays down specific requirements as regards the issue of certificates and the functions of qualified engineers, viz:-

(i) the construction engineer must issue a "preliminary certificate" specifying the permitted water level and any filling conditions as soon as he is satisfied that the reservoir can be wholly or partially filled (Section 7(1));

(ii) further preliminary certificates may be issued, but in not less than 3 nor more than 5 years the construction engineer, on being satisfied that the reservoir can be safely used for the storage of water shall issue a final certificate to that effect unless he provides a written explanation for the delay (Section 7(3) and (4));

(iii) the qualified engineer issuing a certificate is required to send a copy to the enforcement authority (Section 20);

(iv) the inspecting engineer must be "independent", ie not a direct employee of the undertaker nor connected with the construction engineer (Section 10(9));

(v) supervising engineers have a duty to keep the undertakers advised of a reservoir's behaviour, to report in writing at least once a year on matters noted for attention in the final certificate (Section 7(5)) or in the latest report by an inspecting engineer and to recommend to the undertakers at any time if he thinks an inspection is called for in advance of the next scheduled inspection (Section 12).

APPENDIX B

STATUTORY INSTRUMENTS - REGULATIONS AND RULES

SI 1984 No 1874	-	The Reservoirs Act 1975 (Supervising Engineers Panel) (Applications and Fees) Regulations 1984.
SI 1985 No 175	-	The Reservoirs Act 1975 (All Reservoirs Panel) (Applications and Fees) Regulations 1985.
SI 1985 No 177	-	The Reservoirs Act 1975 (Registers, Reports and Records) Regulations 1985. M ?
SI 1985 No 548	-	The Reservoirs Act 1975 (Registers, Reports and Records) (Amendment) Regulations 1985.
SI 1985 No 1086	-	The Reservoirs Act 1975 (Non-impounding and Service Reservoirs Panels)(Applications and Fees) Regulations 1985.
SI 1986 No 467	-	The Reservoirs Act 1975 (Referees)(Appointment and Procedure) Rules 1986.
SI 1986 No 468	-	The Reservoirs Act 1975 (Certificates, Reports, and Prescribed Information) Regulations 1986.
SI 1986 No 853	-	The Reservoirs Act 1975 (Application Fees) (Amended) Regulations 1986.

A Circular (DOE 5/85, WO 8/85) explaining the powers and duties of those concerned with the Reservoirs Act 1975 was published on 27 February 1985. A similar Circular (SDD 3/1985) was issued in Scotland on the same date.

A circular letter explaining the final implementation stages of the Act and the supervision, prescribed information and referee provisions, was issued in England and Wales on 20 March 1986 and in Scotland on 27 March 1986.

DOCUMENTATION FOR RESERVOIRS AND PROCEDURES

N.H. Gimson*

The statutory documentation which is required to be maintained under the Reservoirs Act 1975 by enforcement authorities and public and private undertakers is examined and problem areas are exposed. Some of the procedures required by or related to this legislation are discussed and possible future difficulties raised. Sections are concerned with emergencies and safety procedures. The forms of engineers' certificates and reports are referred to.

INTRODUCTION

In considering this subject, one is struck by a number of contrasts which the Reservoirs Act 1975 exposes. There is, for example, the contrast between the open reservoir with a dam subject to natural floods and the concrete service reservoir which may be little more than 25,000 cubic metres in capacity. There is the difference in the organisation of public water undertakers between England and Wales on the one hand and Scotland on the other. In England and Wales, the local authority is the enforcement authority for the water supply undertaking. In Scotland, the local authority is the undertaker itself for water supply reservoirs and in that role there is no enforcement authority. Above all, there is the contrast between the reservoir which poses a threat to life and property and the reservoir which, although a 'large raised reservoir' under the Act, could cause no serious damage if it failed.

There is also the difference between the position of the public and private undertakers. The major organisations: the Water Authorities and Companies in England and Wales, the Regional Councils in Scotland, the North of Scotland Hydro Electric Board, the CEGB and the like, have a body of expertise on their staffs and very substantial resources. Contrast these with the private undertaker, perhaps an angling association, who in many cases not only did not know of the 1930 Act but has only recently heard of the new Act. Some have even become owners only in recent times and may have purchased the reservoir for a comparatively small sum unaware of the very high financial liability they may have acquired. Even some District Councils are thought to be in this position.

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The paper was written too soon to allow a full account of the experience of the various bodies and individuals concerned. In the areas of the old metropolitan counties in England, the information for the central registers of the successor local authorities, the district councils, has only to be provided by 1 January 1987 and the full provisions of the Act do not operate until 1 April 1987, one year after the remaining areas of the country. It is likely to be some long time before all enforcement authorities have completed their Register and all the potential problems have come to light.

REGISTER OF LARGE RAISED RESERVOIRS (REGULATION 3)

General

The information which is required by Section 2 of the Act to be registered by local authorities (which, for our purposes, are defined as County, Regional, Islands and Metropolitan District Councils) is set out in Regulation 3 of the Reservoirs Act 1975 (Registers, Reports and Records) Regulations 1985. Undertakers were required by Section 24 to give the local authority basic information about the existence of their reservoir(s) but this does not relieve the local authority of the responsibility for including all reservoirs in their area in the Register.

Accordingly, the local authorities have had and will continue for some time to have a demanding task. Starting with very sparse information they have used every means at their disposal to compile a list of large raised reservoirs. Press publicity directed at undertakers produced negligible results but nevertheless by May 1986 local authorities had substantial numbers in their Register and significant numbers remained to be checked over the ensuing months. Sooner or later a desk study is necessary using ordnance survey sheets and most sites and certainly all suspect sites have to be visited, maybe several times. The scale of the task for enforcement authorities can be measured by the fact that at least one has over 100 private reservoirs which have to be investigated. For some of these detailed investigations of capacity may have to be undertaken before entry in the Register can be justified.

Those enforcement authorities with a lengthy Register may be expected to computerise the information, a fact which may not have been appreciated at the time of the legislation but which has since been recognised by the Departments concerned. It would be unfortunate, however, if each local authority worked out its own software. A common bought-in micro computer system seems likely to be the most economic and appropriate.

Several complaints have been received of the inadequate definition of the term 'reservoir'. The first words of the 1975 Act explain that it means a reservoir for water as such. The second sub-section of Section 1 says that the Act extends to any place where water is artificially retained to form or enlarge a lake or loch whether or not use is intended to be made of the water. Some enforcement authorities are puzzling as to whether a reservoir under the Act might be inadvertently constructed when a motorway or other embankment is built with inadequate culvert capacity to release upstream rainfall quickly enough.

Name and Address of Undertakers

Visits may be necessary to establish the undertaker or owner of the various reservoir sites. There can be difficulties. One extreme example which has come to light is where not only the ownership of the reservoir but also that of the dam is split between three parties. A reservoir in Scotland is formed by two dams each in different ownership. However, difficulties experienced by local authorities in this and other aspects of their task seem to vary and to depend on local topography and the type of use to which private reservoirs in the area are put.

Summary of Contents of all Certificates or Reports

By Section 24 of the Act, undertakers are required to submit any certificate given under the 1930 Act and the report (if any) of the last inspection. Copies of the certificates and reports made under the 1975 Act have also to be forwarded to the enforcement authority. But the information prescribed to be contained in the Register is only a summary of each certificate or report.

It is unfortunate that the Regulation did not specify a copy of the complete report. Enforcement authorities can scarcely be expected in present circumstances to provide resources for writing summaries of the many reports, even if it seemed sensible so to do. Not only would it have to be a competent technical person employed to write them but there would still remain the problem of which parts of the report should be omitted. It seems likely that summaries will therefore be dispensed with in favour of retention of a copy of the complete document but at least one enforcement authority was expressing anxiety as to whether it might at some time raise issues of confidentiality.

Category, Capacity and Surface Area of Reservoir; Height of Dam

It is curious that although this information is to be contained in the reservoir records, the enforcement authority is only required to include it if it is 'included in any certificate or report or otherwise known to the authority'. Although there should therefore be no difficulty on this account, the determination of the various factors does raise questions which are referred to later in connection with the statutory records.

REPORTS TO SECRETARY OF STATE (REGULATION 4)

The report under Section 3 of the Act from a local authority to the appropriate Secretary of State has no prescribed form and is only required every two years. The information to be given is limited to any steps which it has been required to take, either as an enforcement authority in respect of other reservoirs, or as an undertaker in respect of its own reservoirs, to ensure that the Act has been complied with. The report is therefore merely to give the Government an opportunity to check that the authority is carrying out its duties responsibly. It does not lead to what some regard as desirable, a national register of large raised reservoirs.

RECORDS (REGULATION 5)

Regulation 5 under Section 11 of the Act prescribes the information to be recorded and the forms which are to be used. The qualified engineer is to determine the 'manner' of recording the information but the layout of the forms does not permit much variation. He is also to determine the frequency of record which at last recognises officially that it is inappropriate to record daily water levels in a distribution service reservoir.

The combination of the wording of Section 11 and that of Regulation 5 might have been simpler. The 'matters' about which 'information' is to be provided are listed in Section 11(i)(a) and (b) of the Act and in Schedule 3 to Regulation 5. The 'information' to be provided about these 'matters' is mostly set out in Schedule 2.

There are a number of points to be made and questions to be raised about the statutory records under Regulation 5 and these are covered in the following paragraphs.

Water Levels and Depth (Part 1)

No inspecting engineer can lay down the frequency of records until the next inspection which may be up to ten years hence. It has been asked whether the weekly level records of the old Form F should be continued until that time. The answer would seem to lie in using the advice of the Supervising Engineer. It is after all the Supervising Engineer whose statutory duty it is to report to the enforcement authority if an undertaker is in breach of the provisions in this regard.

Persons having a function in relation to the Reservoir (Part 3)

The format implies, incorrectly, that inspecting engineers, as opposed to construction engineers, are appointed for a period. In fact, of course, they are appointed for one activity. The same engineer may be appointed for a subsequent statutory inspection but once his task is done, be it certificate or report, he has no further responsibility until the next appointment. It is the supervising engineer who has the continuing responsibility including that to advise the appointment of an inspecting engineer. The statutory record should not appear to take a view as to who this inspecting engineer will be.

Catchment and Rainfall Details (Part 6)

This part of the documentation should normally require completion once only. It is comparatively straightforward but many private owners will feel the need to have the figures produced by a professional adviser, for example perhaps the supervising engineer. Those private undertakers who seek to provide their own figures may find them challenged by a zealous enforcement authority at some stage.

Access, Capacity, etc. (Part 7)

(i) Capacity is the subject which perhaps causes the most difficulty. Fortunately the definition of top water level about which there has been debate in the past and which is now set out in Regulation 2 of the Reservoirs Act 1975 (Registers, Reports and Records) Regulations 1985, and in the preamble to Schedule 2 of that Regulation appears adequate to settle most conjecture. The definition, however, assumes that an overflow of some description exists on the reservoir concerned.

Whether reservoirs will come to light that could not be said to have purpose-made overflows remains to be seen.

The lack of a clear definition of 'reservoir' seems to result in a surprising number of borderline cases, or at least apparent borderline cases on first inspection by the enforcement authority. If a reservoir is clearly a large raised reservoir, enforcement authorities may tend to the view that accuracy in capacity determination is unimportant. However, accuracy in borderline cases is particularly important for the private undertaker for whom much expense may depend on whether his reservoir is covered by the provisions of the Act. In most cases, of course, the 1930 Act should also have prompted such considerations but many private reservoirs and in some parts of the country the majority of them, have not been inspected under the previous Act.

A number of problems arise when seeking to determine the capacity for the purposes of the Act. Probably the most important and the one most affecting the private undertaker whose reservoirs are likely to be open impounding reservoirs is what allowance to make for accumulated silt. This subject has been raised many times and requires guidance from the Departments to avoid variations in approach across the country. Reservoirs exist which are almost completely full of silt.

Several cases are appearing where the natural level of the land is difficult to determine. In any case, deposition and alteration over the years may make the natural ground level irrelevant. It has been said that the term 'escapable contents' would have been more easy of interpretation than the words inherited from the 1930 Act.

In the case of service reservoirs, there may be questions over the interpretation of 'any part of the land adjoining the reservoir'. If the structure is on a significant slope, is the land adjoining the reservoir confined to the periphery of the surrounding embankment? - or the wall of the reservoir? - or could it be a few metres away where the land is lower and the volume stored above that level all the greater?

A reservoir of capacity just below 25,000 cubic metres was found to have a facility for the insertion of stop logs. Another question arose over a cascade of reservoirs each too small to be covered by the Act but so close to each other as to be divided only by the intervening embankment. Many examples of potential disputes of this sort can be expected.

Such points may be regarded as trivial and easily determined by qualified engineers. Disputes on the subject with public undertakers are perhaps unlikely but in the case of some private undertakers there must be a temptation to make the intentions of the Act subservient to financial considerations. Already some are wondering whether, in cases of borderline capacity, it is worth undertaking the expense of a detailed survey in the hope of avoiding greater expense by proving the reservoir is not covered by the provisions of the Act.

(ii) Category of Reservoir - during the operative life of the Reservoirs (Safety Provisions) Act 1930 it was noticeable that whereas the provisions of the Act embraced service reservoirs down to 5 million gallons capacity above the adjacent ground, the documentation stemming from the Act and indeed most of the discussions, explanations and references to the Act related almost exclusively to dams in valleys and other major reservoirs which have the problem of natural floods. The roofed concrete box type of reservoir tended to be ignored. The same has happened with the 1975 Act and it is difficult not to repeat references to this fact when commenting on the new documentation. For example, in part 7, the choice of category on the form is between impounding and non-impounding. Why not the third category of service reservoir? It is of course necessary to distinguish between all three types in the same way that the qualified engineers' panels are distinguished.

What is an impounding reservoir? This question has been asked on various occasions but no satisfactory answer or at least none with the backing of statute has been forthcoming. The term is less than satisfactory as all reservoirs impound. That makes the term non-impounding reservoir even more inappropriate It is said that the lack of definitions either in the Act or as a result of case law, has caused no problems. I think it may now.

So far, there has been no one to challenge the determination by the undertaking or the inspecting engineer of the category of a reservoir but there are in existence reservoirs which have traditionally been inspected as Panel II reservoirs (the equivalent of the new NIR Panel) which could well have been classed as Panel I reservoirs. Enforcement authorities may, indeed should, require to be satisfied on this matter.

There may well be differences of opinion between the inspecting engineer, the enforcement authority and the owner, particularly of a private reservoir being recorded and inspected for the first time. It is however likely that the number of non-impounding reservoirs finally established will form a tiny proportion of the total, leading perhaps to questions as to why a non-impounding panel of inspecting engineers was thought necessary.

An attempt in the past at defining impounding reservoir was 'a reservoir formed by a dam across a valley'. This may require subjective interpretation for some of the less usual types of reservoir or ornamental lakes. Another loose definition which most inspecting engineers may have in their minds is a reservoir whose wall can be subjected to a surcharge due to natural floods. But any open reservoir is subject to this. It is all a matter of degree. A recent non-statutory definition of a non-impounding reservoir in an official document was 'a pumped storage reservoir'. The advertisement requesting applications for engineers to be placed on the non-impounding reservoir panel carried a note as follows 'a non-impounding reservoir means a reservoir not designed to abstract or impede the flow of a watercourse'. It would not be surprising if confusion occurs when it is apparent that the Departments are confused themselves. It is to be hoped that it is unusual in British legislation for a meaningless term to be used in an Act of Parliament and then defined outside the Act in a document which has no force of law.

(iii) Access - it is worth emphasising the importance of completing this information carefully and correctly. Adequate quick access for emergency equipment may help to avoid a disaster otherwise possible from a slowly failing dam.

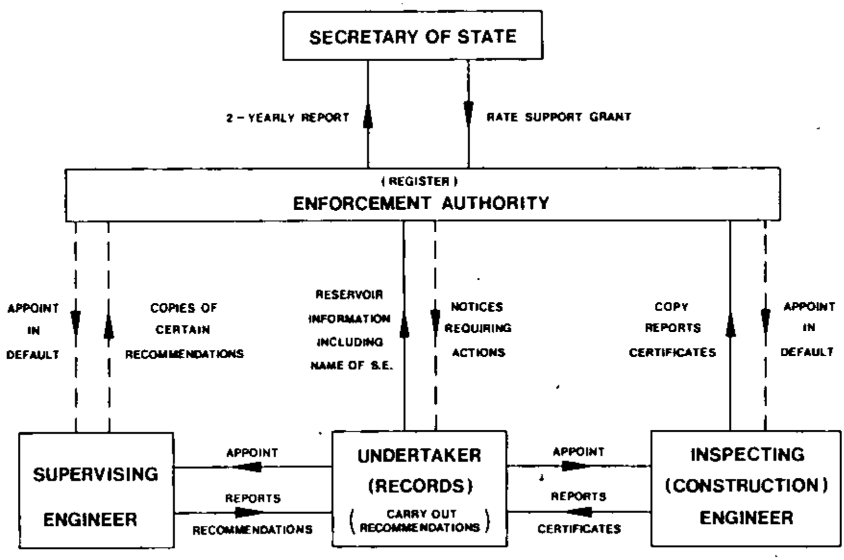
Dam, Reservoir Wall or Embankment (Part 8)

An amending Regulation altered the form of this part of Schedule 2 to the original Regulation by adding information on the levels of the walls above datum. The reason for wanting this is not clear but it will add unwelcome expense for the small undertakers of many reservoirs which have not such records available.

A high proportion of large raised reservoirs are constructed of reinforced concrete but this appears to have been overlooked. The possibility that a reservoir might be constructed of pre-stressed concrete should also have been made clear in the form.

This form also requires figures to be completed to represent the maximum rate of discharge of any draw-off works. There seems little point in seeking great accuracy in such figures as the maximum will only apply when the reservoir is full to TWL (or strictly speaking to maximum flood level). An estimate based on simple hydraulic calculations should suffice. It is common for flow tests to be impracticable at some reservoirs due to flooding of downstream waterways.

FIGURE 1 - Reservoirs Act (1975) - Relationship of the Various Parties



RELATIONSHIPS AND RESPONSIBILITIES

An indication of the external relationships between the various parties covered by the Act is given in Figure 1. For simplicity it excludes the case of a reservoir crossing a county or metropolitan district boundary and where the local authority is itself the undertaker much of the diagram does not apply strictly.

Enforcement authorities seem fairly unanimously to have delegated most of their obligations under the Act to their Chief Technical Officer as there is a continuing technical content in the work although less so once the Register has been completed. The experience in earth embankments of roads engineers can be particularly useful when giving initial consideration to the problem of earth dams.

In large public undertakings local operational management must control the reservoirs and the reservoir keepers, if any, and supply the information for the statutory records. The administrative arrangements for complying with the Act may well depend on the number of large raised reservoirs controlled. Where such reservoirs are numbered in single figures, the approach may be different from that of North West Water who own about 230 but the ultimate responsibility for the reservoirs must remain with line management at whatever level. At district level the engineer responsible should visit each reservoir every one to three months for a general check.

Reservoir keepers who were once to be found resident near most water authority impounding reservoirs and responsible for one or a group of them are less common today. This is a subject of concern to some who see the removal of our traditional first line of defence just when we are seeking to tighten up safety procedures

COMPUTERISATION OF RECORDS

The quantity of information now to be recorded and the need for updating make computerisation inevitable when an undertaker is dealing with anything more than a handful of large raised reservoirs. The statutory forms are published in books and are therefore unsuitable for computers. North West Water has developed a micro-computer system which is able to print out on loose-leaf sheets a full record for each reservoir in direct sequence or in individual pages as required. The system depends on a number of databases and has the facility to search, sort and list any parameter in the required order. The system is menu-driven and work has continued to set up graphics facilities. These will for example show trends in the levels at preset levelling points and leakage flows from up to ten points on each reservoir. Trends in the relationship between rainfall and leakage will also be graphed. The form of the loose-leaf sheets is exactly as the form in the statutory book with the exception that the sheets are turned through 90 degrees for practical reasons and the allocation of space on the sheet has been varied slightly. The writing of a user manual has also been necessary.

Although some of the prescribed records, particularly levels and depths, have to be initiated in the district or in one of the groups of the district it is appropriate for the division to hold the micro-computer which carries the master file. As there are three divisions, the Authority therefore carries all its reservoir records on three micros. Districts carry blank sheets which they fill in as required and dispatch to the division for computer input. They can receive complete print-outs in return. Print-outs suitable for the enforcement authority can also be produced.

As the Authority's new technology advances, the completion of sheets at District level may become largely unnecessary as it should be possible to pass figures direct to the computer by telemetry and the Authority's private regional communication system. A series of micro-computers is eventually envisaged dealing with these and other supply records and, using a networking capability, a total supply picture should be available at any one time.

DISCONTINUANCE AND ABANDONMENT

Sections 13 and 14 of the Act covering the above subjects respectively are two of the more confusing Sections. Neither of the terms is defined but by implication 'discontinuance' refers to the reduction of storage capability below 25,000 cubic metres so that it remains a reservoir but ceases to be 'large raised' and 'abandonment' refers to the reduction of possible storage to nil so that it ceases to be a reservoir at all. As in all matters relating to this legislation, any reference to capacity relates of course to capacity 'above the natural level of the land adjoining the reservoir' and water held or capable of being held below that level is not for consideration.

Both discontinued and abandoned reservoirs must remain on the local authority's register of large raised reservoirs unless and until a certificate under Section 13 (dicontinuance) has been provided by a qualified engineer to the effect that the alteration reducing the capacity has been 'efficiently executed'. Section 14 (abandonment) requires a certificate concerning measures to be taken 'in the interests of safety'. If a reservoir has been abandoned under Section 14, even if its capacity is reduced to zero it must still remain on the register unless it has a certificate under Section 13. One can speculate as to whether it is the intention in the case of abandonment that two certificates should be provided.

Complications may be expected when abandoning one of a cascade of reservoirs. An example is already known of a proposal to abandon a reservoir bringing into question the effect on other reservoirs downstream, at least one of which is in different ownership. In such circumstances, abandonment may need to involve more than one qualified engineer and may produce objections by the downstream owner to upstream proposals. There appears no provision in the Act for this contingency. Supervising Engineers who should make themselves aware on each visit of changes in the catchment would be expected to keep in mind the effect of changes to other reservoirs.

It is worth remembering that an abandoned reservoir may become valuable if a licence to tip is obtainable and the possibility of costs on the one hand and profits on the other may well lead to a good deal of acrimony and possible litigation from which it may be difficult for the qualified engineer to detach himself. It will not have gone unnoticed that the environmental lobby which is always so strongly opposed to the construction of new open reservoirs is just as strongly opposed to their removal or even reduction in size.

MEASURES IN THE INTERESTS OF SAFETY

It seems there may be developing a need for a definition of the above terms. The phrase is used a number of times in the Act as it was of course in the previous Act. In some Sections it is made clear that it is referring to the safety of the dam. It is reasonable to suppose, however, that the Act is not concerned directly with structural safety but only with safety of life and property and where structural safety is not specified it is downstream safety to which the Section refers. A private reservoir undertaker, or for that matter any undertaker, is entitled to ask why an inspecting engineer should require expensive repairs to a reservoir if it is in an area where he feels failure could cause no damage to life or property. He may ask whether in such a situation even Category D in the Engineering Guide on Floods and Reservoir Safety is appropriate. It may be that inspecting engineers themselves vary significantly in their approach to the meaning of 'measures in the interests of safety' on this type of reservoir. If so, we may expect more recourse than hitherto to the provisions for referring the matter to a referee.

The measures in the interests of safety seem throughout Section 14 (Abandonment) to refer solely to the possibility of alterations at the reservoir. No mention is made of the watercourse downstream which may be inadequate for the potentially increased flows when the reservoir is abandoned.

EMERGENCIES AND EMERGENCY POWERS

It is not necessarily sufficient for an undertaker to have a qualified engineer's certificate. That certificate may well list works which should have, but have not, been carried out. Even with a certificate therefore an emergency may exist.

Before work on unsafe reservoirs is undertaken, Section 16 of the Act requires the enforcement authority to appoint a qualified engineer to make recommendations. The authority has no right of appeal against recommendations which its own appointee may make and for which it may have to bear the total cost. It has been pointed out that this is iniquitous when undertakers themselves have such rights against decisions of inspecting engineers.

The view has been expressed that a national procedure, or at least national guidelines for local procedures, is needed to provide for action in emergencies such as a partial or total failure of a dam. At the Greenbooth Dam for instance, where in 1982 a depression in the crest was noticed by a member of the public, rapid draw-down was immediately initiated but there was nonetheless incipient danger. Co-operation between the water authority, the local authority and the police was slow and unsatisfactory, largely due to lack of comprehension. It seems advisable that some contingency plan be prepared for any impounding reservoir having a substantial downstream risk. In the USA inundation maps are prepared to show areas at risk from failure but this can spread undue alarm. A simple procedure, however, agreed between the water authority, the county emergency officer and the appropriate police force should be prepared and kept under review by each of the three bodies.

SMALLER RESERVOIRS

Unfortunately the reservoirs safety legislation missed an opportunity by not seeking to differentiate between reservoirs as to their capacity to endanger human life on failure. Risks downstream are not proportional to the size of the reservoir. The choice of five million gallons in the 1930 Act was meaningless in itself and the 25,000 cubic metres of the recent Act is merely the same figure metricated. There are many reservoirs falling into the statutory category which, because of local geography, could form no threat. There are others below that size which may be a century or more old and could, with a major leak, cause a certain amount of havoc. Such reservoirs often lie on steep urban hillsides.

Undertakers controlling such reservoirs should make up for the perceived deficiency in the legislation by categorising all reservoirs by the potential downstream danger. In any case, all reservoirs smaller than 25,000 cubic metres, most of which will be covered, should be emptied if possible and given a structural inspection every few years. Undertakers should have an inspection cycle laid down, something which has often been lacking previously. Such inspections have the added merit that they allow the cleaning out of accumulated silt, the cause of some of the ubiquitous dirty water problems which water authorities experience.

SAFETY PROCEDURES FOR INDIVIDUALS

Although safety consciousness continues to increase, there has been a sad series of disasters in recent years causing deaths or injuries by asphyxiation or explosive gases. No such accident is known yet to have overtaken an inspecting engineer. Yet it is possible that they are less well trained than many others to deal with hazards in the confined spaces where they must be required to enter from time to time. It is not just the major and well publicised disasters such as at Abbeystead, Carsington and others that we need to consider. There have been many less well known incidents and many near misses.

As an example, a service reservoir when emptied recently for an inspection, was discovered before entry to contain about 7 per cent. of methane. The presence of the methane may be as much of a surprise to other engineers as it was to the staff of North West Water and inspecting engineers generally should take note. Had it not been for a procedure introduced in recent times, deaths may have resulted.

All confined spaces should be classified by the undertaker and signposted accordingly. It is perhaps unlikely that any reservoir would involve a confined space so dangerous as to require a permit to work but inspecting engineers and supervising engineers must be familiar with procedures where methane, carbon dioxide or general oxygen deficiency may occur and must ensure that in appropriate circumstances they are accompanied by properly trained assistants.

CERTIFICATES AND REPORTS

The Regulations prescribing the form of certificates and reports required to be submitted by qualified engineers under the various Sections of the Act were received just before this paper was submitted. There are many more forms than were required under the 1930 Act. This follows from the wider powers under the later Act. No difficulty will however be experienced by qualified engineers in complying with the Regulations.

Also set out in the Regulations is the information to be given under Section 21 by an undertaker to the enforcement authority when proposing to construct, enlarge or re-use a large raised reservoir. Once again the wording relates to dams and to impounding or non-impounding reservoirs but not to service reservoirs.

Copies of all certificates submitted to undertakers are also to be sent to the enforcement authority (Section 20). This is true of virtually all other submissions to the undertakers by qualified engineers. It would no doubt be in the spirit of the legislation for undertakers to inform the enforcement authority on every relevant matter. Certainly they are required by the Act to inform them of the appointment of a qualified engineer for whatever purpose (Section 21). This takes the place of the rather ineffectual requirements of the 1930 Act to publish such facts in local newspapers.

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OPERATIONAL MANAGEMENT - OPPORTUNITIES AND CONSTRAINTS

R GRAHAM SHARP*

The paper discusses the increasing trend for water resource reservoirs being linked with other sources as part of an integrated system. Frequently there are subsidiary uses and objectives in addition to the primary rôle of water resource enhancement, allowing greater benefits to be obtained. At the same time integrated use gives rise to greater complexity in operation and management. To set against such opportunitites there are constraints. Examples are given of some benefits of co-ordinated management and of restraining factors to be taken into account, with reference to the impact on design and operation.

1. INTRODUCTION

1.1 This paper considers some factors relating to the operational management of reservoirs for which water resource augmentation and water supply are the primary functions. This seems a reasonable limitation bearing in mind that this is a joint conference of IWES with BNCOLD.

1.2 The last two or three decades have seen big changes in the way in which water resource and supply reservoirs are managed and deployed, affecting their every day operation. Up to the mid 1960's the great majority of such reservoirs in Britain were operated as single purpose sources for direct water supply. Unless part of a directly associated group, they were usually operated as single independent sources. Above all they were not seen as contributing to the water resources of the whole river catchment in which they were situated.

1.3 With single source, single function operation, output would vary little from day to day; such variations as did arise were on account of fluctuations in demand for the water. There was little perceived need to optimise operations for other objectives as part of a wider catchment based resource system.

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1.4 Several factors have combined to provide both the need and the opportunity for more complex non-uniform operation to optimise usage in an inter-linked source system. Operation in this manner calls for the development of rule curves and other predetermined guidelines related to storage state, time of year and flow conditions. It enables several more or less conflicting objectives to be satisfied, whilst giving priority to the primary functions of each source.

1.5 The increased opportunities for multi-functional and multi-objective operation have been provided by a combination of organisational and technological developments. Water resources and supply are being increasingly planned and operated as part of larger water service systems over wider areas to obtain the greatest benefit from both existing and future works. The application of computing techniques has made possible the analysis and development of pre-determined operating rules. Moreover the introduction of ICA has made feasible the application and monitoring of variable operation.

1.6 Notwithstanding these opportunities for fuller use of reservoirs there remain many constraints on the way they are operated both to ensure efficient performance in meeting the operational objectives and to ensure safety. These constraints can assume greater significance with more ambitious modes of deployment.

1.7 Looking to the future it is hard to foresee other than a continuation of the trend towards complex multi-objective operation as part of an integrated catchment based system. It seems appropriate, therefore to review and take account of these aspects when considering the management of reservoirs.

2. DEVELOPMENT AND USE OF OPERATING CONTROL CURVES

2.1 Operating control curves are a necessity for good management of reservoirs whether viewed as a single direct supply source or as part of a multi-purpose integrated resource system.

2.2 The traditional way of defining the output capacity of a reservoir source is as a single specified daily amount usually described as the reliable yield. This is based on the amount that can just be sustained in a design drought year by the combination of inflow and usable storage over a critical dry weather period (the design drought) after allowing for compensation water releases. Such a definition is convenient, particularly at the development and promotion stage of a new source and in giving a reasonable indication of its size for comparative purposes. However, the derivation of the yield in this way assumes that all usable storage has been drawn on by the date of greatest depletion when inflow once again exceeds draw-off. This would only be possible as a mode of operation if we **knew** in the course of a drought that it would not exceed the design severity (i.e., intensity and duration) for which the ascribed yield applies. In practical terms this means knowing in advance how a drought is going to develop and exactly when it is going to end - knowledge so far denied to mere mortal water engineers to say nothing of their meteorological and hydrological colleagues and mentors!

2.3 ~~This uncertainty~~ about a drought's eventual severity is a very big operational constraint and as such is referred to again later in Section 5 of this paper. Operational staff responsible for maintaining supplies have an instinctive dislike of seeing reservoir storage depleting below about one third full and still reducing. Whilst no-one yet can make a cast-iron prediction of the future of a drought (operations staff and management equally dislike talking in terms of probabilities) it is possible to ease the position to a great extent by introducing pre-determined rule-curves to cut back the output steadily as storage reduces. In this way, should the drought continue, an equilibrium will be approached between inflow and output so that "empty" remains in the future and some useable water remains in store. Such rule curves have been derived and tested by simulating up to some 50 years of hydrological conditions and adjusting the fit until the best balance between risk of failure and output is obtained. They are synonymous in motoring with the sensible application of brakes and gear change approaching a stop sign.

3. USE OF RESERVOIRS IN AN INTEGRATED RESOURCE SYSTEM

3.1 The use and operation of a reservoir as part of a wider multi-source, multi-demand system is undoubtedly the biggest factor to enhance the complexity of the operating regime compared with that applying for a single separate source. In the first place instead of the need to balance the inflow variations against a sensibly constant output we get a situation where the output requirement varies as much as or more than the input, and moreover in opposing directions, i.e. the output is required to increase as inflow reduces during a drought.

Reservoirs for river regulation

3.2 The situation described above applies when the water stored in a reservoir is used wholly or partially for river "regulation" or augmentation of dry weather flows. The stored run-off from the reservoired catchment is used to supplement the natural dry weather run-off from the usually much larger catchment to the downstream control point.

3.3 A good case in point is Clywedog Reservoir on a headwaters tributary of the river Severn - see Fig. 1. This reservoir's primary use is to augment dry weather flows in the river Severn to support several abstractions down the course of the river and still leave an adequate residual flow at the tidal limit.

3.4 We thus have a form of "conjunctive use" between two sources; (i) Clywedog Reservoir and (ii) the river Severn for the remainder of its catchment apart from that impounded at Clywedog and Vyrnwy reservoirs. The river is able to support abstractions except in drought conditions for which the water stored in Clywedog is held in reserve. The latter at such times becomes a vital component. Indeed, the river would not be a "reliable" source without it.

3.5 The river Severn resource system with Clywedog and a small contribution from releases at Vyrnwy, have recently acquired a further element, development of groundwater storage in Shropshire the function of which is similar to that of the regulating reservoir. This further element serves to illustrate a multi-source system involving groundwater used intermittently in conjunction with surface water storage to support river flows and abstractions.

3.6 The incorporation of the Shropshire groundwater element into the resource system has necessitated the formulation of control rules for the use of Clywedog which take account of the differing characteristics of the two storage sources. In particular, the groundwater source has very large storage but is limited in its rate of output by the combination of the aggregate yield of the abstraction boreholes, licence conditions and pump capacity. The reservoir, on the other hand, has by comparison, more limited storage but is capable of large and rapid variations of output by gravity releases to keep step with changing requirements for flow augmentation. The groundwater source has attributes comparable to the stamina but one pace of the long-distance runner, whereas Clywedog has the speed and acceleration of the sprinter.

3.7 To get the most out of these contrasting characteristics, it was necessary to devise operating rules that allow the groundwater scheme to contribute its share of augmentation sufficiently early in a potentially lengthy drought. On the other hand, premature use of the groundwater would involve higher pumping costs and could increase abstraction to an unnecessarily high proportion of the natural recharge.

3.8 These and related complexities of operating a river system with several abstractions and discharges has led the managing water authority to devise comprehensive operating rules and procedures (1). The operating control curves for Clywedog are shown in Fig. 2. These indicate when Shropshire groundwater pumping should commence to supplement regulating releases and when Drought Orders should be sought in extreme conditions. At the "wet" end of the hydrological scale the curves indicate when water can be released for hydro-power and to provide flood retention capacity.

3.9 Fig. 3 compares drawdown in Clywedog over a sequence of years as between actual deployment for river regulation and what would have occurred with direct supply. The same reservoir storage on the impounded catchment when used as a regulating reservoir supports abstractions from the river totalling some three times that obtainable by direct draw-off from the impounded catchment alone.

3.10 Clywedog reservoir and its rôle in the Severn resource system has served to illustrate in the preceding paragraphs the use of reservoirs to augment river flows. But reservoirs can form part of an interlinked source system without this feature; or where it is only a secondary use.

Direct Supply Reservoirs in an Integrated System

3.11 The Derwent Valley reservoir in the headwaters of the Derbyshire Derwent (Fig. 4) are a group of three reservoirs in series which continue to be operated as direct supply sources. However, they are used in combination with other higher marginal cost sources for meeting demands for water over a wide area of South Yorkshire (Sheffield), Derbyshire, Nottinghamshire and Leicestershire. It has been found possible to operate them with variable rates of draw off so as to optimise the total system in respect of output and running costs. Essentially the operational objective is to allow these reservoirs which provide relatively low cost gravity supplies of good quality water to be overdrawn for some 85% of the time. This is possible in the knowledge that other higher cost sources can be brought into fuller use as a drought develops having been under-utilised during the preceding non-drought period. At the same time, output from the Derwent Valley source has to be cut back as storage reduces through the drought period. Thus the main objective is to minimise running costs by fullest use of low marginal cost sources at times of plenty. On the other hand maintaining overall output must take precedence over cost during the more limited periods critical for resources yield.

3.12 It will be evident that (i) this style of operation is only feasible when sources are operated as a system; (ii) it is necessary to have a margin of capacity to attain the required flexibility and (iii) pre-determined operating rules are essential to achieve the non-uniform output ranging from over-drawing to occasionally significantly under-drawing a storage source in relation to its nominal yield. An example of the rule curves currently applied to Derwent Valley storage is given in Fig. 5. These have been established, refined and tested by iterative simulation of their performance through some 50 years of historic hydrological data.

Pump filled Reservoirs

3.13 Reservoirs mainly or partly filled by pumping introduce several other factors if they are to be operated efficiently. Rule curves are needed to make the best use of available electricity tariffs and also to take account of quantities available for abstraction from river flows. This is likely to involve a preponderance of night time pumping; as much summer time pumping as possible to avoid or reduce expensive winter tariff power costs, and careful planning to avoid maximum demand charges.

3.14 Efficient operation with both significant gravity and pumped filling necessitates shrewd assessments of the extent that the variable gravity component can be relied on. Scheduling of the pumped component must allow the cheap gravity component to provide its full share and only to commence heavy pumping when it becomes increasingly evident that support from the pumped component is needed. Rule curves and pump scheduling can go far in providing the best solution, based on simulation of past records of hydrological data combined with assumptions of future use patterns.

3.15 The real pay-off with pump-filled reservoirs comes when they can be used in association with cheaper gravity sources, as referred to in paragraph 3.11 above. This mode of operation is used for the pump-filled reservoirs on the lower Dove in Derbyshire which supply Leicester. In so doing they supplement the gravity supplies from Derwent Valley and water pumped from the river Derwent (Fig. 4). It is possible in most years, to reduce the output of the pump-filled reservoirs by some 10-15% by integrated operation with overdrawn gravity supplies at Derwent Valley. This brings about an average annual saving in pumping costs of over £100,000.

Non Uniform Compensation water releases

3.16 Variable compensation requirements can save unnecessary releases at flows and time of year when they are of no benefit. The water thus saved can be "banked" for subsequent release to give a measure of augmentation for low flows. This approach has been applied successfully on a permanent basis for releases from Lake Vyrnwy and on a time limited basis from the Derwent Valley reservoirs.

4. SECONDARY OBJECTIVES, USES AND BENEFITS

4.1 Increasingly over the past 20 years it has been recognised that reservoirs built primarily for water resources augmentation and supply can be operated to meet other, albeit secondary, requirements. In this way they can meet more than one objective and provide additional benefits. Secondary uses have been developed for older reservoirs as well as newer ones planned and designed from the outset with these in mind.

4.2 Secondary uses and benefits of water resource and supply reservoirs include:-

- recreation both on and around the reservoir, and by means of water released to the associated river for fishing and other water based sports such as canoeing.
- hydro power generation;
- flood retention storage.
- provision for and conservation of the natural environment.

Recreational Uses

4.3 Water Authorities in England and Wales have been required under the Water Act 1973 to consider and make appropriate provision for water related recreational activities and amenities. The Scottish Regional authorities have similarly turned their attention to this aspect.

4.4 Recreational uses may be broadly divided between "active" and "passive", sailing being a common example of the former; bird-watching and picnicking examples of the latter. However the active pursuits have had to be further sub-divided to separate and possibly exclude the more intrusive such as power boating which would otherwise "queer the pitch" for almost all other activities.

4.5 The most important factor from the management point of view is whether any special constraints need to be built into the operating rules for recreational and amenity reasons. So far as the author is aware, it has been generally recognised that reservoir water levels are a function of the combination of in-flow and requirement for discharge and there has been little specific provision for maintaining water levels for recreational and amenity purposes. One exception has been the provision of small amenity dams towards the upstream end of a reservoir to maintain higher levels *there during draw-down of the main volume of storage. It is evident* that in pump-filled reservoirs particularly there can be a conflict of interest which will have to be resolved by guidelines within the operating authority. From a water resource management point of view it may be politic to keep the water level below full for long periods (to reduce pumping head and to save pumping water which may subsequently overflow), but this may run against recreational requirements where margins below top water level are usually disliked.

4.6 Recreational - particularly fishing - considerations are likely to be a constraint both on the rate of release and on the quality/temperature of the released water because of the effect on downstream conditions. This calls for sensitivity in the means of controlling the released water. This in turn calls for careful attention to the valve and draw-off design, including especially the facility for multi-level draw-offs in deep reservoirs.

Hydro power generation

4.7 Hydro power generation, even on a small scale, has been neglected as a by-product or secondary purpose on water resources reservoirs. Recent changes of legislation may, be an encouragement, as intended, to small scale electricity generation but it has to be accepted that the overall economic and operational constraints for both the water authorities as suppliers and electricity boards as receivers of this small-scale power generation are unlikely to make many schemes attractive because of the limited scale of generation that is feasible.

4.8 Clywedog reservoir may again be cited as an example of generation of small scale hydro power as a secondary use. In this case there are two generating sets with an output of 80 KW and 500 KW respectively. The 80 KW set, sufficient for providing the power at the dam site runs all the time from the minimum compensation release of 18 Ml/d. The 500 KW set operates only at those times when either releases of at least that quantity are required or otherwise when such releases can be made in accordance with the rule curves shown in Fig. 3.

Flood Retention Storage

4.9 Apart from the incidental flood retention facility afforded by the well known "lag effect" on all reservoirs due to increased storage associated with higher water levels over the spillway in a flood, few water reservoirs have specific provision for flood retention capacity below the spillway cill level.

4.10 There are clearly circumstances in which such provision would be beneficial. This is especially so where a large upland catchment is impounded and where downstream flooding would be alleviated more cost effectively by flood retention than by improved flood defences.

4.11 Clywedog again offers an example of flood alleviation by retention storage. The Order (2) authorising the scheme contained a requirement to provide flood retention capacity. Retention capacity to stated levels below TWL has to be provided between 1 November and 30 April each winter. This requirement has little impact on the regulating capacity of the reservoir; the only adverse effect could be the possibility, given a dry spring preceding a dry summer, of *starting the regulating season with partial drawdown should in-flows* be insufficient to refill after the end of the flood draw-down period. In practice this has been found to be a rare circumstance.

Conservation of the Natural Environment

4.12 Provision for and conservation of the natural environment in relation to reservoirs and catchment management may be seen as a positive use and benefit or as a constraint according to one's point of view. Either way it is a firm duty placed on Water Authorities in England and Wales (3) "to further the conservation and enhancement of natural beauty and the conservation of flora, fauna and geological or physiographical features of special interest."

4.13 Increasingly Water Authorities see this as a positive rôle rather than an irksome constraint but it has not always been so. Many will recall the several well publicised cases in the 1960's and 1970's where proposals for water schemes were thwarted or radically altered on account of conservation aspects; raising natural lake levels at Ullswater, wild orchids at Cow Green, daffodils at Farndale, and disturbance of a natural wilderness at Swincombe on Dartmoor.

4.14 In recent years because the emphasis has moved from new development to a changed management style of existing works it is now much easier to accommodate nature conservation requirements. Most Authorities make generous provision for meeting these obligations and if they fail it is more from lack of appreciation of particular local conditions than from wanton disregard. Indeed the lessons of 20 ago have been taken so much to heart that there are few instances where specific provision is not made for the welfare of fish and bird life on, around and downstream of reservoirs or where landscaping has not been carried out to blend works (with varying success) into the surrounding scene.

5. CONSTRAINTS ON OPERATION

5.1 The preceding sections have reviewed the opportunities that may exist for operating reservoirs to gain greater and additional benefits. In this section Natural, Design and Operational constraints are considered.

Uncertainty of future weather conditions

5.2 The very limited ability and uncertainty in forecasting future weather is arguably the biggest single constraint on the operation of reservoirs. The operating margins deemed necessary as a consequence of this uncertainty are much larger than the implied limits of accuracy of hydrological calculations based on probability analysis. Operating rule curves are seen as much the most effective way to counter the uncertainty factor. They go far to bridge the gap between theory and practice in the application of probability of natural events.

5.3 Uncertainty has its impact at both ends of the hydrological spectrum. It is the very uncertainty of extreme natural events that dictate the sizing of the maximum flood and the works needed to accommodate it as well as the sizing and operation of storage required to cope with droughts.

Impact on downstream river regime

5.4 The effect on conditions in the river downstream of a reservoir are likely to be a substantial constraint on operational practice, particularly for reservoirs used wholly or partly for river augmentation. The effects can be broadly classified as "quantity" and "quality".

5.5 The river channel downstream is likely to require to be modified to accept large scale releases from impoundment without causing excessive erosion or flooding. The extent of this would be the subject of survey and physical modelling work at the design stage.

5.6 Apart from the capacity to accept maximum releases there may well have to be a constraint on the rate of change of releases both as a safety precaution and to avoid excessive disturbance to the river channel. In any case rapid increases in releases should be unnecessary if the developing weather situation has been properly monitored. A sudden onset of wet weather may call for a speedy reduction in releases but this will seldom cause problems.

5.7 From a design point of view, the need is for flexibility and a wide range of readily controllable draw-off capacity.

5.8 Quality constraints from the downstream regime are primarily concerned with avoiding large temperature contrasts where there are fisheries and fishing, and avoiding or at least minimising releases of bottom water low in dissolved oxygen and high in suspended solids, particularly during the period April - October. Bottom water may be used for winter releases to create flood storage retention. In this way the effect on the downstream river conditions is minimised.

5.9 Rapid Draw-down associated with river regulation and conjunctive use

The higher rates of draw-down associated with variable operation are unlikely to exceed those allowed for in design for rapid emergency emptying. It is nevertheless a constraint to be taken into account should the dam design be critical to frequent rapid draw-down conditions. This factor may be most significant where older earth dams are associated with reservoirs subject to widely variable rates of draw-down.

5.10 Relative draw-down of grouped storage

Grouped reservoirs, in series and/or in parallel have to be operated so as to keep the draw-down of the various elements in balance. This has to take into account not only the relative locations but the differing refill characteristics, the presence of diversions, any pumped filling and the location of draw-offs. As a general rule it is preferable to retain water in the higher reservoirs of a series and in those elements with poorer refill characteristics. Pre-determined guidelines for target proportions of total storage are needed in complex situations.

5.11 Constraints on Pump-filled Reservoirs

As referred to in Section 3, pumped filling must take full account of electricity tariffs. This means diurnal variations in the pattern of abstraction. The installed pumping capacity will have to be assessed on the economics of capital and pumping costs but typically may have to be sufficient to pump the daily quantity in 8 - 10 hours. The diurnal variation involved may be limited by the downstream flow conditions and the effect of such variations on other abstractors.

5.12 Constraints arising from Conflicting Objectives

It will be evident that in meeting more than one objective itself gives rise to constraints in operation. For example if recreational requirements are held to justify a limit on draw-down either in absolute terms or in rate of change, this will inevitably affect the performance for water resource augmentation. It is suggested that relative priorities and objectives should be capable of being changed over the years as circumstances and requirements alter. As an example, recreational requirements could be more fully accommodated for a time when a new source provides spare resource capacity.

5.13 Constraints arising from maintenance

The need for maintenance work on the upstream side of dams, including on the draw-off and over-flow works poses particular but temporary operational problems and may necessitate partial or complete emptying of the reservoir. Experience suggests that the best course is to wait until late July or August before allowing additional draw-down. By that time advantage can be taken of operational draw-down and the requirements for the remainder of the season can be assessed. By careful pre-planning and staging the work should be carried out to allow time for adequate refill by the following early summer. Incidentally, integrated management can help to overcome maintenance problems by judicious switching of the demand load.

6. SUMMARY AND CONCLUSIONS

6.1 This paper has sought to review current practice and trends in operational management of reservoirs primarily used for water resources and supply.

6.2 The theme is one of multi-objective operation as part of an integrated resource system based on total river catchment management.

6.3 To achieve the potential benefits - technical efficiency, economies of operation and environmental enhancement - calls for sophisticated variable operation. This is likely to be in association with other differing sources and reconciling the respective, sometimes conflicting requirements, of the various objectives and benefits sought.

6.4 The high degree of flexibility involved with such variable operation has to be appreciated and provided for at the design stage.

6.5 Flexible operation in the way described needs pre-determined operating rules derived from computerised analysis and simulation. Also automated monitoring and control is moving from the "desirable" to the well nigh essential.

6.6 A full appreciation of operational management considerations and requirements is not only essential in the planning and design of new schemes - something now comparatively rare in Britain - but in the modification and adaption of existing reservoirs.

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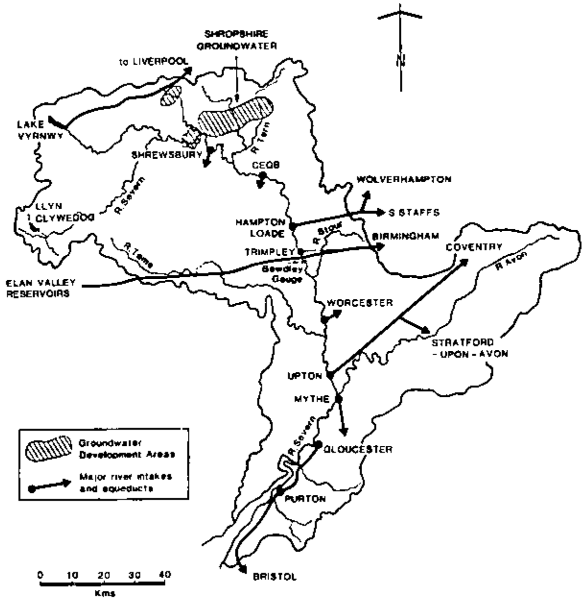
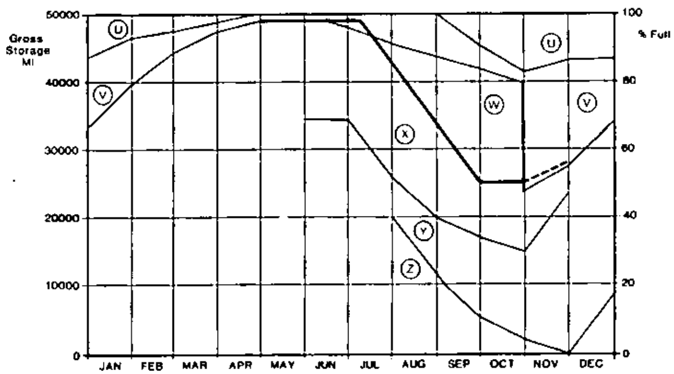


Fig. 1 River Severn Water Resource System



U	Discharge for flood retention, provided downstream river levels are low.
V	Discharge for hydro-electric generation purposes.
U,V	Discharge for river regulation (when required) with all water taken from Llyn Clwedog.
W	Discharge for river regulation (when required) with all water taken from Llyn Clywedog and Lake Vyrnwy.
X,Y,Z	Discharge for river regulation (when required) with water from Llyn Clywedog, Lake Vyrnwy and Shropshire Groundwater.
Y	Seek Drought Order.
Z	Drought Order powers in force (these may extend throughout the next refill season too).

Fig. 2 Clywedog control curves

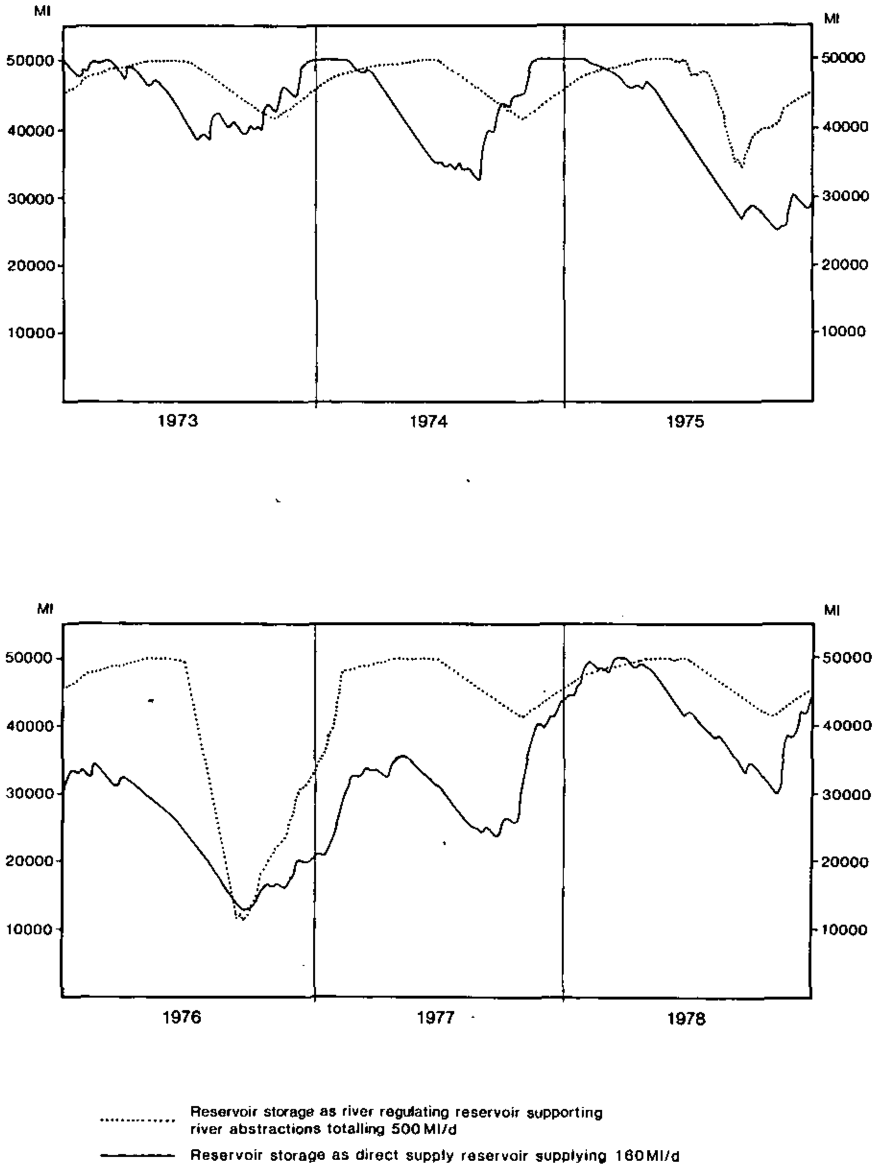


Fig. 3 Comparison of direct supply and river regulation use of Clywedog reservoir

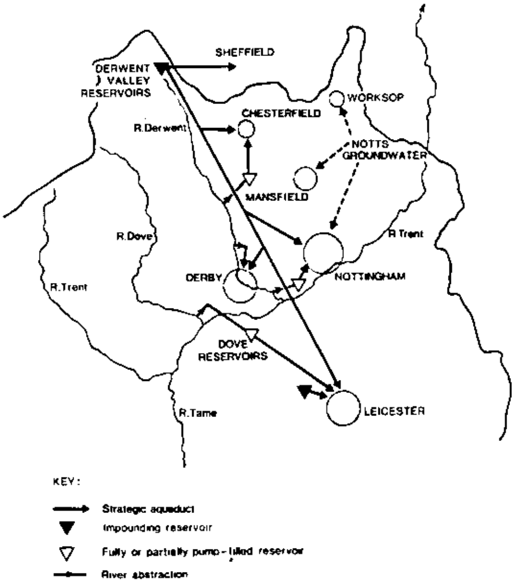
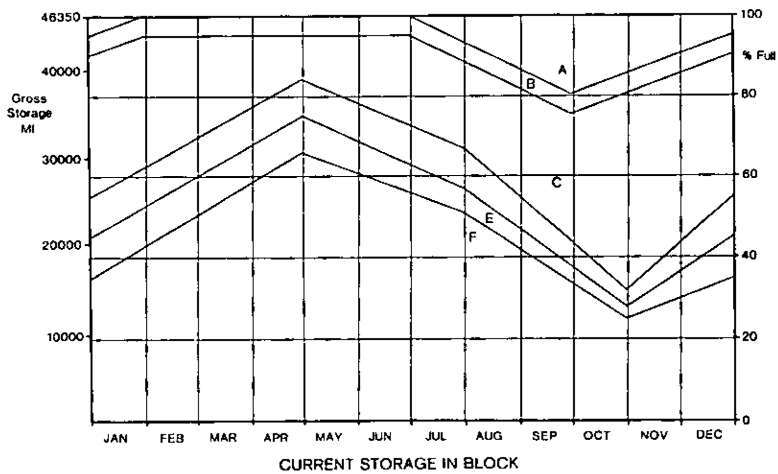


Fig. 4 East Midlands Water Resources System



A	Discharge for flood retention.
A,B	Supply at maximum rate (up to 254 Ml/d).
C	Supply 225 Ml/d.
D	Supply 220 Ml/d ; seek Drought Order.
C,D	Discharge 75 Ml/d compensation water.
E	Supply 175 Ml/d.
F	Supply 150 Ml/d.
E,F	Drought Order powers in force ; discharge 45 Ml/d compensation water.

Fig. 5 Derwent Valley control curves

MISCONCEPTIONS IN THE DESIGN OF DAMS

A D M Penman*

The concept that all dams must contain a waterproof element supported by a stable structure is often misunderstood. Common misconceptions include use of low placement water content to reduce pore pressures in cores and use of computerised stability analyses which ignore progressive failure. There is no means of calculating a factor of safety against internal erosion and piping failure.

Introduction

Design methods have continued to advance as part of engineering progress. Conceptions accepted in one age may be considered unacceptable in another. Advocates of the most modern ideas always feel that they are closer to an absolute truth than their predecessors. At any stage, earlier concepts may be scorned as misconceptions. At any one time, however, there are invariably a variety of conceptions in current use and some will be considered as misconceptions by those who hold other views.

The design principles published only sixty years ago by Wegmann (1927), presumably representing conceptions accepted at the time, would now be dismissed by all designers as misconceptions. In discussing drainage in gravity dams he wrote:

"In Vyrnwy dam in North Wales, a system of drains was constructed in the foundation to relieve the base of the dam from upward pressure that would occur if water leaked under the dam.

It is doubtful whether a system of drainage placed in the dam or its foundations is advantageous, except in special cases. By giving free outlet to the seepage water such a system encourages leakage. Most engineers prefer to construct dams as tight as possible so as to offer the greatest resistance to seepage. Water that passes through a thick masonry dam and only shows itself as a moist spot on the downstream side can scarcely be considered to cause an upward pressure in the masonry. On the contrary, in passing through the pores of the stones and mortar it increases the weight of the masonry and consequently the stability of the dam".

This quotation shows lack of appreciation of the simple concept that all dams must contain two basic features: a waterproof element and a structure that

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will support it safely.

In the past, homogeneous sections of both earthfill and masonry have been expected to perform both functions. Prior to the catastrophic failure of Bouzey dam (20m) on 27 April 1895, (ICOLD 1974), gravity sections had been designed by taking account only of reservoir water pressure on the upstream face and the weight of the dam, limiting compressive stress to about 600kN/m². Improved cements enabled allowable stress to be increased to 1,000kN/m² but the resulting reduced thickness of section was enough to produce instability under the previously discounted uplift pressure. Maurice Lévy (1895) proposed that design should ensure that the vertical compressive stress in the masonry at the upstream face should exceed the pressure from the reservoir water at all levels to prevent ingress of water into the body of the masonry. While this approach would avoid the catastrophy of overturning under uplift pressure, it would not prevent percolation of water through the dam, with risk of damage by acid water, staining and frost damage at the downstream face. A solution was to provide an "impervious" membrane with drainage behind it.

As a remedial measure, percolation through Ringedals dam (33m) in Norway was stopped when it was about 10 years old (Gröner 1933) by building a completely separate reinforced concrete slab as the waterproof element, supported from the original structure of the dam. The slab was made 0.47m thick at the base, reducing to 0.2m at the top and was held 2m away from the upstream face of the original dam (Fig 1) with struts at 2.25m spacing horizontally. Vertical spacing varied from 1.8m up to 3m and the 2m gap was drained at its base.

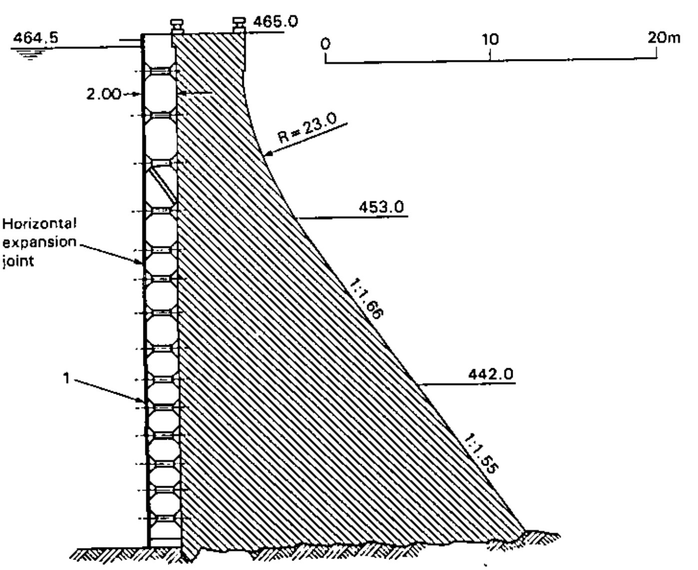

Figure 1 Ringedals dam

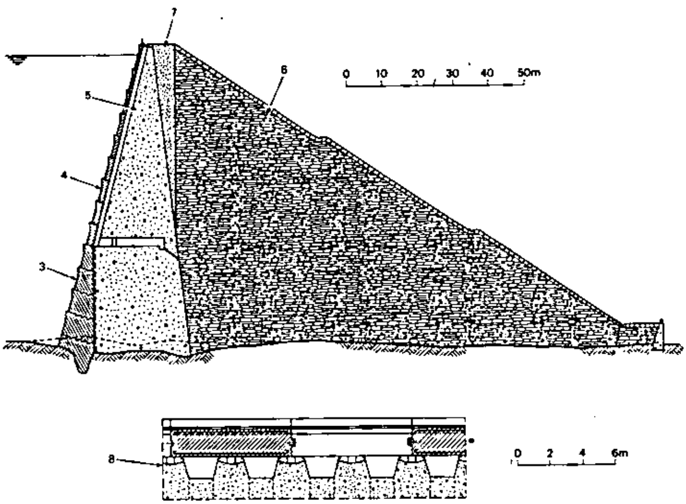

Figure 2 Shing Mun dam

Shing Mun dam (84m), Hong Kong (Fig 2) followed this concept and was designed to have a separate slab supported from the main body of the dam by precast blocks (Binnie 1936). Several Italian dams had faces composed of a succession of jack-arches with vertical support walls at about 2m centres to hold them off the main body of the dam and allow drainage.

This use of separate waterproof faces can be regarded as extreme. In mass concrete gravity dams a more generally used solution was to provide drainage with horizontal galleries interconnected by holes at close centres, all lying on a plane a short distance from the upstream face. A richer concrete was used for the 'slab' upstream of the drains than for the remaining fill of the dam. An example of this approach is shown by Boothwood dam (59m) in England (Fig 3) in which the 'percolation interceptor' was made a special feature. It consisted of units of no-fines concrete, drained through 100mm pvc pipes into the upper and lower galleries. Provision was made for passing a borehole TV camera through these tubes so that if any severe leakage should develop, its position could be determined. The dam was described by Gadd et al (1972).

Use of high cement-content concrete accentuated the problems of heat of hydration and shrinkage on cooling. The complexity of construction in alternate bays and watertight contraction joints increased cost, often making an alternative embankment design a more attractive option, even when foundation conditions were suitable for a concrete dam.

Costs have been reduced by applying the construction methods used for embankment dams. By providing a separate waterproof element, the Alpe Gira dam (178m) was designed with a relatively thin gravity section that would not be subject to water pressure and could be porous with open contraction joints. A low cement concrete, transported in $6m^3$ dump trucks, was spread to 0.8m layers by bulldozer and consolidated by vibrators mounted on tracked vehicles. Open contraction joints were formed by cutting the layers with a hydraulically operated vibrating saw blade mounted on a self-propelled chassis. The dam was faced with 3mm thick sheet steel, backed by drains in case of any leakage. It

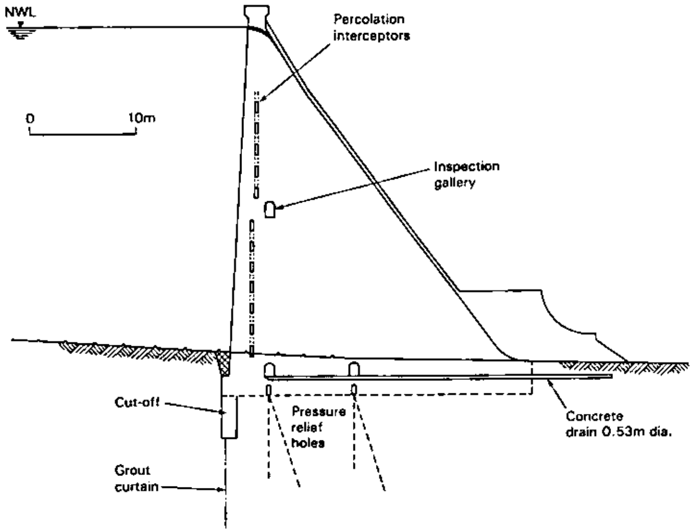

Figure 3 Boothwood dam

was completed in 1964 and has been described by Gentile (1964) and Terracini (1970).

Further reduction of heat of hydration has enabled concrete to be placed by earth moving machinery without contraction joints. An early use of rollcrete was described by Lowe (1962) and special mixes have been developed in preparation for a dam that was to have been built in Britain. Laboratory work and field trials were described by Dawson and Dunstan (1979). Elsewhere several large dams have been constructed by this method. In Japan the roller compacted dam (RCD) method of construction has aroused considerable interest. Shimajigawa dam(89m), completed in 1980 was the first to be built and a number more have followed. Yamauchi et al (1985) have described construction of Tamagawa dam, 100m high, containing $1.14 \times 10^6 m^3$ of concrete, which started towards the end of 1983.

Embankment Dams

Embankment dams of homogeneous section suffered the risk of instability caused by reservoir water reaching the downstream slope. Rotational slips occurred below the point where the phreatic surface met the slope and with clays, high pore pressures could cause deeper seated slips.

The problem was solved by Terzaghi for the Vigario dam (34m) in Brazil by separating the homogeneous section into waterproof and supporting parts with a vertical wall of filter material (Vargas 1970). This ensured that water under reservoir pressure did not enter the downstream shoulder. The dam was re-named the Terzaghi dam in his honour in 1964 (the Mission dam in Canada was also re-named the Terzaghi dam in 1965). This approach was used by Terzaghi to solve the problems at Sasamua (Dixon et al 1958 and Terzaghi 1958). He replaced a central concrete core by a filter drain, as shown by Fig 4.

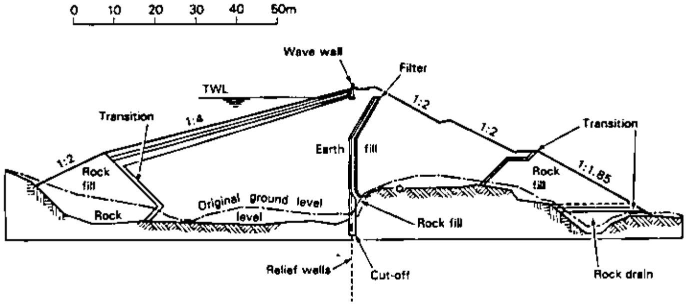

Figure 4 Sasamua dam

Clay Cores

The puddled clay waterproof element of many old British dams is supported by shoulders of more permeable materials on either side. Often a carefully selected fill was placed next to the core so that the permeability of the shoulder fill increased with distance from the core. This provided drainage, often enhanced by chimney drains (built of dry stonework as vertical chimneys, not continuous walls of drainage material that are often given that name) carried upwards through the fill from a drainage mattress at formation.

The more modern central rolled clay cores are also supported by shoulders on either side and drainage is ensured by use of a filter drain in contact with the face of the core. This drain, in the form of a continuous wall, must transmit the thrust from the core to the supporting structural shoulder and be so cleverly designed that water percolating from the core cannot carry clay particles away from the dam through the drainage system.

As we can see from the Maurice Lévy approach, the total compressive stress across any potential plane through the core must exceed the pressure from the reservoir water if hydraulic fracture is to be avoided. Many designers make their clay cores curved in plan so that if they are forced into a straighter shape by the thrust from the reservoir water, they will tighten up, increasing the total stress in the direction of the core axis. This may be a misconception.

Measurements on the downstream faces of two clay cores made with horizontal plate gauges, reported by Penman and Charles(1973) and Penman (1973) showed that during impounding, horizontal movements were negligible. This indicated that horizontal thrust from the clay cores on their shoulders exceeded the thrust from the reservoir water.

To comply with the requirement that the total pressure on any plane passing through the core should exceed reservoir water pressure, the total pressure from the core in an up/downstream direction must approach that of the reservoir water. It therefore seems reasonable for a core that will resist hydraulic fracture to also remain without downstream movement during reservoir filling.

Arching in an up/downstream direction and across the valley reduces the vertical total stress in the central part of an embankment dam and the silo action of a core between the shoulders may cause a substantial further reduction of vertical stress. The horizontal stress σ_a acting in the axial direction (that would act on a vertical plane passing through the core) will be further reduced by the stiffness of the core.

Core material usually has a bulk density more than twice that of water, so the reduction factor from the overburden pressure σ_o to σ_a can be 2 without danger of hydraulic fracture. Unfortunately a strong clay in a narrow core (Penman 1982) can readily cause such a reduction.

Pore pressures

An indication of total stresses may be obtained from measured pore pressures. In a saturated fill any increase of total stress will be carried initially by the pore water. When first placed, a rolled clay core can be expected to contain 3 - 5% air. This makes the pore fluid more compressible and despite the high compressibility of the clay structure, the ratio $\delta u/\delta\sigma = \bar{B} < 1$. Increasing total pressure as height increases causes the air to go into solution in the pore water and can increase $\bar{B}$ to 1. Beavan et al (1977) have given several examples that show δu can be used as a measure of the increase of total vertical pressure $\delta\sigma_v$ in a clay core.

At Derwent dam (36m) a clay blanket was used to connect a central clay core to the upstream toe (this aspect will be discussed below). Pore pressures in the blanket and lower core responded to calculated values of total vertical stress. This displayed arching action, as indicated by Fig 5. During fill placement, pore pressures rose, as shown by Fig 6, but dissipated by drainage during temporary cessations, such as the winter shut-down periods. The values plotted in Fig 5(1)(Rowe 1970) are the sum of the undrained increments and represent the pore pressures that would have developed if the fill had been placed instantly, leaving no time for drainage.

Comparison of the calculated total vertical stress and the measured pore pressures (Fig 5) with the overburden pressure shows clearly the reduction of σ_v in the core due to arching action and the compensating increase of σ_v under the shoulders on either side of the core.

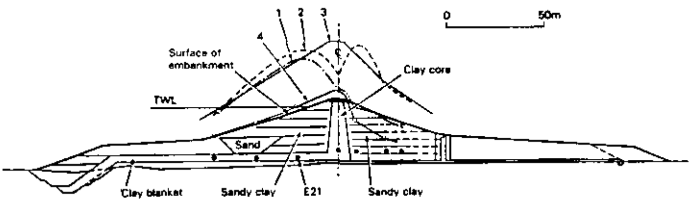

Figure 5 Derwent dam - distribution of pressures in base clay layer

1. Sum of undrained increments of pore pressure
2. Calculated total vertical stress
3. Overburden pressure
4. Pore pressure at end of construction

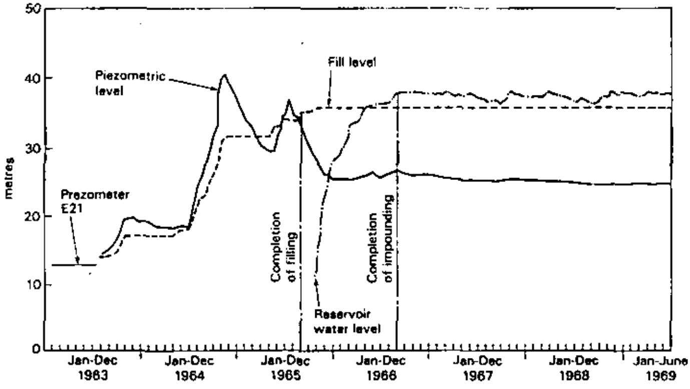

Figure 6 Derwent dam - pore pressures measured by piezometer E21

Similar arching action was displayed at Kielder dam (52m) by the distribution of maximum construction pore pressures, shown by Fig 7 (Millmore and McNicol 1983). It will be noted that the piezometric level in the core, despite the reduction caused by arching and silo action, is above top water level. This was a design feature, following the argument (Penman 1979) that end of construction pore pressures in a core should be at least equal to the pressures to come from the reservoir water to ensure that total pressures will be great enough to avoid hydraulic fracture.

In the past, high pore pressures in the shoulders of earthfill dams have led to instability. In order to avoid this, placement water contents of shoulder fill have been reduced and it is now common practice to place drainage layers in shoulder fill to minimise construction pore pressures. Unfortunately the concept of low placement water content has also been applied to rolled clay cores, and this must be regarded as a misconception.

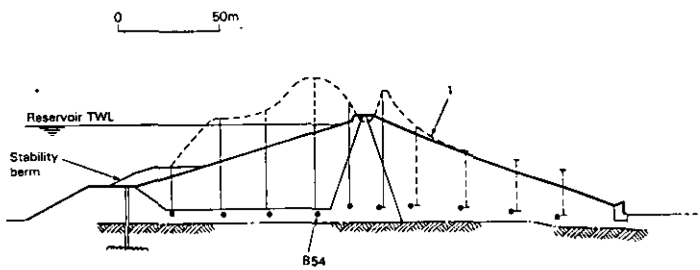

Figure 7
Kielder dam - distribution of pore pressure in base clay layer

1. *Maximum pore pressures developed during construction*
Values of B at piezometer B54 rose to 1.7

It is normal practice to make core width a function of depth to maintain a constant hydraulic gradient. This is accomplished by sloping both sides of the core from a crest width to a larger base width. In some cases, the upper part of the core has been kept of constant width for $\frac{1}{4}$ to $^{1}/_{3}$ of dam height. This appears to inhibit settlement and may contribute to stress reductions sufficient to initiate hydraulic fracture. In two dams that experienced hydraulic fracture cores with parallel upper parts had been used, although there were also other features that would have contributed to reduction of total stresses in the cores.

It should be pointed out that initial pore pressures in a rolled core cannot help but be below atmospheric pressure to give the fill sufficient strength to support the placing plant and rollers. It is usual to express pore pressure in terms of the ratio $r_u = \frac{u}{\sigma_o}$ and initial values of r_u will be negative. Because $\sigma_v < \sigma_o$, r_u can not be expected to rise to 1 in a core. The condition of pore pressure equal to reservoir water pressure will usually be satisfied with an end of construction value of $r_u \approx 0.5$. It is clear from the above discussion that r_u values do not give a direct indication of σ_v.

The overall in-situ permeability of clays is usually higher than that measured on samples in the laboratory because of the natural fabric of the material (Rowe 1972). The reason for puddling clay was to destroy the fabric and mix in any thin layers of sand or silt to make a soft uniform material of low permeability that could readily be pressed tightly into place under the pressure of men's feet.

By contrast, clays for rolled cores often come straight from the borrow pit and so may contain remnants of the original fabric. The action of the heavy placing machinery and rollers can form shear surfaces, e.g.due to bearing capacity failure under heavy wheel loads, so that they tend to have mass permeability much greater than that of puddled clay made from the same borrow. Because the design hydraulic gradient is usually lower than for a puddled clay core, this may be of no consequence, but the fissures and partings can form undesirable faults for the initiation of hydraulic fracture. It is therefore particularly important to ensure that adequate total stresses exist in a rolled core.

A wide core reduces silo effect and increases the contact area with the foundation and abutments. In the extreme it may approach a homogeneous section. Clearly a compromise has to be reached in relation to overall stability. As with other planes through the core, the total pressure across the contact must exceed reservoir water pressure to avoid leakage and risk of erosion. Arching across local irregularities may cause dangerous reduction of total pressure, and it is well known that abutments must be finished to a smooth surface without overhangs. This is often achieved with a concrete or gunite coating. Fissures in otherwise competent bedrock can be particularly dangerous because the core material can form local arches across them and they provide a passage for water that can erode the fill. Silt sized particles can be washed through fissures less than 0.2mm wide. Inspection of a cleaned abutment prior to core placement would normally consider such fissures as closed. A concrete coating can isolate them from the core.

It is usual practice to try to extend the waterproof element of a dam into weathered bedrock in the form of a grout curtain. If it forms a perfect barrier it increases the length of the seepage path for reservoir water and its extent should be designed to reduce seepage flows to non-erodable values and bring leakage within acceptable amounts. In many cases, piezometers placed upstream and downstream of a grout curtain reveal little head loss, showing that the permeability of the foundation has only been marginally reduced by the curtain.

Casagrande (1961) made a particular feature of this aspect and questioned the need for grout curtains in all circumstances.

At Sasamua, the original intention had been to form a grout curtain below the concrete core wall. Many holes had already been drilled when Terzaghi(1958) proposed the radical design change of a filter drain in place of the concrete core wall. He formed the impression that the permeability of the jointed volcanic bedrock was not high enough to justify construction of a grout curtain and used the existing grout holes as relief wells.

The original Lugeon test, intended to determine the need for grouting, applied water to a borehole at a pressure of 1N/mm². If the bulk density of the ground was 20 kN/m³ and the assumption is made that the vertical total stress across a horizontal fissure is equal to the overburden pressure, then application of 1 N/mm² pressure at the top of the borehole can be expected to cause hydraulic fracturing to a depth of 50m. Grout applied under similar pressures would open existing fissures and cause ground heave. Such disturbance of a weathered bedrock is undesirable.

Clearly less disturbance will be caused by a given grout pressure if the vertical total pressure has been increased by the weight of the dam. There are cases where grouting has been carried out during a winter shut-down by boring through the first season's fill, but maximum benefit could be obtained by waiting until the dam is complete. This can be done by providing a gallery near formation level that has been designed to give the headroom needed for the boring and grouting process. Galleries are often provided at the bases of upstream membranes and central asphaltic cores. It is also becoming more common to provide them at the base of rolled clay cores. Such provision not only allows a grout curtain to be made after dam completion, but also allows the decision to be taken about the extent and density or even need for a curtain at a time when it is possible to check on excessive seepage.

Impervious Blankets

Uplift and piping troubles with weirs on deep beds of sand in India were cured by use of a clay blanket upstream, at the end of the last century. The length of the blanket was made sufficient to reduce the hydraulic gradient in the sand under the blanket/weir combination to predetermined design values. These, given by Bligh (1910), varied from 0.05 for silty sand to 0.1 for gravel.

This approach has been used to reduce the hydraulic gradient under dams, as an alternative to a grout curtain, by extending a central clay core to form a clay blanket under the upstream shoulder. The effect on stability is clearly very marked.

A central clay core that is designed to avoid hydraulic fracture, will impose horizontal thrust on the shoulders equal to the thrust from the reservoir. The presence of a clay layer under the upstream shoulder may encourage horizontal movements.

The clay blanket under the upstream shoulder at Derwent dam (36m) was used because of difficulty in forming a central below-ground cut-off. At the site, bedrock lies 55m below ground surface at the deepest point. Deposits above the rock included a 14m layer of laminated clay at a depth of about 13m underlain by a sand and gravel aquifer at artesian pressure giving a head 6m above ground level. The layer was overlain by boulder clay about 4m thick covered by various silty clays and sands. The original intention was to construct a concrete wall in trench as cut-off down to bedrock and work began on its construction from the valley sides. The laminated clay was too soft to support the proposed dam and to avoid a deep seated rotational slip under the shoulders it had to be drained so that it could consolidate and gain in strength as the embankment was constructed. Vertical sand drains 0.3m dia were installed at spacing of 4.3 to 7.5m under the whole embankment to pass through the layer into the aquifer.

Ground water lowering in the deep cut-off trench proved difficult and it became necessary to consider an alternative arrangement with the cut-off terminating in the boulder clay over the laminated clay. To prevent reservoir water passing under the cut-off via the sand drains and aquifer, it was necessary to place the impervious blanket over the upstream drains, i.e. under the upstream shoulder, to key 3m into the boulder clay at the upstream toe (Fig 5). The existing sections of concrete cut-off in the valley sides were connected to the toe key trench.

Both rolled core and 3.7m thick clay blanket were placed at about 3% above optimum water content to give an undrained shear strength, c_u = 90kN/m^2, to minimise risk of cracking under differential settlement. The most critical slip surface passed through the clay core and blanket.

The fill was raised 12.8, 13.4 and 10m to full height during three seasons. High pore pressures during the last season reduced the calculated factor of safety to an unacceptable value and fill placement was stopped for three weeks when 2.7m below crest level. Horizontal movements at the upstream toe increased from about 3mm to 13mm during the last three months prior to completion.

At Kielder dam (52m) it was decided to avoid a major foundation cut-off and control underseepage by use of a clay blanket from the central core to key into glacial till at the upstream toe (Fig 7). It was designed to reduce the hydraulic gradient under the dam to no more than 0.1. With the aim of developing construction pore pressures in the central core higher than reservoir water level to avoid hydraulic fracture, a shear strength specification c_u = 60 to 140kN/m^2 was used. As at Derwent, the critical slip surface passed through the clay blanket and core. Major placement was during two seasons when fill was raised 14 and 22.5m. Towards the end of the second season, high pore pressures caused placement to be stopped prematurely. During the third (final) season, pore pressure increase reduced the factor of safety to 1.28 with the fill more than 7m below crest level. A small toe weighting berm was constructed to ensure stability as construction was completed. During three months before the berm was built, horizontal movements of the upstream toe amounted to 100mm as the bank was raised 7.5m.

These two examples illustrate the care necessary to preserve stability when a soft clay blanket is used under the upstream shoulder. Cases where this feature has led to failure of the upstream shoulders have been discussed by Penman (1986).

Progressive Failure

Stability analyses of slip surfaces commonly use the soil strength parameters c' and φ'. These are determined from shear tests on the soil and usually relate to maximum or peak measured strength. Many soils, when tested at constant rate of strain, show a reduction of strength as strain is increased beyond peak, as indicated by Fig 8. If the testing apparatus can apply sufficient strain, the strength will become constant at a residual value. These residual values give c'_r and φ'_r.

Traditional stability analyses (most of those adapted for calculation by computer) make the tacit assumption that strain along the whole length of the potential slip surface will be uniform. This may be regarded as a misconception. In practice, the strain along most slip surfaces is far from uniform. Bishop (1971) has suggested two types of variation that might occur along a simple circular surface. If the factor of safety used in the analysis is low enough, there is a danger that peak values may be exceeded on some part of the slip surface. The subsequent strength reduction in brittle soils will throw extra shear stress on to other parts of the slip surface and increase strain sufficiently to carry successive sections beyond peak. This aspect has been considered in detail by Skempton (1964 and 1984).

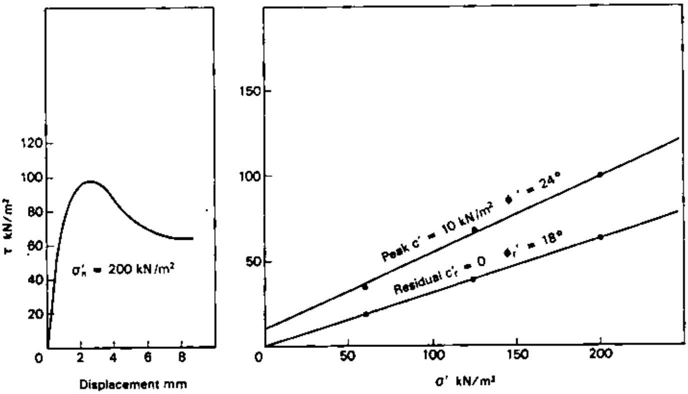

Figure 8 Shear characteristics of a brittle soil

To avoid progressive failure, design must ensure that the peak strength of brittle soils is not exceeded anywhere along the potential slip surface. It would clearly be safe but uneconomic to use φ'_r but a working compromise may be to use φ' and assume that $c' = 0$. Progressive failure can occur in brittle soils when use has been made of c' and φ' in design with too low a factor of safety.

Existing Shear Surfaces

A foundation soil may contain shear surfaces, formed at an earlier time, when the topography may have been different. Part of the surface of a rotational slip in a steep valley side may lie under the much gentler slope found in the valley today. Failures that occurred under ice load may have left slip surfaces in areas where the present topography gives no hint of past instability. Periglacial action can cause creep on relatively flat slopes, leaving surface layers containing large numbers of slip surfaces.

Re-activation of shear on these surfaces can be expected to occur at residual values and design must assume φ'_r and c'_r (probably zero).

Discovery of these pre-formed slip surfaces during site investigation is most difficult. It is almost impossible to detect these surfaces from boreholes and considerable experience is required before they can be identified in trial pits. A knowledge of the geological history of the site may offer the best indication of the likelihood of slip surfaces and their identification may be assisted by some slight change in the properties or quality of the soil above and below them. If the geology of the site gives any suggestion that there may be shear surfaces along which a slip surface might pass, the design assumption should be made that they do exist unless their absence can be proved by site investigation.

Stability analysis used in design must consider slip surfaces that will really pass through the potentially weakest layers. The speed of computer analysis enables large numbers of surface shapes to be checked, but there is always the danger that the programme being used does not allow for all the worst conditions.

Rockfill

At one time, specifications for dumped rockfill required use of only hard, competent rock from the heart of a quarry and inclusion of no small sizes. Weathered rock was wasted and often quarry run was passed over a grizzly to remove fines. It is now recognised that this was a misconception.

Contact forces in large, single size rockfill can be alarmingly high. They are approximately proportional to d^2 (where d represents particle diameter). In fill consisting of 0.5m dia. pieces, an overburden of 100m could produce a vertical force of about 400kN on each contact, whereas a single size coarse sand with 0.5mm dia grains, would have to carry only 0.4N at each contact.

Hard rock (granite, basalt) usually quarries to give very angular shapes. The sharp points of contact are crushed under high loads until the contact area is sufficiently increased so that the strength of the parent rock can provide the necessary support. The effect of water is to reduce the rock strength, so that further crushing will occur when the rockfill is wetted. The open voids between large pieces of rockfill allow them to rotate when further contact crushing occurs, which may result in large overall settlement of the rockfill.

The remedy is to support each individual large piece of rock on a bed of fines, akin to placing masonry blocks on mortar. Use of sufficient fines (down to sand size) not only avoids excessive contact pressures between the larger pieces of rock, but also completely fills the voids and prevents rotation. Compaction at optimum water content by heavy vibrating smooth rollers can produce a very satisfactory fill. No test has yet been devised to determine optimum water content for large size rockfill, but fortunately a slight excess of water is unlikely to be damaging because it can safely drain away. Penman and Charles (1976) have suggested that the limit to the size and amount of fines is when the in-situ permeability of the rockfill is reduced to 1×10^{-5}m/sec. Material with an in-situ permeability much below this value may become subject to construction pore pressures and should be regarded as earthfill.

Dams with Upstream Membranes

If the horizontal thrust from a central clay core on the up and downstream shoulders is equal to the thrust from the reservoir water, then one shoulder should provide sufficient restraint to retain the reservoir. Use of an upstream membrane transfers the water pressure as a thrust on the fill which, if it can have sufficiently steep slopes, need be of no greater volume than one shoulder of an earthfill dam.

The 84m high Shing Mun dam (mentioned above) is a unique example of a type of rockfill dam, built with a section similar to that of a downstream shoulder

(Fig 2). Thrust from the reservoir water on the waterproof slab (4) was transmitted through buttresses faced with pre-cast blocks (8) to a wide concrete thrust block (5), forming the upstream face of hand-placed rockfill (6). A sand cushion (7) was used between the block and rockfill to avoid stress concentrations. The wide drainage gap between the separate waterproof slab and the thrust block ensured that no reservoir water reached the supporting rockfill. This hand-placed rockfill could be expected to be much stiffer than the dumped rockfill, commonly used for upstream membrane dams at that time.

Dumped rockfill earned the early upstream membrane dams a poor reputation because the large deformations that commonly occurred on first impounding often caused severe damage to the membrane. British practice did not favour them. Our first large rockfill dam with an upstream membrane for an impounding reservoir was not built until 1975. The vastly improved behaviour of compacted rockfill has brought them into fashion throughout the world. The highest is the Foz do Areia (160m) in Brazil.

Their weakness lies in the high hydraulic gradient across what is almost a line contact between membrane and foundation or abutment. It is usual to place a grout curtain along this line, often with a gallery incorporated in the grout cap so that further grouting can be carried out if found necessary. It is difficult to design this type of structure without requiring some fill on its downstream side and the compression of this fill under full reservoir loading can cause a sharp differential movement in the membrane as it passes from the well founded, rigid concrete structure on to the fill.

The situation may become much worse if it is necessary to increase the height of the toe structure. This can occur when design, based on a site investigation, has decided on a formation level after removal of topsoil, slope wash and severely weathered material. Construction logistics may require construction of the grout curtain to be amongst the first works on the site, thereby fixing the line for the toe structure. If ground conditions are subsequently found to be worse than assumed and formation level has to be lowered, the toe structure has to be made higher, resulting in a greater depth of fill immediately adjacent to it.

On sites where formation level can be formed in a competent bedrock, the toe structure can be reduced to the form of a slab e.g.(Fig 9) Deer Creek (85m), Salvajina (148m), thereby minimising fill thickness to virtually zero next to the structure. At Khao Laem (130m) where, because it was founded on karst, it was felt desirable to have a gallery, this was built above the slab to avoid any backfilling that would have to carry the membrane, i.e. to eliminate risk of differential settlement.

Often this problem only occurs near streambed, but this is where reservoir head is a maximum and the effects of differential settlements may be most acute.

Instrumentation

Instrumentation is required in order to see if design conceptions approach reality. Type and positions of measurement can only be decided by the designer, who knows the weaknesses and peculiarities of the site and dam. There is a danger that choice and positions for instruments is not given the detailed consideration it deserves. There are cases where instruments have been positioned in neat rows at several levels across a few cross-sections, without reference to potential weaknesses nor full appreciation of the resulting work load of taking and interpreting readings. The cost of instrumentation lies largely in installation and the subsequent reading and analysis of the

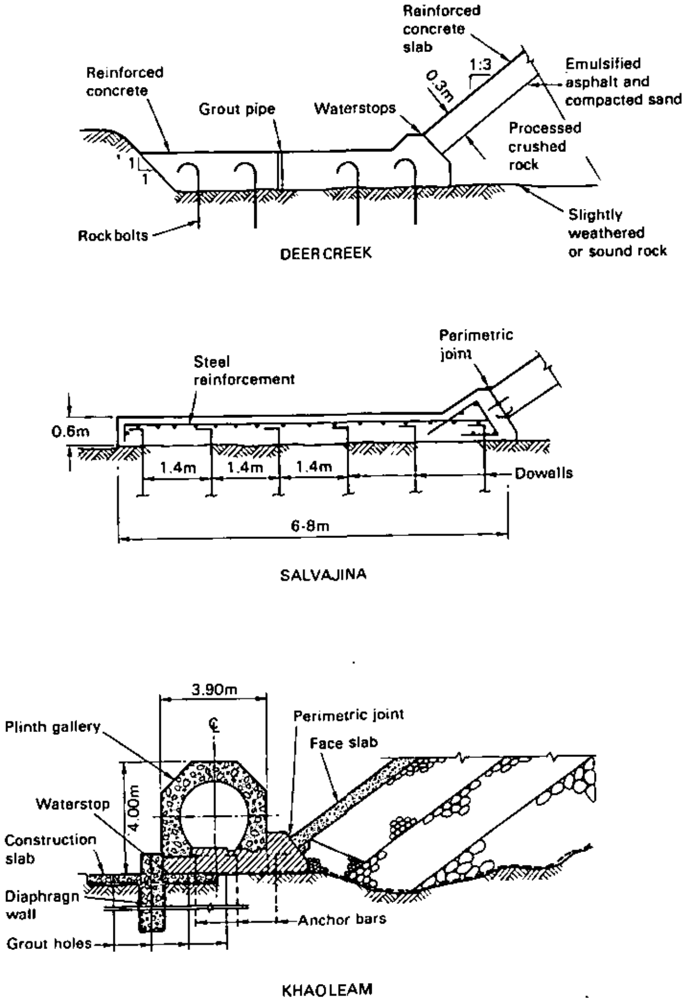

Figure 9 Toe slabs of three large upstream membrane dams

measurements. The cost of the instruments themselves is relatively small and it is always worth using the very best types in the expectation of reliability and long life.

It is becoming usual for instruments to be supplied and installed by specialist firms as sub-contractors to the main contractor. They are thus isolated from the designer and, due to cash flow considerations, often not engaged until a late stage, so that production and supply of some instruments may have to be completed in an unnecessarily short time to suit construction schedule. This does not encourage full trials and repeat calibrations.

As with site investigation, competitive tendering forces prices down and can lead to less than satisfactory results. The vital work of installation may be entrusted to a minimum number of poorly paid staff. Often the required instrument chamber or gauge houses have not been built and the sub-contractor's staff lack the authority to address the problem to the designer or owner. This may cause ommission of the initial and repeated readings so necessary to establish zero conditions. Without reliable zero measurements, subsequent readings may be of limited value.

An apparently simple measurement of considerable importance is that of leakage. The permeability of the waterproof element and the surrounding natural materials ensures a minimum seepage that can be expected to vary with reservoir level. Increasing flows can give early warning of deteriorating conditions.

Measurements that are commonly made of the flows from the drainage systems of the dam may not provide a complete picture. They are often complicated by the effects of rainfall but, more dangerously, they may fail to record leakage escaping through permeable bedrock below the drains. More consideration should be given at the design stage to devising drainage systems that will collect seepage without loss into the downstream foundation, so that reliance can be placed on the measured discharge as a true record of leakage.

Concluding remarks

At the design stage, the type of dam chosen for a new site is most often the embankment type because of its low cost and ability to accept foundation conditions unsuitable for a concrete dam.

Developments in the art and science of geotechnical engineering combined with research studies of the behaviour of large embankment dams that have been carried out throughout the world, have given the designer reliable methods for designing safe slopes. Erosion by seepage and leakage leading to piping failure can be a serious danger and more attention should be paid at the design stage to eliminating this danger.

Perhaps one of the most dangerous design misconceptions is that the factor of safety against dam failure can be obtained from computer analysis of slope stability. There is no known method for calculating a factor of safety against piping failure.

Piping can be said always to originate in hydraulic fracture. If the total pressures across the potential leakage path exceeded reservoir water pressure, the water could only percolate through the voids of the soil and leakage rates could be calculated from soil permeability. Excessive hydraulic gradients could cause loss of material at the discharge face, but this can be protected by filters. The dangerous situation arises when total pressures at any position are not sufficient to avoid hydraulic fracture and high water velocities occur, sufficient to erode the soil.

Sherard (1985) - "believes that there is now essentially incontrovertible evidence available to the profession supporting the conclusion that small concentrated leaks by hydraulic fracturing develop commomly in well-designed and constructed dams -". Fortunately few lead to failure: careful observation allows remedial measures to be taken in time. He refers to many examples of low homogeneous dams that failed rapidly by breaching on first reservoir filling and discusses several cases of large dams, the highest of which to fail was

Teton (93m) in 1976. The discovery of "wet seams" in the cores of several dams adds evidence to the occurrence of hydraulic fracture. That found passing through the wide silt core of Teton during the post failure investigation was particularly intriguing and led to speculation as to whether it could have resulted in failure, had the dam not first failed due to hydraulic fracture in the badly designed key-trench.

First indications of impending trouble may be shown by changes in the rate of seepage through a dam. It is of particular importance to be able to measure accurately the flow of all water passing through a dam. Although measurement is extremely simple and can easily be continuously recorded, it is usually found to be almost impossible to collect all seepage water and difficult to separate the effects of rainfall. Much more consideration should be given to this aspect at the design stage.

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GATES AND VALVES IN RESERVOIR LOW LEVEL OUTLETS LEARNING FROM EXPERIENCE

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Gates and valves used in low level outlets of reservoirs are illustrated. Problems encountered are grouped under headings and are referenced to prototype events or model studies. Conduit hydraulics, flow induced gate vibrations, problems due to two-phase flow, cavitation and slack in gate components are dealt with. Cylinder gates and hollow cone valves are considered separately. Design criteria are deduced.

1. INTRODUCTION

Low level outlets are an essential feature of a dam. They are required for controlled impounding of the reservoir when avoidance of induced seismicity can be a reason, to lower the water level in an emergency or to permit discharge for irrigation abstraction when the water level is below the spillway crest. Scouring of sediment can be another function of bottom outlets.

Low level outlets are controlled by gates or valves or a combination of both. The major difficulties which have been experienced are due to flow induced vibration. Similar problems have been encountered with gates controlling free surface flow but, because of the higher velocities associated with bottom outlets, flow induced vibrations of high head gates tend to be more severe. In addition problems due to two phase flow and cavitation can be present in bottom outlets.

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Lewin (1) classified gate vibration by relating hydrodynamics to constructional features of gates. Kolkman (2) divides vibration phenomena into three types. Naudascher (3) suggests a distinction between three different types of excitation mechanisms, extraneously induced, instability induced and movement induced excitation.

Classification by focusing on the hydrodynamic excitation forces forms an important guide to recognising potential vulnerable features of the design of gates and valves.

The application of practical experience is difficult because few gates, valves and seals are alike in geometry and dynamic behaviour. Personal experiences have the disadvantage that they are random and do not necessarily afford a representative view of the general problem encountered. Therefore, cases from a number of papers and reports have been selected to draw attention to problems which have occurred in gates and valves in low level outlets. From these an attempt has been made to fit them into a scheme for identifying the excitation mechanisms and the flow configurations likely to result in vibration.

2. TYPES OF LOW LEVEL GATES AND VALVES

Control equipment of low level outlets can be divided broadly into three categories; intake gates, control gates and outlet valves.

2.1 Intake gates

1. If vertical lift roller or slide gates are only to be operated under balanced conditions they can be rope suspended. Fig.1

2. The use of Servomotor operated roller or slide gates to permit emergency closure is more usual, even if flow control is located further downstream. Fig.2.

3. Radial gates have been used in some installations. Fig.3.

4. Cylinder gates, are associated with freestanding vertical tower intakes or with morning glory type overflows. Fig.4.

2.2 Control gates

1. Control gates normally take the form of vertical lift slide gates with close coupled servo-motor located in a chamber above. A control gate is frequently backed up by a guard gate. In some installations roller gates have been installed to permit gravity closure. Fig.5.

2. At high heads the jet-flow gate is preferred where the gate slot configuration is designed to induce flow separation in order to avoid intermittent flow attachment. Fig.6.

3. Radial gates are also used as control gates. Fig.7.

2.3 Outlet valves

1. For more limited discharges hollow cone valves backed by butterfly valves are a conventional arrangement. Hollow cone valves as well as hollow jet valves are manufactured up to 3.5m diameter with operating heads up to 250m. This is also about the upper limit of size and head for butterfly valves. Fig.8.

2. Pressure-reducing valves are only suitable as regulating valves in closed pipe systems. Fig.9.

3. SOME CONDUIT HYDRAULICS ASSOCIATED WITH GATES

The introduction of gates into a conduit can cause hydraulic problems of which the following are some examples.

3.1 Gate conduit

The investigation of the bottom outlet of the San Roque Dam in the Philippines (4) demonstrated severe turbulent flow separation upstream of the control gate installation of the type illustrated in Fig.5. This was of a periodic nature causing peak pressure surges. The geometry of the approach section of the tunnel and the transition to the conduits containing the gates affected the pressures in the gate chamber. Chart.1.

The paper also drew attention to the desirability that the cross-sectional area of the conduit at the point of gate discharge should be less than that of the approach tunnel to avoid sub-atmospheric pressures which could limit the opening of the control gate.

3.2 Confluence of jets created by gates in parallel conduits

Where two or more gates are to be installed in parallel then it is necessary to consider the effects brought about by the conveyance of the jets downstream and of any possible combination of the jets downstream and of any possible combination of asymmetrical flow. Problems can result from flow separation, unstable flow, excessive bulking, oblique flow and cross waves.

Koch, in the model study of the bottom outlet of the Randenigala Project in Sri Lanka (5), found that a downstream length of 8m was insufficient for the dividing wall. With velocities up to 43.2m/s flow was separating from the curved face of the dividing wall. In order to guard against low pressures likely to result in cavitation it was necessary to extend and taper the wall by 35m and incorporate facilities for air entrainment.

The bottom outlet of the Mrica Hydroelectric Project (6) has twin conduits, each housing a control and emergency closure gate of the slide type. The dividing wall extends 8.8m downstream of the sill of the control gate with no physical flow separation beyond the wall. The maximum jet velocity was 33.5 m/s.

3.3 Trajectory of jets due to floor offsets

The model study of the drawdown culvert control structure for Mrica (6) showed that the deflectors of the aeration slots in the invert downstream of the gates caused the trajectory of the jet leaving the step to be thrown up to strike the tunnel roof. Omitting the deflectors and modifying the step lead to an acceptable trajectory but with a small decrease in the volume of entrained air.

3.4 Proximity of two gates

The proximity of two gates in a conduit can cause vibration of the downstream gate when the control gate is in an intermediate position and the guard gate is lowered. The jet from the guard gate gives rise to alternating forces on the control gate and in some combination of gate positions there can be turbulent recirculation of flow between the gates. Nielson and Pickett (7) recorded vibration due to this cause and Petrikat (8) mentions a cure of a similar problem by jet dissipators which broke up the discharge jet from the guard gate. Naudascher (9) also draws attention to the danger of vibration due to two consecutively positioned gates in conduit. These are generally transitory problems, however, if considered acceptable they must still be of a level not to damage the structure.

4. VIBRATION OF GATES AND VALVES

4.1 Vibration due to seal leakage

This is probably the most frequent cause of gate vibration. The mechanism of self-excitation due to seal leakage is explained by Petrikat (8) and Lewin (1). The explanation of the hydrodynamic effect differs in the two papers. Reference (8) gives in his paper an example of vibration due to the top seal for a low level vertical lift gate (Bharani Dam) and Krummet (10) discusses a similar example (Chart 2). Petrikat also quotes the example of vibration of a radial gate for a bottom outlet due to leakage of the top seal (Chart 3). Kolkman (2) records vibration of a sphere valve before the seal is inflated and Mitchell (11) vibration due to leakage through a reverse radial gate.

Current practice is to use neither a musical note type of seal (Chart 2) nor an L shaped seal (Chart 3) but a centre bulb seal Fig.10, pressurised by the upstream water level.

With the top seals of gates in tunnel a high velocity flow is established as soon as the seal ceases to make contact with the fixed seal plate. If the gap is narrow there will be a fluctuating pressure which can cause gate vibration. It is therefore important that the shape of the portal and the seal arrangement is such that the gap is rapidly increased as the gate is lifted.

4.2 Vibration due to the shifting of the point of flow attachment

There are few recently recorded examples of vibration due to the shifting of the point of flow attachment for high head gates, Fig 11, although cases still occur in gates in open channels. This may be due to the acceptance that a sharp cut off point should be provided at the lip (1, 10, 12) and the lip tapered back at 45^{0} or more as is shown for the jet-flow gate in Fig.6. A recent model study (13) showed that a 50^{0} taper reduces the hydraulic downpull forces, compared with those which occur due to a 45^{0} taper of the bottom of the gate. Although it appears to be established that the gate lip must have a sharp cut off point, the effect of a wide edge still continues to be discussed (9).

4.3 Vibration due to flow gap width variation

This is a self-exciting phenomenon which occurs with high velocity flows under small gate openings which can be triggered by a vertical movement of the gate. The movement is translated into a momentary pressure change which can reinforce the initial movement of the gate. The inertia of the water under non-stationary flow contributes by causing a pressure rise in the conduit. The gate movement will then either be amplified under resonance conditions or damped out by a combination of positive hydrodynamic and mechanical damping.

The mathematical analysis of vibration under these conditions has been set out by Abelev (15), references (2, 14) and others. In many cases it is possible to calculate whether negative hydrodynamic damping occurs, or if it is compensated by stronger mechanical damping. Negative damping indicates an unstable system. While the resonance frequency of a gate system can be computed, it is more difficult and sometimes impossible to calculate the dominant exciting frequencies. Two possible sources of disturbing frequencies are the vortex trail shed from the bottom edge of a partly opened gate and the pressure waves that travel upstream to the reservoir and are reflected back. Both of these can be estimated.

In Kolkman's opinion (2) the resonance frequency should be higher than the forcing frequency by a factor of 3 or more.

4.4 Vibration due to a free shear layer

The model of the Split Yard Creek Control Structure in Queensland, Australia (16) indicated a well defined shear layer at the gate shaft opening. This was subject to apparently periodic oscillations. Two control gates were located upstream of the shaft and in the fully open position were subject to flow induced excitation which appeared as "beats" because one frequency component of the excitation was close to the natural frequency of the gate assembly. The problem was cured by a combination of leading and trailing edge ramps at the intersection of the shaft and the tunnel.

Reference (1) illustrated the possibility of a free shear layer at the lip of a gate causing fluid dynamic excitation and Martin et al (17) give further examples.

4.5 Unstable flow through small openings

Reference (1) suggested that the dynamic behaviour of gates at small openings is due to pressure fluctuations which in turn are caused by discharge fluctuations. The pressure fluctuations exert a force on the lower edge of the gate causing an extra vertical force.

Kolkman (2) concluded that the width of the leakage gap should be preferably twice or more the width of the gate edge. Although restrictions as to the minimum gate opening are, as a result,often stipulated by gate manufacturers, the authors have found that in many cases the mechanical damping characteristics of the gate have prevented vibration at small openings.

5. TWO-PHASE FLOW

Where air can be introduced into a conduit severe pressure fluctuations can occur at the control gate due to the build up of stagnated air under high pressure at the conduit crown upstream of the gate. The air, which is uniformly distributed at the head race tunnel, accumulates to form air pockets due to the relatively low velocity of flow in the conduit and the long distance upstream from the gate. The air pockets stagnate at the upstream side of the skin plate until they are partially drawn under the gate. When the pressurised air is released it reaches atmospheric value almost instantaneously with explosive force.

A particularly severe problem of this type was noted in a model study by Rouve and Traut (18), Chart 4. In the discussion on the paper Kenn pointed out that air-entraining water flows are notoriously difficult to model, except perhaps when tested with full-scale velocities. Because of scaling problems pressure fluctuations in prototypes may prove less severe than those suggested by model tests.

Neilson and Pickett (7) record severe vibration of a reverse radial gate which was attributed to the collapse of large vapour cavities near the gate. The gate acted as a control valve for a high lift lock with a maximum differential head of 28.1m.

This type of problem can only be solved by venting upstream as well as downstream of the gate.

Singh et al (19) reported the dislocation of a bulkhead on a tower type intake due to air compression. Air entraining vortices had formed at the intake under some lower reservoir levels and the subsequent operation of the emergency gate 200m downstream of the portal caused the surges which increased the pressure on the trapped air driving it up the intake in an air/water spout which dislocated the bulkhead.

6. CAVITATION AND EROSION

The causes of cavitation and its effect on engineering structures have been well documented. The immense damage to the tunnels at Tarbela in 1974 has been attributed to cavitation. With high velocities and the potential for sheared flow adjacent to conduit boundaries regions of low pressure can be set up with pressures close to that of incipient cavitation. Small surface irregularities can be sufficient to drop the pressure to a level to initiate cavitation.

Considerable cavitation damage was reported by Wagner (20) due to high velocity flow up to 41 m/s. This was due to poor alignment of the liner joints, projecting joint welds and minor ridges and depressions in the paint coating. Offsets as little as 0.8mm into the flow produced marked damage and the degree of damage increased with larger offsets. Depressed surface offsets of 6mm produced paint removal and minor pitting. Laboratory studies were conducted by the Bureau of Reclamation (21) to establish the velocity - pressure relationship for incipient cavitation at offsets with rounded corners and sloping surfaces that protrude into the flow. These may be used as guidelines for establishing tolerances for surface irregularities of linings downstream of gates.

The greater resistance of stainless and high nickel steels to cavitation damage is documented by Wagner (20) and others, although cavitation damage to stainless steel occurred downstream of the regulating gates at the Dartmouth Dam low level outlet (22).

Cavitation conditions can arise at pier nosing where piers divide several gate passages, especially under asymmetric gate operating conditions. Anastassi (4) gives an equation for an eliptical shape of the pier nosing to reduce pressure fluctuations for symmetrical flow and a modified equation for a smaller eliptical shape for asymmetric flow conditions. The paper also notes that the taper of the pier must be gradual to prevent flow separation.

Cavitation at gate slots was investigated by Ball (23) and others (24, 25). Flow past the gate slots results in a decrease in pressure on the conduit walls immediately downstream from the slot and recirculatory flow within the slot. Cavitation erosion can occur downstream from the slot at high velocity flow. The ratio of the slot width to depth is one of the parameters as well as the conduit geometry downstream from the slot. Undesirable pressure conditions on the conduit walls can be improved to some degree by offsetting the downstream edge of the slot and tapering gradually back to the original tunnel wall alignment. Fig.12.

In the situation where a hydraulic jump downstream of a gate is contained within a concrete tunnel, considerable erosion to the invert can occur due to the recirculation of debris within the jump.

7. SLACK IN GATE COMPONENTS

When a gate opens or closes the inertia of the water creates regions with an increase or decrease in pressure. Excitation can also be brought about by flow velocity fluctuations at constant gate openings which will vary across the opening.

Vertical lift roller gates can be subject to hydrodynamic pressure conditions so that the uppermost wheel or wheels are on the point of unloading. Reference (2) gives an example of this type where a strong rotational vibration occurred, centred on the lower wheel shaft.

Excessive slack in mechanical gate components such as guide wheels, pivots, hoist chains should be avoided. Where clearances are essential it is important that the component be preloaded. An example of preloading of the guide wheels of a surge shaft gate is shown in Fig.13.

8. CYLINDER GATES

Cylinder gates are used where the controlling gates must operate in a shaft or intake tower. They are used as shut off gates and for regulating the intake. Vibration and cavitation problems have been recorded, particularly at small openings. The gates are guided by rollers operating on tracks fixed to the tower walls and therefore have little mechanical friction to overcome hydrodynamic excitation. Long operating stems or suspension chains of cylinder gates result in low resonance frequencies (see section 4.3). Vibration in some gates appears to have been due to lack of a sharp cut off point at the lower lip (see section 4.2). Vibration which has been experienced at low gate openings is consistent with the variation of hydraulic downpull forces due to unstable flow (see section 4.5).

Ball (26) conducted a model study of a high head cylinder gate which demonstrated that cavitation could occur due to the preliminary gate seat design and also vibration of the gate. Bixio et al (27) found asymmetric pressure distribution on the shell of the gate due to unsteady flow through the eight openings of the intake. Negative pressures were recorded under emergency closure conditions, especially at the lower edges of the gate. Martin and Wagner (28) investigated the vibration of the intake tower at the Keechelus *Dam Outlet Works in the State of Washington* and discovered that it was due to flow surges at the intake and not the cylinder gates which were designed to control the flow.

9. HOLLOW CONE VALVES

Two major types of hydrodynamic problems have been experienced with hollow cone valves:

9.1 Vane failure

This has been attributed to a number of causes but the most likely one is hydroelastic instability causing vibration normal to the chord of the valve and twisting about the longitudinal axis. Destructive resonance occurs at a critical velocity at which the flow couples the two forms of vibration in such a way as to feed energy into the elastic system. Possible modes of vibration for a hollow cone valve are shown in Fig.14. Mercer (29) suggests a parametric value incorporating a coefficient depending on the ratio of shell to vane thickness and the number of vanes. Valves with a value less than 0.115 have operated successfully and valves with a value greater than 0.130 have failed.

Neilson and Pickett (7) reported a major vane failure of a 2740mm diameter hollow cone valve. The failure was of the fatigue type. Mercer's parametric value of the valve as originally constructed was 0.176.

Falvey (30) cited severe vibration of two 2135mm hollow cone valves. The observed 85Hz frequency correlated well with estimates of its natural vibration frequency based on the paper by Mercer. The valve opening in the prototype had to be restricted to a maximum of 80%.

9.2 Shifting of the point of flow attachment

As the hollow cone valve is opened the flow control may shift from the sleeve to the valve body, Fig 15., and intermittent attachment and re-attachment may occur resulting in severe vibration (7). Under these conditions the opening of the valve, that is the sleeve travel, has to be limited.

9.3 Seal failures

The authors have experienced a failure of the nose cone seal of a hollow cone valve within nine months of commissioning a 2750mm diameter valve. A 300mm long section of the seal was torn out. The cause was not established.

Deterioration of the seal between the sleeve and the body of the valve is a more common problem. In most cases it does not lead to vibration of the valve, but a leakage in the form of a jet can erode the metal faces.

10. CONCLUSIONS

Problems which have occurred with gates and valves in low level outlets have been listed in this paper. The headings can be used as a preliminary check list by the practising civil engineer. Although the examples are not comprehensive, they are indicative of the problems which can be encountered. The headings may suggest that the possible problems are of equal importance but by far the most frequent ones are due to flow induced gate vibration. Kolkman (2) classifies these into three types:

- Vibration modes with flow-gap width variation due to gate vibration.

- Vibration modes with constant flow-gap width; often high frequency plate vibrations are observed.

- Vibrations occurring at gate positions where flow is wavering between reattachment and full separation.

The first can be analysed theoretically in many cases, as mentioned in section 4.3 of this paper. In the other two types, self-exciting vibration is probably due to a mechanism involving a fluctuating discharge coefficient induced by the "added mass flow" of the vibrating gate. Lewin (1) suggested design guide lines to avoid vibration problems due to the last classification.

Design criteria which can be deduced from the examples and from the authors' experience:

- unstable flow separation and reattachment should be avoided.

- low pressure regions must be vented by admitting air. Ventilation should extend to the invert of the conduit.

- leakage gaps should be at least 1.5 times the width of the gap edge and preferably twice or more.
- slack in mechanical components such as wheels, pivots etc should be eliminated by pre-loading.
- resonant frequencies of gate hoist systems should be significantly above forcing frequencies.
- water column separation should not hit gates or gate members.
- low pressure zones at gate slots and those due to manufacturing or construction tolerances should be avoided.
- dead zones in conduit should be examined for the possibility of air accumulation.

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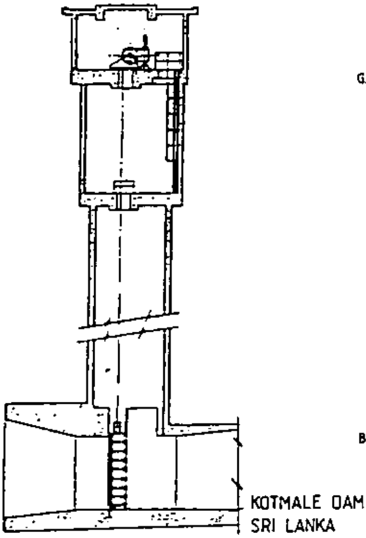


FIGURE 1. INTAKE GATE ROPE OPERATED

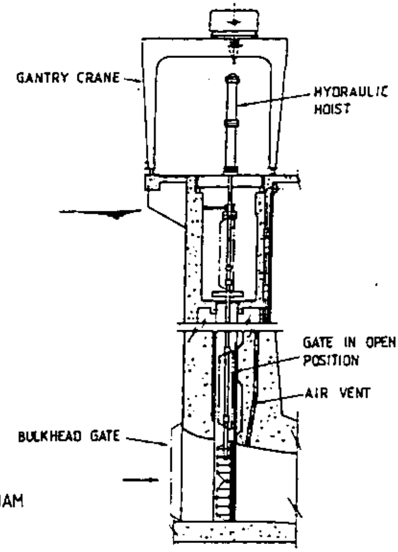


FIGURE 2. INTAKE GATE SERVO-MOTOR OPERATED

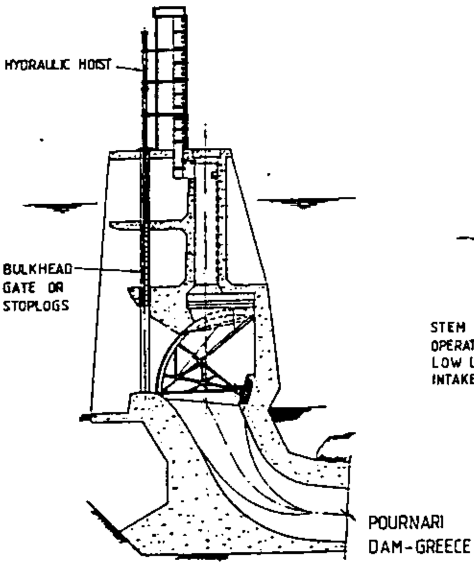


FIGURE 3. INTAKE GATE OF THE RADIAL TYPE

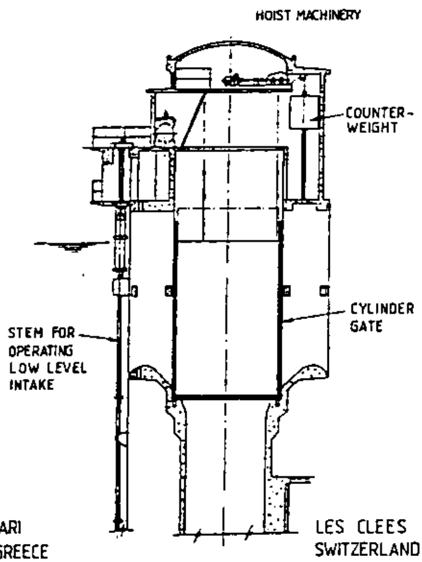


FIGURE 4. INTAKE GATE OF THE CYLINDER TYPE

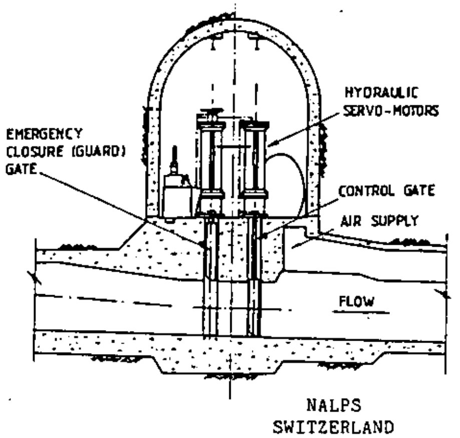


FIGURE 5. CONTROL AND EMERGENCY CLOSURE GATES OF THE SLIDE TYPE

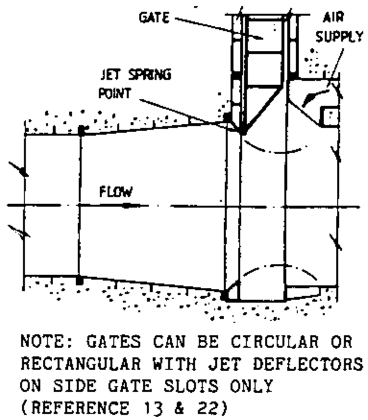


FIGURE 6. CONTROL GATE OF THE JET-FLOW TYPE

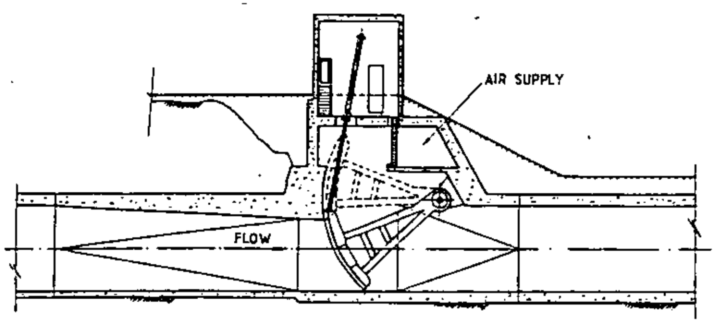


FIGURE 7. CONTROL GATE OF THE RADIAL TYPE

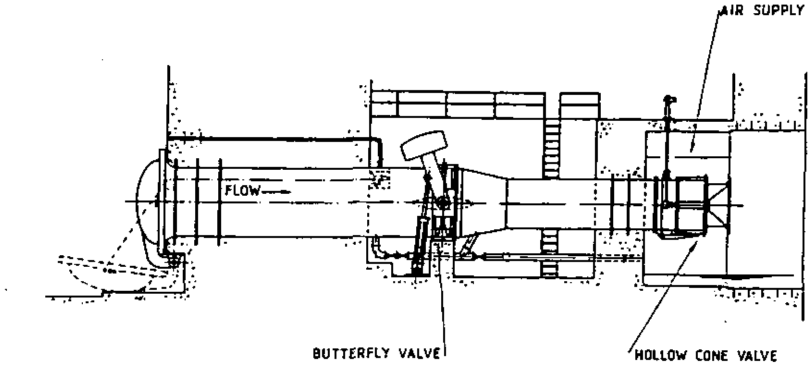
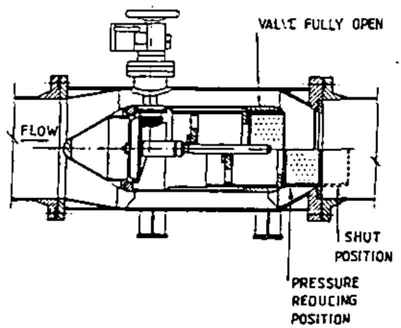


FIGURE 8. ARRANGEMENT OF BOTTOM OUTLET VALVES
BUTTERFLY VALVE AND HOLLOW CONE VALVE



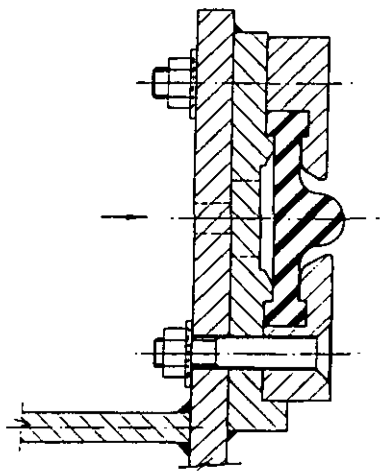
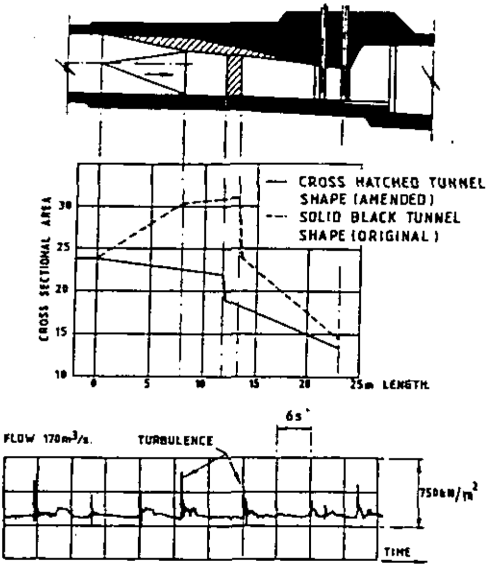


FIGURE 9. PRESSURE - REDUCING VALVE

FIGURE 10. PORTAL SEAL FOR A HIGH
HEAD GATE - CENTRE BULB SEAL

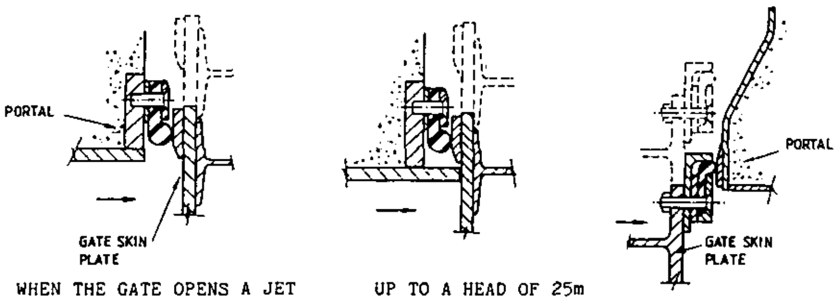


ORIGINAL AND AMENDED CROSS SECTION OF THE GATE APPROACH CHAMBER PRIOR TO SEPARATION OF THE FLOW INTO THREE SLUICEWAYS. THE ORIGINAL SHAPE CAUSED FLOW SEPARATION AND INTENSE TURBULENCE RESULTING IN PEAK PRESSURES IN EXCESS OF 500 KN/m^2 AT THE CROWN OF THE CHAMBER.

6 GATES 1.4m WIDE x 3.1m HEIGHT.
UPSTREAM TUNNEL : 5.5m DIAMETER
BOTTOM OUTLET DISCHARGE CAPACITY $500m^3$ /s MAXIMUM HEAD ON GATES : 92m.

AFTER ANASTASSI (4)

CHART 1. BOTTOM OUTLET OF THE SAN ROQUE DAM - FLOW SEPARATION AND TURBULENCE WITHIN THE CHAMBER UPSTREAM OF THE GATE FLUIDWAYS.



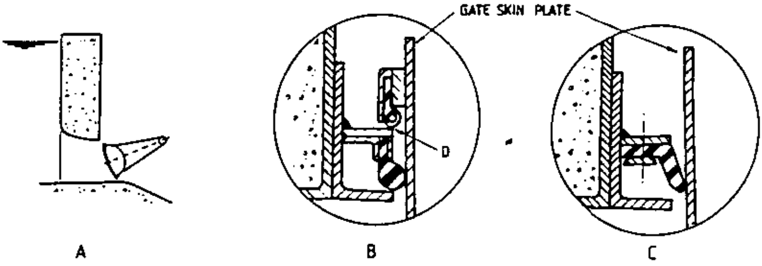
WHEN THE GATE OPENS A JET SHOOTS PAST THE SEAL. THE PRESSURE EXERTED ON THE SEAL AND THE PULSATING PRESSURE ON THE SEAL CONTACT PLATE CAUSED VIBRATION OF THE GATE.
FREQUENCY 35-50 Hz

UP TO A HEAD OF 25m THE SHIELDING PREVENTED VIBRATION. WITH GREATER HEADS (30m), VIBRATION OCCURRED. THE SEAL WAS DESTROYED. A STEEL REINFORCED HARD RUBBER SEAL PREVENTED VIBRATION

WITH THE SEAL FIXED TO THE GATE VIBRATION IS ONLY POSSIBLE DURING THE FIRST 100mm OF HOISTING

AFTER KRUMMET (10)

CHART 2. VIBRATION OF BOTTOM OUTLET GATES DUE TO TOP SEAL ARRANGEMENTS.

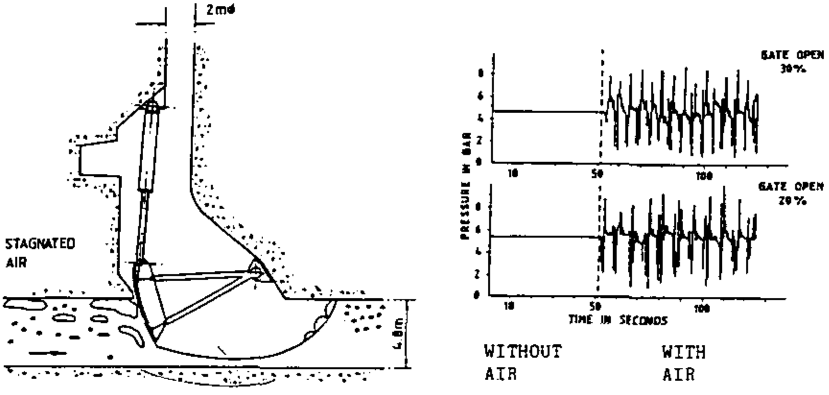


VIBRATIONS OF THE GATE OCCURRED WITH GATE OPENING OF 5-10mm. FREQUENCY 6.34Hz, AMPLITUDE 0.2mm. WITH A SMALL GAP, PRESSURE BUILDS UP ON THE SURFACE D AND LIFTS THE GATE. THIS REDUCES THE PRESSURE IN THE SPACE BETWEEN THE TWO SEALS AND THE GATE LOWERS. THE ACTION IS THEN REPEATED.

VIBRATIONS DO NOT OCCUR IF AN L-SHAPED SEAL IS SUBSTITUTED AS 'C'
HEAD ON GATE CILL 21.5m GATE OPENING 6.3m

AFTER PETRIKAT (8)

CHART 3. VIBRATION OF A BOTTOM OUTLET GATE DUE TO THE TOP SEAL



PRESSURES AT SKIN PLATE

IN MODEL TEST (SCALE 1:25) SEVERE PERIODIC PRESSURE FLUCTUATIONS WERE OBSERVED UPSTREAM AND DOWNSTREAM OF THE GATE. THE GATE VIBRATED AT A MODEL FREQUENCY OF 0.2-0.7Hz. PRESSURE FLUCTUATIONS OCCURRED AT GATE OPENINGS OF 20-40% AT TWO-PHASE FLOW CONDITIONS ONLY.

AFTER ROUVÉ AND TRAUT (18)

CHART 4. TWO PHASE FLOW BELOW A RADIAL GATE

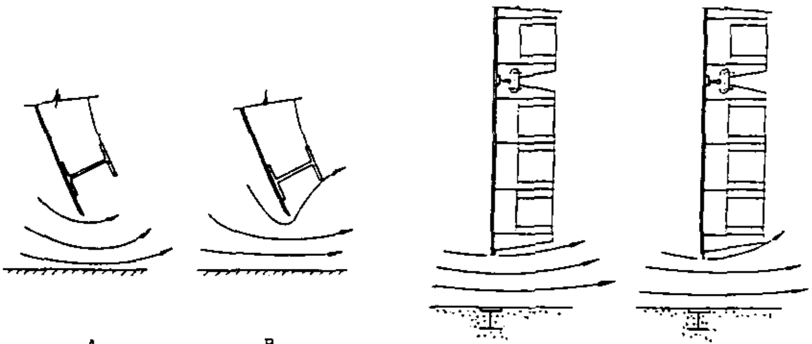


FIGURE 11. SHIFTING OF THE POINT OF FLOW ATTACHMENT - THE FLOW OSCILLATES BETWEEN CONDITIONS A AND B

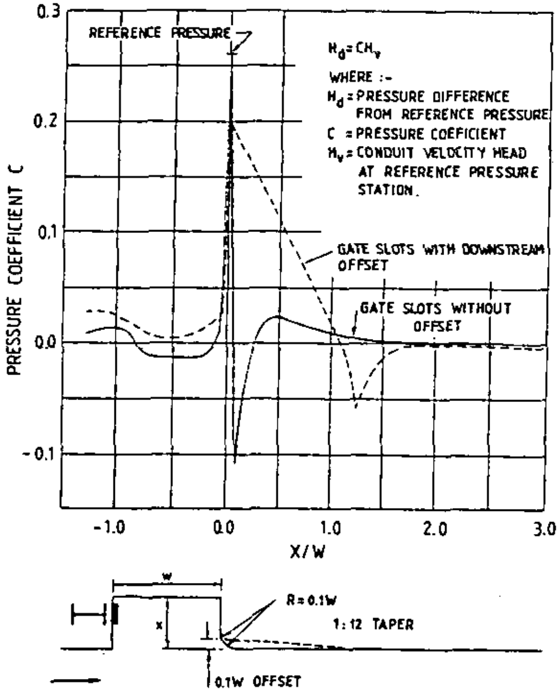
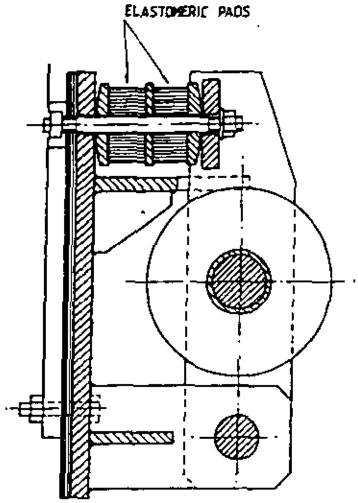


FIGURE 12. GATE SLOTS - PRESSURE DIFFERENCE DUE TO AN OFFSET DOWNSTREAM OF A GATE SLOT



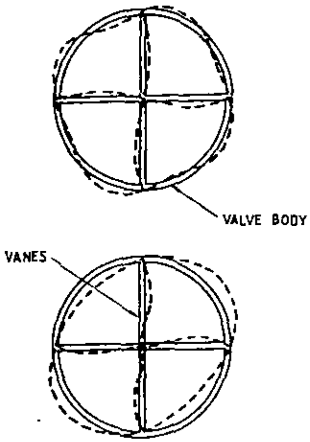


FIGURE 13. EXAMPLE OF PRELOADED LATERAL GUIDE ROLLER FOR A HIGH HEAD GATE

FIGURE 14. POSSIBLE VIBRATIONAL MODES OF HOLLOW CONE VALVES

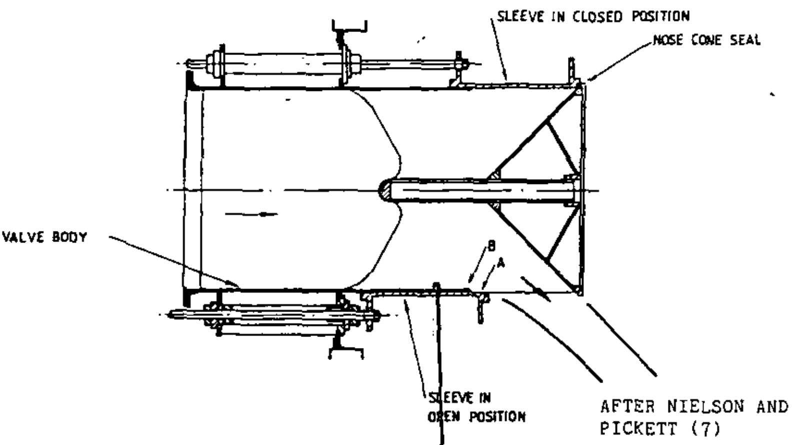


FIGURE 15. VIBRATION OF HOLLOW CONE VALVE DUE TO THE SHIFTING OF THE POINT OF FLOW ATTACHMENT - OSCILLATING BETWEEN A AND B

INSPECTIONS UNDER RESERVOIRS ACT 1975

W.P. McLeish*

The system of regular inspection of reservoirs in Great Britain established by the Reservoirs (Safety Provisions) Act 1930 has been improved. Requirements of the Reservoirs Act 1975 relating to inspections, qualification of civil engineers to make inspections for various purposes, to the enforcement of any recommendations made in the interests of safety, and for continual surveillance of reservoirs are reviewed. Some aspects affecting the keeping of records and making inspections are discussed.

INTRODUCTION

1. Defined terms and passages from the 1975 Act (1) used in the text or Tables are identified by double quotation marks; sections of the Act are referred to in the text as, for example, Section 7(2) or only S7(2); relevant Statutory Instruments are referred to as, for example, S.I. 1986 No. 468. Table 1 summarises the scope of panels of "qualified civil engineers" for the time being; Tables 2 and 3 illustrate the complexity and variety of inspections and certificates for which the Act provides; while they should be a useful guide for those concerned with inspection of reservoirs, reference should be made to the Act and relevant Statutory Instruments(2) and (3) for their full terms.

2. Following failures of small impounding reservoirs in Great Britain at Dalgarrog, North Wales and Skelmorlie, Ayrshire, both in 1925 with loss of life, the 1930 Act (4) came on to the statute books. That Act has contributed to safe performance of reservoirs but experience disclosed various defects and pointed directions for improvement. An interim report on floods affecting reservoir practice, published by the Institution of Civil Engineers in 1933 (5) formed essential guidance for engineers concerned with safety of reservoirs.

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3. After the Edinburgh Congress in 1964, Mr. J. Guthrie Brown, then President of the International Congress on Large Dams, wrote to the Institution of Civil Engineers recommending setting up a special committee to review the 1930 Act; it first met in 1964 and reported in 1966.

4. Nine years later the Reservoirs Act 1975 received royal assent; it makes further provision against escape of water from large reservoirs or from lakes or lochs artifially created or enlarged. But it was not until eleven years later, on 1st April 1986, that the Act was substantially implemented (except for certain provisions relating to Greater London and the metropolitan counties). The objective is to protect persons or property against an escape of water from a reservoir.

5. In 1978, the interim report on floods (5) was replaced by "Floods and reservoir safety" (6) which was reviewed in 1983 (7); it is particularly relevant to inspections made under the Act.

MAIN PROVISIONS OF THE 1975 ACT AFFECTING INSPECTIONS

6. In this paper the term 'inspection' covers any inspection of a "qualified civil engineer" which leads to preparation of a certificate or report whether in that role or in the role of "construction engineer" S6(1), "inspecting engineer" S10(1), or "referee" S19(2). Because the recommendations made by any of them affect the supervising engineer some reference to that role is made for completeness.

7. Some doubt has been expressed as to the exact meaning of safety which is nowhere defined in the Act. In dealing with emergency powers, however, S16(1) states, "Where it appears to the enforcement authority, in the case of any large raised reservoir, that the reservoir is unsafe and that immediate action is needed to protect persons or property against an escape of water ....". The implication is that it is the duty of "qualified civil engineers" making any inspection under the Act to do so for the protection not only of human life, but also property generally.

8. If aggrieved by any recommendation as to measures to be taken in the interests of safety or the time of the next inspection an undertaker may refer a complaint to a referee under Section 19. An enforcement authority appears not to have that option in cases where they appoint and receive such recommendations direct from qualified civil engineers.

9. The 1930 Act system of panels with appointments for life has been replaced by new panels of qualified civil engineers where appointments are limited to five years only. While an engineer may submit an application for re-appointment, it is clear that status as a duly appointed "qualified civil engineer" will have to be watched by individual engineers.

10. The scopes of the several panels as presently constituted are shown in Table 1.

TABLE 1 - Panels of "qualified civil engineers" to act in relation to reservoirs to which the Act applies

Panel name	Scope of panel
A.R. (All Reservoirs Panel)	To design and supervise the construction and alteration of, to inspect and report upon, and to act as supervising engineer for all reservoirs, and to act as referee under section 19 and for the purposes of section 16 of the Act relating to emergency powers of enforcement authorities.
N.I.R. (Non-impounding Reservoirs Panel)	To design and supervise the construction of, to inspect and report upon and to act for the purposes of section 16 relating to emergency powers of enforcement authorities, for all reservoirs which are not impounding reservoirs, and to act as supervising engineers for all reservoirs.
S.R. (Service Reservoirs Panel)	To design and supervise the construction and alteration of, to inspect and report upon and to act for the purposes of section 16, for all reservoirs which are not impounding reservoirs and which are constructed of brickwork, masonry, concrete or reinforced concrete, and to act as supervising engineers for all reservoirs.
Sup (Supervising Panel)	To act as supervising engineer for all reservoirs.

Reservoirs within ambit of Act

11. Under Section 1 the Act applies to a "raised reservoir", which becomes a "large raised reservoir" "if it is designed to hold, or capable of holding, more than 25,000 cubic metres of water above that level" i.e. "the natural level of any part of the land adjoining the reservoir" S1(1).

12. The Act provides for registration of large raised reservoirs. It defines a large raised reservoir in terms of storage in the words used in Section 1(1), being measured from the bottom up without specifying an upper limit. In practice, the criterion for registration appears to be by determination of reservoirs where the stored volume of water (does it include silt and mud?) below "top water level", being a term defined in S.I. 1985 No. 177, exceeds 25,000 cubic metres. "Top water level" means, in relation to a reservoir with a fixed overflow sill, the lowest crest level of that sill, and for a reservoir the overflow from which is controlled wholly or partly by movable gates, syphons or otherwise, the maximum level to which water may be stored exclusive of any provision for flood storage."

13. Inspecting engineers may well become embroiled in argument, in borderline cases or in regard to Discontinuance of large raised reservoirs S13(1), as to whether flood storage above "top water level", lawfully brings a reservoir within the ambit of the Act.

14. It is also of moment that in the case of Abandonment under Section 14 the criterion for abandonment does not refer to a *given volume* but *is "to secure that the reservoir* is incapable of filling accidentally or naturally with water above the natural level of any part of the land adjoining the reservoir or is only capable of doing so to an extent that does not constitute a risk". In the context of property at least, perhaps all reservoirs constitute a risk.

15. Some small reservoirs, or former "large raised reservoirs" with lowered, sometimes inadequate spillways, may have storage less than 25,000 cubic metres below "the lowest crest level of a fixed overflow sill", i.e. "top water level" but have large to even very large flood retention storage above the lowered overflow level.

16. The Act, being concerned with protection of persons or property might be thought to be so whether storage is above or below an overflow level.

<u>Responsibility of "qualified civil engineers" engaged in inspection of reservoirs</u>

17. *Unlike systems which apply elsewhere, in Great Britain* the 1975 Act like the 1930 Act before it, relies on the ability, experience and judgement of a single "qualified civil engineer". This should not be construed as implying that an individual can be master of all the technologies affecting reservoir safety. But *an individual civil engineer shoulders* the responsibility for evaluating all the matters necessary and making "any recommendation he sees fit as to measures to be taken in the interests of safety". Such measures have the force of law. While here and there in the Act a particular requirement may have a shift of emphasis, there is no doubt about the main duty.

<u>Timing of Inspections</u>

18. Periodical inspections must be carried out at the times given in Sections 10(2) (periodically as specified at intervals not exceeding 10 years) and 26(1) (as soon as practicable for reservoirs constructed prior to the 1930 Act and never inspected). A report of an inspection should include any recommendation the inspecting engineer "sees fit to make" as to the time of the next inspection S10(3). Timing of an inspection may also result from a recommendation to the undertaker by the supervising engineer at any time "he thinks" that such an inspection is called for. Wording indicates different orders of judgement, on the one hand, (sees fit to make) implies considered judgement while on the other (thinks) suggests - if *in doubt recommend an inspection. Inspections under other* sections of the Act will arise from time to time in particular cases - abandonment, re-use, investigation by referee etc.

19. In addition to any inspection implied in giving certificates relating to construction or enlargement of reservoirs, a qualified civil engineer must be employed to carry out inspections and make reports as set out in Table 2. Some repetition and overlapping occurs but all references are included to facilitate understanding and to illustrate shades of difference which arise.

TABLE 2 - Inspections (other than those relating to construction or enlargement of reservoirs)

Type of Inspection/ by	° Section of Act/Outline of duties and Comment
A Periodical by "independent qualified civil engineer ("the inspecting engineer")"	
°	10(3) To report the result of inspection "as soon as practicable after an inspection", including any recommendations "he sees fit to make as to the time of next inspection, or as to measures that should be taken in the interests of safety".
°	10(4) To "include in his report a note of any such matters" (that need to be watched by the supervising engineer) "during the period before the next inspection."
°	10(5) To issue a certificate (Table 3, E).
	Note: Under 10(6) if "any recommendation as to measures in the interests of safety" is included, report may be subject to reference to a referee, subject to which undertakers "shall as soon as practicable carry the recommendation into effect under the supervision of a qualified civil engineer."
°	11(2) to direct on "intervals" and "manner" in which undertakers shall "keep a record in the prescribed form."
°	26(2) in a case where a large raised reservoir was constructed before commencement of the 1930 Act "then on the first inspection of the reservoir under this Act the inspecting engineer shall annex to his report drawings and descriptions giving, so far as he can, the like information of the works actually constructed .... as would have been annexed to a certificate under Section 7(6)"; i.e. "detailed drawings and descriptions giving full information of the works actually constructed including dimensions and levels and details of the geological strata or deposits encountered in trial holes or excavations made in connection with the works".
	Note: There appears to be an inconsistency between Section 26(2) and the note relating to the Annex to a report under Section 10, S.I.1986 No. 468 p.12.

TABLE 2 (contd.)

B
Supervisory
by "qualified civil engineer ("the supervising engineer")"

- 12(1) "...to supervise the reservoir and keep the undertakers advised of its behaviour in any respect that might affect safety, and to watch that the *provisions of section 6(2) to (4)" (i.e. provisions* of certificate of "construction engineer" for new, enlarged or restored reservoirs respectively) "or section 9(2)" (i.e. provisions of certificate of "qualified civil engineer" for re-use of an abandoned large raised reservoir) "... and of section 11" (i.e. relating to keeping of "a record in the prescribed form", and installation and maintenance of instrumentation) "are observed and complied with and draw the attention of the undertakers to any breach of those provisions."
- 12(2) to watch, "so long as any matters are noted as matters that need to be watched by him (the supervising engineer) in any annex to the final certificate ... or in the latest report of an inspecting engineer, to pay attention in particular to those matters and to give the undertakers not less often than once a year a written statement of the action he has taken to do so."
- 12(3) relating to recommending to the undertaker "that the reservoir be inspected under section 10 ... if at any time he thinks that such an inspection is called for."

C
Other inspections or actions of qualified civil engineers which may involve inspections

(i) by "qualified civil engineer" (qualified for the purpose of the relevant section)

- 8(2) regarding powers of enforcement authority in event of non-compliance (of undertaker) with requirements as to construction or enlargement of reservoirs to appoint a qualified engineer "to inspect the reservoir and make a report on the construction or alteration and to supervise the reservoir until he gives a final certificate for the reservoir under this section"; report shall include "any recommendations he sees fit to make as to measures to be taken in the interests of safety" 8(3); and under Sections 8(4) and 8(7) to issue certificates subject to 8(5) and 8(6) (Table 3).
- 9(1) Before an abandoned reservoir is brought back into re-use "to inspect the reservoir and report on it" and, 9(4), to issue certificates subject to 9(5).
- 9(8), 10(8) and 14(5) regarding inspection which may be required on being consulted by enforcement authority "as to the time to be specified in the notice" to be served on an undertaker under the relevant section.

TABLE 2 (contd.)

- 10(6), 15(2) regarding supervision of the carrying into effect of "any recommendation as to measures to be taken in the interests of safety", and giving a certificate that the recommendation has been carried into effect (Table 3, F and I).
- 13(1) regarding design or approval and supervision of an alteration for Discontinuance of a large raised reservoir, and giving a certificate S13(2) that the alteration has been efficiently executed (Table 3, G).
- 14(1) regarding Abandonment, to "report as to the measures (if any) that ought to be taken in the interests of safety to secure that the reservoir is incapable of filling accidentally or naturally with water above the natural level of any part of the land adjoining the reservoir or is only capable of doing so to an extent that does not constitute a risk."
- 16(3) Relating to "recommendations as to the measures to be taken" in the exercise of emergency powers, in the case where an enforcement authority proposes to exercise those powers; and in regard to supervising the carrying into effect of any measures so taken.

(ii) by referee being an "independent qualified civil engineer"

- 19(3) acting with powers "after investigating the complaint ... to make such modifications as he thinks fit in the report containing the recommendation complained of" under 19(1).
- 19(1) as may be appointed under section 19(2) (see also S.I. 1986 No. 467) in the case of the undertaker being aggrieved as to:-"measures to be taken in the interests of safety" recommended by an inspecting engineer or "an engineer acting under section 8, 9 or 14", or a recommendation "as to the time of the next inspection" by an inspecting engineer.

  8 regarding non-compliance as to construction or enlargement of reservoirs and recommendations by "the construction engineer" as to "measures to be taken in the interests of safety."

  9 regarding re-use of abandoned reservoirs and recommendations by "a qualified civil engineer" as to "measures to be taken in the interests of safety".

  14 regarding Abandonment of large raised reservoirs and a report by "a qualified civil engineer" "as to the measures (if any) that ought to be taken ...to secure that the reservoir ... does not constitute a risk." (See (i) 14(1) above).
- 19(4) To give a certificate (Table 3, J).

20. In respect of all matters to which the Act applies, certificates must be issued as listed in Table 3.

TABLE 3 - Certificates to be issued under the Act following specified stages of construction and inspection of reservoirs

Name of Certificate To be issued by ° Section of Act/Remarks
A Preliminary Certificate by "construction engineer"
° 7(1) relating to any reservoir or addition to a reservoir "specifying the level up to which it may be filled and the conditions (if any) subject to which it may be so filled;"
° 7(1) preliminary certificate may be superseded "from time to time ... by the issue of a further preliminary certificate varying the previous certificate, whether as to water level or as to conditions."
° See *+ over page.
B Interim Certificate by "construction engineer"
° 7(2) relating to an addition to a large raised reservoir "specifying the level up to which it may be filled until the issue of a preliminary certificate, and the conditions (if any) subject to which it may be so filled;"
° 7(2) interim certificate may be superseded "from time to time ... by the issue of a further interim certificate varying the previous certificate, whether as to water level or as to conditions."
° See * over page.
C Final Certificate (with annex giving a note of "matters (if any) that need to be watched by a supervising engineer during the period before there is an inspection of the reservoir under Act") by "construction engineer"
° 7(3) If not less than 3 years after a preliminary certificate is first issued, "the construction engineer is satisfied that the reservoir or, as the case may be, the reservoir with the addition is sound and satisfactory and may safely be used for the storage of water," specifying "the level up to which water may be stored and the conditions (if any) subject to which it may be so stored", with annex 7(5), relating

TABLE 3 (contd.)

to "matters (if any) that need to be watched by a supervising engineer...".

° See *@+ below.

* 8(4) relating to preliminary, iterim and final certificates when acting under powers of enforcement authority in event of non-compliance as regards construction or enlargement of reservoirs,

@ and 8(5) and 8(6) with qualifications regarding timing of issue, and reservoir being sound and satisfactory, relating to final certificate only.

\+ 9(4) relating to preliminary and final certificates subject to 9(5) in connection with powers of an engineer acting in regard to re-use of abandoned reservoirs; with qualification regarding reservoir being sound and satisfactory.

D
Certificate of Efficient Execution of Works under section 7(6), 8(7)
(with annex comprising "detailed drawings and descriptions giving full information of the works actually constructed, including dimensions and levels and details of the geological strata or deposits encountered in trial holes or excavations made in connection with the works.")
by "construction engineer"

° 7(6), 8(7) "for any reservoir or addition to a reservoir ... as soon as practicable after the completion of the works and in any event not later than the giving of the final certificate."

E
Inspecting Engineers Certificate under section 10(5)
by "independent qualified civil engineer" "the inspecting engineer".

° 10(5) With report of periodical inspection, "stating that the report does or does not include recommendations as to measures to be taken in the interests of safety and, if it includes a recommendation as to the time of the next inspection, stating also the period within which he recommends the inspection should be made."

F
Certificate under section 10(6) as to the carrying out of safety recommendations
by "qualified civil engineer"

° 10(6) When under supervision of a "qualified civil engineer", "any recommendation as to measures to be taken in the interests of safety ... has been carried into effect."

TABLE 3 (contd.)

G
Certificate under section 13(2), on Discontinuance
by "qualified civil engineer"

- 13(1) "to design or approve and to supervise the alteration";
- 13(2) on being satisfied "that the alteration (to render a large raised reservoir incapable of holding more than 25,000 cubic metres of water) has been completed and has been efficiently executed."

H
Certificate under section 14(3), on Abandonment
by "qualified civil engineer"

- 14(2) and 13(1) "to design or approve and to supervise the alteration";
- 14(3) stating that a report made under 14(1) does or does not make recommendations "as to the measures (if any) that ought to be taken in the interests of safety to secure that the reservoir is incapable of filling accidentally or naturally with water above the natural level of any part of the land adjoining the reservoir or is only capable of doing so to an extent that does not constitute a risk."

I
Certificate under section 15(2) as to the carrying out of Safety Recommendations
by "qualified civil engineer"

- 15(2) in cases where the undertaker fails to comply and the enforcement authority causes any recommendation to be carried into effect under reserve powers by appointment of a "qualified civil engineer"; certifying that "any recommendation as to measures to be taken in the interests of safety ... has been carried into effect."

J
Referees Certificate under section 19(4)
by "independent qualified civil engineer"

- 19(4) Relating to cases where undertakers are aggrieved by any recommendation of inspecting engineer or engineer acting under sections 8, 9 or 14 with respect to "measures to be taken in the interests of safety or as to the time of the next inspection," and when a referee is appointed under 19(1);
- 19(4) "stating that decision does or does not modify the report" which is the subject of the undertakers being aggrieved.

Note:

(i) Any reports of inspections and certificates must be in the prescribed form, S20(1) and S.I. 1986 No. 468.

(ii) Under Section 20(4) (a to d), where any certificate; any report by an "inspecting engineer" and other specified instances involving inspections; a decision of a referee in specified cases; any written explanation by a "construction engineer" with reasons for deferring giving a final certificate are delivered by the engineer in question to the undertakers S20(2), the engineer must within 28 days send a copy of it to the enforcement authority. Additionally, a copy of drawings and descriptions annexed to a report of an inspecting engineer for the first inspection under this Act of a "large raised reservoir" constructed prior to commencement of the 1930 Act (1st January 1931) must be sent, with a copy of the report, to the enforcement authority by the inspecting engineer; this applies "...whether or not the report is stated in the inspecting engineer's certificate to include a recommendation as to measures to be taken in the interests of safety." S26(3). See note in Table 2A under S26(2).

° This is an important provision affecting inspecting engineers who, with the implementation of the Act, may encounter an increasing number of reservoirs in this category; it may prove to be a relatively costly matter for the undertakers in some cases.

(iii) Similarly under Section 20(4)(e), in the case of documents in specified instances, relating to advice given by the supervising engineer to the undertakers which includes a recommendation "to have the reservoir inspected under Section 10", the supervising engineer must within 28 days send a copy of it to the enforcement authority.

PERIODICAL INSPECTIONS

General

21. Periodical inspections must be carried out by "independent qualified civil engineers". It will be necessary for inspecting engineers to satisfy themselves that they meet the definition of "independent" given in Section 10(9), namely:-

"(a) that he is not in the employment of the undertakers otherwise than in a consultant capacity; and

(b) that he was not the engineer responsible for the reservoir or any addition to it as construction engineer, nor is connected with any such engineer as his partner, employer, employee or fellow employee in a civil engineering business."

22. The inspecting engineer's main responsibility and duty in carrying out a periodical inspection under Section 10 is to:-

- make any recommendations he sees fit to make:

  - as to the time of the next inspection
  - as to measures that should be taken in the interests of safety

- under Section 10(4) to note the matters (if any) that need to be watched by the supervising engineer during the period before the next inspection

- under Section 11(2) to give directions as to the intervals and manner in which the undertakers shall give information in keeping a record in the prescribed form.

23. While safety of persons and property must be paramount, considerations of practicability and effectiveness of monitoring, in terms of reliability of results, will influence the framing of such recommendations, notes and directions. These aspects should be kept in mind throughout the making and reporting on inspections.

Appointment of inspecting engineer

24. Under the 1930 Act an inspecting engineer's appointment was at-an-end following inspection and submission of a report. A similar arrangement could apply under the 1975 Act. Wording of Section 11(2) regarding keeping of the record "..in such manner as may from time to time be required by any directions of the construction engineer or inspecting engineer" is consistent with successive engineers acting in each role. Such wording would also appear to fit a different case of a 'term appointment' of an inspecting engineer, which arrangement would be consistent with the format of Part 3 of the 'Prescribed form of record' relating to inspecting engineer "From.... To....". Strangely, the supervising engineer has no "From.... To...." provision. Is this an oversight?

25. Whatever the format of the 'prescribed form of record' indicates, both inspecting and supervising engineers will want to know, with certainty, the time duration of their responsibilities. It may well be in the interests of safety and of minimising the costs of implementing "measures in the interests of safety" to accommodate interim advice by an inspecting engineer in a lawful manner. Cases are already arising as to the proper means by which the frequency of keeping records of water level and other monitoring data can be reduced from requirements of the 1930 Act to take advantage of the provisions of the 1975 Act.

Facilities for inspection and information to be furnished

26. Under Section 21(5) undertakers are required to afford supervising and inspecting engineers all reasonable facilities and to furnish them with the statutory record (8), copies of

any statutory certificates with annex if any, inspection reports with annex if any, and such further information and particulars as each may require.

27. Facilities furnished should also include those required in connection with health and safety, clearance of vegetation, and lighting and access to all parts of the works.

28. Statutory records are frequently incorrect and completed in a variety of ways. Documentation for the 1975 Act will breed its own confusions, for example:-

- in Part 3 of the prescribed form of record regarding the duration of appointments of inspecting and supervising engineers who will wish to ensure that their responsibilities, obligations and liabilities are not unknowingly extended and do not follow them from neglect to appoint a successor;

- also, in Part 7 of the record, doubt is likely to arise regarding the capacities "at top water level" and "between the lowest natural ground level of any land adjoining the reservoir and top water level";

- in Part 11 the heading "Details of any unusual events, such as seismic activity .... "suggests that only such large scale events are intended to be listed. No doubt guidance from inspecting engineers will be sought.

  The author suggests that while earthquakes and extreme floods are obvious matters for record, so too would be not so large floods which overtopped the dam or any part of the overflow arrangements, or any unexplained movements of land or emergence of springs nearby but not necessarily in "surrounding land" and such as, at first sight, might affect the stability of the reservoir. It is preferable to err on the side of excess and record any event, even such as an animal falling through the surface, with dates and brief description of the circumstances: the *understanding of mechanisms of behaviour of dams and* reservoirs often involves 'detective-like' work in which even the smallest clue may play a vital part.

  Other difficulties and doubts exist in regard to completion of the record; others will emerge with experience.

29. Inspecting engineers require data additional to, and distinct from, that contained in the statutory record to enable reservoir safety to be effectively evaluated. Such data would include, for example, relevant reports of site and laboratory investigations, model studies, head/discharge characteristics of valves and gates, and publications and drawings; it would of valves and gates, and publications and drawings; it would also include data required *for flood estimation and routing* and stability analysis, any geological information available, and all records of monitoring..

Floods and reservoir safety

30. With regard to floods, the 1983 Report of the Working Party to review The Floods Guide (7) concluded that the Guide did not require amendment at this stage ... but that, in the meantime, certain aspects should be clarified as summarised below:-

> The Guide is not mandatory and panel engineers, in consultation with owners should use their own discretion in selection of the "Design Flood" being that which, if exceeded, is likely to cause a breach. The "community" referred to for Category A is considered to be one of not less than ten persons; and there are special cases where floods smaller than the Probable Maximum Flood would be appropriate, e.g. for some flood retention reservoirs or farm dams.

31. Evaluation of what is likely to cause a breach is no easy matter. Most engineers concerned with reservoirs have direct experience of quite large floods or can read about them. But even major recorded floods have relatively short return periods. To exercise judgement as to cause and effect on that level, where one may have a 'feel' for the scale of events, is on a plane altogether different from doing so in relation to flood events which transcend actual experience.

32. In the case of earth embankments, leakages not infrequently occur near top water level where drying out and cracking of cores may occur. Monitoring arrangements are often insufficiently descriminating to separate changes in leakage flows from the extraneous contribution from rainfall; this leads to there generally being little or no reliable data of core integrity near and above overflow level. The temptation is to relate the head required to discharge a design flood to that actually available without closely deliberating the consequence of such a head on the embankment. In any event, what actual evidence is available to demonstrate performance at high heads?

- Until much more information is available to guide engineers in this respect, the author advocates caution in exercise of judgement on tolerance to overtopping; to build up data surveillance and photography of reservoirs in severe as well as congenial conditions is recommended. This can be achieved by seasonal variation of the time of visits both of inspecting and supervising engineers from year to year, and making special efforts to witness severe events of precipitation, wind and temperatures, making the soonest practicable visit following reported freak conditions, and recording the circumstances.

Monitoring and Surveillance

33. The interval between inspections may be as long as ten years, which is a long period to continue to amass monitoring

data relating to movements, settlements, leakages etc which may be significant in relation to safety, but often are not. Embarrassingly large piles of data exist, analysed and interpreted with varying degrees of effectiveness, or even lying gathering dust without more than cursory attention.

34. Monitoring is expensive and, even when prescribed with the best intentions, may produce data which is not susceptible to clear interpretation. Space limitations prevent discussion in depth; comment is thus restricted to substance. It is for the construction or inspecting engineer to note matters which a supervising engineer has the duty to watch, or to give directions as to the intervals and manner in which data should be collected and recorded by the undertaker.

° The author suggests a minimum of physical monitoring coupled with a continual perceptive surveillance using the simplest available means of obtaining evidence of performance. While sophisticated equipment will be necessary in some cases, basic instrumentation should be limited to identifying general and anomalous features of performance which may affect safety, keeping in mind that there is no clear line separating evidence of the safe and unsafe and that particular features measured at the surface may be only poor reflections of significant, but deep seated problems. Any possibly adverse feature identified should then be instrumented and observed with sufficient frequency and detail to establish the character and significance of the anomaly. Thereafter monitoring and surveillance should be increased, or preferably decreased, to suit particular cases.

The character of monitoring and surveillance should be a continual flexible awareness of performance and defects within, in particular instances, limits recommended by the inspecting engineer. This is no easy matter of specification of arrangements which have to continue and be effectively and economically targeted on safety for up to ten years. An arrangement for a supervising engineer to consult an inspecting engineer from time to time is indicated in the interests of maintaining effectiveness.

Reports

35. The prescribed form SI 1986 No. 468 Regulation 4, differs in significant respects from that applicable to the 1930 Act. It excludes, but does not preclude, a 'statement as to certificates and reports of previous inspections furnished to the engineer' and a 'general description of inspection made and the conditions found'; in relation to particular matters to which a report must refer, no specific reference is now made to 'leakages', 'movement of reservoir banks or walls' or to 'any alteration in top water level' (meaning a change to be recommended).

36. The latter can be covered by a recommendation but it would have been better to have a prescribed heading under which to marshal evidence and make the case for such recommendation. None of the other exclusions fall naturally under other headings.

- The author considers that each of the excluded items is important in the context of safety and should be included in reports as explicit headings, along with a section for 'any other matters' which do not fall naturally under any of the headings. Also, the inclusion of photographs are a useful complement to description of conditions found.

37. Inspecting engineers, "when appropriate" should give findings as to compliance with obligations to maintain a record. The prescribed form of record is clearly a format only. Undertakers will have to produce loose-leaf books adopting the prescribed format, to facilitate expansion. As mentioned already, different interpretations and methods of keeping the record will arise and individual inspecting engineers will have to make up their own minds in giving findings regarding undertakers' compliance with S(11) "Recording of water levels etc."

- The author suggests a symposium in say three to four years time to review experience on documentation and inspections generally.

REFERENCES

1. Reservoirs Act 1975, London, U.K.
2. Statutory Instrument 1986 No. 468 - Reservoirs Act 1975 (Certificates, Reports and Prescribed Information) Regulations 1986.
3. Statutory Instrument 1986 No. 467 - Reservoirs Act 1975 (Referee) (Appointment and Procedure) Rules 1986.
4. Reservoirs (Safety Provisions) Act 1930, London, U.K.
5. Institution of Civil Engineers, London, 1933 "Floods in relation to Reservoir Practice".
6. Institution of Civil Engineers, London, 1978 "Floods and reservoir safety : an engineering guide".
7. Institution of Civil Engineers, London, 1983 "Floods and reservoir safety - Report of the Working Party to review the Guide".
8. Thomas Telford Ltd, London, 1985 "Prescribed form of record for a large raised reservoir".

THE ROLE AND TRAINING OF SUPERVISING ENGINEERS

N.J.Ruffle*

The appointment of Supervising Engineers is a requirement of the Reservoirs Act 1975. The paper examines the relevant provisions of the Act and discusses the identity, training and duties of the Supervising Engineer including his relationship with the undertaker (or owner) and the Inspecting Engineer.

INTRODUCTION

In 1966 an ad hoc committee of the Institution of Civil Engineers submitted to the Home Office proposals for a revision of the Reservoirs (Safety Provisions) Act 1930. They pointed out in paragraph 15 that:

> "Under the existing Act, the qualified engineer who has designed and supervised the construction of a reservoir, must issue a Preliminary Certificate before it may be filled with water either wholly or partially. When construction is complete he is required to issue a Final Certificate, after which, under the Act, he has no further responsibility for the oversight of the reservoir. Once a Final Certificate has been issued, the Act does not require the condition of the reservoir to be considered again by a qualified engineer until the time comes for the first of the periodical inspections. This may be for as long a period as ten years from the time when the Preliminary Certificate was issued; and subsequent inspections are permitted to be ten years apart. This means that a reservoir may not be the responsibility of any qualified engineer for several years after completion and during the period between periodical inspections; yet it is significant from a study of the record of slide failures suffered by earth dams in all parts of the world that a quarter of these failures occurred during the first year after construction, a quarter during the next four years, and the rest fairly uniformly over the next 35 years."

To ensure sufficient oversight of dams in the years immediately after construction, the Committee recommended that the final certificate should not be issued within 3 years of the preliminary certificate. They also proposed a maximum period of 5 years for issue of the final certificate to prevent

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undue delay in the provision of the design details accompanying the certificate of execution of works which must be issued not later than the final certificate. They further proposed that the first of the periodical inspections should take place within 2 years of the final certificate instead of within 10 years of the preliminary certificate.

All the recommendations have been taken into the 1975 Act but in paragraph 18 of the report there was an additional observation which stopped short of a specific recommendation but contained an idea that was developed into a major provision of the 1975 Act. They said:

> "The Committee further considers that the undertaker should be under a statutory obligation to nominate a chartered civil engineer to be responsible for the oversight of his reservoirs. In the event of any significant change occurring in the behaviour of the reservoir this engineer would be required to inform the Panel engineer who undertook the last statutory inspection."

The notion of a Supervising Engineer is thus at least 20 years old and can be viewed as a permanent extension of the supervision expected of the Construction Engineer. It pre-dates the current tendency to manage with fewer resident reservoir staff and it would not be right to associate one closely with the other. The appointment of a Supervising Engineer does not lessen the need for members of the workforce to visit high category dams at least every few days, particularly after storms, and to report to him on any subsidences, wave damage or apparent leakages.

References in the 1975 Act to the Supervising Engineer occur in a number of sections and for convenience these have been summarised in abbreviated form in the Appendix.

APPOINTMENTS

Supervising Engineers must be qualified civil engineers who have been appointed jointly by the Secretaries of State for the Environment, Scotland and Wales, after consultation with the Reservoirs Act Consultative Committee of the Institution of Civil Engineers, to any one of four panels. Although one of these panels is specifically a panel of Supervising Engineers, members of the other "superior" panels are also empowered to act as Supervising Engineers and some may do so, particularly in respect of small privately owned low category reservoirs. Applications for the Supervising Panel were invited in December 1984 and, by March 1986, 244 appointments had been listed. Of these, half are employed by the Water Authorities and Water Companies, the Scottish Regional Councils, British Waterways Board, and the North of Scotland Hydro-electric Board. A quarter are with firms of consulting engineers. The remaining quarter mainly give private addresses and their employment is not evident. Some no doubt intend to offer their services in a private capacity. It is to these and to panel members with firms of consulting engineers that the owners of the many private amenity reservoirs can be expected to look for their Supervising Engineers.

Undertakers, including private owners, should by 1 April 1986 (1 April 1987 in the areas of the previous GLC and Metropolitan Counties) have appointed a Supervising Engineer to every large raised reservoir and informed the enforcing authority to that effect. Unlike Inspecting Engineers, Supervising Engineers do not have to be independent of the undertaker, nor do they have to be independent of Construction or Inspecting Engineers. Whether they are engaged full-time or part-time on Supervising Engineer's duties is a matter for the undertaker. The majority can be expected to have other duties for most of the time.

The Act calls for a Supervising Engineer to be employed at all times when a reservoir is not under the supervision of a Construction Engineer. It can be inferred that he should be generally available in the area and a replacement would have to be arranged in the event of say a prolonged absence from the country.

TRAINING

Although the Act makes no mention of training there is a clear need for instruction in what Supervising Engineers should look for and include in their reports. Additionally they should, of course, become conversant with the various powers and duties conferred upon them by the Act. The Northumbrian Water Authority designated Supervising Engineers in 1979 and they were given training by a Panel 1 consulting engineer. Subsequently the Water Industry Training Association has been running 4-day courses which include the practical exercise of inspecting and reporting on two impounding reservoirs. It is understood that some 200 engineers have attended these courses to date.

Supervising Engineers should possess sufficient expertise to notice whether anything unusual has occurred at a reservoir before it can cause a major hazard. For this purpose they need to be experienced engineers and they need to acquire a basic understanding of the design and functioning of dams of various types. In particular they need to familiarise themselves with the details of the dams to be in their care. Knowledge of the faults that dams may develop is relevant, as are the tell-tale signs of developing problems that are known to experienced Inspecting Engineers. They must above all look for signs of change and although they can examine only the faces of the dam structure, even deep-seated problems can be expected to produce surface indications. The significance of seepages and other drainage flows, their qualitative examination and measurements, is another area where an understanding is needed. Perhaps even more important is an understanding of the available measurements of uplift pressures. The more modern dams commonly have instruments installed for the measurement of pore pressures, settlements and tilts and an appreciation of the significance of the readings and their variations is also necessary.

DUTIES

When a Supervising Engineer has been appointed to supervise a reservoir he carries a statutory responsibility to keep the undertakers advised of its behaviour in any respect that might affect safety. It is worth noting also that this responsibility emcompasses the whole of an impounding reservoir including the hillsides as well as the dam. Supervision should also involve reporting on maintenance requirements whether or not they can be seen to be connected with reservoir safety. Proper maintenance of the dam and ancillary works, even where classed as cosmetic, can have an indirect effect on safety. Where the Supervising Engineer is an employee of the undertaker, there may be advantage in including the responsibility for the carrying out of the maintenance work in his duties.

He must also watch that other provisions of the Act are observed and he must draw the attention of the undertakers to any breach of those provisions. First, they concern the requirement that a new or enlarged reservoir, or an abandoned reservoir to be brought back into use, must be used for the storage of water only in accordance with a certificate of the Construction Engineer or an Inspecting Engineer as appropriate. Second, they concern the requirement that records prescribed in the Act and by the Inspecting Engineer must be kept. The Supervising Engineer has to satisfy himself that the necessary instruments are being maintained and that the prescribed information is being recorded satisfactorily and is up-to-date.

The Supervising Engineer must recommend to the undertaker that a reservoir should be inspected by an Inspecting Engineer if, at any time, he thinks that such an inspection is called for. The undertaker must act on this recommendation, and arrange an inspection. Any such recommendation for an inspection or for any other action to be taken, or any drawing of attention to a breach of the provisions in the preceding paragraph, must be copied by the Supervising Engineer to the enforcement authority. In that respect he can be seen to possess power over the undertaker who may be his direct or indirect employer. Whilst no doubt ordinarily tactful and avoiding over-zealousness, he must be prepared to be resolute if the occasion demands!

If the Construction Engineer in his final certificate or the Inspecting Engineer in his report have noted matters which need to be watched, the Supervising Engineer must provide a written statement to the undertaker on his related action, at least once a year. Such matters may include the reading of instruments or observations on survey lines. He may or may not be expected to make the observations and recordings himself, but he must in any case make it his business to ensure that these tasks are being properly performed.

INSPECTIONS AND REPORTS BY THE SUPERVISING ENGINEER

The Act contains no guidance on the frequency of inspections by the Supervising Engineer which is therefore a matter for agreement between the undertaker, the Supervising Engineer himself and the Construction or Inspecting Engineer. In straightforward cases it may be considered sufficient if there is a detailed inspection once a year followed by a written report to the undertaker. Although external examination of covered service reservoirs, including embankments and under-drainage outfalls, together with the operation of valves can be carried out annually, internal inspections may be troublesome to arrange and a lower frequency may be acceptable, perhaps once in two or three years.

There also ought to be intermediate visits by the Supervising Engineer for more cursory inspections. When confidence is high that nothing of moment is likely to be seen, it is the author's view that 3 to 6-monthly visits should be sufficient even for large, high category reservoirs, provided that alert people are around every few days. For low category reservoirs the frequency of visiting may be more relaxed. Considerable room for opinion exists on the extent to which reservoirs should be visited by Supervising Engineers and others, but a concensus will no doubt emerge.

Detailed inspections will include close examination of the dam or reservoir structure and all the accessible ancillary structures, noting their condition and observing any structural movements or changes in the width of any cracks. The reservoir draw-off and scour valves will be operated over their full travel.

The report can usefully follow a pre-designed format if only to ensure that no aspects are overlooked. Where a reservoir is generally in order and little needs to be written about its condition, there is something to be said for preparing a check list specifically for that reservoir, for marking on each occasion. Specific reference should be made to all matters noted by the Inspecting Engineer as worthy of continuing surveillance. Recommendations for maintenance and other work to be carried out in the year ahead should be summarised.

RELATIONSHIP WITH THE INSPECTING ENGINEER

An Inspecting Engineer is appointed to make an inspection and report after which, in normal circumstances, his appointment ends. The next inspection may be up to 10 years later and may be carried out by a different engineer. In the intervening period continuity will be provided by the Supervising Engineer. It is important that the Supervising Engineer should accompany the Inspecting Engineer on his inspection, taking the opportunity to discuss the subject and to absorb his thinking as he goes about the task.

During the period between scheduled inspections the Supervising Engineer can call for an additional inspection as referred to earlier. It is however conceivable that a query may arise which does not call for a full scale inspection and can be dealt with by consultation, probably with the Inspecting Engineer who carried out the previous inspection. There is no direct reference in the Act to a consultation of this sort but it is a course of action that is likely to have appeal to the undertaker.

CONCLUSION

The innovation of the 1975 Act in introducing Supervising Engineers consolidates opinion held over many years and is a logical measure to correct a deficiency in the earlier Act. Although the additional costs involved may cause some private owners to view this development with limited enthusiasm, undertakers generally can be expected to regard it as a codification of good practice.

APPENDIX

REFERENCES IN THE ACT TO SUPERVISING ENGINEERS

S.7(5) Construction Engineer to note matters to be watched by the Supervising Engineer.

S.10(2)(c) Undertakers to arrange an inspection by an independent qualified civil engineer (Inspecting Engineer) at any time the Supervising Engineer so recommends.

S.10(4) Inspecting Engineer to note matters to be watched by the Supervising Engineer.

S.12(1) Supervising Engineer to be employed at all times when a reservoir is not under the supervision of a Construction Engineer.

Supervising Engineer to be a qualified civil engineer (as defined in S.4(1)).

Supervising Engineer to supervise the reservoir and keep undertakers advised of its behaviour in respect to safety.

Supervising Engineer to watch that the reservoir is not used for the storage of water except in accordance with a certificate of:-

- S.6(2) Construction Engineer in case of a new reservoir.
- S.6(3) Construction Engineer in case of an enlarged "small" reservoir.
- S.6(4) Construction Engineer in case of an enlarged "large" reservoir.
- S.9(2) Qualified civil engineer in case of the re-use of an abandoned reservoir.

Supervising Engineer to watch that instruments and information referred to in S.11 are being maintained and recorded respectively.

S.12(2) Supervising Engineer to watch matters noted in a final certificate for the latest report of an Inspecting Engineer and report in writing at least annually to the undertakers on his related action.

S.12(3) Supervising Engineer to recommend to the undertakers an inspection at any time he considers it is called for.

S.12(4) Enforcement authority to require the undertakers to appoint a *Supervising Engineer if it appears none has been appointed.*

S.20(4)(e) Supervising Engineer to send copy to the enforcement authority of his advice to the undertaker recommending an inspection or any other action, or drawing attention to a breach of S.6(2), S.6(3), S.6(4), S.9(2) or S.11.

S.21(3) Undertakers to notify enforcement authority of appointment of a Supervising Engineer and when appointment ceases.

S.21(5) Undertakers to afford the Supervising Engineer all reasonable facilities and provide stated information.

THE SIGNIFICANCE OF PROBLEMS AND REMEDIAL WORKS AT BRITISH EARTH DAMS

J A Charles*

Substantial civil engineering works are being carried out at many dams amongst the ageing population of British earth dams. These include works to increase overflow capacity and also remedial works associated with drawoff arrangements, embankment slope stability and internal erosion. The long term performance of earth dams is reviewed in relation to the diagnosis of causes of deterioration. The significance of rates of settlement and leakage is discussed. Problems of slope stability and internal erosion are examined and case histories described.

INTRODUCTION

The history of the provision of an adequate supply of unpolluted water to the towns and cities of Britain has been closely associated with the construction of impounding reservoirs. Figure 1 is based on data from the British section of the World Register of Dams (1) which lists dams greater than 15 m in height. Figure 1a shows that there is a large preponderance of dams of the embankment type and that there has been a fairly steady increase in the number of large dams over the last 150 years. However when the construction of large dams during the last 30 years is examined, it is found that there has been a rapid decrease in new construction from a post war peak late in the 1950s (figure 1b). In marked contrast to this downturn in new construction there appears to have been a significant increase in civil engineering works being carried out on existing dams. It is this latter trend that forms the background to this paper and some of the factors which may have contributed to it will be briefly examined.

(a) Safety standards have been reviewed. There has been a general increase in awareness of safety considerations in many different areas of society as recent Health and Safety at Work legislation bears witness. In the field of dam safety, two important developments have been associated with the Flood Studies Report (2) and the new Reservoirs Act (3). The 1975 Flood Studies Report (2) has led in some cases to revised estimates of floods that have necessitated increased overflow capacities at existing dams. Implementation of the 1975 Reservoirs Act (3) is likely to lead to closer supervision of dams and more recommendations for remedial works. The exceptional hazards posed by large reservoirs would seem to justify this increased conservatism.

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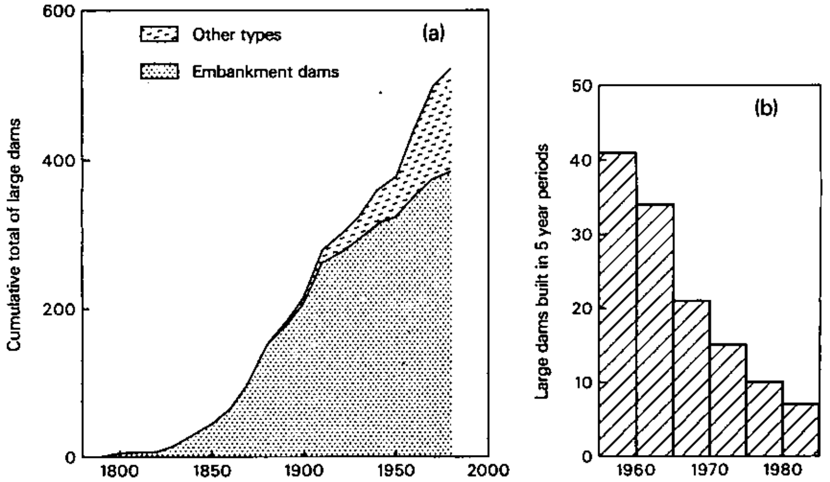

Figure 1 Large dam construction in United Kingdom (derived from World Register of Dams (1) not including tailings dams).

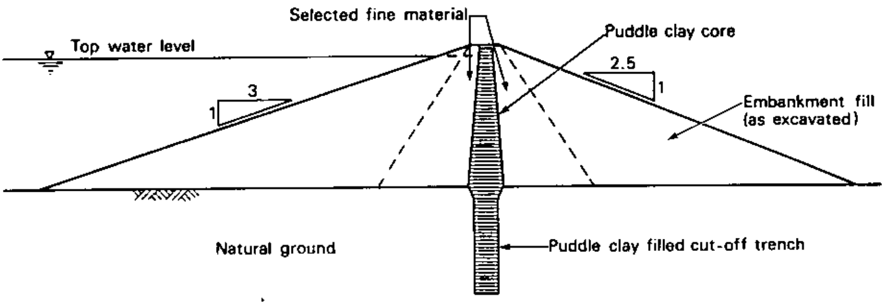

Figure 2 Typical cross-section of puddle clay core dam.

(b) The problems and risks have tended to increase. The average age of British dams is increasing. A substantial proportion of the total stock was built in the last century. The majority of dams in the United Kingdom are old earth embankments built before the theories of modern soil mechanics were widely understood and before modern heavy earth moving plant was available. Such structures may be liable to various forms of deterioration and ageing processes. Although in recent years there have been no failures involving loss of life, a number of serious incidents have shown that such fears are not groundless.

In addition to these two factors the increase in civil engineering work carried out on old earth dams may also have been assisted by the change in the pattern of ownership of many large dams. Since the reorganisation of the water industry in 1973, a substantial proportion of British dams have been in the ownership of a small number of regional water authorities with large engineering staffs. This may have encouraged a more informed and uniform approach to maintenance, improvement and remedial works.

In the appendix some 70 cases have been listed of civil engineering works carried out on old earth dams during the last 20 years and for which some published information is available. Two categories of work can be distinguished. Firstly there are works resulting from reassessment of risks and safety standards. These are mainly concerned with increasing overflow capacity and freeboard. Essentially they are improvement works. Secondly there are works resulting from the deterioration of old dams. This category can include a wide variety of works including replacement of outlet pipes, stabilisation of slopes, repair of cores and in the most extreme cases complete reconstruction. These latter types of remedial works are the subject of this paper.

LONG TERM PERFORMANCE AND DETERIORATION

As earth dams age in service, some deterioration must be expected. It is the nature, extent, rate and consequences of deterioration that need to be examined. Before doing this it is essential to have some understanding of the construction of British earth dams. Information on these historical matters has been presented elsewhere (4) (5) and only a few salient facts will be highlighted here.

For over 100 years, from the middle of the nineteenth century to the middle of the twentieth century, a fairly standard embankment design was used for most earth dams. This is illustrated in figure 2. A narrow central core of very wet puddle clay was supported by shoulders of a more granular fill. Often cohesive selected fill was placed next to the core. Where necessary a deep cut off trench was excavated and filled with puddle clay. In later dams the trench was commonly filled with concrete. It was common practice with early dams to take outlet pipes or culverts through the earth embankment. With later dams an outlet tunnel was driven through natural ground.

A recent survey of unsatisfactory performance of UK earth dams (6) indicated that the primary causes of in-service failures and serious incidents (ie. excluding construction failures) were as follows;

external erosion (overtopping)	24%
internal erosion (seepage, leakage)	55%
slips and slides	14%
other causes (eg. mining subsidence)	7%

This clearly suggested that internal erosion is the major threat to old British earth dams.

An examination of the works carried out at earth dams in the last 20 years listed in the appendix indicates that a considerable proportion of the work has been concerned with arrangements for safely passing floods. In relation to deterioration of the dams three particular types of remedial work have been commonly undertaken concerned with (a) outlet arrangements, (b) slope stability and (c) repair of cores. Table 1 summarises the major types of works carried out at the dams listed in the appendix. Obviously in many cases more than one category of remedial work was carried out and therefore the percentages add up to more than 100%. Other remedial

TABLE 1 - Analysis of main types of remedial work at 70 dams listed in appendix

Type of remedial work	No. of cases	%
Overflow structures	36	51
Drawoff facilities	15	21
Slope stabilisation	12	17
Core/membrane repair	18	26

works have included new wave walls, upstream slope protection and demolition and reconstruction.

It is interesting to compare the situation in Britain with that in Northern Ireland where, although the Reservoirs Act, 1975 does not apply, a six million pound programme of investigation and works has been carried out by the Northen Ireland Department of the Environment (7). All but 3 of the 64 publicly owned reservoirs have embankment dams and about half of these were built in the nineteenth century. For all the dams data was assembled in a standard form and an external examination was carried out. At 21 dams where there were signs of apparent deterioration, soil samples were taken and tested and piezometers were installed. As a consequence of these investigations, berms were added to the downstream slopes of 4 of the dams. Restoration of original freeboard by raising core and crest was necessary at 20 dams. Fourteen spillway systems were enlarged or reconstructed. Instrumentation was installed in a number of the more important dams.

OBSERVATION, INVESTIGATION AND DIAGNOSIS

It is desirable to detect deterioration at an early stage, and then to investigate it adequately and diagnose the cause so that effective and economical remedial works can be designed and executed. Thus the stages prior to the design of remedial works can be described as observation, investigation and diagnosis.

(i) Observation. The most important type of observations frequently may be the visual observations of the reservoir keeper or other maintenance personnel who visit the dam on a daily basis. Localised settlement and damp patches on the downstream face may be detected at an early stage. The new panel of supervising engineers (3) will also have a major role here. These visual observations should be supplemented by regular monitoring of crest settlement and leakage flows.

(ii) Investigation. An investigation of the internal condition of the embankment may be carried out because observations have indicated some form of deterioration. For example in a category A situation (reservoirs where a breach will endanger lives in a community (8)) it might be considered desirable with large dams to monitor the situation through the installation of piezometers and to carry out appropriate soil testing.

(iii) Diagnosis. In some cases this may be elementary. If the outlet arrangements for a 100 year old dam are cast iron pipes laid through the embankment, it may be considered that this alone is sufficient cause to carry out remedial work. Localised settlement may clearly point to internal erosion. Movements on the slope may indicate an incipient slip. However in some situations it may be more difficult to diagnose the cause of the problem or indeed whether there is a problem. An unusually large rate of settlement of the embankment crest or an increase in leakage might not be readily explicable and there might be a number of possible causes.

SETTLEMENT

It is relatively simple to monitor the settlement of the crest of an embankment using precise surveying techniques. It is not always easy to interpret the measurements and diagnose the cause of the settlement as settlement may occur due to a number of processes.

(i) Primary consolidation of the puddle clay. At the completion of construction of the embankment it is probable that there would be excess pore pressures that would dissipate only gradually to steady seepage values. Extended periods of reservoir drawdown could cause further primary consolidation of the core. A clay foundation would also consolidate over a long period.

(ii) Volume reduction of upstream fill on first filling. Poorly compacted fill would be liable to collapse compression when first inundated and would be likely to cause some crest settlement.

(iii) Secondary compression of puddle clay and shoulder fill. Small creep movements would continue under conditions of constant effective stress after primary consolidation was completed in the fill materials and the foundation. Major fluctuations in reservoir level will cause stress changes in fill and foundation that may significantly increase these creep movements.

iv) Slope instability. Crest settlement could be due to the onset of shear failure in one of the embankment slopes.

(v) Erosion. Where localised settlement occurs the cause may be obvious but erosion could also cause a more general form of settlement.

Settlement due to causes (i) and (ii) should be completed during the early years of the dam's life. It is clearly important to determine whether settlements measured many years after the completion of a dam can be attributed to cause (iii) or whether serious problems such as cause (iv) or (v) are indicated. To do this it is necessary to obtain some indication of the magnitude of the crest settlement that might be attributable to secondary compression in the puddle clay and creep in the embankment shoulders and foundation.

For clay soils a coefficient of secondary compression (c_α) is defined by

$$c_\alpha = \frac{\Delta\varepsilon_v}{\log (t_2/t_1)} \qquad (1)$$

where $\Delta\varepsilon_v$ is the vertical compression of a soil sample under constant effective stress in one-dimensional compression between times t_2 and t_1 after the load was applied. For normally consolidated clays values of c_α typically range between 0.005 and 0.02. Limited information on remoulded puddle clays indicates a range between 0.001 and 0.004. The results of an oedometer test on a puddle clay are plotted in figure 3. Organic soils and highly plastic clays can have larger values. Granular fills can exhibit similar creep compression and for poorly compacted fills the logarithmic creep rate is typically between 0.002 and 0.007. As the soils in the field are not laterally restrained to the same degree that they are in an oedometer and as some stress changes will occur in the field due to fluctuations in reservoir level, it can be expected that settlement in the field might be a little larger than that indicated by laboratory tests. (There is evidence that major drawdown of a reservoir can cause significant settlement.) Consequently if we define a settlement index (S_I) analagous to c_α such that,

$$S_I = \frac{s}{1000\ H \log(t_2/t_1)} \qquad (2)$$

(where s is the crest settlement in mm measured between times t_2 and t_1 since the completion of construction of the embankment at a section of the dam H metres high) then we might expect that likely values of S_I would range between 0.003 and 0.01. Some typical values measured at dams which are apparently behaving quite satisfactorily are given in table 2.

TABLE 2 - Crest settlement of some central core dams

Dam	Height m	Date completed	Period of measurement	Settlement index S_I	Clay core
Cwmwernderi	22	1901	1981-85	0.009	puddle
Challacombe	15	1945	1981-85	0.008	puddle
Selset	38	1959	1959-70	0.004	puddle
Llyn Brianne	90	*1971*	*1971-85*	*0.004*	*rolled*

The settlement of Selset during the first 11 years following the completion of construction has been derived from Bishop and Vaughan (9) and Penman (10). The general embankment fill at Selset would have been much better compacted than in the earlier puddle core dams. Llyn Brianne dam, a rockfill dam with wide central rolled clay core, has been included in the table for comparative purposes.

It is suggested that where measurements of crest settlement give $S_I > 0.02$ the possibility of some other mechanism besides creep causing the settlement should be seriously considered if the large value cannot be accounted for in some obvious manner such as continuing primary consolidation of a deep clay foundation.

SEEPAGE AND LEAKAGE

It might be expected that measured rates of seepage and leakage through earth dams would give a good indication of condition and performance. However there can be considerable difficulties. If seepage rates are calculated for puddle clay core dams based on the low measured values of permeability of puddle clay (circa 10^{-10} m/s), then very small rates are obtained. Even for a 30 m high dam they would typically be smaller than 0.01 litre/s. Actual leakage rates are of course generally much greater than this. In some old dams there is no facility for collecting and measuring leakage. Even where such facilities do exist it may not be obvious where the water is coming from. Generally leakage will be influenced by both reservoir level and rainfall and this can make leakage measurements difficult to interpret. Nevertheless an increase in leakage rate that cannot be related to an increase in reservoir level or rainfall should be regarded as significant evidence that a serious problem may be developing as, of course, must the appearance of turbid water.

Leakage observations have often provided evidence that serious problems have developed. After the 24 m high Lower Lliw dam was completed in 1867 it was found that leakage varied between 1.4 and 2.9 litre/s depending on the rainfall (4). However in 1873 turbid water flowed from the downstream drains at a rate of 26 litre/s and there was visible settlement of the embankment. Similar problems had occurred at the 22 m high Grizedale dam completed in 1866 (11). In 1867 water in a spring at the foot of the embankment became muddy and leakage was estimated at 45 litre/s. In both the above cases major repairs were required to the cores. Kennard has described how grouting the core of the 11 m high Lower Slade dam some 70 years after it was built reduced the leakage from 5 litre/s to less than one third of this value (12). After the reservoir was filled at 48 m high Balderhead dam, leakage increased to 60 litre/s (13). On refilling subsequent to repairing the rolled clay core the leakage was less than 10 litre/s.

Leakage is unlikely to be so excessive that the ability of the reservoir to impound water is seriously affected. Usually the main concern is that leakage will lead to internal erosion of the embankment or its foundation. It could also lead to high pore pressures and instability in the downstream shoulder.

SLOPE STABILITY

The survey of unsatisfactory in-service performance of UK earth dams (6) indicated that only 14% of the cases were due to slips and slides. This may be because the stability of puddle core dams is generally most critical at the end of construction. Consequently stability failures occurred more frequently during construction than subsequently with the reservoir impounded. Nevertheless stability failures have occurred during the working life of puddle core dams and present a serious hazard to reservoir safety. Slips may be located entirely within the embankment fill or pass through both embankment and foundation. Post construction upstream slope stability is usually at a most critical stage either during first filling the reservoir or during a subsequent rapid drawdown. In both cases the slip will occur with the reservoir significantly below top water level and the embankment is unlikely to be breached. A failure of the downstream slope is potentially much more serious. Downstream slopes of British earth dams are typically 1 vertical in 2.5 horizontal. The outer material is often more granular in character than the fill next to the puddle clay but this is not necessarily so. It would not usually have been very well compacted.

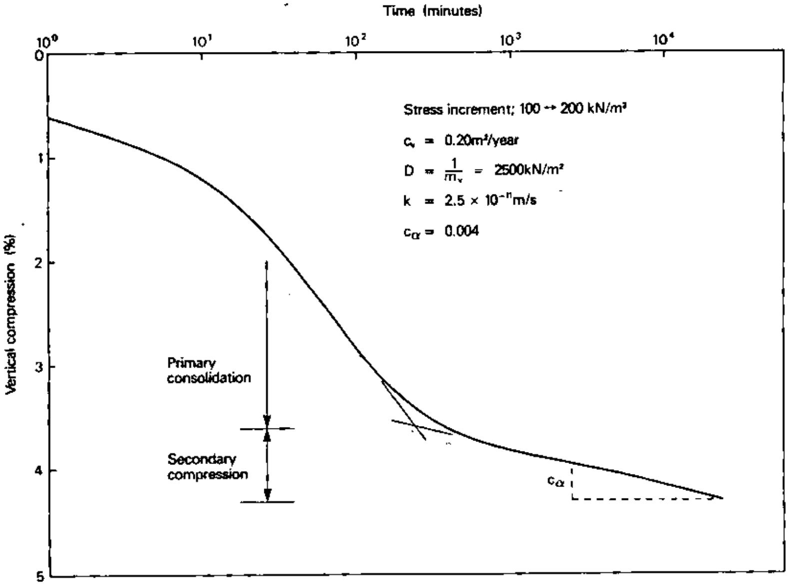

Figure 3 Consolidation of puddle clay in oedometer (PL = 24%, LL = 44%, w = 36%).

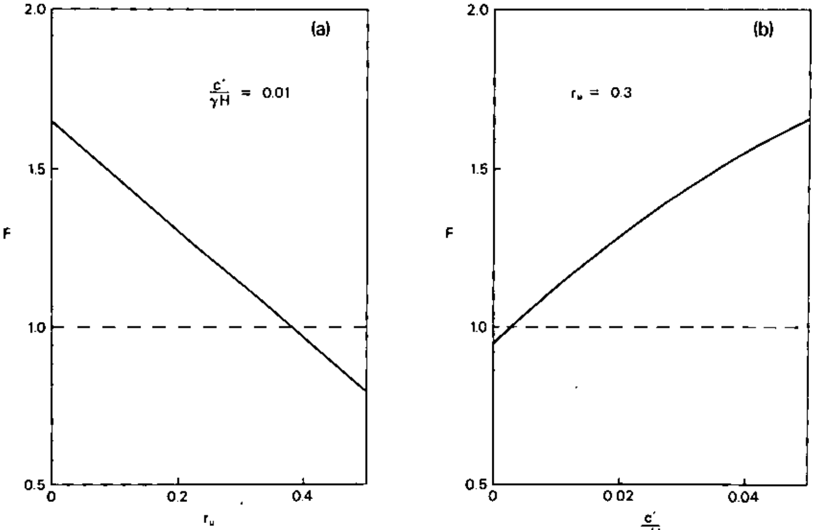

Figure 4 Stability of 1 in 2.5 slope in soil with ϕ' = 30° built on strong foundation (derived from (18)).

Measured values of shear strength parameters for some downstream fills are quoted in table 3. Typically the angle of shearing resistance ($\emptyset'$) is about 30° and there may be a small cohesion intercept (c'). Figure 4 is based on the Bishop and Morgenstern stability charts and illustrates how the factor

TABLE 3 - Shear strength of embankment fill

Dam	Date completed	Height m	Downstream slope	Shear strength parameters			Ref
				c' kN/m^2	$\emptyset'$ degrees	$\frac{c'}{\gamma H}$	
Boddington	1820	6	1:2.5	0.3	31	0.003	(14)
Burnhope	1936	40	1:2.5	0	35	0	(15)
Brent	1835	7	1:2.5	8	26	0.06	(14)
Combs	1805	16	1:1.6	9 9	31 24	0.03	(16)
Stanford	1928	10	1:2.5	5	28	0.03	(17)
Tringford	1829	7	1:2.0	0	33	0	(14)

of safety of a 1 in 2.5 slope decreases as the pore pressure ratio (r_u) increases. (r_u is the ratio of pore pressure to overburden pressure.) With zero pore pressures in the downstream slope of a fill with $\emptyset' = 30°$ and $\frac{c'}{\gamma H} = 0.01$, F = 1.65. However an increase in average r_u value to 0.38 reduces F to unity. Thus the stability of the downstream slope may be critically affected by a rise in pore pressure due to leakage through or around the core. Figure 4b illustrates the significance of the cohesion intercept on the calculated factor of safety. This makes stability particularly difficult to assess as the cohesion intercept is notoriously difficult to determine reliably.

Where the stability of a slope is not considered to be adequate either because the calculated factor of safety is close to unity or because signs of incipient stability failure have been observed, then remedial measures must be undertaken to increase the stability. These can be divided into two categories; firstly drainage measures to reduce pore pressures in the downstream fill and secondly fill placement to flatten the slope or weight the toe of the embankment. Where a reservoir is of little economic significance it may of course be preferable to remove the dam.

In November 1984 a shallow slip occurred at the downstream toe of Lambieletham dam and led to an emergency drawdown of the reservoir. The dam was located near St Andrews and had been built in 1899. It had a puddle clay core, an upstream slope of 1 in 3 and a downstream slope of 1 in 2.5. It had a maximum height of 15 m. The embankment was subsequently demolished in October 1985 and the Building Research Establishment was permitted to investigate the embankment during demolition.

It was found that instability of the lower part of the downstream slope was due to high pore water pressures within the downstream fill. However the puddle clay core appeared to be in good condition and no evidence of leakage through the core was found. The undrained shear strength, moisture content, plastic and liquid limits are plotted against depth below crest in figure 5. The undrained shear strength was measured both by field vane tests with a 20 mm x 40 mm vane and by laboratory undrained triaxial compression tests on 38 mm diameter samples sheared at an axial strain rate of 0.5% per minute. The vane tests gave values of undrained shear strength about 10 kN/m² greater than the triaxial tests. Both sets of results indicated an increase in strength with depth of between 1 and 2 kN/m² per metre depth. The clay was of low to intermediate plasticity and the results plot above the A-line on a plasticity chart. The particle size distribution showed 31% of clay size particles and 32% silt size. The strength and plasticity properties of the puddle clay were fairly typical of a puddle core in good condition (5). There was no evidence of the extreme variability in properties that can characterise puddle cores that have been subjected to erosion.

Another possible source of water in the downstream fill was leakage around the scour pipe or from a fracture in that pipe. Again no evidence of this was found. The scour pipe had been embedded in a concrete block where it passed through the clay core. When this was excavated during demolition, some water issued from under the block. However much larger volumes of water were found to be coming from the north west valley side into the downstream fill and it was concluded that it was this water that had saturated the lower half of the downstream shoulder of the embankment. It is noteworthy that remedial works had been carried out in 1934 to prevent leakage at the north west end of the dam. However leakage continued and from 1941 to 1969 the reservoir was kept 2.2 m below top water level.

The downstream fill material was very variable but much of it could be classified as very silty sand. Undisturbed samples were not obtained but drained triaxial compression tests on reconstituted samples in a loose condition gave $c' = 5$ kN/m², $\phi' = 30°$. A stability analysis based on these strength parameters and the observed pore pressures gave a factor of safety of 1.28 for the downstream slope. However the value of the factor of safety was strongly dependent on the cohesion intercept (c'). A significant reduction in c' could reduce F to unity. With zero pore pressures in the downstream fill the factor of safety would probably be greater than 1.5.

EROSION AND CORE REPAIRS

Only 26% of the dams listed in the appendix have had repairs to their watertight elements. Considering the high incidence of problems connected with internal erosion (6) this may be a little surprising. The repair of an eroded puddle clay core is one of the most difficult types of remedial work. Where a core has settled, raising its level should not be unduly difficult. However where a core has been subject to internal erosion at depth, repairs will be both expensive and difficult. In earlier years repairs were attempted by deep excavations in timbered trenches and shafts, followed by repuddling. Lower Lliw was repaired in 1879 in this manner (4). Similar work was also carried out at Grizedale (11). In more recent years the approach has been to adopt some form of remedial grouting or to replace the core with a new diaphragm of plastic concrete using slurry trench techniques. Four methods can be distinguished.

(a) Injection grouting eg. Lower Slade, constructed 1900, repaired 1971 (12); Greenbooth constructed 1961, repaired 1983 (19); Walshaw Dean Lower constructed 1907, repaired 1982 (5).

(b) Thin grout screen installed by injecting grout into void formed by vibrating I section beam into puddle clay core eg. Banbury constructed 1903, repaired 1973 (20).

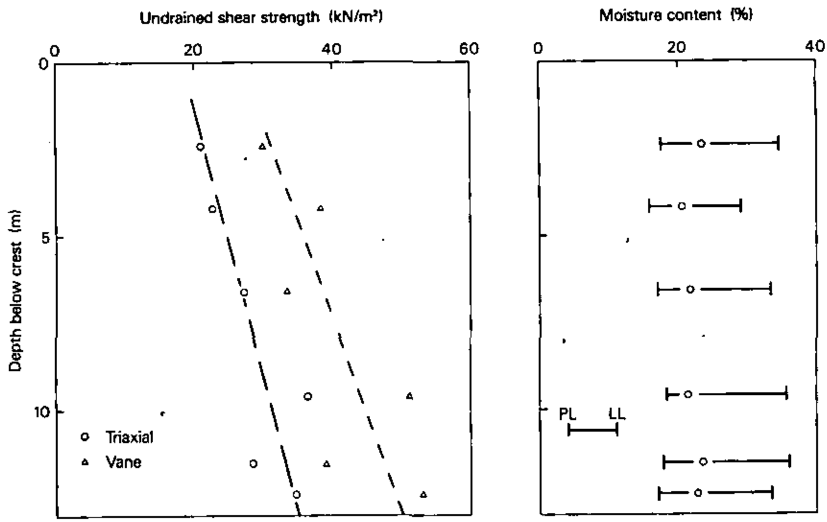

Figure 5 Properties of puddle clay core at Lambieletham dam.

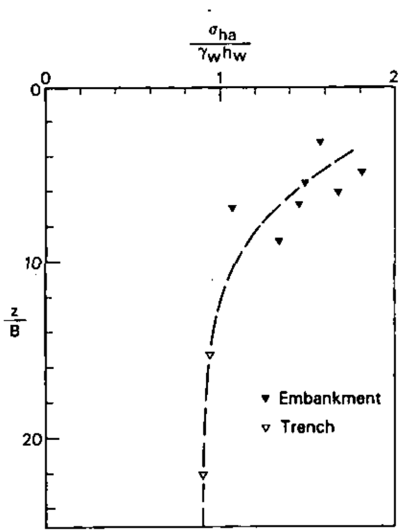

Figure 6 Susceptibility to hydraulic fracture of puddle clay core dams (measurements in puddle clay trench were made at Walshaw Dean Lower).

(c) New core of self-hardening bentonite cement slurry eg. Doffcocker Lodge constructed 1870, repaired 1985 (21).

(d) New core of plastic concrete using slurry trench technique eg. Lluest Wen constructed 1896, repaired 1972 (22); Withens Clough constructed 1894, repaired 1972 (23).

The repair of Balderhead dam (13) in 1968, only four years after its completion, included both grouting of the core and a new core of plastic concrete in the badly eroded area. It should be noted however that Balderhead had a rolled clay core not a puddle clay core.

In considering the repair of puddle clay cores several questions need to be answered. Does the core need to be repaired and if so will it be sufficient to grout the core or should a new core of plastic concrete be provided? What mixes should be used for injection grouting and for plastic concrete? The satisfactory long term performance of such remedial works will depend on properties of the repair materials such as strength, deformability, permeability, density, erodibility and their compatibility with the surrounding puddle clay. With some techniques the required properties of the repair materials may be largely governed by problems of placement rather than long term performance. Beier and Strobl (24) have studied the erodibility of various types of material used for cut off walls. A preliminary test carried out at the Building Research Establishment on a mix for a self hardening bentonite cement slurry showed permeability increasing from an initial value of 1×10^{-8} m/s to 2×10^{-7} m/s over a 50 day period under a low hydraulic gradient. There is a great need for well documented case histories of the repair of puddle clay cores which include information about long term performance subsequent to repair.

Hydraulic fracture of the core by the reservoir water pressure may be a mechanism which leads to erosion of puddle clay cores. The Building Research Establishment has carried out investigations of a number of dams to determine the total stresses and pore pressures within puddle clay cores (5). It has been proposed that the ratio $\frac{\sigma_{ha}}{\gamma_w h_w}$ can be used as an index of susceptibility to hydraulic fracture (where σ_{ha} is the horizontal earth pressure in the direction along the axis of the dam, $\gamma_w h_w$ is the full reservoir head at that depth). It would seem unlikely that reservoir water pressure could cause hydraulic fracture when σ_{ha} is greater than $\gamma_w h_w$ (this assumes that the vertical stress is larger than the horizontal stress). Figure 6 shows some correlation between $\frac{\sigma_{ha}}{\gamma_w h_w}$ and core geometry represented by $\frac{z}{B}$ where z is the depth below crest and the core width is 2B. It is clear that hydraulic fracture is particularly likely in deep narrow cutoff trenches where $\frac{z}{B}$ is very large. Where hydraulic fracture does occur, the seriousness of the situation will depend partly on the erodibility of the puddle clay and also on the filter properties of the fill or foundation downstream of the core.

Walshaw Dean Lower dam, located north west of Hebden Bridge, is the lowest of a group of three reservoirs. The three dams were constructed between 1901 and 1907. Walshaw Dean Lower has a maximum height above foundation level of 22 m, the upstream slope is 1 in 3 and the downstream slope 1 in 2. The central puddle clay core extends below rock head in a very deep 3 m wide puddle clay filled cut-off trench which terminates 40 m below crest level.

It is believed that problems arose soon after impounding began and that there was major loss of material. Bowtell (25) stated that in February 1908 leakage from the lower reservoir increased progressively and the reservoir was emptied. There appears to have been a series of leaks and repairs between completion of construction in 1907 and final commissioning in 1915. It is believed that 1 m of settlement has occurred since the end of construction. A settlement rate of more than 1 cm per year indicated little reduction in the rate of settlement. This corresponds to a settlement index $S_I = 0.08$ based on the 22 m height of the embankment. This is much larger than could be accounted for by secondary compression and creep in the fill materials.

A site investigation was carried out in late 1980 and boreholes in the clay core showed some softening of the clay below original ground level. There was evidence of small lenses or water paths through samples around which the clay was very soft. Borings through the downstream foundation showed that the cut-off trench had been excavated through a stratum of highly fissured sandstone. The puddle clay plotted above the A-line on the plasticity chart and varied from intermediate to high plasticity. The particle size distribution of a sample from 30 m below the crest had a clay fraction of 44%, a silt fraction of 35% and a coarse fraction of 21%.

In 1982 remedial grouting was carried out. A cement grout was used in the rock foundation immediately downstream of the cut-off trench and a cement bentonite grout in the core. The reservoir water level was drawn down for most of 1982. Before grouting took place spade-shaped total earth pressure cells were installed in the puddle clay filled cut-off trench at a section where the embankment had a height of only 13 m above original ground level and the clay filled trench terminated 40 m below the crest (5). Cells with associated pneumatic piezometers were installed along the centre-line of the trench at 23 m, 28 m and 33 m below crest level. The cells at 23 m and 33 m measured σ_{ha}, the cell at 28 m measured σ_{hud} (horizontal earth pressure in upstream/downstream direction). Grouting in the proximity of the cells did lead to an increase in the horizontal pressure but this increase soon decayed away and there was little evidence that grouting had permanently increased the horizontal earth pressures in the clay filled trench. On completion of the remedial grouting, the reservoir was refilled. The measured horizontal earth pressures were smaller than the pressure due to the full reservoir head (fig 6). This indicated that the puddle clay was still susceptible to hydraulic fracture but, if the grouting of the rock foundation downstream of the trench was effective, erosion should be halted.

CONCLUSIONS

1. Remedial and improvement works to the ageing population of British earth dams are now subjects of considerable importance involving both major expense and public safety.

2. A substantial proportion of the works carried out in the last few years has been related to the reassessment of safety standards in relation to flood estimation and overflow capacity.

3. In future a greater proportion of civil engineering work may be remedial work associated with the ageing and deterioration of the earth dams and in particular drawoff works, slope stability and core repairs.

4. It is essential that deterioration is detected at an early stage; frequent visual inspections and regular monitoring of settlement and

leakage are particularly important. The new panel of supervising engineers will have a major role.

5. Diagnosis of the nature of the problem may be difficult eg. crest settlement could be due to a variety of causes. It is useful to compare measured crest settlement with what could reasonably be attributed to secondary compression of core and fill materials. The comparison is assisted by the use of a logarithmic settlement index (S_I).

6. Downstream slope stability may depend critically on pore pressures. Leakage could therefore seriously affect slope stability. In high risk situations it may be prudent to carry out soil strength tests and install piezometers to monitor pore pressures so that stability can be evaluated.

7. Hydraulic fracture by reservoir water pressure may initiate the erosion of puddle clay cores. Deep clay filled trenches may be particularly susceptible. The repair of eroded puddle clay cores is both expensive and difficult. Research is needed into the effectiveness of remedial measures.

ACKNOWLEDGEMENTS

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APPENDIX

REMEDIAL WORK ON SOME BRITISH EMBANKMENT DAMS, 1966-1985.

Key

Date of work:	c (date) = circa
	(date) c = contract let
	(date) t = tender stage
Cost index:	all prices adjusted to 1985 pounds.
	a = less than £100 000
	b = £100 000 - £250 000
	c = £250 000 - £500 000
	d = £500 000 - £1000 000
	e = more than £1000 000
References	vn = notes provided on BNCOLD visit
	pc = private communication

Dam	Location	Height m	Date built	Remedial work			
				Description	Date	Cost index	Ref.
Aldenham	Elstree	8	1795	Sections of upstream and downstream slopes stabilised following slips	1976		26
Auchengaich	Helensburgh	28	1945	Embankment regraded	1977		1
Balderhead	Darlington	48	1965	Plastic concrete diaphragm wall constructed in zones of core damaged by erosion; elsewhere core grouted.	1968		13
Banbury	Lea Valley, north London	10	1903	Thin grout screen constructed in existing core	1972 -73	b	20
Barrow no. 1	Bristol		1850	Wave wall heightened; access track upgraded to prevent scour	1982	a	27
Barrow no. 3	Bristol	17	1897	Overflow constructed; toe drains and leakage measurement installed	1984	a	27
Barrow Compensation	Bristol	12	1864	Spillway reconstructed with overflow cill lowered; works to seal leakage and refurbish drawoff shaft.	1982	b	27
Blaenant Ddu	Swansea	24	1878	Demolished	1978		28
Brushes	Oldham	20	1870	Raising of crest and puddle clay core; construction of wave wall; refurbishment of valve shaft; enlargement of spillway.	1985t		29a
Buckieburn	Denny	23	1905	Rockfill toe wall constructed and slope flattened following downstream slip; new weir and spillway constructed	1970 -71	c	30
Calf Hey	Blackburn	20	1859	Modification of overflow; reconstruction of wave wall; replacement of section of drawoff tunnel and pipes within existing tunnel	1984t		29b
Castle Howard Great Lake	Malton	6	1798	Leakage repaired by excavation and clay puddling	1967		12
Celyn	Bala	51	1965	Riprap added to upper sections to replace and reinforce original rock which was weathering rapidly			31
Chew Magna	Bristol	9	1850	Auxiliary overflow constructed and embankment raised by 1 m following overtopping in 1968			27
Clubbiedean	Edinburgh	17	1850	Embankment and foundation grouted	c 1975		4
Clydach	Pontypridd	9	1900	4 trenches dug in downstream toe and filled with filter material, following discovery of high water table	1970		32
Combs	Chapel-en-le-Frith	16	1805	Rockfill buttress placed on downstream slope, following slip; wave wall built	1982 -83		16
Coulter	Biggar	26	1908	Puddle clay core and foundation grouted	1976 -77		pc
Cowlyd	Dolgarrog	14	1920	Rockfill placed over 115 m length of downstream slope	1974		33
Cwmtillery	Ebbw Vale	10	1895	Spillway replaced, culvert lined, core and foundations grouted following mining subsidence	1972		32

Dam	Location	Height m	Date built	Remedial work			
				Description	Date	Cost index	Ref.
Daer	Dumfries	42	1956	Part of upstream slope refaced; overflow works reconstructed	1980 -85	e	34
Daff	Greenock		1921	Auxiliary overflow constructed; new wave wall built	1982	b	34
Deep Hayes	Leek	18	1848	Dam lowered	1981	d	35
Doffcocker Lodge	Bolton	7	1870	Diaphragm wall constructed using self hardening bentonite cement slurry	1985	a	21
Dowdeswell	Cheltenham	12	1886	Spillway reconstructed	1985		vn
Dunoon no. 2	Dunoon	10	1915	Breached	1982	a	34
Earlsburn (2)	Stirling			Reconstruction of overflow channels; repairing and strengthening embankments	1982c	d	36a
Earnsdale	Blackburn	24	1863	New spillway constructed; uPVC pipe threaded through cast iron outlet pipe	1976	d	37
Elsecar	Barnsley			Construction of new spillway	1985c	b	36b
Elslack	Skipton	22	1931	Dam and spillway reconstructed	1972		1
Greenbooth	Rochdale	35	1963	Foundations, shoulders and core grouted following depression in crest	1983	c	19
Greenfield	Oldham	20	1902	Gated auxiliary spillway constructed	1980 -82		vn
Greenside	Clydebank	15	1897	New overflow works constructed; wave wall built	1983 -84	b	34
Hewenden	Bradford	12	1840	Cracks in overflow tunnel filled with cement grout	1971		37
Hollingworth	Glossop	21	1854	Top water level lowered	1968		1
Hollingworth Lake	Rochdale	11	1800	Fill placed to improve upstream and downstream slope stability; overflow works reconstructed; drawoff systems improved	1985		29c
Jackhouse	Blackburn	20	1869	Breached	1984		vn
Kielder	Newcastle-upon-Tyne	52	1982	Concrete facing blockwork repaired, following storm damage	1984		38
Kinder	Stockport	29	1912	Deepening of spillway channel and enlargement of discharge culvert	1985t		36c
Kype	Strathaven	17	1904	Overflow works improved; wave wall constructed	1982	b	34
Lambieletham	St Andrews	15	1899	Demolished	1985	a	pc
Litton (Upper and Lower)	Bristol			Overflow works reconstructed	1983 -84	d	27
Lluest Wen	Maerdy	25	1892	Plastic concrete diaphragm wall constructed; overflow works reconstructed; intake and outlet pipes renewed; new rockfill toe placed.	1971 -73	e	22
Llugwy	Dolgarrog	15	1920	Reconstructed	1975		1
Logan	Strathaven	17	1901	Overflow works reconstructed	1978		1

Dam	Location	Height m	Date built	Remedial work			
				Description	Date	Cost index	Ref.
Lower Lliw	Swansea	27	1867	Dam demolished and completely reconstructed	1976 -78	e	4
Lower Roddlesworth	Blackburn	25	1857	New valve shaft constructed;, tunnel lined; new valves installed	1972 -73	d	39
Lower Slade	Ilfracombe	11	c 1900	Core and foundation grouted	1971		12
Megget	Selkirk	56	1982	Displaced riprap replaced	1984	a	29d
Mitchell's House	Accrington	20	1868	Installation of vertical drainage system	1985t		29e
Mixenden	Halifax	14	1873	Construction of syphon overflow and outfall structures; construction of culvert	1985t		36d
Munnoch	Irvine		1877	Overflow lowered by 1.65 m	c 1980	a	34
Neuadd Lower	Merthyr Tydfil	16	1884	Improvements to overflow	1985		40
Pick-Up Bank	Lancashire	20	1849	New overflow constructed down middle part of embankment; polyethylene pipe threaded through existing cast iron drawoff pipe	c 1975		37
Rake Brook	Blackburn	26	1857	New valve shaft constructed, tunnel lined; new valves installed	1972 -73	d	39
Redmires Upper	Sheffield	16	1854	Outlet pipes lined	c 1977		vn
Stanford	Rugby	10	1928	Crest and downstream slope reinforced with revetment mattresses; drawoff and tunnel repaired; overflow reconstructed	1983 -84	c	17
Tamar Lake	Cornwall	8	1825	Drainage layer, berm and relief wells installed	1970		12
Toddbrook	Whaley Bridge	21	1840	New overflow constructed over section of embankment; new wave wall built; embankment and foundation grouted			pc
Torduff	Edinburgh	26	1851	Remedial work following settlement of slopes attributed to settlement of clay filled trench	c 1981		4
Upper Carno	Ebbw Vale			Spillway widened; clay core and embankment raised; new wave wall constructed	1984 -85	b	pc
Walshaw Dean Lower	Hebden Bridge	24	1907	Core and foundations grouted	1982		5
Warley Moor	Halifax	12	1872	Remedial work to overflow works, replacement of pipes and valves, raising level of north embankment	1985 c		36e
West Corrie	Kirkintilloch	17	1895	Drainage installed in downstream slope	c 1972		41
Winscar	Barnsley	52	1975	Foundation grouted, minor crack in asphaltic membrane repaired	1977 -79		31
Withens Clough	Halifax	22	1894	Plastic concrete diaphragm wall constructed; new drawoff tunnel and shaft constructed	1971 -72	e	23
Worsbrough	Barnsley	8	1804	New overflow and drawoff works constructed	1981 -83	d	42
Yeoman Hey	Oldham	21	1880	Gated auxiliary spillway constructed	1980 -82		vn

Bill McLeish

- HK inspections - 1930/1975 Act ?
- JTS relations - exclusive?
- overseas experience

Ted Haws

- SR v NI process
- overseas experience