Reservoir safety and the environment

Proceedings of the eighth conference of the British Dam Society held at the University of Exeter, 14 – 17 September 1994

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Organizing Committee: C. Beak, Dr J.A. Charles, J. Hinks, Dr A.K. Hughes, A. Macdonald (Chairman) and G.A. Yarwood

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Preface

The biennial conferences of the British Dam Society have a central role in the work of the society. Following the successful conference held at Stirling in June 1992, it was decided to hold the eighth conference at Exeter in September 1994. Two key issues of concern to the dam engineering community in the United Kingdom and worldwide are addressed at this conference, namely reservoir safety, particularly related to existing structures, and environmental aspects of existing and proposed schemes.

As the average age of the worldwide stock of dams increases, more emphasis is being placed by the engineering profession on the requirements for safety monitoring, assessment and remedial works to existing structures. Since the 1930s, the United Kingdom has had legislation covering reservoir safety, and setting standards for the monitoring and inspection of structures. Papers contained in these proceedings summarise recent experience in safety analysis and remedial works on both embankment and concrete dams.

Natural hazards are of particular relevance to reservoir safety and a number of papers review the impact of floods or seismic events, not only on the main dam structure and its ancillary works, but also on the area downstream which would be affected by a dam failure. The importance of hazard ranking and contingency planning for disaster scenarios is discussed.

The need to appreciate both positive and negative influences of a reservoir on the environment was emphasised by Dr W. Pircher, President of the International Commission on Large Dams, when he presented the Geoffrey Binnie lecture at the seventh BDS Conference. A session of the eighth conference is devoted to environmental aspects of reservoir construction and management. A number of papers describe experience worldwide on topics such as ecology, sedimentation, socio-economic influences, and leisure use of reservoirs.

The papers in this book cover a diverse range of subjects in the field of reservoir engineering, and should be of value to all involved in reservoir design and construction, in the United Kingdom and worldwide.

A Macdonald
Chairman
Organising Committee

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The asphalt membranes at Colliford and Roadford Reservoirs

J.D. EVANS, BScTech, FICE, FIWEM, Resident Engineer, South West Water Services plc, and A.C. WILSON, MICE, MIWEM, Principal Engineer, Babtie Shaw & Morton

SYNOPSIS
The paper refers to the asphaltic concrete membranes constructed on the upstream face of two dams in south-west England and describes their design and construction and performance and maintenance experiences to date.

INTRODUCTION

1 There are only six dams in the United Kingdom which have asphaltic concrete membranes as the waterproofing element on the upstream face. Two of these are South West Water's strategic reservoirs at Colliford in Cornwall (Johnston and Evans 1985) and Roadford in Devon (Johnston, Wilson and Evans 1994). Colliford, impounded in 1983, stores 29,000 Ml of water while Roadford completed in 1989 has a capacity of 37,000 Ml and together they provide 55% of the total raw water storage in the region.

2 The embankment at Colliford is 30m high and 520m long and is constructed of about 183000m^3 of sandwaste, a residue from a china clay pit 2km from the site. Roadford's embankment is 40m high and 430m long, built of 1,000,000m^3 of low grade rockfill which was quarried from the reservoir area upstream of the dam (Wilson and Evans 1990).

3 The overall area of the membrane at Colliford was 8850m^2 and that at Roadford 22350m^2 (Duncanson and Johnston 1988). Specialist sub-contractors were used to supply and lay the asphalt on the dams, Bitumarine from Holland at Colliford and a consortium of Alfred McAlpine Quarry Products and Walo Bertschinger A.G. from Switzerland at Roadford. At both sites the cost of membrane construction was about £53/m^2 at 1993 prices.

4 Membranes were chosen because at both locations there is a lack of suitable clay to form an impermeable core. They have further advantages:-

i) obviation of expensive armouring of the upstream face.
ii) minimum placing costs resulting from forming the embankment with a single material.

Additionally at Colliford the membrane prevents pollution of the reservoir from kaolin being leached from the sand fill. At Roadford, the membrane

Colliford Dam

Roadford Dam

Figure 1 Connections to cut-off

prevents acid from the oxidation of pyrites in the fill polluting the reservoir and keeps the embankment as dry as possible thereby minimising long term reduction of strength due to degradation.

DESIGN AND CONSTRUCTION

5 At both dams the membrane consists of two layers of bituminous material laid over a drainage blanket placed on the face of the embankment fill. The lower binder layer is 70mm thick and provides a foundation for the 80mm thick dense course. While the dense layer is virtually impermeable, the binder layer has a permeability greater than 1×10^{-4}m/s to ensure that any seepage through the dense layer reaches the underlying drainage blanket. Additional reinforcement was provided at the connection to the upstream toe. The membrane is completed by a seal coat to provide protection against ultra violet light. (Fig 1)

6 The performance criteria for the membrane was specified at both dams and it was the responsibility of the contractor to design the asphaltic mix. A wide variety of British, American and Continental standards were used to ensure that the asphalt met the critical requirements of flexibility, slope stability, permeability and durability. At Colliford the Von Asbeck test apparatus was used in the laboratory to demonstrate that the asphalt remained impermeable under water pressure when subjected to a deflection of 50mm in the centre of a 500mm circular sample. At Roadford a deflection of 60mm over a 180mm length in an asphalt slab showed the flexibility of the material and its ability to cope with shear without cracking. The bitumen content of the dense material was 4.7% at Colliford and 4.0% at Roadford.

7 The seal coat used at Colliford was a two layer Flintkote cold emulsion applied by spray to give a surface with an "orange peel" finish. The first layer contained mineral fibres which helped to fill small interstices on the face of the asphalt. At Roadford a hot applied mastic was laid from a trolley car equipped with a squeegee spreader. 33% of polymer modified bitumen with 67% of filler was used in the mastic to reduce the problem of "elephants noses" (runs) and "elephant skin" (surface tension cracking). However a shallow crazing pattern is apparent on the final surface.

8 The area close to the connection with the concrete cut-off, where the highest differential settlements may be expected, is likely to be the most critically stressed. The top surface of the concrete structures has a groove and a large surface area to give good contact with the asphalt to ensure a satisfactory seal. The top curve of the structure and its inclined downstream face helps to reduce differential settlement in this location.

9 Only very small settlements occurred at Colliford where sandwaste had been used for construction. At Roadford a zone of the same material was placed between the toe structure and the rockfill. As the sandwaste has a compressibility of about one third of that of the rockfill it forms an ideal transition zone to minimise strain on the asphaltic membrane. The inclined

Figure 2 Upstream toe details

downstream face of the toe structures made it possible for large vibrating rollers to achieve satisfactory compaction close to the face.

10 To give added protection at the toe, a Hypofors bitumen sheet, reinforced with nylon and polyester fabrics, was sealed to the face of the asphalt at Colliford. At Roadford an additional 50mm layer of dense asphalt was used which contained 0.5% more bitumen to allay concern that the extra thickness would stiffen the membrane in the area where the greatest flexibility was required.

11 At both sites the asphalt was mixed in a mobile plant and laid by a paver winched up and down the face from a crest machine which carried the paver during longitudinal movements. At Colliford where there is a shorter slope due to the smaller dam height and greater inclination (IV:2H), the paver was fed by crane and skip. At Roadford a separate tipping trolley was used on the IV:2.25H slope to deliver asphalt from the crest machine to the paver.

12 Construction at Colliford took place during the cold and showery Spring of 1983 and this caused some problems at times in ensuring that the asphalt was placed at optimum temperature. The asphalt was laid in 28 working days at an average rate of 615m^2/d. The membrane was completed in a three month period. In contrast the membrane at Roadford was laid at an average rate of 1850m^2/d during the warm dry Spring of 1989. The good weather was a significant factor in the overall completion time of just over 2 months.

13 Testing to ensure that the asphalt had been compacted to below the required standard of 3% air voids was carried out with a nuclear densimeter. At Colliford over 800 tests indicated an average air void content of 1.6% while at Roadford 2546 tests gave an average of 1.1%.

IMPOUNDING

14 Impounding at Colliford commenced in July 1983 and the reservoir overflowed for the first time in August 1986. Overflow continued until May the following year and from January to May in 1988 and 1989; since then the water level has remained below the spillweir. Roadford was impounded in October 1989 and the reservoir rose to within 150mm of the spillweir in May 1991, the highest level reached to date. No overflow has occurred, primarily, because surplus water has been released from the reservoir through the hydrogeneration facility to take advantage of the current non-fossil fuel tariff.

15 At both Colliford and Roadford any seepage through the membrane is collected in the underlying drainage layer and discharged into the inspection gallery where flow can be monitored (Fig 2). In addition at Colliford there is a toe drain immediately downstream of the cut-off which can collect water from the drainage layer due to the high permeability of the sandwaste fill. At Roadford there is a series of pressure recording wells, fitted with valves which are normally kept closed, these can act as shallow drains for the foundation immediately downstream of the cut off.

Figure 3 Membrane drainage flows

16 Initially, during impounding, there were only intermittent drips noted from the membrane drains at Colliford. However during the period from December 1985 to June 1986 a flow occurred from two drains on the west side which was accompanied by an increased flow from the adjacent toe drains. In general there were only small flows from the toe drains until the reservoir level rose to within 3m of TWL.

17 At Roadford small flows were observed from the membrane drains shortly after impounding. During the first winter when the reservoir water level had reached 83% of the total depth, the total flows had become significant. Likewise flow from the pressure recording wells had increased with the rising water level.

18 These events at both reservoirs gave rise to concern that there may be some imperfection in the asphalt and similar investigations were used at both reservoirs to try to establish the reasons for the flows. Bacteriophage (a virus specific to bacteria) was introduced into the reservoir water in the vicinity of the dams. Serratia marcescens was used at Colliford (Martin 1988) and enterobacter cloacae at Roadford. At Colliford the phage was detected in the membrane and toe drains 24 hours after dosing with peak levels occurring after 48 hours. At Roadford the phage was not detected until 6 days after dosing and then in only low concentrations suggesting dilution from another water source.

19 Inspection of the surface of both membranes was carried out by divers but no sign of any defect was found.

20 At Roadford analysis of water samples from the under-membrane drains was also carried out to try to establish the source of the flow. At first sight these investigations appeared to indicate a reasonable connection between the reservoir and the drainage system perhaps related to small cracks in the asphalt. However the subsequent seasonal pattern of the flows (see Figure 3) seems to indicate a different source for the water and that any defects that may have existed in the flexible asphalt have "healed" after it has been bedded on to dam the face by water pressure.

MAINTENANCE

21 During the spring of 1987 a broad greenish white line was observed on the membrane just below the top water line at Colliford. When the reservoir was drawn down it was found that the Flintkote emulsion seal coat was rough and discoloured over a width of about 300mm. Closer inspection revealed that the Flintkote seal coat had become "pimpled" with projections 1-2mm above the general level and 5-10mm across. In some places the pimples had dried and lifted from the asphalt although the Flintkote itself was still soft and exhibiting the self healing effect intended.

22 Investigations established that the cause was a growth of *Stigeoclonium tenue* algae which had occurred in the shallow water and adhered to the membrane. Following draw-down the algae had died and shrunk causing bituminous material to be drawn upwards causing the pimpling effect. It was also thought that the spray application of the seal

Figure 4 Rainfall reservoir level and membrane settlements

coat may have resulted in a thicker coat than necessary being applied and leaving a rougher surface which would encourage retention of water borne sediments and growths.

23 There had been no previous experience of this effect on emulsion seal coats and recent inspection of some dams on the Continent had shown the Flintkote to be in perfect condition after 20 years service. The use of a cold applied system was considered to be superior because any moisture trapped in the interstices would be absorbed into the emulsion and not cause bubbles to form.

24 Remedial works consisted of pressure washing a strip about 1m wide which although it removed the discoloration failed to remove any Flintkote even in areas where flaking had occurred previously. After the application of a super penetration primer, a coat of Flintkote No. 3 and one of No.5 were applied by brush to give a smoother surface than had been obtained originally by spraying. Although there was no contractual liability the remedial works were carried out free of charge by the main contractor with materials provided by the emulsion manufacturer.

25 Since 1987 there has been little further deterioration of the original seal coat and the repaired strip is in perfect condition. There has been no sign of a recurrence of the algal growth in spite of the reservoir remaining at levels 2-3 metres below TWL for long periods.

26 With experience at Colliford, it was perhaps no surprise to see a "line" just below the water level during the Spring of 1991 at Roadford. This was found to consist predominantly of decaying algal cells including *Diatoma*, *Tabellaria feocculosa*, *Synedra*, *Gomphonema*, *Chrysococeus* and *Tribonema*. No damage to the seal coat was obvious on subsequent draw down and other less distinctive "algal lines" were observed in following Spring periods. As the mastic seal coat was applied during a period of dry weather, there was little chance of moisture being trapped. The timescale may be too short to conclude that it is more durable than an emulsion.

27 During inspections at Colliford it was noticed that the mastic seal between the membrane and the precast concrete wave wall units was settling. By 1993 this amounted to 30mm in places and weeds were becoming established in the joints. The weeds were removed and the groove topped up with a bitumen mastic similar to the original specification at a cost of approximately £1,300.

PERFORMANCE (DRAINAGE FLOWS)

28 As described previously, flows occurred from the membrane drainage systems during impounding. At Colliford there have been no further flows from the membrane drains themselves but flows have continued from the toe drains. The majority of the flow occurs from the flanks at the point where the gradient of the valley becomes flatter. There is a marked seasonal variation influenced by rainfall with the peak occurring in the spring of the year (Fig 3).

29 At Roadford, flows from the membrane drains were initially located mainly at the toe of the west flank but subsequently other areas provided the majority of the flow. Currently the maximum flow is occurring from two drains which are located at the lowest point of the cut off in the valley bottom. With flow practically ceasing during the summer periods despite little variation in reservoir level, the seasonal trend is obvious.

30 The reason for the flows is not clear but the seasonal pattern would seem to rule out leakage through the asphalt membrane. At Colliford it is considered that the majority of the water collected by the upstream toe drain

is from rainfall percolating through the sand and ground water from the dam foundation. At Roadford both rainfall and ground water may be involved, but the mechanism that allows the water to emerge from the membrane drains rather than through other systems cannot be explained at present.

PERFORMANCE (SETTLEMENTS)

31 At Colliford overflow settlement cells were provided under the asphalt to measure any deflections. These have indicated only small movements of up to 6mm as was expected from the virtually incompressible sandwaste fill. During the underwater survey a 4m long straightedge and a powerful torch was used to identify depressions but none were found.

32 Overflow settlement cells were also used at Roadford (Evans and Wilson 1992) together with a special electrolevel measurement system installed immediately under the asphalt (Tedd, Price, Evans and Wilson 1991). There is no confidence in the data from the cells which indicate movements of ±300mm; the errors are probably related to tubes of too small a diameter being used over the long distance to connect to the gauge house downstream. The results from the electrolevels appear to have given accurate measurements of the deflections that have occurred at the toe of the dam.

33 The electrolevel measuring system was specially developed by the Building Research Establishment for use at Roadford. It consists of articulated stainless steel box sections each containing a small curved glass tube partially filled with an electrolytic fluid which acts as a transducer providing an output voltage proportional to the tilt angle. The instruments extend 14m up the face at the centre of the dam and 11m at the east and west flanks.

34 Settlements have increased in proportion to reservoir head and reached a maximum of 100mm at the highest point up the slope when the reservoir was at its highest level in 1991 (Fig 4). Since then there has been little movement although a recovery of about 7mm was detected when the reservoir was drawn down by about 3m. Of particular interest were the deflections at the concrete toe structure and at the sandwaste/rockfill interface where settlements were 5mm and 6mm respectively. This indicates that the sandwaste has been effective in reducing settlement at the most vulnerable area of the joint with the concrete cut-off and that the deflections can easily be accommodated by the flexibility of the asphalt.

CONCLUSION

35 The use of asphalt membranes to waterproof the embankments at Colliford and Roadford Dams has been an unqualified success. Construction proved to be easy because placing fill and membrane laying were not required to proceed simultaneously. Costs were not excessive considering that no clay core or rip rap material was economically available. In service, the membranes have proved to be waterproof and their performance has

given no cause for concern. In the 10 years since Colliford was completed virtually no maintenance has been required although a possible construction defect had to be rectified. No deterioration has been observed during 4 years at Roadford but the time scale is probably too short to draw definite conclusions. Nothing that has occurred at either site to date gives an indication that the asphalt membranes will not achieve the long life expected from this type of construction.

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Discontinuance/abandonment of Killamarsh and Woodall Reservoirs under the Reservoirs Act 1975

P.A.S. FERGUSON, MA, SM, MICE, Director, and D.A. McFADYEAN, BSc, MICE, Senior Geotechnical Engineer, Sir William Halcrow & Partners Ltd

SYNOPSIS Constructed in the 18th Century, neither of the two reservoirs are now required to serve their original function and Woodall Reservoir became severely damaged by mining subsidence in the 1980s.

The removal of the reservoirs from the jurisdiction of the Reservoirs Act (1975) entailed lowering the top water level in Killamarsh Reservoir by 1m to reduce its capacity below 25,000m^3 and providing a low level overflow at the empty Woodall Reservoir to prevent storage of water in times of flood. Other works were carried out at both reservoirs to preserve and enhance existing amenity uses and at Killamarsh Reservoir to dispense as far as possible with future maintenance requirements.

BACKGROUND

1. Located 24km NE of Chesterfield, both reservoirs were constructed to the design of Brindley in the 18th Century to serve the Chesterfield Canal, sharing a common feeder channel leading away from the reservoirs below Woodall. Their general arrangement is shown on Fig. 1.

2. They possessed many other common features, both originally having:

- embankment dams, built of locally won boulder clays and Coal Measure Shales
- low level draw-offs beneath the centres of the embankments
- spillweirs situated on the right abutments of the dams, with downstream arrangements leading overflows to either the local streamcourse or the canal.

3. Killamarsh Reservoir is on the County Brook, which enters the River Rother, downhill of the Chesterfield Canal, at the Rother Valley Country Park. Woodall Reservoir is on a tributary of the County Brook, which joins the latter about 500m downstream of Killamarsh Reservoir.

4. The Chesterfield Canal in the locality of the reservoirs fell into disuse in 1908 when the nearby Norwood Tunnel collapsed, and the requirement of the reservoirs to feed the canal ceased. It is probable that maintenance of the system of feeder channels from the reservoirs also ceased then as details of them are extant and traces on site are now vestigial. The channel from

Route of Public Bridleway

FIGURE 1 ; GENERAL LAYOUT

Killamarsh to Woodall is still identifiable, as is the means of diverting flows from Killamarsh down the County Brook, but the arrangements at Woodall are now difficult to elucidate, particularly the feeder channel to the canal, which can now not be found. However it is apparent that a direct connection between the reservoirs and the canal no longer exists.

5. Woodall Reservoir, the larger of the two, was used by the Nether Green Angling Club from shortly after the First World War until the reservoirs were purchased by British Coal in the 1980's, when mining of the Clowne Seam beneath both was planned; they have remained in the ownership of British Coal ever since. In the event, only Woodall was undermined; it was emptied beforehand and has remained empty ever since, owing to the damage that was caused to the embankment dam by the undermining. The Clowne seam was worked during 1981 by longwall methods at a depth of 88m below the dam with a mined thickness of 1-2m. The predicted subsidence of the dam was 1.08m; to our knowledge actual subsidence was not measured. The subsidence damage to the dam occurred concurrently with the mining beneath it and was visible as several deep cracks of 37-50mm width in the western half of the dam with downthrows of 75-100mm. North-south fissuring was also seen in a steep exposure of fine grained sandstone to the east of the dam. About two weeks after mining beneath the dam, new north-south cracks were observed immediately to the west, but with no worsening of the previous damage. The Nether Green Angling Club transferred its activities to Killamarsh Reservoir.

6. The works initially carried out by British Coal in the 1980s to keep Woodall Reservoir empty comprised the installation of a 580mm dia uPVC pipe inside the existing 760mm internal diameter brick low-level draw-off through the original valve chamber, removal of the original valves and installation of a gate valve in the chamber, demolition of the original brick overflow pipe through and downstream of the chamber, excavation of a channel to the downstream watercourse and conclusive severance of connections with the feeder taking flows to the canal.

7. At Killamarsh, leaks associated with the low-level draw-off had caused concern from time to time and remedial measures, the details of which are not all available now, were carried out. This included replacement of the original timber pipe downstream from the valve shaft (the Neptune) with a concrete pipe at a higher level. Works associated with this included the provision of a valve on this pipe at its upstream end in the Neptune; puddling against the outside face of the brick-lined Neptune in the excavation for the new pipe; blocking off of the collapsed timber pipe from within the Neptune and repairs to the upper parts of the Neptune. Fig. 2 shows a cross-section of these arrangements through the dam. Despite all of these works, there were still leaks emanating from the Neptune or the drawoff pipes and these appear never to have been satisfactorily cured. The pipe from the reservoir bed into the Neptune has no controls on it and therefore the water level in the Neptune is the same as the reservoir level. The upstream end of the pipe is heavily silted, but did unblock when the valve on the downstream pipe was last opened in the 1980s, with a resultant emptying of the reservoir and loss of fish stock.

8. The channel from the overflow weir at Killamarsh had been eroded over

FIGURE 2 : CROSS SECTION THROUGH KILLAMARSH DAM

the years and had become deeply incised, undercutting the right bank at a sharp left-hand bend and oversteepening both banks over much of its length; the resulting cliffs were a hazard to public safety. Wave protection on the upstream face of the embankment consisted of stone blocks; they had been displaced over the years and no longer adequately protect the face. Both reservoirs have become a local amenity, with a bridlepath passing below Killamarsh dam and along the crest of Woodall dam; the path compounding the damage to this dam.

WORKS UNDERTAKEN

Works to Lower the Killamarsh Reservoir Top Water Level

9. The works carried out to lower Killamarsh Reservoir top water level by 1m, reduce maintenance requirements, improve safety and preserve the amenity were as follows:

10. A new weir control structure, as shown on Fig. 3 was constructed on the site of the former brick and stone spillweir. This new structure has a sill level (88.30m AOD) 1m below the original sill level thereby lowering the top water level by 1m. This is designed to discharge at a maximum rate of $5.1m^3/s$. Based on calculations made following the last inspection in 1988 under Section 10 of the Reservoirs Act (1975), the design discharge capacity is equivalent to the peak outflow from the reservoir for the PMF. This also accords with a requirement of the National Rivers Authority (NRA), that the outlet capacity of the existing weir should not be exceeded.

11. The structure has been provided with galvanised mild steel open mesh flooring and handrailing as a safety precaution. A galvanised mild steel grille prevents unauthorised access and limits passage of large debris into the overflow pipe. Timber steps and a lockable hinged opening in the mesh flooring allow access to the structure for maintenance.

12. Rock filled gabion baskets are positioned around the top of the control structure to retain the backfill placed to the channel sides of the original structure. The surface of this backfill is protected with a 'Tensar' mat, topsoiled and grass seeded to help maintain the stability of the slope.

13. Flows from the new overflow weir are discharged into a pipeline laid on the bed of the original deeply incised overflow channel and the channel has been backfilled so as to remove the hazard of the cliffs on either side.

14. The pipeline comprised 65m of 900mm diameter pipe laid at gradients varying from 1 in 8 at its upstream end to 1 in 11. Flow velocities in the pipeline are supercritical and therefore manholes at bends in the pipeline are not appropriate. Instead, preformed pipebends and concrete thrust blocks were provided.

15. An impact type energy dissipating outfall structure and channel protection were constructed at the downstream end of the overflow pipeline. The impact wall and stilling basin created by a raised end sill are designed to absorb the energy of peak flows with a velocity of the order of 8.6m/s.

FIGURE 3 ; NEW WEIR CONTROL STRUCTURE AT KILLAMARSH

FIGURE 4 ; GENERAL ARRANGEMENT AT WOODALL

16. Flows from the outfall structure enter the County Brook at approximately right angles to the Brook and its banks adjacent to the structure were provided therefore with protection comprising random, coursed and mortared stonework on a gravel/lean mix concrete bedding for a distance of 4m beyond the outfall structure.

Works To Reduce Maintenance Requirements at Killamarsh Reservoir

17. Stone pitching was placed along the length of the embankment dam to ensure long term protection against wave action along the new waterline. The original stones still remaining were used in the construction of a toe wall which stabilises a layer of compacted DoT Type 1 granular sub-base and loose stone pitching up to 300mm in size, to a nominal thickness of 300mm.

18. The absence of controls on the inlet to the Neptune, its uncertain condition and proneness to silting-up, coupled with the gradual deterioration of the Neptune shaft itself were the greatest potential maintenance liabilities following discontinuance. In view of the consequences of the last attempt to lower the reservoir level using these draw-off arrangements, when much of the fish stock was lost, and the expense that would be incurred in replacing the inlet pipe to the Neptune and reducing the maintenance liability of the Neptune itself, it was decided to abandon these draw-off arrangements. The absence of a low-level draw-off was considered acceptable in view of the large freeboard of the reservoir (about 1.5m, taking account of run-up and waves above maximum water level during the PMF) and the capacity of the outlet weir, which together substantially diminish the risk of an occurrence that would require emergency drawdown of the reservoir. Nevertheless, to cater for this unlikely event it has been stipulated that a pump, with a capacity of 175 l/s against a head of 2.5m, is kept on stand-by locally and maintained in working condition. The abandonment of the Neptune was accomplished by filling it with granular aggregate and by grouting the outlet pipe for a distance of approximately 12m downstream of the valve which was left in place and closed. This was in order to guard against the possible failure of the outlet flap valve (causing potentially rapid loss of water from the reservoir). The ungrouted continuation of the outlet pipe remains free draining to prevent build up of groundwater in the immediately surrounding embankment. A gravel-surrounded land drain continues as far as the streamcourse. The Neptune shaft was capped with a 250mm thick reinforced concrete slab and covered with topsoil.

19. The downstream embankment between levels of about 87 and 89m AOD was buttressed over a length of about 5m on either side of the Neptune with free-draining stone wrapped in geotextile and capped with topsoil. This was done to improve the stability of the embankment adjacent to the Neptune to cater for any future rise in water levels in the embankment arising from any increased leakage from the Neptune as it deteriorates with time.

Works to Preserve and Enhance Amenity at Killamarsh Reservoir

20. These works included:

- Removal of soft mud and organic matter exposed by the lowering of the water level. The mud was deposited at the north west

corner of the pond and sown with grass seed and some bull-rushes transplanted from the Woodall pond. The latter were recommended by the NRA as a means of filtering incoming flows.

- Provision of a stone-filled sump 3m square in plan and protected with Terram filter material just upstream of an inlet channel formed through the deposited mud. This was requested by the NRA to filter and allow the deposition of silts carried by the run off from an adjacent access road and farmland.
- Clearance of weeds and vegetation from the reservoir, mainly in the region of the incoming County Brook to improve inflows and help prevent localised flooding.
- Construction of 35 no timber angling platforms along the new waterline.
- Reinstatement of the path below the embankment dam and as far as the outfall structure with DoT Type 1 granular aggregate overlying Terram geofabric.
- Reinstatement of the temporary site works access road as far as the west end of the embankment dam with DoT Type 1 granular aggregate overlying Netlon geofabric, for the use of the Angling Club.

Works to Woodall

21. Since the reservoir was emptied during the 1980's, the banks and beds have become occupied with natural marshlands, trees and other vegetation. In order to prevent impounding behind the embankment dam, when inflows exceed the capacity of the 580mm dia uPVC outlet drain, a notch was cut through the embankment and a control structure and channels constructed in this notch to carry excess flows to the existing channel which had been excavated for the 580mm uPVC pipe outfalling downstream of the embankment. The control structure culvert was designed such that discharges to the downstream watercourse will not exceed the discharges which would originally have occurred when the reservoir was operational, as stipulated by the NRA. The maximum flow in a 100 year design storm is $2.5m^3/s$ in the culvert with $1.1m^3/s$ through the 580mm diameter pipe. If the pipe should become blocked, the culvert is designed to carry the full flow. Details of these and other minor works to preserve the amenity and improve safety are described as follows:

Works to Prevent Impounding Woodall Reservoir

22. Fig. 4 shows a general plan of the works and indicates the extent of earthworks and recontouring of the embankment and adjacent slopes.

23. A total of approximately $1900m^3$ of embankment fill and adjacent ground was excavated during the formation of the notch. Most of this material was used to infill the redundant Woodall weir and intake channel or was deposited at the upstream faces of the east and west ends of the embankment dam and then trimmed to merge with surrounding contours. The slope angle of the cut either side of the control structure was 1:5 to allow continued use as a bridleway and all areas exposed by the earthworks were grass seeded. Some $700m^3$ of the excavated material was transported to Killamarsh where it was used to fill over the pipeline laid in the overflow

channel.

24. The invert level of the reinforced concrete control structure culvert was set at 79.0m AOD. Cellular concrete lined open channels were constructed using "Grassblocks", underlain with geotextile and covered with topsoil and grass seeded. The underlying geotextile (Geolon 15) will allow grass roots to pass through which should enhance the long term strength and durability of the channels.

25. To prevent revetment blocks being dislodged as a result of high flow velocities during floods, the gradient of the channel downstream of the culvert was kept flatter than the natural slope by the provision of two 0.5m high steps formed from gabions.

26. The existing channel downstream of the valve chamber for the 580mm uPVC pipe was protected by gabion baskets on its banks and a layer of 300mm nominal size rip-rap laid on A20 size gravel on its bed, where the channel from the culvert now joins it. A brick culvert (the feeder from Killamarsh) in the left bank, downstream of the junction, was demolished to permit the laying of gabion baskets; it was filled with silt and clay. Opposite, the mouth of a similar brick culvert on the right bank was stowed with free draining stone and gabions placed in front of it. The partly-blocked inlet of this culvert was found in the overflow channel of the original overspill weir just upstream of stoplogs across the streamcourse and was obviously provided to divert overflows into the canal feeder. The inlet was blocked with locally available debris.

27. The channel downstream of the new control structure joins the streamcourse to the County Brook approximately 20-30m downstream of the gabion protection works.

28. The 580mm diameter uPVC pipe has an invert level of 77.40m AOD at its upstream end and 76.21m AOD at its outfall below the valve chamber. The gate valve was left fully open, allowing the 580mm diameter pipe to drain normal flows through the reservoir and ensure that the new overflow channel will only operate during particularly heavy floods. The chamber was repointed and its cover reset on two courses of brickwork. A short length of handrailing was erected above the brick headwall on its downstream side.

Works to Preserve and Enhance the Amenity of Woodall Reservoir

29. Tree and shrub planting was subsequently carried out on the cut slopes either side of the control structure principally in the area of woodland downstream of the embankment dam where some trees were removed to allow access for the construction of the works.

30. The bridleway was reinstated for a length of 160m along the whole length of the embankment dam crest to a width of 2m using Terram geotextile and Type 1 granular aggregate. The bridge over the redundant backfilled spillway was dismantled.

31. The bridleway between Killamarsh and Woodall was also reinstated and improved in certain locations where natural watercourses cross the path. In

these low spots land drains surrounded with large stone and gravel, were installed beneath Terram geotextile and Type 1 granular aggregate.

32. Vegetation was cleared from the original path along the length of the west (left) bank of the reservoir. This allows more access for local people alongside the wetland habitat which has become established in the base of the reservoir.

CONCLUSIONS

33. The works described above enabled two redundant reservoirs to be removed from the jurisdiction of the Reservoirs Act whilst enhancing the amenity and safety of the area. Environmental improvements were made where these could be done in a cost effective manner.

34. The necessary abandonment of the derelict Neptune continues to leave Killamarsh Reservoir without a ready means of emergency drawdown. Accordingly, a procedure was prepared for implementation in the event that drawdown becomes necessary.

35. Subsequent to the works described, certificates of discontinuance were issued for the two reservoirs. It is believed that the reservoirs and surrounding lands may be passed from British Coal Corporation to the local authorities who will further develop the amenity value of the area, but without the statutory liabilities of the two reservoirs.

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Internal erosion problems in some old embankment dams in France

J-J. FRY, EDF-CNEH, France, P. BRUN, CNR, France, and P. ROYET, CEMAGREF, France

SYNOPSIS : Dams built in France during the first part of the 20th century are homogenous according to the Frontard design. a certain number of them failed during the first years after drawdown, others are still in operation, but some of them show internal erosion and seepage problems. Two types of embankment fills are particularily sensitive : alterated granite fill founded on fissured granite, and silt bank on gravel foundation. Descriptions of some historical cases are treated and the repair works are described.

INTRODUCTION

Internal erosion is a tricky problem which heavily affects dam safety. Up to now, more than two hundred small dams, below than 15m, have been built in the south-west part of France per year. Not all benefited from the necessary design rules. So it is hardly surprising that one of these small dams, St Julien des landes, 8m high, have failed by piping, in that particular case along the culvert when the reservoir raised 3m during first filling. Except for this kind of failure due to a design error, no failure has been reported many years after completion, but a lot of problems have been noticed and some expensive repairs have been necessary. Two kinds of dams are particularly sensitive to internal erosion : altered granite fill on fissured granite foundations and silt fill on gravel foundations. The historical cases encompassed in this paper illustrate the importance of the problems encoutered.

1. ALTERED GRANITE FILL AND FISSURED GRANITE FOUNDATION

1.1 Lavaud-Gelade embakments

1. The Lavaud-Gelade embankment is a 22 m high homogeneous embankment, built during the Second World Wair in 1942 out of granitic arena with a 40 % fines content, with no filter or drains, and originally with steep slopes : H/V = 1.5/1. This corresponds to the Frontard design which indicated that a clayey-sandy earth (between 1/2 sand and 1/2 clay and 2/3 sand and 1/3 clay) may be used to build an homogeneous earthfill with steep slopes and limited height. The fact that the borrow areas were small and that part of the fill was placed at night during wartime resulted in a wide range of grain sizes. A cutoff trench was dug to a depth of 2.5 m to watertighten the

first few metres of the foundation,composed of fissured granite. The depth of 2.5 m resulted from the lack of mechanical and financial means, as deeper works on the right bank were advisable.

2. During first filling, leakage appeared downstream in the foundation and there were wet patches on the downstream face. It was decided to grout the right bank with clay (because of the lack of cement) in order to watertighten that bank, to dig 10 drainage trenches in the gravel and to add a downstream shoulder to reduce the risk of instability (fig. 1). The leakage was reduced but the wet patches on the downstream side did not totally disappear.

before refurbishment: (1) granitic arena sand (2) facing masonry
1944 refurbishment (3) drainage trench (4) gravel shoulder

Fig.1. The Lavaud-Gelade dam after 1944 refurbishment

3. Presently stability is being re-evaluated. The reservoir water level has been lowered by 1.5 m with no financial consequences, in order to limit the extend of the wet patches that are still visible 1 to 2 m above the shell. It is remarkable to note that the embankment has suffered very little displacement. The flow computation also shows high pressure gradients imposed by highly anisotropic permeability in the fill and permeability of the order of 10^{-5} m/s in the foundation, related more to the initial weathering of the foundation than to internal erosion. The leakage flows have decreased and are now stable. Slight variations in the reservoir water level can be noted.

4. In this case it is interesting to note that there are not clear signs of internal erosion, despite the absence of any filter and 50 years in operation. The fines content and the nature of the clay would explain this apparent stability. The study underway should clarify this point.

1.2 The Matemale dam

5. The Matemale dam is a 37 m high homogeneous gravel moraine dam, completed in 1959. The foundation is composed of sandy and gravely alluvium due to granite erosion, granitic arena and fissured granite. After first filling in 1960, a progressive drop in piezometer readings from 1960 to 1967

was observed, followed by a sudden rise in 1969. This structure has little settlement, except that settlements at the base of the dam were higher than in other areas. The hydraulic monitoring system was composed of 9 USBR piezometric cells and 4 cross-arms around the central section. Between 1970 an 1978 the piezometric curve dropped regularly, but after the reservoir had been emptied twice, the level of the piezometers in the centre rose abruptly at refilling.

Fig.2: The Matemale dam. Piezometric level in 1960, 1979 and 1991

6. Consecutive to these phenomena, site investigations were undertaken using drillholes with sampling, injection of radioactive tracers and installation of piezometers. The study confirmed the presence of alternating fine-poor and fine rich layers, a slight decrease in the percentage of fines under the piezometric surface, and the highest concentration of fines around the filter.

7. A movement of fines was the origin of these piezometric changes. The weak content of fines, 13 % < 0,08 mm, creating excessive permeability (10^{-6} m/s) must be considered as inferior to the lower limit for earthcore. The grain size, typical of morainic or alluvial gravel in mountains, is not self-filtering, and the lack of sand particles between 0.05 and 0.5 mm destabilizes the small clay content, which can migrate between the grains.

8. On the other hand, the drop in the piezometric levels can be explained in part by the role of the vertical drains, played by the cross-arm and the open piezometers slotted from top to bottom. Thus the flow was connected to the permeable foundation (10^{-5} m/s) and increased gradients, going over 2 in some places, threatened to wash out the soil. Furthermore, natural clogging upstream from the embankment decreases flow and the effect of the cross-arms.

9. the cross-arms and the piezometers were filled with bentonite-cement grouting. Thus the piezometric levels rose and the risks of erosion decreased. No impervious facing was considered necessary, as since the pressure had stabilised.

1.3 the Nepes dam

10. the nepes dam is a multiple arch buttress dam, built in the middle of a valley on sound granite in 1945. However, on the bank, the rock is under a 6 to 15 deep alluvium. Therefore two 13 m dikes were built to link the dam to the banks. The core is made of 10 cm thick reinforced concrete over a concrete cutoff; The fill is granitic clayey sand (fig. 3).

Fig. 3 : The Nepes Dam: Piezometric head before works

11. In 1990, after refurbishing, piping was revealed on the left bank. The presence of a sand deposit was visible at the toe of the bank. Moreover, the discharge collected on this bank was 100 l/mn, ten that on the right bank.

12. Site investigations carried out on the basis of the geophysical studies have shown that the cutoff was not on the sound rock. Installation of 12 pore pressure vibrating wire cells revealed the existence of high piezometric levels in the foundation. Finaly, drillholes, with confirmations by penetrometers, revealed some gravely layers in the old alluvium deposed by an old river in the foundation. The sand was carried out across the gravel.

13. A 630 m^3 grouting wall was placed to stop these flows. The grouting continued to 290 m^3 of bentonite-cement (C/W=0.4) and 100 m^3 of cement, in order to reduce the discharge 7 times, halve the pore pressure and reach 1.10^{-6} m/s with on-site permeability test.

1.4 The Sapins dam

14. The sapins dam is a 16 m high embankement built of granitic arena in 1977-78 with a horizontal drain at the contact with the foundation and a vertical drain that was not connected to the horizontal one and that was positioned 2 m below the normal water level. In 1988, the beginnings of piping, leakage, and surface sliding were observed on the downstream part

(1) Upstream riprap (2) granitic arena (3) chimney drain
(4) Drainage blanket (5) toe rockfill (6) slurry wall
(7) rockfill (8) Grout curtain (9) piezometer

Fig. 4 . The Sapins dam

15. The first pore pressure measurements had shown a progressive rise upstream from the drain in 1980. but the measurements were wrong and abandoned. The discharge into the horizontal drain was always stable and very high : 10 l/s. However, the discharge in the vertical drain reached a maximum in 1982 at 1.5 l/s and then decrased to reach a stable level of 1.5 l/s after 1984.

16. The excavation of two trenches showed that the sliding only affected the topsoil and that there was flow at the point where works had been broken off during the winter of 1977-78. After all of these observations, plugging of the vertical drain or its exit, followed by progressive saturation of the fill downstream, could be diagnosed.

17. To guarantee safety in the short-term, an inverted filter was installed along with a rockfill shell. then, during the summer of 1989, a grout wall was poured down to the foundation. Three years after the reservoir was re-filled, the total flow has been drecreased 10 times and the downstream piezometric head lowered by 4 m.

1.5 The Saint-Pardoux dam

18. The Saint-Pardoux dam is a homogeneous embankment 19 m high built between September 1974 and May 1975, protected by granite rockfill shells and founded on granite that is weathered at the surface and cracked.

19. the water from two springs discovered during the works was collected at the bottom of the excavations. As soon as the reservoir was filled, the water level dropped and additional grouting was undertaken. Since then,

another spring has appeared on the left bank and wet patches can be seen on the fill on the right bank. Inspections in 1991 revealed high piezometric head on the right bank that fluctuates very quickly with variations in the reservoir water level, while the water collected from the springs has tended to increase since 1981, reaching 40 l/min while the drainage blanket discharges 10 l/min (fig. 5).

Before works
(1) Compacted granitic sand (2) Rockfill (3) Drainage blanket
(4) Alluvium (5) Grout curtain (6) Piezometric level
(7) Seepage area

After rehabilitation works
(1) Slurry wall (2) Grouted connexion (3) grout curtain
(4) Drainage shafts (4) Lowered piezometric level

Fig. 5 . The Saint-Pardoux dam

20. It is clear that the fill is segregated and the foundation has not been sufficiently treated. The water level was therefore lowered by 4 m, and in the space of four months, reinforcement works were done; A diaphragm wall measuring 2300 m² crosses the fill and the first 2 m of the rock ; it was placed using a "tube à manchette". a single-line grout curtain in the foundation required 100 tonnes of cement from drillholes spaced 1.5 to 3 m apart. A drainage curtain on the left bank abutment, drainage wells downstream, and about 20 piezometers have been added.

21. The rise in the reservoir water level has been the opportunity to check on the effectiveness of those works : the piezometric head in the fill has been lowered by 5 m in the left bank, 7 m in the valley, and 3 m in the right bank. The wet patches have disappeared downstream, but the spring upstream is still discharging 35 l/min.

1.6 Lessons learnt

22. Widely varying grain sizes and the difficulty in watertightening make weathered granite ground hard to work with. The high permeability of fissured and weathered granite means the foundation must be accorded special treatment.

23. The economic design of small dams built with granitic arena or alluvial material favours homogeneous fills. A vertical filter placed at the normal water level and connected to a three-layer blanket is absolutely necessary. After removing the decompressed ground, grouting and surface treatment of the first few metres of weathered granite greatly reduce the risk of piping in the foundation.

24. As concerns construction, the variability of the materials requires a certain number of precautions. With a fines content of over 20 %, a borrow area providing twice as much material as required for the fill, water content higher than w_{opn}, and very strict visual and mechanical inspection during construction are all mandatory conditions for proper achievement. If several of these conditions are not fulfilled, another method for watertightening must be studied.

2. SILT BANKS

2.1 The banks on the Rhone river

25. Legislation passed on 27 May 1921 called for the development of the Rhone river from Switzerland to the Mediterranean, and the project is now nearing completion. The CNR (Compagnie Nationale du Rhône) has held the concession granted by the national government since 1934, and has had to build approx. 500 km of embankments 15 m high or less. Thus long experience has been acquired in design, construction and use of such embankments.

26. Twenty-five accidents have been reported. Although some caused concern, none resulted in failure of the embankment. Among them, 10 involved leakage associated with the appearance of a sinkhole or settlement, and transport of fines. They all occured in silt banks. In fact, two types of banks were built, depending on the material available : silt banks and gravel banks. Although the silt banks have the advantage of being much more watertight, they are nonetheles more vulnerable to internal erosion at the contact between the silt and the gravel. A few examples taken from these cases are presented hereafter (fig.6).

27. The silt banks have a core of compacted silt resting on the silt of the foundation ; rolled gravel and rockfill protective layer covers them upstream and downstream dumped gravel placed over a drain plays the role of a filter (fig. 6). The core, 90 %OPN compacted, has average permeability 10^{-8} m/s. The gravel banks don't have a core, but the permeability of the different materials increases from upstream to downstream.

2.2 Bernier-cordon sinkhole

28. The development creates head of 11.4 m and a 12 km reservoir.

The problem was located at PK 106.83, in an area where fines transport was reported over a distance of 100 m, requiring specific inspection.

29. In June 1991, eight years commissioning, a sinkhole was discovered during an inspection. The 800 litre hole was on the crest, near the downstream slope. A gravimeric survey revealed two zones with negative varitations in gravity. But these anomalies were of low amplitude and were not directly related to the sinkhole. The piezometric profile installed afterwards on the zone gave no indication of abnormal hydraulic behaviour.

(1) foundation gravel (2) foundation silt (3) compacted silt
(4) compacted gravel (5) dumped gravel (6) drain (7) riprap

beginning of the erosion

(1) preferential flow in local zone of very pervious gravel
(2) silt deposits

development of the erosion

(3) vertical silt decompression: regressive propagation of arch effect

yield state of erosion

(4) sinkhole apparition

fig. 6. Silt banks on the Rhone river: internal erosion process

30. The origin of the sinkhole was the fact that during construction the silt had been placed in direct contact with the drain, although in the original design filter conditions were intended to be ensured by the gravel (d15=0.5 mm, with d85=0.25 for the silt). Thus materials breaking away and fines transport at the contact between the materials occur horizontally to cause local decompression of the silt, which propagates along the horizontal conduits and vertical shafts, according to the weak areas encountered (fig. 6).

2.3 The Vallabregues incidents

31. The Vallabregues plant in the departement of Gard has head of 11 m and a reservoir 18 km long. Since impounding in 1970, the embankments have chronically suffered from accidents, due to the heterogeneity of the silt and its density. Several of these accidents are worth nothing.

32. During impounding, high hydraulic head downstream and strong flows with fines transport were noted on several profiles. Additional decompression drillholes solved these problems of internal erosion.

33. Several years later, the same phenomena re-appeared. At PK 252.5, for example, leakage over a distance of 90 m occured on the downstream slope and caused a beginning of regressive erosion. A shell was built to stop this phenomenon. Site investigations revealed a mud lens 3 m below the side channel that imprisoned the piezometric head of the underlying gravel and that extended 4 m above the level in the side channel. This type of accident shows the difficulty in identifying all the geological anomalies over long lengths.

34. Where the natural layer of silt was too thick, gravel drains were installed on the silt to link the upstream and downstream shells in order to avoid excessively high uplift in the upstream shell during emptying. These profiles suffered several incidents : leaks at the top of ghe downstream slope, higher piezometric head in the drains than in the foundation gravel. Insufficient permeability of the downstream shell resulting from penetration of silt in the drains would explain those incidents.

35. Sinkholes and settlements relating to leakage were evidence of internal erosion. Two such cases are described hereafter.

2.4 Sinkhole at PK 255.7 on the left bank of Vallagregue embankment

36. During a visual inspection in February 1992, in an area of leakage discharging into the side channel, a sinkhole measuring 50 cm in diameter was discovered full of water on the road on the berm; A horizontal hole led into the sinkhole and brought in leakage water. The piezometers in the area indicated no anomaly.

37. To avoid external erosion, drainage trenches and a shell at the level of the road were installed. After a vain attempt to use chemical tracers, geophysical testing was done by the company CPGF. Electrical tracing of saline flows showed significant anomalies localised in the downstream part the area under study, less than 5 m from the shore. Overall electrical tracing,

after massive injection of sodium chloride, confirmed the location of the entry of the leak, in the vicinity of the sinkhole that had been discovered. A speed of 6.10^{-2} m/s was calculated from the 3D images of the transfert of the saline solution (fig. 7).

Fig. 7. 3D transfert of the saline solution under Vallabregue bank

38. The clearly localised inlet area was successfully plugged with clay. A 90 % decrease in the leakage was achieved at the bottom and the leakage at the berm was completely stopped.

2.5 Sinkhole at PK 250.6 on the left bank of Vallabregue embankment

39. In April 1976, settlement several metres wide on the upstream facing was filled in. In May 1991, a sinkhole 70 cm wide and 3 m deep was discovered perpendicular to the slope in the wooded upstream face, with a leak at the foot of the downstream side. The repair work consisted in removing material from that area and compacting silt that was brought in. Vertical settlement measurements later showed that more settlement had

occured. In October 1993, a sinkhole measuring about 1 m appeared in the same place.

40. Through tracing, the source of the leakage was located in the Rhone, 10 m from the bank; The speed at which the water was seeping could be estimated at 2.5 10^{-2} m/s. Flow was located at the interface between the silt and the gravel and continues to remove silt. An attempt to plug it with clay failed. A survey of geophysical and thermal measurements is underway to more accurately determine the path of the leaks. A watertightening process using bentonite geocomposite is being studied.

2.6 Embankments on the Rhine

41. EDF is responsible for 300 km of embankments between Kembs and Strasbourg. Built between 1930 and 1975, they are either of gravel, from very gravely alluvium (10 to 30 % sand) with slopes between H/V: 2/1 and 2/3, or - for the majority of them - of earth core encased in gravel with heights from 12 to 15 m. They are watertightened by an upstream lining of concrete slabs 10 to 15 cm thick, with joints.

42. Surveillance takes the form of a monthly reading of the piezometers placed every 250 m, with closer spacing on special points (water intake, outlet, leak etc.). Because of the high permeability of the gravel (10^{-2} to 10^{-3} m/s), heavy leakage is detected by peculiar variation in piezometric readings in comparison to the general trend. The leaks are measured monthly and a detailed inspection every year on foot makes it possible to locate new leaks.

43. In general the structures behave well. However, work is needed to restore watertightness and stop erosion. The erosion results in sand and silt movements and settlement of the berm. The path of leaks is detected through various techniques. Unlike the case for the Rhone, fluorescein injected into the Rhine or into the piezometers gave good results. This could be explained by the high permeability of the gravel in the foundation. Inspections by divers, a denser piezometric network, penetrometer, temperatures, and geophysical investigations are commonly used techniques to locate the path of leaks.

44. these defects are dealt with after an attempt to locate the point of departure of the leaks visually by a diver in the river bed. Four techniques are used as set out below (prices date from 1984) :

- dumping materials : this is the first method used, as it is the least expensive. However, it is not always effective. It costs £ 90/m of embankment plugged with mud, while earth is twice as expensive,
- grouting : grouting is often done through pipes to avoid grouting large quantities and damaging the watertightness. Installing the pipes in drillholes is preferable to hammering them in, as they are more easily plugged in the second case. A two-line costs £ 65/m^2,
- Thin curtains : this commonly used technique consists in hammering in sheetpiles for a heigh of 12 to 15 m and then pouring bentonite-cement as the sheetpile is being withdrawn (sometimes voids are discovered in the embankment when the sheetpiles are being driven in). The success rate is over 50 % but if flow is very concentrated, it breaks through the curtain as

it is being placed. A square metre of curtain costs £ 20 plus the costs of installing and removing plant, which amount to £ 25,000,

- sheetpile curtain : this technique is used when the preceding ones have proven ineffective. The sheetpiles must be reinforced at the top. Cost is £ 63/m^2.

2.7 Banks on the Isère river

45. In 1989, for construction of the St-Egrève dam, EDF heightened up to 11m the banks on the Isère river that had been built of silt and gravel, downstream from Grenoble between 1900 and 1950. During impounding, some problems appeared in the fill or the foundation. The works undertaken varied depending on the type of problem :

- high piezometric head and fines deposited in the side channel : relief wells ,
- wet patches on the downstream face : placing a downstream shell, composed of a sand filter (0-10 mm), screened gravel (10-120 mm) serving as a drain and shell, and a rockfill toe. The total cost of the operation amounted to £ 25/m,
- excessive leakage : cutoff wall excavated by power shovel (£ 35/m) or Kelly grab (£ 50/m).

46. Impounding lasted one year and there have been no major problems since then.

2.8 Lessons learnt

47. these exemples show the difficulty in interpreting the mechanisms and the random character of occurence of these internal erosion accidents. These problems are infrequent, occur only gradually, and are limited by the natural pligging of the embankments after floods.

48. The problems are detected visually : the inability of piezometers to detect such phenomena in silt is noteworthy.

49. The mechanism behind these phenomena is the appearance of high flow speeds in the foundation gravel, erosion of the silt at the contact with that gravel, decompression of weak area, appearance of sinkholes or new flow paths with erosion.

50. The search for these flows always begins with underwater site investigations and the thermal, electromagnetic, geoeletric, gravimetric and tracing methods, in their order of interest, have all given excellent results. Several methods should be used together.

51. The main part of repair consists in rendering the upstream face of leaks watertight. this solution appears to be preferable to internal watertightening in the body of the embankment through grouting.

CONCLUSIONS

52. In this analysis, two families of embankments suffering from recurring problems have been identified, and the problems observed and remedies found have been described. a more detailed analysis of these cases could consitute the beginning of an European data bank on internal erosion that would provide better knowledge of that phenomenon and the means to control it.

Photographic monitoring in the safety assessment of dams and reservoirs

P.G. KALAUGHER, MA, BSc, MICE, FGS, CEng, CGeol, Senior Lecturer in Civil Engineering, School of Engineering, and P. GRAINGER, BSc, DIC, MSc, PhD, FGS, CGeol, Senior Lecturer in Engineering Geology, School of Engineering, and Director of the Earth Resources Centre, University of Exeter

SYNOPSIS. In the safety monitoring of dams and other reservoir structures, visual inspection plays an important part in the identification of potentially hazardous conditions. A photographic monitoring system provides the means of recording the observations from successive visits. An optical instrument has been devised which allows an immediate comparison on site between a 35 mm colour transparency taken earlier and the scene it represents. The transparency is fused stereoscopically with the directly viewed scene. The impression is close to that of normal vision except that differences in detail which have occurred since the transparency was taken are immediately obvious.

INTRODUCTION

1. Visual inspection plays an important role in the monitoring and safety assessment of dams and other reservoir structures where a careful watch has to be kept for minor changes which could warn of incipient hazards. Surface deformations may be the first indication of internal erosion of an embankment dam, for example, but these could occur between instrumented sections within the dam or between survey targets on its surface and hence be missed by conventional measurements. A satisfactory visual inspection of all the available external surfaces and features of a dam provides a comprehensive, qualitative record which allows the detection of differences, but only if there is sufficient confidence in relying on the inspecting engineer's memory, backed up by notes or drawings. The success of such inspections also depends on continuity of inspecting personnel over many visits.

2. The monitoring system advocated here was developed to enhance the visual observations made during the inspection visit by viewing and comparing the previous and current images on site. When the engineering structure and its previous record are seen together, their images stereoscopically fused, the visual effect is so realistic and compelling that the full capabilities of the human visual system are available to detect differences between the two images (Kalaugher, 1985). The system provides a form of time-lapse viewing which, by decoupling visual memory, has particular

advantages if another engineer has to take on the responsibility of inspection.

THE PHOTOGRAPHIC MONITORING SYSTEM

3. The monitoring instrument, which incorporates a 35 mm single lens reflex camera, is mounted on a surveying tripod and has provision for fine adjustments to rotations about both the horizontal and vertical axes for accurate alignment.

4. During a site visit suitable positions are established from which the photographic monitoring gives optimal coverage of the relevant features of the dam or reservoir. Colour transparencies are taken with the camera in the monitoring instrument and these form a record of the initial state of the scene or structure. On subsequent visits, the transparencies are placed one at a time in the instrument for comparison with the appropriate direct view. The direct view of the scene is observed through the camera with one eye and, for both horizontal and vertical format photography, this viewfinder image can be fused stereoscopically with its previous state as recorded in a transparency taken on a previous visit and viewed simultaneously with the the other eye.

5. Any differences between the two images are, or can be made, immediately obvious. Some differences can be detected because the two eyes receive non-matching visual input from parts of the scene and therefore cannot be fused in those areas. Other differences may require occlusion of the eyes in turn or will be revealed by anomalous stereoscopic depth as explained in paragraph 22.

6. The method is essentially a field method: the reasons for any detected changes can be explored then and there, sometimes by making use of the ability to vary the stereoscopic depth by changing the direct viewing position.

7. If sufficient differences are detected to warrant the taking of a repeat photograph, the camera is already in place with the correct orientation. The new photograph is available for later field comparisons and also, when viewed with the original photograph, forms an office record for confirmation of the differences observed in the field. It must be emphasized, however, that most of the visual effects are more strongly developed during in-situ viewing.

USE OF SYSTEM FOR DAM SAFETY MONITORING

8. Satisfactory stereoscopic combination of the two images relies on there being sufficient recognisable visual texture in the field of view covered by a monitoring transparency. Only then will there be a strong enough tendency for the two images to merge (the "compulsion for fusion" reflex). Compulsion for fusion is strong for masonry, brickwork, concrete and rockfill all of which have texture which is likely to remain consistent at the

local level for many years. Most dams and their associated structures display a variety of well-textured surfaces. For the purposes of monitoring, the stronger textures are exploited to overcome the visual difficulty of matching areas of weaker texture, such as grassed slopes.

Embankment dams

9. Widespread movement or deformation of an embankment dam can be recognised by repeated surveying of widely spaced markers, or may be visually obvious because of distortion of a previously smooth curve or straight line. The recognition of local deformation is difficult if the surface material has such rough outlines that interpolation between surveyed points is inappropriate. In dam slopes that are unvegetated and constructed of coarse-grained material (gravel, rockfill or rip-rap) the rough outlines of the individual particles form ideal visual markers for comparison between the previous image and current view. This richness of texture allows photographic monitoring to reveal gross deformations and small relative movements.

10. When used as wave protection on the upstream face of an embankment dam rip-rap can provide a strong indication of subsidence resulting from scouring out of material beneath the rip-rap, from internal erosion within the dam or from general distortion of the dam. Wave protection formed of stone pitching or concrete slabs is also ideal for photographic monitoring. A series of transparencies, taken during calm conditions, with different water levels in the reservoir would help to reveal distortion in the upstream slope whatever the method of wave protection. This technique, using water levels as horizontal markers, would be particularly appropriate for dams with an asphaltic concrete upstream membrane.

11. Coastal protection works have been monitored photographically (Grainger & Kalaugher, 1991). The protection has included rip-rap and this has been photographed along the line of the defences from an elevated position using either a 50 mm or a 200 mm focal length lens. Relative movements of individual blocks and areas of general subsidence, where underlying sand has been scoured out, can be detected.

12. The deterioration of the rip-rap material by cracking, breaking or wearing of blocks is also shown up by this form of monitoring, by direct comparison of before and after states.

13. With decreasing grain size of the material forming the downstream slope of an embankment dam it becomes more difficult to correlate detail between the two images. The finer grains are too small to give recognisable texture and are likely to be disturbed by rain-wash. When, as is commonly the case, the slopes are protected by vegetation it is impossible to match the detail which constantly changes through growth, other seasonal changes or mowing.

14. For monitoring slopes of fine-grained material, with or without vegetation cover, it is necessary to make the best use of any more permanently textured features to form "hard" visual borders to areas of weaker texture. By careful choice of photograph stations and fields of view, distortion of the slope may be recognised against spillway channels, wave walls, instrument houses, steps, or drains on the slope or on berms. If closer control is required, texture can be introduced by adding marker posts or "landscaping" boulders.

Concrete and masonry dams and related structures

15. Concrete and masonry dams and all other structures associated with reservoirs provide sufficient permanent texture for photographic monitoring. In gravity or buttress dams differential movement can be detected at the joints between blocks. In arch dams stress cracking and distortion resulting from yielding of the abutments can be monitored.

16. The recommended use of photographic monitoring in structural inspection generally (Kalaugher, 1987), is to provide a record so that changes in detail can be recognised. The extent of cracks and changes in their widths, efflorescence, staining, spalling and other signs of deterioration including alkali aggregate reaction can all be monitored successfully. To enhance the resolution of detail on more distant surfaces, such as the face of a dam viewed from downstream, telephoto lenses can be used in the initial photography and for the monitoring. In addition to its use for dams, photographic monitoring is suitable for spillways, stilling basins, draw-off towers and wave walls.

USE OF SYSTEM FOR MONITORING RESERVOIR MARGINS

17. Besides its use as wave protection on the upstream faces of embankment dams, rip-rap or armourstone often performs a similar function in the prevention of erosion of vulnerable parts of reservoir rims. It may also form scour protection to spillway channels, stilling basins and to river banks downstream of the reservoir.

18. Any unprotected lengths of reservoir margins and river channels downstream of dams are recommended for photographic monitoring for signs of erosion. The large lengths involved and irregular nature of the features lend themselves to a visual rather than a metric approach. The development of small-scale erosional features, such as runnels on embankment dams or their abutments, should be monitored in the same way.

19. Their erratic displacement patterns and lack of hard geometrical features make landslides difficult subjects for monitoring by conventional surveying methods . The photographic monitoring system has been used extensively for in-situ monitoring of coastal landslides for hazard zonation purposes (Kalaugher & Grainger, 1990) in steep slopes of rock and in gentle slopes of soil. Naturally occurring or reservoir-induced

landslides on the valley sides surrounding reservoirs should be checked by this method.

20. The advantage of in-situ photographic comparison, over conventional surveying, is that in the vast majority of cases no movement will have occurred and this fact will be recognised instantly. A complete survey would otherwise be needed to demonstrate no change, at a far greater cost.

THE MONITORING INSTRUMENT

21. The optical monitoring instrument incorporates a Nikon F3 single-lens reflex camera and is compatible with a standard tribrach (with an optical plummet) for mounting on a surveying tripod.

22. The instrument is designed to enable:

(a) the scene to be viewed through the camera optics by one eye while the previously taken colour transparency of that scene is viewed by the other eye and the two images stereoscopically fused.

(b) fine adjustments to be made to rotations about horizontal and vertical axes so that the coverage of the direct view and the transparency can be matched exactly.

(c) an aerial image of the direct view to be formed in order to avoid the image being degraded by being captured on a focusing screen.

(d) the camera to be focused, in spite of the absence of a focusing screen, by elimination of parallax.

(e) viewing to appear quite natural in that when a standard (f = 50 mm) lens is fitted to the camera the instrument acts as a x1 telescope.

(f) telephoto lenses to be fitted to give more detailed coverage of selected areas.

(g) the two images to be rotated together to positions other than the right way up.

In this way the false (or time) parallaxes that result from displacements occurring since the time of photography can be brought parallel to the observer's eye-base. The false parallaxes lead to "anomalous" depth effects (Kalaugher, 1985) which can be confirmed if the sense of depth reverses when the images are rotated to inverted positions.

(h) both horizontal and vertical format photography to be taken.

23. Direct viewing through the camera optics has the following advantages:

(a) when setting up for monitoring, alignment to match the field of view of the transparency is greatly simplified when the transparency is fused with the directly-viewed scene.

(b) while monitoring is in progress, the camera remains correctly aligned. Repeat photography can be taken at a time when the natural lighting conditions are most suitable.

24. Monitoring will normally be carried out with an eye-base effectively reduced to zero by re-occupying the photograph station so that the usual stereoscopic depth is eliminated. The only stereoscopic depth observed would then be that resulting from false parallaxes. With its perspective fixed at the time of photography, the transparency can be combined with directly viewed perspectives that differ slightly from its own. By moving the equipment to one side or the other of the survey nail marking the photograph station, the direct view will have an altered perspective. Stereoscopic depth can thus be enhanced or reversed.

25. It is possible to measure the amount of displacement of any object which appears in the transparency and in the real scene. In cases where access is safe and feasible, the magnitudes of large displacements can be determined by locating in the real scene the former position of a recognisable feature (as recorded in the transparency) and measuring by tape to its new position. The smaller displacements, which result in anomalous depth, can be calculated, approximately if the magnitude of the anomalous depth can be estimated in the field, or more accurately by viewing the original and repeat photographs in a micro-photogrammetric system, such as an Adam Technology MPS-2 (Uren & Thomas, 1992; Stirling, 1990). The simplest and least time-consuming techniques of photogrammetry can be used since the before and after photographs will have exactly the same coverage.

DISCUSSION

26. Photographic confirmation may be not be necessary for identifying the first occurrence of an obvious distortion but, once the initially smooth curve or straight line has been disrupted, memory becomes less reliable for judging whether or not there have been further changes. Combination of the direct view with a transparency, taken after the initial distortion, will allow monitoring to be continued with confidence. Similar considerations apply to the appearance and subsequent development of landslide scarps on the reservoir rim.

27. The monitoring photographs, taken under controlled conditions, themselves represent condition surveys. If suitable stereo-pairs are taken in conjunction with the monitoring photographs the usefulness of the coverage is increased as topographic surveys can then be produced photogrammetrically in an instrument such as the MPS-2.

28. In the vast majority of inspection visits the conclusions are that there have been insignificant changes and that no follow-up action is therefore required. In-situ photographic comparison will greatly improve the confidence in such a decision. Where significant movements or changes are detected (which could have been overlooked by visual inspection alone) photography is repeated during the same visit. This photography, when compared in the office to the original, can be used for photogrammetric

measurement of the changes and to convince others that there really is a crisis. It may be decided that monitoring will have to be more or less continuous if accelerating changes are indicated. Decisions, such as whether or not to lower the level of water in the reservoir after unprecendented flood flows have passed through the spillway and stilling basin, can be taken rapidly on site. For use during emergencies it is sensible to have coverage on a wider scale in addition to photographs taken for monitoring specific parts of the reservoir works. The dam and its associated structures can then be monitored from a safe distance.

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Examples of problems involving clay cores affecting dam safety

Professor A.D.M. PENMAN, DSc, CEng, FICE, Geotechnical Engineering Consultant

SYNOPSIS. Troubles with clay cores usually involve cracking or loss of material. Traditional puddled clay cores made excellent seals but could lose material into rock fissures etc. Strong rolled clay could suffer hydraulic fracture. Bad design could lead to slips. Three case histories are considered.

INTRODUCTION.

1. The impact of the embankment dam on the environment is least of all dam types. Made, as it is, from local fill, it can often be covered by local vegetation so that the reservoir it impounds merges into the countryside as a natural lake and offers fishing, dinghy sailing and other outdoor activities that may previously have been unobtainable in the area. This factor, combined with its excellent safety record and low cost, has made it the world's most popular type of dam (Penman 1986).

2. From Telford's time until the middle of the 20th century, the traditional British design used a waterproof element of puddled clay. Originally it was used as below ground cutoff, extended as a central clay core, supported by earthfill shoulders. The clay was puddled, ie mixed and kneaded with added water by hand or in a powered pugmill: a steam powered machine was specified by James Simpson in 1864 (Binnie 1981). This was to break up any natural structure of fissures and sand partings, to produce a homogeneous soft plastic heavy material. It was compacted into place by the heels of the puddle gang, in trench below ground and between shutter-boards in the fill. It formed a flexible, plastic wall of about twice the weight of water, which effectively sealed into the valley sides, minimising leakage from the reservoir.

HYDRAULIC FRACTURE.

3. The low permeability of a fine grained material like silty clay relies on the particles being held close together by a total pressure greater than the

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pressure of the percolating water, which is then forced to flow through the minute pore spaces between particles. If the water pressure exceeds the total pressure, it can push open a passage between the particles, thrusting them aside and creating a fracture, the sides of which may erode as water streams through. The principle that, to prevent the ingress of water into the upstream face of a masonry dam, the compressive stress should exceed the reservoir pressure at all levels, was proposed by Maurice Lévy in 1895. This concept was not applied to clay cores until Lofquist(1951), discussing arching action and the low earth pressures measured in the narrow cores of two dams, suggested that the low pressures were not a good feature and might lead to the formation of horizontal cracks through the core. In 1957 he re-emphasised this aspect, suggesting that the vertical total pressure in a clay core should exceed reservoir water pressure at all levels. He made a re-assessment in 1992, suggesting that water under pressure against a homogeneous clay permeates into it and that an initial irregularity, crack or feature is necessary to initiate hydraulic fracture. This may be a contributory factor to the relatively satisfactory performance of puddled clay cores in contrast with those of harder, rolled clay.

4. Balderhead (48m) was one of the first major British dams to use a rolled clay core, compacted at or slightly below Proctor optimum water content. It suffered hydraulic fracture on first reservoir filling in 1966. There were similar occurrences in the cores of the two Norwegian dams, Hyttjiuvet (93m) and Viddalsvatn (75m) (Penman 1986). The risk has been reduced by use of more flexible, softer rolled cores, with the clay specified in terms of strength rather than water content (Kennard et al 1979).

MATAHINA DAM.

5. This 85m high embankment dam with slightly sloping rolled clay core supported by rockfill shoulders has been described by Galloway(1967 & 1970): a longitudinal section is shown by Fig.1. The weathered greywacke core showed typical w_L=30 and I_p=10. In common with rolled clay cores of that time, it was placed fairly dry as confirmed by the negligible measured construction pore pressures and high value c_u=230-300kN/m². The columnar and regularly jointed hard ignimbrite rock forming the abutments was not amenable to being trimmed to a smooth contact surface for the core. Benches formed naturally as the rock was excavated to remove weathered material and some were purposely made wider so that they could be used as roadways during construction. The intention

Fig. 1. Lonitudinal section of Matahina dam.

was that no benches should be left wider than 1.5m. As the core rose towards wider benches, they were trimmed back or partially filled with a fillet of concrete sloped at 45° so that the horizontal part of the bench would be no more than 1.5m. At higher levels this was reduced to 0.75m.

6. Unfortunately during first filling in 1967 when the reservoir level was 5m below TWL, there was a sudden increase in flow from the main drains, which reached a peak of 570l/s and became discoloured. Two weeks later, the weight of a bulldozer punched through the roof of a cavity above the right abutment, which proved to be a chimney of loose material extending down to a bench 12m below crest. Sherard (1973) used this case history as one of his examples of dam cracking. It was argued that it was caused by a concentrated leak through a differential settlement crack in the core associated with a bench on the abutment. Differential settlement would rotate the direction of principal stresses adjacent to the irregular abutment and reduce the minor principal stress sufficiently to allow of hydraulic fracture: it was not necessary to assume that an open crack had developed prior to erosion by escaping water. The zone was excavated and repaired.

7. Twenty years later, the dam was shaken by an earthquake of 6.6 magnitude which produced vertical and horizontal accelerations at the right crest of 3.20 and 7.65m/s² and at the left crest of 3.80 and 3.86m/s² (Gillon & Newton 1991). Leakage increased, but power generation was resumed 14 hours after the earthquake on completion of repair to transformers. At the left abutment, the ignimbrite settled 5mm increasing to 8mm, during the following 12 weeks while the crest settlement was 73mm increasing to 80mm.

8. Investigations were made at both abutments by inclined boreholes in the core and while some caving zones were found and there was evidence of some leakage, reservoir level was not lowered below a minimum for power generation. Additional piezometers

were installed that showed a head loss of only 4m through the core. Leakage measurements made at a weir on the main drainage outlet were found to under register because of losses into the ground water and weir flows ceased when the reservoir was lowered to minimum operational level.

9. Ten months later a sink hole appeared in the crest over the left abutment. Subsequently the reservoir was lowered and power generation had to cease. Detailed investigation showed core damage to a depth of 20m below the crest. During construction, the diversion tunnel had been permanently closed and there was no bottom outlet. Provision was made for reservoir lowering by a 3.66m diam. tunnel with an invert 44m below crest. At times of low river flow, it could lower the reservoir to about 37m below crest.

10. After considerations of grouting and diaphragm walls, it was decided to make a permanent repair by rebuilding both abutments to a) produce smooth contact surfaces and b) soft, flexible clay adjoining them so that a high enough contact pressure would be developed between clay and abutment to avoid further risk of hydraulic fracture. The investigation had revealed no signs of core damage below that associated with the benches at 27m and 32m below crest on the left and right abutments respectively. Accordingly it was agreed that the depth of excavation would only be sufficient to remove all damaged material, thereby leaving a few metres of freeboard above lowered reservoir level. Protective membranes were placed over the downstream rockfill in case increased river flows raised reservoir level, which could have flooded the exposed core but not endangered the dam.

11. Existing stresses induced by the full height dam below the depth of 30m were considered sufficient to minimise risk of hydraulic fracture below this depth where the deep borehole investigations had shown no signs of damage. The excavated core clay was stock piled on a level site downstream of the dam. A study was made of the possibility of adding bentonite to increase the I_p of the clay, but because some additional fill was required to replace contaminated core material, nearby clay pits were tested to find sources of fatter material. Eventually two sources were chosen and a trimix of excavated and new clays was mixed to produce a clay that had slightly improved properties and a higher water content. Following British practice, a strength rather than a water content specification was given for placement. To produce sufficient flexibility and softness, c_u=75kN/m² with two thirds of test results to be in

the range 65 to 85kN/m². Within 1m of the abutment face the most plastic, fine material was chosen with c_u=40 to 65kN/m².

12. After excavation to full depth, the abutments were trimmed and re-concreted, with reinforcement and rock anchors. On the left abutment there were 5 steps, the largest 8m wide, 28m below crest. The condition of the core was studied during excavation and vertically above each of the 5 steps there was a zone of cracking and major damage about 6m wide. Loss of material from these zones had caused the chimney which produced the sink hole at crest level. Below some of the steps, gaps had formed where the core had pulled away from the concrete. The abutment shapes before and after remedial work are shown by Fig.2.

Fig.2. Abutment shapes before and after remedial works.

13. An additional 49 piezometers were installed in the new core fill to check that sufficiently high pore pressures were developing during backfilling. In a wet core, pore pressures reflect total pressures and low pressures can be due to low total pressures caused by arching rather than due to drainage of pore water. Penman (1979) suggested that to avoid risk of hydraulic fracture, measured piezometric levels should be at or above reservoir level on first filling. In general this was achieved and the repaired core-abutment contact has continued to show satisfactory behaviour.

CARSINGTON DAM.

14. The failure of this 35m high embankment dam, nearing completion on 4 June 1984, provided a vivid example of the influence of core shape, position and condition on dam stability. The events relating to

the failure have been eloquently described by Coxon (1986). The Engineers, who had been designing dams for more than 100 years, used the traditional slopes of 1 on 3 upstream and 1 on 2½ downstream, but instead of a central puddled clay core over a concrete cutoff or grout curtain, they used a core of asymmetric section, with a large upstream toe projection (the "boot") extending under the shoulder fill as indicated by Fig 3. Bedrock was a grey

Fig.3 Section of Carsington dam 125m north of major section.

carboniferous mudstone with a layer of soft yellow clay at the surface. This had been eroded in the valley bottom and was not present under the foundation at the major dam section on which most of the stability analyses were carried out. On either side, however, the layer of yellow clay was largely left in position to protect the underlying shales from softening in the period between stripping and filling, with the argument that it would consolidate rapidly under the weight of dam fill. At a distance of 180m north of the major section, the ground surface sloped downwards at about 1 on 10 towards upstream. The upstream shoulder was founded on the "boot" and the in situ layer of soft yellow clay. This provided an excellent path for a slip surface through the core, boot and yellow clay, where the collapse began. The drag down from the sliding material caused adjacent sections to fail: a process which continued to and beyond the major section until a dam length of 380m had slipped.

15. The yellow clay and underlying weathered mudstone from the borrow pit were used for the core. Undisturbed samples taken from the clay in situ and from the core both exhibited brittle properties. This post-peak strength reduction encourages progressive failure. The dam collapse gave an incentive for renewed study of the problem and has produced new methods of analyses, eg. Potts et al (1990); Chen et al (1992).

16. Weather limited clay placing to approximately the months of April to October. The dam was almost brought to full height during two years and only a

few metres height remained to be placed during the 3rd year (1984). Concern had been expressed about stability and a small berm was built against the upstream toe of the major section during the 2nd season. Displacement monitoring was then begun (Aug.1983) to assess the stabilizing effect of the berm. The maximum rate of horizontal displacement at the major section was about 15mm/m rise of fill during the remainder of that season and some continued movement occurred during the winter shutdown.

17. During the winter shutdown 1983/4, lines of 5 settlement pegs were placed across the "crest". Prior to the last placing season, four conditions were noted: 1) piezometer CU1 ceased to function and behaved as though the connecting tubes were damaged, 2) pore pressures measured by one piezometer in the core had continued to rise after fill placement ceased, 3) a blockage developed in a USBR cross-arm settlement gauge on the dam axis, 4) the settlement pegs showed settlements of the core and upstream shoulder, as shown by Fig.4. These conditions could

Fig.4. Conditions at end of the 1983/4 winter shutdown.

be interpreted as being caused by movements along a slip surface forming through the core and the boot. Equally they could be interpreted as malfunctioning of instruments and settlement with displacement of the soft clay core. This interpretation was preferred by the Engineer.

18. Many factors contributed to the slip and re-analysis has given consideration to pre-formed slip surfaces in the yellow clay layer caused by periglacial action in the past and progressive failure, accentuated by the presence of the "boot", which acted as a stress concentrator. Its detrimental effect is illustrated by consideration of the part of the dam downstream from a vertical centre-line

through the core. The downstream shoulder, founded on the same layer of yellow clay on the other side of the slight knoll, sloping slightly downwards towards downstream, was supporting the equivalent of half of a traditional type of symmetric central core. Movements of this shoulder, with its steeper outer slope of 1 on 2½, were measured by horizontal gauges and found to be small. Creep during the last winter shutdown was only 2mm.

19. The indications are that the downstream shoulder remained stable and it is probable that with a symmetric central core, the upstream slope would also have been stable. Reconstruction of the collapsed length of the dam, described by Banyard et al (1992), used a central, traditional type of core.

YONKI DAM.

20. This 55m embankment dam in Papua New Guinea, had an asymmetric clay core with a vertical downstream face and an upstream face sloping at 2v on 1h, similar to that at Carsington but without a "boot". It was placed at a strength specification c_u=50 to 130kN/m² with 80% in the range 60 to 120kN/m². The residual soil, used for core and fill, was brittle: peak values of c'=0, ϕ'=28°, decreased to ϕ_r'=11°.

21. When fill level had reached 4m below final crest, blockages occurred in the inclinometer tubes I10 and I9 at positions that could have been on a potential slip surface as shown by Fig.5, with failure of the electrical piezometers ME3, 9 and 12. The similarity with pre-failure conditions at Carsington caused the Engineer to wisely halt construction. Rates of horizontal movement given by the inclinometers were 3mm/m rise of fill at the toe (I8) and 15mm/m at the crest (I10).

22. During the subsequent 4 week stability study, critical pore pressures dissipated to such an extent

Fig.5. Section of Yonki dam.

that it became safe to complete construction while maintaining detailed observation of movement and pore pressure for which extra instrumentation was installed. The blockage in I9 was found to be due to fill that had dropped down the tube and could be washed out. A 200mm long x 31.5mm diameter T.V. camera lowered into the tubes enabled the vertical closure of the telescopic joints to be measured and the blockage in I10 was found to be due to compression of a closed joint caused by settlement and was not due to shearing action. The rest period while the re-assessment was made gave time for pore pressure dissipation and the dam was completed without further incident and is behaving satisfactorily.

CONCLUSIONS.

23. By the beginning of February 1984, the instrumentation at Carsington showed strains sufficient to block the cross-arm gauge and to break a piezometer lead, with surface deformations that could be interpreted as the beginnings of a rotational slip movement. Had these movements been accepted as an indication of impending failure, there was time to build a substantial stabilising berm before the final few metres were added to crest height. It is quite understandable that when similar instrumentation malfunctions were observed at Yonki, it was thought that this dam was following the pattern of Carsington. Fortunately, in the event, this proved not to be the case, but it emphasises the difficulty of interpreting such events.

24. Although stability failure during embankment construction may have serious economic consequences, dam failure at the later operational stage with reservoir full can have much greater consequences for public safety. One of the most common causes of failure at this later stage is internal erosion which in some cases is associated with hydraulic fracture of the dam core. Hydraulic fractures at Matahina caused sufficient loss of material for swallow holes to appear at the crest. Had this been allowed to continue, it could eventually have lead to breaching. This experience demonstrated the need for smooth abutment surfaces and a soft flexible heavy core to ensure adequate total contact pressure not only against the abutments but also across any potential leakage surface through the core to prevent the ingress of reservoir water. The very bold decision was made to carry out the extensive work of replacing the end portions of the existing core and the continuing satisfactory behaviour of the dam has justified this major work.

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Remedial works to clay cores of UK embankment dams

P. TEDD, PhD, MICE, J.A. CHARLES, PhD, DSc(Eng), FICE, and
I.R. HOLTON, MSc, Building Research Establishment

SYNOPSIS. Most embankment dams in Britain have a central clay core and the long term durability and performance of these internal cores is of major concern for reservoir safety. Where deterioration occurs it may be expensive and difficult to repair; furthermore sudden deterioration could imperil the safety of the dam. This paper reviews remedial works to central clay cores using data from the BRE Dams database. The approaches adopted to rectifying defective cores are classified and the effectiveness and durability of the different remedial methods are addressed. Particular emphasis is given to the possible increased use of self-hardening slurry trench walls to repair cores.

INTRODUCTION

1. For well over 100 years British embankment dams were built using a traditional form of construction incorporating a central core of puddle clay. It is clear that the engineers who designed and built these dams recognised that the puddle clay core was the primary element ensuring the satisfactory performance and safety of the dam. Most British embankment dams were built before the modern concepts of filter design had been formulated. The narrow central cores of puddle clay which were commonly used were not protected by filters although it became customary to place a selected fill on either side of the core which in many cases may act as a filter. Some of these dams also had a clay filled cut-off trench, but in the later dams concrete filled cut-off trenches were used. The long term performance of the clay core forms the vital element in the safety of the dam. Serious incidents which have sometimes led to major remedial works to cores have been summarised by Charles and Boden (1985) and Charles (1986, 1989).

2. Remedial works may be required to deal with defects resulting from some fault in the core as built, some process of ageing or of deterioration. A clay core may deteriorate due to a number of different causes. Normally the rate of seepage through a clay barrier is very small and the scope for chemical deterioration is limited. Dispersive clays are not common in Britain. Physical deterioration is more common and may occur due to either surface effects such as drying during prolonged drawdown or processes located at depth such as internal erosion.

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3. The need for remedial works to the core is not just to make the dam more watertight but is principally related to maintaining the safety of the dam. There are three main hazards which could lead to catastrophic failure with an uncontrolled release of the reservoir water; internal erosion, external erosion and slope instability of the downstream fill or foundation. The remedial works to the core can be associated with one or more of these hazards. It should be noted that the two British dam failures which have caused large loss of life, Bilberry and Dale Dyke have both been attributed to internal erosion.

4. This paper reviews the methods that have been used to deal with defective cores. Although the emphasis is on the methods adopted to rectify problems at depth within the core, it should be recognised that many dams have had minor remedial works carried out to their cores as part of works to raise the dam and to increase the freeboard. Some of the major remedial works involving grouting and diaphragm walls are now several decades old and their durability can be assessed.

TYPES OF REMEDIAL WORKS

5. Although puddle clay cores have generally performed well, at some dams remedial works have been required to reduce leakage and to repair damage caused by internal erosion. The BRE dams database (Tedd et al, 1992) contains records of incidences of leakage associated with the core for 67 dams. The severity of the leakage has varied. The number of incidences of turbid leakage is small and not all of these were associated with erosion of the core; many were associated with leakage into draw-off tunnels and shafts. Repairs at some dams were carried out shortly after first filling, but many repairs have been undertaken during the last two decades when the dams were 100 years old or more.

6. The BRE dams database has been used to identify case histories and analyse the nature and frequency of remedial works. A wide variety of solutions have been adopted where problems have occurred with clay cores. Four approaches can be distinguished:

(A) construction of new watertight element at different location,
(B) raising of central clay core,
(C) repair of central clay core,
(D) improvement of drainage.

7. The first category (A) is the most radical type of solution and has usually involved the installation of an impermeable blanket on the upstream face of the dam. The fourth category (D) is the least radical solution in which the presence of leakage is accepted and the remedial measures seek to ensure that the flow of water is dealt with in such a way that it does not present a hazard to the safety of the embankment. Category (B) covers all those cases where the core is raised but is not repaired at depth. The need for raising may or may not be associated with some malfunctioning of the core. Category (C) refers to the cases in which the core is repaired. Several different repair techniques are in common use.

8. The Appendix lists the dams on the BRE database where remedial works have been undertaken to repair the core (category (C)). It is likely that many more remedial works have been carried out than those recorded in the database, particularly older dams where problems developed shortly after their construction and instances of dam raising. However it is believed that the following brief descriptions of the various methods give a representative view of remedial works to the central clay cores of British dams.

(A) CONSTRUCTION OF NEW WATERTIGHT ELEMENT AT DIFFERENT LOCATION

9. Problems with a central core may be overcome by, in effect, abandoning the core and constructing a new watertight element at a different location. This new watertight element has usually been an upstream clay blanket but the most extreme form of remedial work to a core was that used at Woodhead dam following severe leakage problems. A second embankment was completed in 1876 downstream of the first with an entirely new puddle clay core within it. The area between the two cores was infilled.

10. Five dams are known to have had upstream clay blankets added to them as the central core or draw-off works have proved unsatisfactory. Bateman placed upstream clay blankets on three dams, Torside, Holmestyes and Ogden in the 1850s. Most recently Barrow Compensation had a clay blanket added to it in 1982 to increase the seepage path through the dam.

11. Although the construction of a new upstream watertight element may appear to be a simple, if expensive, remedy for a defective core, there are several problems with this approach. A surface clay layer is more exposed than an internal core and is much more likely to dry out and crack during periods of reservoir drawdown. In his autobiography, Telford (1838), describes how he abandoned the use of puddle in exposed positions because of his observations of the effect of weathering. Bateman had major problems at Torside. Wave protection will be necessary and slope instability also could be a problem. Some of these difficulties could be overcome today by using a geomembrane instead of clay. However one major problem remains because the new upstream barrier cannot be easily linked to the old cut-off under the central core.

(B) RAISING OF CENTRAL CLAY CORE

12. Many clay cores have been raised on one or more occasions but these are often quite minor works. The raising is likely to have been associated with a perceived lack of freeboard. This may have occurred because of gradual settlement of the dam arising from consolidation of the core and cyclic loading of the dam due to reservoir fluctuations (Tedd et al, 1990). The freeboard could also be insufficient due to the adoption of more stringent flood requirements. Many cores of dams have been raised several times. Some leakage problems near the top of embankments are reported to be associated with the junction of the original core with the raised core.

(C) REPAIR OF CENTRAL CLAY CORE

13. Most remedial works fall in this category, in which an attempt is made to rectify the situation by repairing the core *in situ*. Generally the watertightness of the core is improved by some form of sealing material. There are several different ways of repairing the central core and these can be classified by the means used to install sealing material in the core; excavation, displacement or injection. The Appendix lists dam cores repaired by these different methods.

(C1) EXCAVATION OF CORE AND INSTALLATION OF SEALING MATERIAL

14. Several techniques involve excavation, or partial excavation, of the clay core followed by the installation of some form of sealing material. Clay and three types of slurry trench wall have been used.

(C1a) Replacement with clay

15. In the early days of puddle clay core dams, excavation using timbered trenches and subsequent replacement or repuddling of the clay was the principal method used to repair a faulty core at depth. This approach was used at Grizedale in 1869 and at Lower Lliw in 1880. When leakage occurred through the puddle clay core at Doe Park (Binnie, 1981), a trench was sunk immediately downstream of the core keeping the reservoir level above the bottom of the trench. Although the method was successful in identifying the location of leakage and enabling repairs to be made, it was considered too hazardous and was discontinued. Even if the observation of leakage at Doe Park was rightly considered to be too hazardous, the method still had the advantage that the nature and location of the erosion phenomena could be identified. Fissures in the puddle clay filled with coarse material were found at Lower Lliw, Neuadd Lower and Rhodeswood.

16. Replacement with clay is now only used when the core is being raised or when the problem is near the top of the core. The method was labour intensive and it would be very expensive to repair a core at depth using this technique today. Furthermore, although an eroded core can be repaired in this way, the situation that produced the erosion may still be present. The new puddle clay may be no more able to resist the erosive processes than was the original clay. An example of this is provided by Lower Lliw where problems recurred in 1883 only a few years after the repair work.

(C1b) Diaphragm wall single phase method

17. As far as is known, a self-hardening bentonite cement wall using the single phase method has only been used at Doffcocker Lodge (Connery, 1985) to replace the core within the embankment. This type of wall has, however, been used as a cut-off for the remedial works at Cod Beck (McLeish et al, 1987) and as a cut-off during the construction of a number of dams such as Alton Water Reservoir (Hetherington et al, 1975) and Kielder.

18. The technique involves excavation of a trench, typically 0.6m wide, under the support of a self-hardening bentonite/cement slurry which is left in the trench to set. Both the walls at Cod Beck and Doffcocker Lodge were excavated in panels approximately 3m long, down to a maximum depth of 24m and 9.5m respectively. The mix used at Doffcocker Lodge was reported to be 40kg of bentonite and 270kg of cement per 1000 litres of water. The properties of the hardened slurry at Doffcocker Lodge have not been reported except to say that the unconfined compressive strength was specified to be in the range 150 to 200kPa.

19. This type of cut-off wall has been used extensively to control lateral migration of pollutants and gas from landfill and contaminated land sites. It is estimated that in excess of 80 walls have been constructed some of which are up to 1.5 kilometres long. Excavation for many of the cut-off walls has been by hydraulic backhoe type excavator, but for depths in excess of 11m a diaphragm wall grab or cutter is used. The long term performance of these walls as containment barriers is currently being studied by BRE (Tedd et al, 1993) and it is useful to summarise some of the merits of the system and the properties of the hardened material. A general description of slurry trench cut-off walls and the properties of the material is given by Jefferis (1993).

20. Mixes used are generally in the range of:

Bentonite	40 - 60 kg
Cementitious material	100 - 350 kg
Water	1000 kg

The cementitious material usually consists of a blend of cement and granulated ground blastfurnace slag. It has been found that the use of slag gives a more impermeable material and that the cementitious content can be reduced to 90-150kg. The optimum percentage of the slag as cement replacement in the mix is generally between 60-80% of the total cementitious material. Below 60% there is little improvement in permeability and above 80% the material can be excessively brittle. Although the use of bentonite above 40kg gives a marked decrease in permeability, above 55kg the slurry may become so thick that it becomes unpumpable. The best design appears to be to use the maximum possible bentonite with the minimum cement content. Permeability of 10^{-9}m/s can be easily achieved with such mixes but if a reasonably flexible wall is to be achieved these permeabilities may take several months to be reached.

21. The hardened slurry exhibits brittle type behaviour at low confining pressures with a strain at failure of 0.2% to 2%. Only at confining pressures in excess of the unconfined compressive strength will plastic type behaviour be exhibited. Undrained triaxial tests give strengths in the region of 500kPa. The unit weight of the hardened material is typically between 11 and 12 kN/m^3.

22. The material has a very high moisture content, usually in excess of 250% and sometimes up to 400%. As a result it is very susceptible to drying shrinkage and will crack if not covered. Shrinkage cracks commonly occur at

the top of these walls typically down to a depth of 300mm, but cracks down to a depth of 1m have been found in a wall left uncovered for a few weeks. Therefore it is good practice to remove the top part of the wall and replace it with compacted clay. Once the cracks due to drying have been formed it appears that the material does not re-swell sufficiently to seal the cracks when resaturated.

(C1c) Diaphragm wall double phase method

23. The diaphragm walls used in remedial works to clay cores have in most cases been formed using the two stage method and plastic concrete. In the two stage method the excavation is initially supported by a bentonite slurry which is subsequently displaced by plastic concrete. Details of the mixes used in the slurry trench walls have been given by Charles (1989). The main advantage of the double phase method over the single phase method is that the material forming the cut-off wall is stronger and likely to be more erosion resistant.

24. The two stage slurry trench wall method is known to have been used at five dams where the clay core or parts of it have been replaced with plastic concrete. Although slurry walls have only been used at a relatively small number of dams, the scale of the remedial works and the age and size of dam has varied considerably, (see Appendix). Relatively deep walls have been constructed at Balderhead (46m), Lluest Wen (35m) and Withens Clough (27m). However smaller scale operations have been carried out as at Blenheim Lake where the wall was only 5m deep (Wade, 1993). Slurry walls tend to be associated with large scale operations and in the USA walls greater than 100m in depth have been constructed.

25. As far as is known all the slurry walls have performed satisfactorily. It should be noted that at Asprokremmos Dam in Cyprus, where a diaphragm wall was used as the cut-off, a 500mm deep gap was found between the bottom of the wall and bedrock and special measures were needed to seal this. The problem was attributed to the use of a concreting tremie pipe which was too short (Taylor, 1985).

(C1d) Replacement with slurry in open excavation

26. At two dams, Little Deny and Woodburn (Durward, 1993), bentonite/cement/pfa slurry described as a plastic concrete was placed into open trenches which were supported with trench sheets and struts. At Little Denny, the trench was excavated down to approximately 3m depth, 2m either side of the spillway. At Woodburn the trench depth was approximately 2.5m and 150m long. This approach has more in common with clay replacement (category C1a).

(C2) DISPLACEMENT OF CORE AND INSTALLATION OF SEALING MATERIAL

27. Instead of excavating clay from the core, a sealing element may be

installed within the core by displacement techniques. The action of displacement may cause some remoulding of the clay which is likely to be beneficial.

(C2a) Sheet piling

28. One of the simplest approaches to remedial works is to drive sheet piling into the core. Sheet piling is known to have been driven into 12 dams, but at least half of those dam probably do not have clay cores. Usually steel sheet piling is used, although corrugated asbestos sheets were used to a depth of 2.5m at Lockwood reservoir (Ray and Bulmer, 1982), but there were problems.

29. Although the method is relatively cheap and simple it is of limited applicability. Leakage may occur through the clutches of the sheet piles. Generally sheet piling has been used to solve leakage problems near the top of the dam and is not known to have been used to depths greater than 5m.

(C2b) Vibrated beam wall

30. The vibrated beam wall is the method where a thin grout screen is installed by injecting grout into the void formed by a vibrating I section beam. At Banbury the method was used to a depth of 7m in 1973 and has proved successful for at least 20 years. The system may be classed as a displacement and injection system. The method is used extensively in Austria and Germany to form permanent cut-offs and has been used to down to depths of 25m (Barker and Esnault, 1992).

(C3) INJECTION GROUTING

31. Within the last 30 years grouting has become the most common form of remedial treatment carried out to repair clay cores and foundations of dams. Grouting can be used to deal with general problems of leakage through the core or with more localised leakage problems, for example, those associated with the draw-off tunnel.

32. Pressure grouting was invented and first applied in 1802 by a French engineer, Charles Berigny, who named it the "Injection Process". Its first use in dams was in 1876 by Thomas Hawksley to seal fissures with cement grout in the rock beneath Tunstall dam in Weardale. In 1877 and 1878 the same process was used at Cowm dam in Lancashire.

33. The first use of grouting for remedial works to embankment dams in the UK was at the Walshaw Dean reservoirs which were leaking to the extent of more than 50 l/s when cement grout was used by H P Hill (Barns, 1927). Boreholes were put into rock below the deep cut-off trench and injected with a grout of 20:1 water cement ratio which was decreased to 10:1. 1386 tons of grout were used. This was not remedial work to the core or cut-off as such but rather to the foundation to prevent erosion into the foundation. Grouting of fissure rock foundations for modern dams during construction is now established practice to create grout curtains or blankets, (Bruce, 1982).

34. Various developments in grouting methods and materials have taken place and have been used in the repair of cores. Early grouts were cement grouts and were only capable of penetrating fissured rock and gravels. There has been an increased use of bentonite cement grout and more recently microfine cement grouts which can penetrate down to silt size.

35. *Tube à manchette* grouting was invented by Ischy in the 1930s and has been described in detail by Glossop (1961). It was first used at the Bou Hanifia dam in Algeria. *Tube à manchette* grouting allows grout to be placed at specific locations and is known to have been used to repair the cores of at least four dams. The particular advantage of this method is that once the tube is in place, it can be used repeatedly with a whole range of grouts.

(D) IMPROVEMENT OF DRAINAGE

36. The amount of water leaking through a core is unlikely to be so large that it causes any significant problem for water storage. The significance of a leak is related to its potential for causing internal erosion or instability. An alternative approach to attempting to repair a defective core is therefore to improve drainage so that water leaking through the core is safely conveyed by drains with filter protection. This type of approach is adopted in the design of modern dams but it is not easy to install such systems in existing dams. Nevertheless improved drainage has been used in several remedial works such as the systems installed in near horizontal holes drilled from the downstream slope at Cod Beck in 1986 (McLeish et al, 1987) and in near vertical holes drilled from the crest at Mitchell's House in 1986 (Rofe et al, 1990). Where drainage has been installed it is not always known whether the water being drained was leakage through the core.

CONCLUDING REMARKS

37. The loss of water through a leaking clay core is not likely to be of concern in itself. It is the risk that the leaking water will cause internal erosion of the core and downstream slope instability that causes concern and, in particular, that erosion could lead to a sudden catastrophic failure of the dam. When clay cores malfunction, effective, economic and durable remedial works are required. Access to the crest of the dam will be required for the heavy equipment needed for the remedial work. It will usually be necessary to draw the reservoir down to ensure safety during the works. Although the Appendix lists many remedial works, it is not complete and it may contain cases which are not strictly relevant to the present subject e.g. foundation grouting, drainage not associated with core leakage. However it gives a good impression of the types of works which have been carried out. In the early years of puddle clay cores the remedy for defects was likely to involve either an upstream blanket or excavation and repuddling of the central core. In recent years grouting has been the most common form of remedial work to defective clay cores.

38. The construction of a new watertight element on the upstream face has

obvious advantages in that it obviates the need to identify the cause and location of the problem in the core, which can be difficult. One substantial disadvantage is that it will not be possible to link the new watertight element with the old cut-off under the dam centre-line.

39. Excavation and refilling is suitable for shallow repairs. This approach is labour intensive and would not now be economic for deep repairs. It had the advantage that the eroded core could be examined but the disadvantage that the repuddled clay was liable to further erosion. Two phase slurry trench walls using plastic concrete solve this problem and have been used at a number of dams.

40. The widespread use of single phase slurry trench cut-off walls as containment barriers may lead to them receiving more attention as possible solutions for repairing clay cores. One major difference in their use in embankment dams is that the hydraulic gradient will be large, of the order of 50. A single phase slurry wall can be used to a great depth. It can replace unsuitable or softened clay and fill voids with more erosion resistant material while still supporting the existing core. However the double phase wall is stronger and more erosion resistant than the single phase wall.

41. Sheet piling does not involve excavation of material and the method of installation is well understood. Leakage through the clutches remains a hazard. The grout screen also does not involve excavation of material and the grout fills any voids in addition. These displacement techniques have the advantage that they remould the adjacent clay.

42. Injection grouting is the only method available to seal a fissured foundation. Core grouting can be used for localised repair and to a great depth. The efficacy and durability of grouting puddle clay cores could be questioned.

43. While the concept of dealing with a defective core by improved drainage is sound, in practice it will be difficult to achieve this in a satisfactory manner. It may be used in addition to other methods in some cases.

44. Quality control seems certain to be an increasingly important consideration for all remedial methods. This will involve materials testing before and after placement, control supervision during construction and performance testing of the completed structure. The long term performance and durability of the various methods should be investigated further.

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APPENDIX: REMEDIAL WORKS TO CENTRAL CLAY CORES AND CUT-OFFS

C1. EXCAVATION OF CORE AND INSTALLATION OF SEALING MATERIAL

Dam	Date built	Height m	Repair date	Reference
C1a. Replacement with clay				
Castle Howard GL	1798	6	1967	Kennard, 1972
Doe Park	1866	18	1864	Binnie, 1981
Grizedale	1866	22	1869	Arthur, 1911
Island Barn	1911	9		Cronin, 1951
Lliw Lower	1867	24	1880	Binnie, 1981
Neuadd Lower	1884	16		Deacon, 1902
Rhodeswood	1855	21	c1855	Bateman, 1884
Rivington Lower	1857	18	1932	Binnie, 1981
Roddlesworth Up.	1865	20	1904	Binnie, 1981
C1b. Diaphragm wall single phase method				
Doffcocker Lodge	1870	6.5	1985 (9.5)	Connery, 1985
Cod Beck	1953	23	1987 (24)	McLeish et al, 1987
C1c. Diaphragm wall double phase method				
Balderhead	1964	48	1968 (46)	Vaughan et al, 1970
Blenheim Lake	1760	7	1981 (5)	Wade, 1993
Earlswood Com. B.		5		
Lluest Wen	1898	20	1973 (35)	Twort, 1977
Withens Clough	1894	22	1972 (27)	Arah, 1975
C1d. Replacement with slurry in open excavation				
Little Deny	1888	18	1988 (3)	Durward, 1993
Woodburn	1894	11	1982 (2.5)	Durward, 1993

( ) depth of wall

C2. DISPLACEMENT OF CORE AND INSTALLATION OF SEALING ELEMENT

Dam	Date built	Height m	Repair date	Reference
C2a. Sheet piling				
Burbage	1904	20		
Castle Howard G.L.	1798	6	1938/45	Kennard, 1975
Drayton	1804	7	1971	
Loch an Add East	1835	2	1967	
Loch an Add West	1835	1	1967	
Lockwood	1903	9	1980	Ray & Bulmer, 1982
Pebley	1776	9	1980	
Randieford	1938	11	1985	
Saddington	1799	9	1940	
Shon Sheffrey	1895	14	1984	
Wilstone No.3	1839	8	1946	

C2. DISPLACEMENT OF CORE AND INSTALLATION OF SEALING ELEMENT

Dam	Date built	Height m	Repair date	Reference
C2b. Vibrated beam wall				
Banbury	1903	10	1973	Ray & Bulmer, 1982
East Warwick	1897	6		
King George V	1912	8		
West Warwick	1897	6		

C3 INJECTION GROUTING

Dam	Date built	Height m	Repair date	Reference
Grouting of core				
Balderhead	1964	48	1968	Vaughan et al, 1970
Barkin House	1908	7	1963	
Barrow No.1	1850	8	1990	
Blenheim Lake	1760	7	1968	
Castle Howard	1798	6	1948	Kennard,1975
Church Wilne	1971	10	1973	
Clubbiedean	1850	17	c1975	Binnie,1981
Coulter	1907	24	1977	Gallacher,1988
Cwmtillery	1895	13	1954/73	Little,1975
Daill Loch W.	1932	8	1985	
Dubbs	1869	7	1933	
Earnsdale	1863	24	1976	
Greenbooth	1962	35	1983	Flemming & Rossington, 1985
High Newton	1878	6	1957	
Lluest Wen	1896	20	1915	Little,1975
Lluest Wen	1896	20	1973	Twort,1977
Loch na Bric W.	1932	6	1985	
Pentwyn	1863	10		Binnie, 1987
Rhodeswood	1852	21	1974	
Shon Sheffrey	1895	15	1984	
Torduff	1851	26	c1980	Binnie, 1981
Walshaw Dean Low.	1907	22	1982	Charles and Watts, 1987
Walshaw Dean Mid.	1907	24	1941	Wood, 1946
Wideford	1908	11	1984	McLeish et al, 1987
Withens Clough	1894	22	1972	Arah, 1975
Grouting of foundation				
Church Wilne	1971	10	1973	
Clubbiedean	1850	17	c1975	Binnie, 1981
Cowm	1875		1879/84	
Greenbooth	1962	35	1983	Flemming & Rossington, 1985
Lower Slade	1889	11	1971	Kennard, 1972
Walshaw Dean Low.	1907	22	1911-15	Bowtell, 1979
			1982	Charles and Watts, 1987
Walshaw Dean Mid.	1907	24	1908/41	Wood, 1946
Wideford	1908	11	1984	McLeish et al, 1987
Withens Clough	1894	22	1972	Arah, 1975

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Environmental assessment of reservoir schemes

A.J. ELDER, BSc(Eng), CEng, MICE, MIWEM, Partner, Rofe Kennard and Lapworth, Director, ARK Associates

SYNOPSIS. The requirements of Environmental Assessment form a logical part of the design process and should be welcomed by reservoir developers as a way of demonstrating the benefits of well designed schemes. Dam designers are perhaps better placed than others in the industry to carry out assessment because of experience gained before the word "environmental" became fashionable. The paper describes assessment work carried out on three schemes in the 1970s, 1980s and 1990s.

INTRODUCTION

1. The aim of this paper is to describe the practical use of environmental assessment in relation to reservoir schemes with particular reference to two projects also involving mineral extraction. Reservoirs cover large areas of land, are often in or near areas of outstanding natural beauty or special scientific interest and frequently have a major landscape impact. The issues raised cover a wide spectrum and are of relevance to other water sector schemes as well as developments generally.

2. The main tenor of the paper is to welcome environmental assessment legislation and to stress that developers should have nothing to fear from the requirements. Assessment carried out well should ease the planning process, should reduce the risk of planning refusal and should result in better schemes. Even where a formal assessment is not required, an environmental appraisal should demonstrate to any interested parties that the proposals have been properly formulated.

PAST EXPERIENCE - ARDINGLY RESERVOIR 1970s

3. Assessment covers a systematic study of the likely effects of a development, consultation with interested parties and reporting on the study to the appropriate body. Clearly, even before current legislation many schemes included such work but it was not called environmental assessment. In my experience, promotion of reservoir schemes has always included appraisal of the impacts, although in the past engineering design was probably taken to a reasonably detailed level before impacts

were assessed. As an example, I have selected the Ardingly Reservoir Scheme in Sussex which was promoted approximately 20 years ago and was constructed between 1976 and 1978. Rofe, Kennard & Lapworth (RKL) were the consultants for the joint clients, Southern Water Authority and Mid-Sussex Water Company. It is interesting to quote from the paper 'Planning & Promotion' presented to an ICE branch meeting by Forster, Beazley & Kennard (1977):

> *'The promotion of a reservoir scheme these days can attract opposition from many quarters and from the very first approach to landowners for access for ground exploration, every effort was made to explain the need and consequences of the proposal. With the decision to proceed with the scheme these efforts were stepped-up to embrace all interested parties. In preparation for this exercise, Dame Sylvia Crowe was retained as Consultant Landscape Architect which enabled many difficult questions to be answered positively. The project raised great interest locally and such details as were known were displayed on the Southern Water Authority stand at the South of England Show at Ardingly. Any questions which were asked and could not be answered at the time were noted and written replies sent at a later date.'*

4. The paper goes on to describe the negotiations with one of the landowners, the National Trust, following which their gardens at Wakehurst Place were extended to include a water garden at the upstream limit of the reservoir. Attention was also drawn to the regular meetings of the working party whose brief included 'to incorporate in the proposed works suggestions put forward by other interested parties during the consultative procedures.'. This illustrates a sound addressing of environmental assessment principles long before the term became fashionable.

RECENT EXPERIENCE - BLASHFORD LAKES

5. More recently the Blashford Lakes Scheme (Eastwood, Elder & Pearson, 1981) has been promoted by Wessex Water with phase 1 gaining planning consent prior to present legislation and phase 2 following the present EA regulations. The project is of particular interest being immediately adjacent to the New Forest on one side and next to the River Avon Site of Special Scientific Interest on the other. The scheme comprised the creation of water supply reservoirs from worked out gravel pits which themselves had been excavated over a 40 year period. There had thus been time for recreational users to have become well established, one of the lakes had a national reputation as a prime coarse fishery, and for areas of nature conservation interest to have been created; one lake was being developed by a local conservation group.

6. RKL were appointed to carry out a feasibility study in 1984 and immediately assembled a team of engineers, landscape architects and ecological specialists to carry out site surveys, collect data, consult with interested parties and formulate design proposals. This work culminated in a feasibility report which recommended that six of the ten existing lakes, four of which were still being worked by gravel companies, should be developed for water supply. The report stressed the need for the design to take into account ecological and landscape enhancement and amelioration measures and for a management strategy to be formulated for all the lakes. Subsequently, a planning application was successful for phase 1 of the scheme, including use of three of the lakes for water supply and including a river intake, for refilling the lakes, and a water treatment works. Construction took place between 1987 and 1990. The close liaison with the planning authority, New Forest District Council, continued during construction when quarterly liaison meetings were held to keep them informed of progress and issues encountered.

7. The phase 2 works for development of three more lakes included a formal environmental assessment which was a logical progression of the work carried out in the previous seven years. This phase had a greater visual impact because of the raised embankments but this was mitigated by continuing the philosophy of sensitive landscape design, appropriate grading of embankment slopes and tree and shrub planting. An additional issue was the local concern of seepage from the reservoirs causing flooding problems. There was a history of localised flooding in the area which had to be studied and it was then demonstrated that this was due to poor drainage of surface water and not to high groundwater levels. Controlled reservoir seepage would not, therefore, affect the local flooding problem. Planning consent for phase 2 was granted early in 1992.

PRESENT EXPERIENCE - TESTWOOD LAKES

8. Promotion of the Testwood Lakes Scheme commenced after implementation of EA legislation and it is, therefore, a good example of current practice.

9. The scheme was proposed by the Hampshire Division of Southern Water Services Ltd to create bankside storage for their existing River Test Intake at Testwood and to provide water to meet peak demands in drought conditions. The existing facility comprised an intake with a licence to abstract up to 136 Ml/d subject to maintenance of a minimum residual flow in the river. Water was treated at the adjacent Testwood Water Supply Works.

10. The site had been identified as having potential for water storage in resources studies carried out over 30 years ago and had been safeguarded, along with other sites, in local authority plans. In 1990 Southern Water appointed RKL to carry out a feasibility study into creation of the storage on land which they had already purchased. The limits of the site varied considerably from the area investigated in previous studies.

11. The site is shown on Fig 1 and is bounded by the M27 motorway to the north, the River Blackwater and the Test Valley to the east, a country lane and residential properties to the west and an industrial estate to the south. In addition, major gas mains, overhead power lines and a footpath cross the site. Work on the 6 month feasibility study started on a number of fronts with a team set up to cover engineering landscape ecology, geology, hydrology and other disciplines. Site works were put in hand covering:

Ground Investigation
Topographical Survey
Botanical, Invertebrate and Bird Surveys
Visual Impact

Fig. 1. Testwood Lakes Scheme – Proposed Development Plan

12. The site surveys were supported by data collection and desk studies, including consultations with English Nature, Hampshire Wildlife Trust, angling clubs, local authorities, the NRA, public utilities, the DoT and MAFF. The information gained and issues raised were taken into account in developing the outline design which formed part of the feasibility report. Specialist studies included hydrological modelling of the rivers to assess the impact on flooding of loss of part of the flood plain.

13. The feasibility report recommended creation of 2000 Ml of water storage held in three lakes by excavation to a 10m depth and construction of perimeter embankments up to 6m high. The development would include removal of 1 million m^3 of gravels and over 100,000m^3 of peat from the site. It was acknowledged that some of the hedgerows, trees and woodland must be retained and that new ecological habitats should be created. A balance was, therefore, proposed with approximately 80% of the 58 hectare site being disturbed to create 23 ha of water. The embankments were wider than required for engineering reasons to enable sympathetic landscaping and flatter slopes around the lakes.

14. The report concluded that promotion of the scheme would require a formal environmental assessment which should include more detailed studies to:

Justify the need for the scheme
Confirm that no better alternative ways of meeting the need were available
Assess traffic and noise impacts on local residents
Assess implications on wintering wildfowl and wader breeding areas

15. The recommendations of the feasibility report were approved by Southern Water in late 1990 and the decision made to pursue promotion of the scheme. The formal environmental assessment was thus put in hand.

16. The Environmental Statement describes the assessment and was compiled as a single bound document supported by 13 background reports. Each report covered specialist areas of the assessment and was produced by different members of the project team. The titles of these reports were:

1. The Planning Background
2. The Need for the Scheme
3. Construction and Operation
4. Extraction Proposals
5. Geological Assessment
6. Alternatives
7. Assessment of the Rownhams Site

8. Landscape
9. Ecology
10. Hydrology and Water Quality
11. Traffic and Access
12. Recreation
13. Noise

17. The thirteen individual reports were produced to a standard format by seven different organisations which illustrates the multi-disciplinary nature of the team. It is worth stressing that coordination of the team was based around regular monthly meetings chaired by Southern Water where all members could report on progress and discuss details. In addition, team members met separately to liaise on design matters or consult with outside parties. It is clearly vital that not only is an effective project management framework set up but that the individuals are capable of working in a team.

18. The main statement provided a summary of the 13 background reports but technically the complete set of documents formed the Environmental Statement. A separate Non-Technical Summary was produced as required by the regulations. Each background report also included its own non-technical summary.

19. The statement was compiled under five headings as follows:

1. The Planning Applications
2. Defining the Issues
3. The General Issues
4. The Environmental Impact of the Testwood Proposals
5. Conclusions

20. Part One summarised the proposals and pointed out that the scheme was the subject of two linked planning applications: one for the mineral extraction (to be determined by the County Council) and one for the reservoir construction (to be determined by the District Council).

21. Part Two referred to relevant statutes and regulations, listed all consultees and the results of consultations from which the issues were defined.

22. Part Three summarised factors relating to: the need for the scheme, including security of supply, demand forecasts, population growth, leakage control; and alternatives both to the concept of water storage and to the site itself.

23. Part Four covered the environmental impact of the Testwood proposals including both mineral extraction and reservoir construction. The primary impacts were visual/landscape, ecology, hydrology, recreation, traffic and noise. Secondary factors considered but for which

no significant impacts were identified were archaeology and the historic environment, agriculture, accident risk, air and climate, and employment.
24. Part Five summarised the conclusions from the assessment of impacts.

THE ISSUES

25. It is not the purpose of this paper to describe all the issues included in the environmental statement but there were a number which are of particular interest and relate directly to the water cycle.
26. Water Area – The decision that the water area on the site should not be maximised but that certain woodland and hedgerow features should remain was based on the desire to create a balanced and attractive landscape and to provide areas for nature conservation and public access including car parking. This approach can be seen to demonstrate Southern Water's duty under the Water Industry Act which requires undertakers to further conservation and enhancement, to allow public access to land and to make sites available for recreation.
27. Need for the Scheme – The primary need for the scheme was the requirement to have security of supply and half the storage ie 1000 Ml was allocated as being retained to meet emergency needs. Such an emergency could be a failure of the Testwood source due to pollution of the River Test causing the intake to be shutdown or a failure of another source where there is no provision for storage. The secondary need, for which the remaining 1000 Ml would be available, was to enable Southern Water to meet increasing demands in the 21st century. The demand forecast took account of population growth forecast by the County Council, the continuing increase in per capita consumption, pipe network leakage reduction targets and the assumption that all new properties would be metered and their demand would, therefore, be reduced.
28. Alternatives – The main alternative to the Testwood site was at Rownhams, located in Test Valley Borough, where an area had also been allocated for reservoir development in local authority plans. This alternative was studied in a considerable amount of detail so that it could be compared fairly with Testwood to demonstrate which was the better on environmental grounds. It was concluded without doubt that there was a greater adverse impact for a Rownhams development. Despite these conclusions, representatives of Test Valley Borough Council continued to lobby for Rownhams to be promoted in place of Testwood; they were evidently keen for the water area to form part of a proposed country park.
29. Hydrology – The embanked reservoirs would result in loss of part of the flood plain which carries the Rivers Blackwater and Test. Computer modelling was carried out which demonstrated that the increase in depth of flooding at various return periods would be less than 50mm. Such a value was accepted by the NRA as being of minor significance, although

the owners of Broadlands Lake were particularly concerned about the possibility of fish being flushed out of the lake during flood events.

30. Hydrogeology – Excavation up to 10m below ground level and lowering of groundwater by approximately 8m was identified as a major issue. Concern was expressed about the impact by adjacent landowners, the NRA and the wildlife trust which managed the Site of Special Scientific Interest downstream. The perception that lowering of groundwater levels would be detrimental was taken up by many people although the reasons for the concern were varied:

> the NRA's interests were that groundwater lowering should not reduce river flows and that Southern Water should not be inadvertently abstracting groundwater for public supply when the lakes were drawn down;
>
> the two major landowners were concerned that lake and river water levels should not be affected because of the potential impact on fishing;
>
> the wildlife trust were worried that a drop in groundwater levels on the SSSI could destroy wetland habitats or result in salinity changes;
>
> the project team was also aware that dewatering would introduce the risk of settlement of the motorway embankment.

31. The mitigating measures proposed were a cut-off trench at the north of the site to limit drawdown beneath the motorway and a comprehensive monitoring network around the site. The network included boreholes to monitor groundwater levels fortnightly with loggers in two of the holes and survey points on the M27 embankment to detect settlement or lateral displacement prior to, during and after construction. Although borehole pumping tests had been carried out, it was acknowledged that drawdown from large scale pit dewatering could not be predicted with accuracy. The agreed approach was to monitor levels as the work proceeded and, if necessary, extend the cut-off trench down the site.

32. Since completion of the environmental assessment a groundwater model of the site and surroundings has been developed using MODFLOW finite difference software. The model has been calibrated against groundwater levels taken on site in the steady state and during borehole pumping tests. The model has been used to evaluate the effects of dewatering individual gravel pits, the mitigating impact of cut-off measures and the operational use of the reservoirs. Estimated dewatering pumping rates were used to design settling lagoons and produce figures for obtaining NRA discharge consents.

33. Noise – The impact of noise during construction was partly mitigated by the fact that background levels were on the high side because of the existing major roads. However, noise bunds were proposed around the gravel processing plant and between the site and the residential properties. This kept noise level to less than 10 dB(A) above the background level.

PROGRAMME

34. The Testwood Lakes Feasibility Study was carried out in 6 months between March and October 1990. The formal environmental assessment commenced in late 1990 and the planning application was submitted in August 1991 with planning committee approval being given in January 1992.

CONCLUDING REMARKS

35. I have referred to three reservoir schemes in this paper and have described the formal environmental assessment carried out for Testwood Lakes scheme to demonstrate EA procedures in practice. It is not a new requirement for the water industry to have to study environmental issues with reservoir promotion but the legislation now requires that the work is carried out within a formal framework. High quality environmental assessment should enable good schemes to be promoted successfully and should help to rebuff the glib statements made by some politicians and officers of regulatory authorities that reservoir developments are undesirable.

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Environmental implications of constructing a concrete reservoir within a 100 year old dam

A.J. ELDER, BSc(Eng), CEng, MICE, MIWEM, Partner, Rofe Kennard and Lapworth, Director, ARK Associates, M. FLETCHER, BEng, MSc, CGeol, FGS, Regional Manager, Rofe Kennard and Lapworth, and P.R. RICE, BSc, Lecturer, University of Derby, Consultant, ARK Associates

SYNOPSIS. Monkswood Reservoir comprises an earthfill embankment with a berm on the downstream face. The reservoir is supplied by springwater from natural springs on the surrounding valley slopes. It is situated within an Area of Outstanding Natural Beauty (AONB) adjacent to a Site of Special Scientific Interest (SSSI). For water quality reasons, diversion of the springwater to a holding tank adjacent to the reservoir was investigated and subsequently implemented. The holding tank was sited within the dam berm after careful consideration of the environmental and geotechnical constraints.

INTRODUCTION

1. The 100 year old Monkswood Dam has been storing water from adjacent springs since construction in 1896. In 1990, the owners Wessex Water required storage to be transferred to a roofed tank to safeguard the high quality source water from pollution or algal blooms. A feasibility study was instigated to assess the appropriate size of structure and possible locations.

BACKGROUND

2. The embankment dam that retains Monkswood Reservoir is approximately 160 metres long, with a maximum height of 15 metres at the centre of the valley. The original drawings of 1893 show the embankment consisted of a puddle clay core supported by inner zones of "clayey material" and outer zones of "sandy or mostly stony material". The cross section shows the embankment was founded on "oolitic sand, peat and alluvial deposits". Beneath the toe on each side of the dam, the foundation material was apparently replaced with "dry rubble". According to the drawings a cut-off trench follows the line of the embankment and extends 12 to 17m below original ground level. Later ground investigation has confirmed that the dam is founded on Lower Lias Clay which the relevant 1:10560 scale British Geological Survey Sheet indicates is "foundered and landslipped strata".

Reservoir safety and environment. Thomas Telford, London, 1994

3. Monkswood Dam has had a history of problems ever since work commenced on site. During construction the design slopes were flattened and since completion numerous other problems have occurred including a number of hillside slips, deformation of the outlet tunnel (steel strutting was erected in 1928), seepage and subsidence of the downstream slope. Several of these incidents are referred to in the paper on failures of UK embankment dams by Charles and Boden 1985. This describes the slips occurring during construction in 1896 and the downstream slope subsidence in 1931.

4. Various investigations have taken place over the years and remedial works have been undertaken including grouting of the core, installation of light sheet piling along the crest and major works to the bywash channel (which carries surface drainage around the perimeter of the reservoir to prevent pollution of the stored spring water). The most recent works were the addition of an emergency spillway in 1990 involving lowering of a 15m length of the crest. The site clearly has had an extremely interesting history.

APPRECIATION OF THE ENVIRONMENTAL ISSUES AFFECTING THE SITE

5. The spring water holding tank was to be located on land owned by Wessex Water Services within the existing Monkswood Reservoir compound as the adjacent land formed a Site of Special Scientific Interest (SSSI). The location was largely determined by space and geotechnical constraints. Rofe, Kennard & Lapworth and subsidiaries, ARK Associates, environmental consultants and RKL Geotechnical Ltd, geotechnical specialists, worked closely together from the early stages of the scheme to ensure environmental issues were identified and dealt with, as far as possible, in the design phase. An environmental appraisal was carried out prior to the detailed design of the holding tank. The tank location is shown on Fig. 1.

6. It was evident from the outset that the most significant environmental issue to be addressed would be the visual impact of constructing a concrete holding tank in this attractive rural landscape of rolling hills and pasture land. Indeed Monkswood Reservoir falls within the proposed extension of the Cotswold Area of Outstanding Natural Beauty. An assessment was carried out to determine the potential visual impact of the holding tank within the undulating terrain of the St Catherine's valley. From site surveys, it was determined that the holding tank would be the focal point in the valley for three properties and would be visible from more distant views to users of three public footpaths in the vicinity. The design of the holding tank was therefore orientated towards ameliorating these potential impacts. The ecology of Wessex Water's operational land was not a significant issue as very few trees were present except for a small orchard at the toe of the dam which was valued

by local residents. The principle vegetation of the site was grassland which was relatively species poor showing characteristics typical of disturbed habitats.

7. The prime concern of English Nature and local interest groups was that the adjacent SSSI should not be affected and that no surplus excavated materials should encroach on the SSSI (although much of the land was owned by Wessex Water).

8. Other environmental concerns related to effects during construction including potential pollution of the local stream and traffic use of the narrow country lane which was the only access to the site. A number of influential property owners resided in the valley which fact concentrated the minds of those involved in the development, including local councillors and council officials.

Fig. 1. Monkswood Springs Holding Tank – Tank Layout – Single Cell

MINIMISATION OF VISUAL AND ENVIRONMENTAL IMPACTS

9. Geotechnical constraints prevented concealing the holding tank by locating it underground, although it was possible to reduce its impact by partially burying the structure. Measures were incorporated into the design of the exposed parts to integrate the new structure into the surroundings. It was decided to face the walls of the holding tank with natural stone from a local source. With time and weathering this would resemble the stone walls of nearby farm buildings and be in keeping with the character of the valley. The flat roof of the structure would also be visible from elevated views, so it was decided to topsoil and seed it with a grass seed mix that would help the roof blend in with the grass covered reservoir embankment and surroundings.

10. In addition to incorporating design measures to minimise environmental impact of the scheme, other landscaping measures were undertaken. A detailed plan was prepared showing the locations for landscaping planting that would partially screen the holding tank from views and further integrate it into the natural environment. The planting was carried out upon completion of construction and included strengthening the orchard at the base of the berm, planting shrubs and climbers in front of the tank to break up the line of the most visible east face and strengthening the hedge line that follows the southern boundary of the site. Care was taken to select species that were suitable for this location and were already present in the vicinity. Species planted included Oak, Ash, Hawthorn, Elder and Hazel.

11. In order to minimise the disruption caused by the movement of vehicles on the narrow country lanes surrounding the site, material excavated for the holding tank was used as backfill to landscape the structure into the existing dam face. This not only prevented the need to remove the material from the site but also softened the line of the structure and its visual impact.

GEOTECHNICAL CONSIDERATIONS

12. The geotechnical implications of siting a concrete service reservoir partially buried within a berm are complex. The designers were conscious of the chequered history of the dam and maintenance of adequate embankment stability was an essential priority at all times. Accurate assessment of existing and predicted stability of the berm and embankment and the adjacent hillside required comprehensive site investigation (comprising information study, walkover study, walkover surveys and ground investigation) to obtain reliable geotechnical parameters to incorporate into the analysis.

13. The two design options were founding the tank on Lias Clay (directly or on a layer of fill) or construction on a piled foundation. Besides reducing the risk of deep excavations, the second option would enable disturbance to the dam structure to be minimised. A conservative

and practical foundation design was subsequently adopted to support the structural load of the concrete service reservoir. Verification of the pile design parameters was carried out by initiating a programme of pile loading tests on selected piles during construction.

14. Hazard assessment of all site investigation, temporary and permanent works was carried out to minimise pollution risk and superficial environmental damage.

15. There were geotechnical requirements specified for the temporary storage and re-use of excavated materials given limited storage areas within the reservoir compound area. A policy of minimum off-site removal of spoil was adopted and careful re-use of bermfill material was carried out where appropriate. Excavations were trimmed to safe batter slopes within the berm fill material. Topsoil was temporarily stored within stable designed stockpiles and reused during landscaping of the works.

DESIGN SUMMARY

16. RKL had historical data available from Monkswood Reservoir including drawings in connection with inspections under the Reservoirs Acts, contract documents prepared for emergency spillway works and survey drawings. The main factors affecting the design of the holding tank were:

- the need to ensure stability of the dam and hillside both during and after construction
- provision of a suitable foundation
  minimising visual intrusion
- provision of a cost effective design meeting the operational needs of the water company

17. There was a balance to be achieved in assessing the appropriate level for the 2 Ml capacity tank to ensure satisfactorily hydraulic performance, to aim for balanced cut and fill, to minimise the risk and cost associated with deep excavations and yet to minimise exposure of the structure above ground.

18. The final design of a reinforced concrete structure founded on in-situ bored concrete piling was procured through separate piling and civil works contracts. The former was carried out in March/April 1992 and the latter between May and November 1992. The completed tank, prior to shrub planting, is shown on Fig. 2.

Fig. 2. Monkswood Springs Holding Tank

CONCLUSIONS

19. Construction of a large concrete structure within the berm of an old earth dam with a history of settlement and leakage problems provided a challenging project for engineers. That the Site should be within an area of great natural beauty and adjacent to a site of special scientific interest introduced a further range of problems to be solved and introduced other disciplines to the project team.

20. The environmental and geotechnical constraints were satisfied by detailed, co-ordinated environmental assessment and site investigation. Careful and sensitive design and liaison between environmental, geotechnical, civil and structural specialists during the design, construction and landscaping of the works ensured a comprehensive environmentally and geotechnically satisfactory engineering solution.

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Reservoir development in semi-arid countries — a human ecology perspective

J.W. FINDLAY, BSc, CEng, MICE, Principal Engineer, Babtie Shaw & Morton

SYNOPSIS. It is widely recognised within the relevant reservoir development professions that dam based schemes can produce significant ecological benefits and disbenefits. While environmental impacts play an important part in assessment of UK schemes, one has to go to developing countries to fully appreciate the stark choices that sometimes have to be addressed. The author draws on experience in the North West Frontier of Pakistan, where an institutional strengthening project was carried out for the Overseas Development Administration (ODA) within the Small Dams Directorate of the Provincial Government.

INTRODUCTION

1. The paper has been prepared as one dam engineer's perspective of human ecology in relation to reservoir development. Human ecology is defined as the study of relationships between people and their environment; the paper looks specifically at changes in soil and water regimes in the semi-arid areas of the North West Frontier Province of Pakistan and the effect of these on human beings. This viewpoint is based on observation during engineering activities rather than a specialist scientific study and also refers to general environmental aspects of reservoir schemes. The author has been involved with dams and reservoirs in the UK and Nigeria as well as Pakistan and uses this experience to highlight some of the contrasts in environmental impact between developed and developing countries.

2. The environmental impact of reservoirs rarely seems to be out of the news in the present era of strong environmental lobbies and global communications. In relation to the developing world the public at large has been encouraged to believe that any scheme involving dams, particularly large ones, is doomed to failure either through crippling sedimentation of the reservoir, generation of seismic activity or the misery of enforced resettlement for thousands of natives from the reservoir basin. Certainly for large reservoir schemes in general, negative or adverse impacts tend to be borne locally to achieve regional and/or national benefits. The informed dam engineer knows that there is a sound basis for concern on these subjects and that mistakes have been made in the past, also that there are many other environmental topics that require careful consideration in the development of a scheme. These include deteriorating water quality, spread of waterborne disease, inequitable spread of benefits and destruction of habitat. However, the engineer and his professional colleagues in the development sector are encouraged by the knowledge of benefits that a carefully planned and implemented scheme can bring. Protection from persistent flooding,

availability of water in all seasons, the prospect of irrigated agriculture, opportunities to develop fishing and stabilisation of population movements are but some of these.

ENVIRONMENTAL IMPACT IN THE UNITED KINGDOM

3. In the UK there is a relative abundance of water and the days of major reservoir schemes may be coming to a close, certainly it appears to be proving increasingly difficult to locate sites for even urgent storage needs. However, the country is blessed with a legacy of water supply schemes based on dams and reservoirs that suffer a minimum of environmental problems, initially due to the suitability of conditions in this country but latterly through careful public consultation and planning. This allows the relative luxury of considering some of the finer points of environmental impact in the knowledge that the basic quality of life will not be greatly affected if a scheme cannot proceed. Thus engineers find themselves shaping their structures aesthetically and producing water based ecosystems that enhance the environment and increase the quality of life beyond the immediate needs for water. This is a valuable addition to the skills of the water specialist.

4. The environmental areas most commonly affected are listed below:

Landscape and visual
Archaeological
Fisheries and fish migration
Downstream water quality
Resettlement of inhabitants
Construction activity effects
Tourism/recreation and its effect on locals

It is worthwhile expanding on some of the problems and solutions associated with these.

5. Landscape and visual effects can be addressed most readily with embankment type dams. Surplus or unsuitable material may exist as overburden in borrow areas and can be used cost effectively to shape the embankment or site in general. At Carsington Dam in Derbyshire, failure of the original structure made available considerable volumes of disturbed fill that was suitable for landscaping and this was incorporated in the embankment to produce gently sloping curved embankments that blended in with the surrounding topography and reduced the overall impact of the dam on the landscape. On the same scheme, the need for stability berms allowed a village bypass road to be sited there rather than on the crest where it would have been more visually intrusive.

6. At Kielder dam in Northumberland normal draw down would have exposed extensive areas of the solum at the head of the reservoir. A secondary dam (Bakethin) was constructed to overcome this visual impact. This was also used to enhance the environment by creating a nature reserve.

7. At Carsington Dam a major part of the planning process related to the handling of site traffic and the prevention of construction period pollution. A traffic control scheme ensured that only those routes suited to the nature and volume of site traffic (particularly quarry products) were used and that other, minor roads, were off limits. Extensive rainfall run-off control

measures ensured that the exposure of aggressive leachate generating strata did not result in changes to the natural condition of the local streams which formed the upper reaches of a valuable fishing river.

8. At Roadford Reservoir in Devon the dam was constructed of rockfill won on site. The quarry was sited on the reservoir shoreline and when worked out was shaped to minimise intrusion into the surrounding landscape.

9. All these matters are important to the British public in general and to specific interest groups in particular.

10. In the present water supply situation in Britain while projected water shortages requiring a new reservoir might lead to restrictions if the scheme is not implemented, these would probably affect activities at the upper end of the quality of life scale - restricted car washing, garden watering etc. As such the environmental impacts of the scheme might reasonably be judged as unacceptable.

11. However, it should be recognised that in developing countries where the population is far from static and climatic conditions exact a heavy toll on life and property, different values have to be placed on the need for and consequences of a water storage scheme.

STARK CHOICES IN DEVELOPING COUNTRIES

12. In contrast to the UK examples described above, similar impacts in semi-arid developing countries would be of much less concern to the general public. In developing countries water supply schemes generally set out to provide a reliable source of water where none existed previously. Availability of potable water is generally restricted to a very small percentage of the population and the priority is therefore to have a basic supply for direct consumption and for cooking with requirements of hygiene coming next. The scale of need and demand is thus at the opposite end of the spectrum to that of developed countries.

13. In rural northern Nigeria population density is not high but away from the major rivers surface water is scarce or non existent for much of the year. It is the role of women in this area to collect water and during the dry season they can spend up to six hours walking to and from the nearest source. Wells can only be dug to a limited depth and often the groundwater drops below this level in the dry season. Boreholes are expensive and difficult to maintain and the strata may not always be able to support a reliable supply. Reservoirs offer a cost effective solution to these problems and in the face of these the adverse impacts reduce in significance. Certainly land is lost but it is rarely at a premium.

14. In North Western Pakistan the priorities are different but no less critical than those in Nigeria. Much of the land is mountainous and ownership of good agricultural land within a region is often concentrated within a few important families. As land passes from father to sons it has to be parcelled according to their share resulting in vast numbers of small plots that are difficult to work in an economic fashion. With a high population density and severe pressure on land there is often insufficient cultivable land available to enable even subsistence farming under rainfed conditions. There is a tremendous need to increase agricultural production, food supply and therefore the welfare of the people.

15. In Pakistan women do not perform the same role as in Africa in terms of seeking out water. This is due to the very close family community within

which women move and which would preclude trips on their own outside the village. Clothes washing is still done at the river or pond as in Nigeria but by the girls of the village. Mature women are largely confined to the family compound.

16. In many areas of NWFP water bearing strata could be developed by boreholes to give substantial and reliable yields but, although having a better developed infrastructure than Northern Nigeria, problems of maintenance and running cost still dominate the cost/benefit arguments of groundwater development. Electricity in particular is prohibitively expensive in anything other than modest quantities, even if it is available on the grid system - this despite the region being dominated by Tarbela one of the worlds great hydro schemes. However, most family compounds are served by a well and domestic water is relatively easily obtained. While this is a major health care bonus, undernourishment, particularly of children is still a problem. Such is the pressure of population on available land that irrigated agriculture is often the only means by which even subsistence farming can be carried out.

17. In such a social climate the prospect of surface water irrigation schemes is welcomed with open arms almost irrespective of its sustainability.

RESERVOIR DEVELOPMENT - EXPERIENCE IN NORTH WEST PAKISTAN

18. The North West Frontier Province of Pakistan (NWFP) is a mix of mountains and plains lying between the Hindu Kush the Karakorams and the Punjab. The River Indus runs through the northern mountainous regions and the start of the great Indus plain forms the province's eastern boundary to the south (Fig 1). Although touched in part by the north western extremities of the Indian sub-continent monsoon, much of the province is very dry and with extensive areas of barren sandy soil qualifies as semi-arid in many parts.

19. NWFP has an extreme climate, The mountain communities in the North can be cut off by snow for six months of the year and even in Peshawar at only 450m above sea level and 34° N winter frost is not uncommon. However, it is at the opposite end of the scale that the real problems occur and in July the vale of Peshawar is one of the hottest places in the world with mean temperatures approaching 40°C and peaks reaching 48°C or more. To further emphasise its position at one of the great geological convergences it is also subjected to regular earth tremors. The west of the province forms part of the largest tribal society in the world, The Pathans, and although the provincial capital Peshawar is a large city most of the population live in small tightly knit agricultural communities. The province is less well developed than most of Pakistan with fewer and poorer roads and virtually no industry. NWFP also bore the greatest load during the Afghan War when it was not only in physical proximity to the fighting but was home for up to three million refugees.

20. Pakistan is famed for its irrigation network and boasts two of the worlds most famous dams, Mangla and Tarbela. The latter lies entirely within NWFP but despite its massive irrigation releases presently contributes nothing to the agriculture of the Province, all the water passing down the Indus to the headworks of the great irrigation systems on the Indus plain. There are however plans to change the situation by diverting some of the

Figure 1: Map of North West Frontier Province

water from the reservoir by tunnel to the west. Tarbela is also a major hydro-electric plant.

21. The first of Pakistan's major dams was built at Warsak on the Kabul river close to the NWFP border with Afghanistan (near the Khyber Pass) and like Tarbela was designed for both hydro power and irrigation. The Kabul meets the Indus on the border between NWFP and Punjab Province at the head of the Attock gorge and provides a dramatic illustration of one of the main characteristics of NWFP rivers. At this point the Kabul emerges from the Vale of Peshawar one of the most productive agricultural areas in Pakistan and at its confluence with the Indus produces a dark silt laden plume in the glacier blue of the larger, more vigorous stream. However, all is not as it first appears.

22. Although the Indus at this point has just emerged from the mountains and appears the epitome of a snow fed upland river, it has been changed by man. Tarbela Dam lies only fifty kilometres upstream and the ninety kilometre long Tarbela lake traps a large proportion of the vast quantities of geological erosion products carried by the river. These products are generally a result of its dash from the Himalayas through the steeply incised valleys of the Karakorams, although soil erosion due to deforestation also plays a part. The campaign to slow the inevitable progress of the deposits towards the turbine and irrigation intakes is a critical one in terms of productivity for Pakistan but one which they must ultimately lose as the Indus performs its ancient role of erosion and transportation.

23. The Kabul River at Attock carries with it a sediment load derived from soil erosion and the colour indicates the scale of the loss in terms of organic material and nutrients. Warsak Dam lies approximately 100km upstream from the confluence and has already lost its storage capacity through sediment build up, no longer trapping sediment and with limited scope to generate power or attenuate floods, although still providing the headworks for an irrigation canal system. However, much of the silt load on the Kabul has a solution in terms of conservation practices to reduce erosion of soils.

24. These two rivers illustrate the perpetual problem for reservoirs in NWFP. There will always be sedimentation and some of this will be unavoidable. However, although a dam and its reservoir can be designed to cope with the effects of sedimentation they do nothing to limit it and can even accelerate soil loss if planned and used unwisely.

25. Pakistan is a populous nation and although NWFP is not in the same league as the teeming provinces of Punjab and Sind its topography concentrates the feeding needs of its people on a relatively small area. This same topography and the everpresent symbols of Tarbela and Warsak produce a common belief amongst laymen that a programme of dam building would solve many problems; principally water availability, failed crops, flooding and others such as lack of employment and population migration.

26. NWFP is naturally blessed with the water resources of several major perennial rivers and in the past these have been used to create extensive irrigation areas, notably the Swat system that nourishes much of the productive Vale of Peshawar. These schemes have run for many decades and are based on river diversions and intakes. Although avoiding much of the sedimentation problem associated with reservoirs it is still extremely difficult to avoid material entering the canal system with consequent loss of conveyance capacity.

27. However, water resources are not distributed uniformly across the Province. The perennial rivers are limited to the mountains to the north, where winter snows and glaciers provide reliable sources, and all join the Indus not far below Tarbela. The remainder of the province has only seasonal river flow and limited rainfall to depend on, the monsoon rains only touching the North Eastern parts of the province where perennial rivers already exist. Attempts have been made to dam the seasonal rivers or to divert flows to offline storage and although some irrigation is carried out from these they suffer from unreliability of river flow as well as the ever present sedimentation.

28. The need for water in NWFP is generally for irrigation rather than public supply, most communities having access to a well or borehole for domestic use. Although there are a number of large and successful canal systems fed by river offtakes, in many parts of the province agriculture is dependent on sparse and unreliable rainfall. Often when rain does come it is intense and inconsistent in its coverage resulting in some communities losing their crops through drought while others have their fields washed out by torrents. Generally speaking, most of the practical diversion weir sites on perennial rivers have been exploited and although some areas have good potential for aquifer development the cost of drilling and running borehole fields is prohibitive. Thus for many communities a reservoir represents the only prospect of improving agricultural production and hence the quality of life.

SUSTAINABILITY OF SCHEMES

29. Considering the foregoing it can be seen why provision of dams for storage or diversion of water, for irrigation or water supply is perceived by the NWFP population at large as a major contribution towards the improvement of living conditions in their province. The reality is more complex and consideration has to be given to a range of factors that influence the life of dam schemes such as hydrology, engineering performance, operational control and social behaviour. These can severely restrict the benefits of a scheme.

30. Even small dams have a major effect on their immediate environment and with some large schemes the consequences of their creation have more regional implications. A small scheme creates a body of water where none existed before and apart from its functional aspects may affect local social perceptions as land values change and the amenity of the water body becomes apparent. The largest schemes may displace whole communities or ethnic societies and change their way of life permanently, apart from the impact that the sheer physical size of the reservoir can have on communications, such schemes can even induce minor climatic variations.

31. All dams affect the physical and biological characteristics of the reservoir area in that surface water distribution will be varied from the natural patterns, groundwater levels will change, plant life may be lost in the reservoir area and so on. As noted above the human ecology of the area will also change. However, the principal aim of a reservoir scheme is to improve the quality of life of the population as a whole. If this is at the level of basic provision such as water or nutrition then some of the finer points of environmental impact may have to be kept in perspective. Certainly an overall balance should be sought when identifying, assessing, planning or implementing a scheme.

32. The hydrology of an area determines whether there is a reasonable prospect of storing a given volume of water from the rainfall run off within the catchment. In NWFP the paucity of long term hydrological records and the variability of the climate make decisions on the siting of reservoirs particularly difficult. One of the principal functions of the project carried out for ODA was to improve the availability of existing hydrological records and to initiate a programme of data acquisition. There remains however a considerable risk that a given catchment will not provide sufficient yield to operate a planned reservoir. Evaporation loss from water surfaces is also a significant factor particularly when a reservoir is shallow.

33. Sedimentation has already been mentioned as a major influence on reservoir life in NWFP and has the potential to completely negate the value of a reservoir within a few years irrespective of original storage volume. Hydrographic surveys of existing reservoirs, although difficult to carry out accurately, represent the most reliable method of estimating future sedimentation.

34. Many of the potential dam sites in NWFP lie in valleys filled with deep deposits of sands and gravels. The generally free draining nature of soils and the associated lack of cohesive material for blanketing results in seepage and leakage being a major concern.

35. Proper operational control of dam based irrigation schemes is crucial to achievement of planned benefit. The system has to be operated to distribute water in the correct proportions throughout the command area and over the full growing season. Sadly the tribal society and family hierarchies of NWFP lead to abuse of the system to satisfy perceptions of historical rights. This leads to uncontrolled depletion of reservoirs to keep water flowing to important land owners at the head of the system, often to the extent of overwatering crops. The fields at the tail of the system suffer intermittent and ineffective irrigation as a consequence.

36. It is obviously essential to ensure that a scheme is sustainable. There is no point in investing valuable resources to create a facility that will cease to function before it has produced the changes that remove the initial need. While some dam types can be considered to have an almost indefinite structural life in the human timeframe, their associated reservoirs are often more vulnerable. An irrigation scheme requires several years of water availability before the cropping pattern becomes established and in the case of orchards this period may extend to eight or ten years. Generation of income from such schemes sufficient to improve quality of life and permit saving of surplus funds will take much longer. Without such funds alternative means of support can not be developed. If a scheme cannot be sustained for that length of time then there is no prospect of the community becoming independent.

ENVIRONMENTAL IMPACT OF RESERVOIRS

37. Almost any scheme that reliably extends the use of limited water resources for irrigation in a semi-arid area can be said to have an ecological benefit. In the strict sense it is an agricultural benefit in that more can be grown on the area commanded by the source but, if properly implemented, it is likely to bring with it improved management of soil and water and a reduction in the pressure of the population on the environment.

38. As noted in the introduction dams and their reservoirs do not always create the impression of being a benefit to the surrounding environment. From the experience gained on the ODA project in NWFP it is possible to make a few generalised statements that put a balanced perspective on dam development in semi arid areas.

39. Natural conditions in such areas are harsh. Plant and animal life is sparse and subjected to dramatic variations in climatic conditions that prevent secure establishment. Humans eke out a living as best they can and rarely if ever progress beyond subsistence level. Soils are poor and erodible and rainfall is unreliable.

40. Land is levelled as far as possible, by hand or basic machinery when available, to maximise the benefit of the rains that come. Every available area of workable soil is shaped from the landscape from terraces of a few metres width contouring the steepest slopes through to more conventional field layouts, but all of these are prone to loss of soil or destruction through the unpredictable nature of precipitation and its run-off. Even generally flat areas are deeply incised with gully erosion.

41. Stream beds dissect the landscape, dry for much of the year but capable of wreaking havoc when their hilly catchments intercept an intense rainstorm. The resulting floods not only destroy crops but can wash away whole fields that have been painstakingly built up on river banks from the more fertile material. Attempts are made to harvest some of the flows by building offtakes to feed small canals or in some cases to block off the river channel and induce reasonably controlled flooding of flatter areas, but these are only temporary features that are easily destroyed and have to be rebuilt time and again.

42. Dams do not solve all these problems but with careful planning they can reduce some of the adverse impacts. The greatest advantage is that the available water can be stored for controlled release thus increasing the effectiveness of application to irrigated crops as well as prolonging the period of availability. Under the best conditions this gives a controlled soil water balance that can produce the crop yields necessary to significantly improve agricultural production and justify the cost of the scheme. The other major contribution is in the reduction of severe floods with savings on loss of valuable soils and field areas or in some cases in reductions in the amount of protective work that is required.

43. Care does have to be taken that the irrigated area is prepared to receive the enhanced quantities of water and that the application of irrigation does not lead to excessive uncontrolled run-off with consequent loss of fertile soil and gully erosion of levelled areas.

44. The greatest obstacle to dam development is sedimentation. The construction of a dam across a stream prevents the natural progression of stream and bed load material downstream. This not only reduces the storage capacity of the reservoir but disrupts the balance of erosion and deposition downstream that under some circumstances could result in aggravated deepening of stream beds.

45. The permeable nature of dam foundations in such areas is likely to permit seepage flows from the reservoir and this will manifest itself as higher groundwater levels and perhaps an area of increased vegetative growth. Together with the general raising of groundwater levels around the reservoir basin and perhaps even within adjacent drainage areas, the production and reliability of wells and boreholes could also improve.

46. The drowning of an area of valley will obviously result in the loss of some vegetation, however, in the regions described this is likely to be a loss that will be offset by the greater shoreside growth that the presence of water should support. Within limits of scale, animal life might reasonably be expected to migrate to the margins of the reservoir without major disturbance and may also flourish with the availability of surface water. It is possible that some undesirable organisms may be established in the revised ecosystem and this should receive careful consideration at the scheme assessment stage. The body of water itself may offer the opportunity to establish an aquatic ecosystem with potential for introducing fish stocks and therefore a further source of nourishment and income in the area.

47. The availability of water in the area will inevitably change the perceptions of the local population with regard to use and misuse of water. Wastage will occur and drainage will become a consideration. Standing water may increase and lead to stagnant pools thus creating a new and undesirable ecosystem with its attendant health implications. As with the other aspects of water based rural development, education, training and support are crucial to success.

CATCHMENT CONSERVATION

48. No discussion of reservoirs in NWFP would be complete without reference to catchment conservation. Although creation of a dam and reservoir has no direct effect on conditions within the catchment area supplying the water, there are many secondary considerations.

49. The problem of erosion continues without the presence of a dam downstream. Throughout the world but particularly evident in the foot hills of the great south asian mountain ranges the pressure of population is putting intolerable strains on the land as trees are felled to create areas for crops and for use as fuel for heating and cooking. As this proceeds the ability of the terrain to resist erosion diminishes and greater and greater quantities of soil are exposed, disturbed and ultimately washed away to cause further damage downstream. Despite the lack of roads or mechanised equipment the ancient forests of NWFP can be seen emerging from the valleys as logs or baulks of timber strapped to the sides of camels. Even the brush and scrub that holds the marginal soils on steep slopes is cut and transported by donkey to the nearest town. These resources are not being replaced but are often the only cash crop that the hill people can use to eke out the limited subsistence farming that the skimpy soil and steep slopes allow. It is a vicious circle and reservoir development can aggravate it by raising standards of living downstream and creating further social divisions between upstream and downstream communities leading to even greater efforts to harvest the timber. In some cases construction of the dam could improve access that would allow a more commercial approach to the felling. It is therefore essential that schemes include specific provision for catchment conservation, not only to prevent early sedimentation of the reservoir but to reverse the trend of deforestation and provide alternative means of income for the catchment communities.

CONFLICTING PRIORITIES

50. In conclusion it can be stated that there are likely to be many conflicting priorities in the development of reservoirs in semi-arid countries. The environmental impact of a scheme as it is currently interpreted publicly in the UK may not be particularly significant in the face of extreme need. There can be little doubt that the provision of a reliable source of stored water that is capable of supporting irrigated agriculture in an area where the population barely survives on subsistence farming should be given more consideration than the visual intrusion of the dam or indeed the loss of a relatively small area of marginal habitat through flooding. The more specific ecosystem balance of the area should however be considered carefully to ensure that the soil/water relationship is improved particularly from the viewpoint of human ecology. The overriding provision should always be that the solution proposed is sustainable. However, there should be no illusions about the practicalities of the dam engineering and the reliability of the reservoir yield. The problems associated with both aspects could lead to wasted resources and rapid disillusionment of the populace if the scheme fails to live up to expectations. Dams in areas such as the drier parts of NWFP appear tremendously attractive at first sight offering the opportunity to harness hill torrents and simultaneously solve the twin problems of uncontrolled flooding and lack of perennial water supplies. Sadly, sedimentation and potential leakage act with variable catchment runoff to make many initially encouraging sites unrealistic.

51. The NWFP study tackled these problems with a multi disciplinary team that sought to establish the importance of an integrated approach to small dam development for irrigation, bringing together engineers, agronomists and sociologists at the initial stages of a project. The reports and guidance notes prepared by the team emphasise this approach and it is encouraging that these are in use within a strengthened small dams department in NWFP. However, there is no simple solution to the problems of semi-arid agricultural areas in NWFP, certainly uncontrolled reservoir development is unlikely to help. The answer may lie in low cost detention structures and in developing economical means to tap the water held in river bed sediments behind such structures. Further study of this topic was recommended in the final report of the ODA project.

Some sedimentation and water quality problems of the Volga River reservoirs

V.N. GOLOSOV, PhD, Senior Research Scientist, and
N.N. VINOGRADOVA, PhD, Senior Research Scientist, Moscow State University, Faculty of Geography, Laboratory of Soil Erosion and Channel Processes

SYNOPSIS. Using the Universal Soil Erosion Equation adapted for Russia, we have calculated erosion rates for the basins of the Volga river reservoirs and have marked map " Soil erosion on the arable lands of European part of Russia ". After that we have calculated the total volume of eroded soil and some pollutants. As the result it became possible to choose area with different level of erosion rates on the watersheds of each reservoirs.

INTRODUCTION

The Volga is the largest river in the European part of Russia. It occupies vast territory of forest, forest-steppe and steppe zones. Several dams were constructed during the period 1937-1981. They formed an uninterrupted cascade of 8 large reservoirs (Fig.1). Their total volume is 150 km^3 and total area is about 20 000 km^2 . Length of bank line is about 13 500 km . 36% of bank line are subject to erosion. The main characteristics of reservoirs are presented in Table 1. The Volga river reservoirs accumulate sediments and pollutants of different origin. It is possible to divide them into the three major groups:

1. Sediments and pollutants of industrial and agricultural plants, runoff from urban areas;
2. Sediments and associated pollutants, which were formed as the result of surface runoff from cultivated areas;
3. Sediments , which were formed as the result of bank abrasion and bottom erosion in the reservoirs.

It is more easier to determine the point sources of water pollution, which belong to the 1 and 3 groups. But it is more difficult to find sources of agricultural pollution , because agricultural pollutants comes from numerous of separate agricultural tracts, belonging to a regional watershed. Pollution from any given tract (as by herbicides, excessive fertilization, etc.) is relatively small and often it is difficult to identify and measure it. But the effect of pollution from numerous tracts results in a general lowering of water quality through pollution of runoff waters, especially in large reservoirs. Excessive sediment load resulting from soil erosion is the major source of downstream

water pollution.

STUDY SITES AND METHODS OF INVESTIGATION.

The major goal of our research is to pinpoint territorial position of the main non-point sources of pollutants and sediments for the basins of the Volga river reservoirs and to determine their influence on sedimentation and water quality throughout the whole basin (except the Kama river basin upstream the Low-Kama Dam).

There are many chemical, machine-building, metallurgical, textile factories and plants situated on the banks of the Volga river and its tributaries. There are severe problem of river pollution with poorly treated waste waters. But this is merely technological problem, which can be solved by using more up-to-date systems of sewage treatment.

It is more difficult to provide the control of entering of sediments and accompanying pollutants from cultivated areas. We used the method, which was elaborated for determination of zones with different intensity of soil erosion in the Laboratory of Soil Erosion and Channel Processes (Larionov, 1984). A modernized form of the Universal Soil Loss Equation (Wishmeier & Smith, 1978) was used to calculate soil-loss from rainfall Soil loss from melting snow was calculated according to the model of State Hydrological Institute (Instruktsiya ..., 1978). A picture of the erosion potential of relief (the most variable factor) was obtained from the large scale topographical maps with further statistical processing by computer. This made it possible to obtain the most realistic descriptions of slope length and gradients, average values for soil-loss on large areas, as well as the distribution of land classified according to the severity of erosion within these areas.

The applicability of empirical formulae and the compatibility of their results have been checked several times by field studies using the correlative deposit method; the Cs-137 method (Yakimova,I.V. ,1988. Quantitative relief factor estimation for mapping erosion-prone lands. Thesis, Moscow State University) and data from hydrological monitoring sites and comparison with data from regional research (Golosov ,Ivanova, Litvin & Sidorchuk, 1992).

The results of calculations were compiled in 1: 1 500 000 map of erosion-prone regions for European part of Russia. Then we calculated annual soil-loss from the cultivated lands for each river basin of the Volga river system and for each reservoir catchment. Some of the results of these measures are presented in the Table 2.

We used the value of soil erosion intensity as an index of intensity of entering pollutants from agricultural lands. This index was chosen because most part of pollutants entering from slopes to the river channel, are transported together with soil particles. Except the traditional water pollutants (pesticides, fertilizers et. al.) isotope ^{137}Cs became now one of the major pollutants in the European part of Russia after

Fig. I . Map of Volga basin.

Chernobyl incident. ^{137}Cs is an artificial radionuclide with a half life of 30.17 years, which was released into the stratosphere by past testing of thermonuclear weapons and deposited as fallout. In most agricultural environments ^{137}Cs reaching the soil surface as fallout is strongly and rapidly absorbed by clay minerals (Livens & Baxter, 1988). Subsequent lateral redistribution of ^{137}Cs occurs in association with sediment particles in response to erosion, transport and deposition processes (Walling & Quine, 1991). After Chernobyl incident large quantity of ^{137}Cs had fallen together with precipitation on vast territory of Russia, including the west part of the Volga river basin (Fig.2). So sedimentation zones of reservoirs are areas with high level of pollution with ^{137}Cs.

RESULTS AND DISCUSSION.

The major results of sedimentation rates in the Volga river reservoirs are presented in Table 3. In column 4 we included only sediments, which had entered the reservoirs on the section of river between two dams plus sediments from tributaries and minus sediments, which had entered the low pool from the upper pool. Accurate data about river sediment discharge for the Saratov and the Volgograd reservoirs are absent.

Sediment river discharge is the main component (29% & 39% from total sedimentation) only for the Ivan'kovo and the Uglich reservoirs. Abrasion processes are the principal source of sediment for reservoirs of low and middle reaches of Volga river (Tabl.3). Intensity of sedimentation increase along the cascade length from 0.17-0.25 cm per year for forest zone reservoirs till 0.60 cm per year for steppe zone reservoirs (Zakonov, 1989).

Erosion rates are high enough on the arable lands inside Ivan'kovo reservoir catchment (Tabl.4). Maximum values of soil loss are observed in the Vazuza river basin, which drains Smolensk-Moscow upland. But now sediments from the Vazuza river can not reach the Ivan'kovo reservoir because of the construction of dam of the Vazuza reservoir, used for supplying of Moscow with drinking water.

Erosion rates on the slope of the Uglich reservoir basin are almost equal with erosion rates in the Ivan'kovo reservoir basin (Tabl.4). Maximum soil loss is observed in the Dubna river basin (Tabl.2). But specific sediment discharge is lower here due to small area of arable lands. The small coefficient of delivery ratio is typical for this section of the Volga river. It is connected with large distance between cultivated lands and river channels.

Different situation is observed in the Rybinsk reservoir basin. As a rule steep slopes of river valleys are cultivated here first of all. This facilitates delivery large quantities of sediments to the bottom of river valleys. But total area of arable lands is extremely small. So total amount of sediment is relatively moderate.

Table 1. Main characteristic of the Volga reservoir

No	Reservoir	Natural zone	Year of construction	Volume ($10^6 m^3$)	Length (km)	Depth (m)
1	Ivan'kovo	forest	1937	1120	113	3.4
2	Uglich	forest	1939	1245	120	5.0
3	Rybinsk	forest	1940	25420	250	5.6
4	Gorki	forest-steppe	1955	8700	434	5.5
5	Cheboksary	- " -	1981	14200	340	6.2
6	Kuibyshev	- " -	1955	58000	510	9.0
7	Saratov	steppe	1968	13000	340	6.7
8	Volgograd	steppe	1960	31450	524	10.1

Table 2. Erosion rates from cultivated lands for some typical river basin of the Volga river reservoirs.

River basin	Reservoir	Area of cultivated land (%)	Erosion rate (t per hectare per year	Specific sediment yield (t per km^2 per year)
Vazuza	Ivan'kovo	60	10.2	610
Tvertsa	Ivan'kovo	33	4.8	160
Medveditsa	Uglich	45	4.3	190
Dubna	Uglich	28	9.8	270
Mologa	Rybinsk	13	5.1	66
Suda	Rybinsk	10	4.0	40
Unzha	Gorki	15	4.0	84
Kostroma	Gorki	38	5.6	210
Vetluga	Cheboksary	22	4.3	95
Zusha	Cheboksary	67	6.7	454
Upa	Cheboksary	55	8.4	460
Cheptsa	Kuibyshev	60	11.2	730
Civil	Kuibyshev	62	8.4	520
B.Cheremshan	Kuibyshev	45	3.0	133
Syzranka	Saratov	72	5.3	385
Tereshka	Saratov	83	4.3	355
Chardym	Volgograd	83	3.0	246
Eruslan	Volgograd	88	1.5	129

Table 3. Some sources of sediment and rate of sedimentation in the Volga river reservoirs.

Reservoir	Basin area (10^3 km^2)	Annual erosion volume, thousands t		$\frac{W_2}{W_1}$ (%)	Input in reservoir (%)	
		slope W_1	river W_2		river sediment	bank abrasion
Ivan'kovo	40.57	12 495	195	1.6	29	66
Uglich	60.043	4 109	78	1.9	39	58
Rybinsk	146.0	8 080	541	6.7	18	80
Gorki	229.0	15 355	453	3.0	18	82
Kuibyshev	1210.0	186 355	6295	3.4	14	85

Table 4. Relationship between specific sediment discharge and rate of sedimentation .

No	Reservoir	Specific sediment yield t/km	Rate of sedimentation thousands t per km per year	Phosphorus concentration* milligramme per litre
1	Ivan'kovo	3.08	2.09	< 1
2	Uglich	2.11	2.55	2.5
3	Rybinsk	0.94	0.95	1.5
4	Gorki	1.85	2.35	1.2
5	Cheboksary	3.13	-	-
6	Kuibyshev	3.05	6.97	2.7
7	Saratov	2.51	-	3.0
8	Volgograd	2.68	-	3.0

* According Litvin & Kiryuhina (in press)

Growth of soil erosion intensity is observed in the Gorki reservoir basin. Specific sediment yield is slightly higher here then in the Upper Volga basin (Tabl.4).

Vast basin of the Cheboksary reservoir is characterized by large differences of soil loss rates, which are connected with different landscapes and with changes of area of cultivated lands. Large part of the basin is situated in the forest-steppe zone with high percent of arable lands. High level of erosion rates led to small river aggradation, especially in the Upper Oka river basin (Golosov & Ivanova , 1993). As the result zone of accumulation had increased within small river basins. This process decreased the delivery of soil particles in large river channels and reservoirs.

High percentage of arable lands is more typical for middle and low reaches of Volga river basin. From the other hand heavy rains occur here more often. Both these circumstances result in the growth of sedimentation rates in the reservoir (Tabl.3).

Especially dangerous situation is connected with growth of use of fertilizers and ^{137}Cs delivery in river channel. Maximum of cesium-137 precipitation is observed in the Oka river basin, especially in the Zhizdra and the Upa rivers basins (Fig.2). Our field observation in these region had shown, that about 30 percent of ^{137}Cs can be transported downstream in association with suspended sediment , which was eroded from the surface of drainage basin. As the result, the input of ^{137}Cs in sedimentation zone of reservoirs may be extremely high since suspended sediment can transport very easy on the long distance. So total radioactive emanation of sediments can exceed permissible levels, especially in the Cheboksary and the Kuibyshev reservoirs.

Two main factors affect the pollution of the Volga reservoirs with fertilizers: specific sediment yield of reservoir basins and quantity of the fertilizer per area unit For example, the concentration of phosphorus in reservoir water increase from the Upper Volga reach to the Low Volga reach (Tabl.4). The Uglich reservoir is the exception of this rule. This fact can be explained by very high level of fertilization in the Moscow region farms.

CONCLUSION.

The above presented results indicate that sedimentation rates in the Volga reservoirs are not very high due to relatively low erosion rates, as well as high levels of redeposition of sediment within river basins . Maximum values of sedimentation rates are observed in the Kuibyshev reservoir.

The soil erosion map of the Volga basin can help to define areas with maximum soil-loss , where water and soil conservation works should be implemented first of all. It is especially urgent for territories with high levels of pollution with ^{137}Cs. The influence of fertilizers on the water quality are higher for reservoirs of the Low Volga.

Fig. 2. A generalised map of fallout inputs of Chernobyl-derived caesium-I37.

Because in the last years the loss of fertility of chernozem in the steppe and forest-steppe zones of the Volga river basin is observed, it is possible to expect wider use of fertilizers in these areas. This can provoke higher levels of water pollution in the low Volga reservoirs if the water and soil conservation works would not be adequate.

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Environmental assessment — legislation and planning requirements within the European framework

M. HENDRY, BA, MRTPI, MIHT, Partner, Adams Hendry, Chartered Town Planners and Environmental Consultants

SYNOPSIS This paper describes the legal foundations on which environmental assessment of development proposals is based and the planning context within which decisions are made. It highlights a number of practice points that arise from the requirement to assess the environmental impact of proposals, including reservoirs and dams, before consent can be granted. The paper concludes that environmental assessment should be integrated into the processes of project design and consultation with expert bodies and the public. It is suggested that this approach will result in schemes that have an acceptable impact on the environment.

INTRODUCTION

1. Every major development goes through at least three decision-making processes. The developer (who is as likely to be a government department or statutory undertaker as the conventional 'speculator') makes decisions about location, design, programme and cost. Pressure groups, whether they are people living close to the project or national organisations concerned with a particular issue, decide whether they need to safeguard their interests by objecting to the proposal (or supporting it) or attempting to influence its design or location. Finally, the consenting authority (national or local government, or statutory regulator) decides whether the proposal is acceptable and if it should go ahead. The courts provide a fourth decision-making forum if things go wrong.

2. These decision-making processes are increasingly concerned with environmental matters. The environmental impacts of major developments can become issues of national political significance: witness Twyford Down, Oxleas Wood and the Channel Tunnel Link. Future examples might include M25 widening. Local planning authorities make their decisions on planning grounds, but like central government they are political organisations, usually with environmental agendas geared towards local circumstances. The proposals will be judged against the background provided by statutory structure and local plans and regional planning guidance.

ENVIRONMENTAL ISSUES

3. Along with the transport and infrastructure projects referred to above, reservoirs and dams are amongst the most potentially controversial developments. They almost always have a dramatic immediate impact on

the landscape and usually lead to changes in water tables, flood patterns, aquatic and terrestrial ecology, and human activity. The environmental impacts may be negative or positive (probably a mixture of both). It is likely that they will be significant and long term.

4. If promoters of such major developments are insensitive to the views of local people and pressure groups, and do not understand the environmental criteria that the local planning authority or regulator will apply to their proposals, they should expect confrontation, delay and a much greater chance that their scheme will be rejected. It is, therefore, surprising how many promoters still think about the decision-making processes in discrete phases. First they design the scheme, then (if they have to) they assess its environmental impact. Unfortunately, environmental assessments added as an afterthought often sacrifice objectivity in order to justify the proposals and come unstuck when subject to scrutiny. This approach invites controversy and confrontation. A much better strategy is to incorporate environmental assessment within the design and consultation process, so that when the scheme is submitted to the regulators, it is known to be acceptable in environmental terms

The Role of Environmental Assessment

5. Environmental Assessment (EA) is the process by which information about the environmental effects of a project is systematically gathered and presented. Used to best effect, it plays a central part in the generation and comparison of options, design of the project, and analysis and presentation of its likely effects. EA also provides the basis for auditing and monitoring the environmental effects of development. The EA process is intended to prevent or minimise adverse environmental impacts arising from potentially harmful development.

6. An Environmental Statement (ES) is the document, or series of documents, which sets out the results of the assessment. The ES provides information about the development and its likely impacts, which has to be taken into account by the decision maker before the grant of consent.

7. The environmental consequences, or impacts, of major development proposals have been a central concern of the British planning system since its establishment in 1947. However, until 1988, environmental impacts were explored in an unstructured and often ad hoc manner. Since 1988, European Directive 85/337/EEC ("The assessment of the effects of certain public and private projects on the environment") has formalised the need for information gathering, consultation and assessment.

8. An effective environmental assessment will be based on the foundation of the requirements and objectives of the EC Directive, which I deal with first below. The framework given by national legislation is the next topic- especially the Town and Country Planning (Assessment of Environmental Effects) Regulations 1988. Finally, the paper considers how the legislation helps define the fabric of an assessment - that is what goes into an individual assessment and how it is presented.

The EC Directive

9. The EC Directive requires assessment of specified types of development to be carried out before development consent is granted. The Directive's objective is as follows:

> *"The effects of a project on the environment must be assessed in order to take account of concerns to protect human health, to contribute by means of a better environment to the quality of life, to ensure maintenance of the diversity of species and to maintain the reproductive capacity of the ecosystem as a basic resource for life."*

10. Relevant projects are defined as

> *"the execution of construction works or of other installations or schemes or other interventions in the natural surroundings and landscape including those involving the extraction of mineral resources".*

11. In the UK such proposals mostly require planning consent. There are some major exceptions, to which I shall refer later. In most cases, however, there is a clear link between environmental assessment and the town and country planning system. The Directive requires that assessment should be an integral part of the consent procedure.

12. The Directive says that the purpose of the environmental impact assessment is to

> *"identify, describe and assess in an appropriate manner, in the light of each individual case ... the direct and indirect effects of a project on the following factors:*
>
> 1. *human beings, fauna and flora* (i.e. life)
> 2. *soil, water, air, climate and the landscape* (i.e. the setting for life)
> 3. *the inter-action between these two* (i.e. changes in the environment)
> 4. *material assets and the cultural heritage* (i.e. objects like buildings but also less tangible matters such as a stretch of countryside made famous by great literature)

13. The developer has to provide at least the following:

1. A description of the project comprising information on the site, design and size of the project. (Do not be deceived by the apparent simplicity of this requirement, many assessments fail because they are not clear about these straightforward matters)
2. A description of the measures envisaged to avoid, reduce and, if possible remedy significant adverse effects. (This assumes that the adverse effects have been correctly identified.)

3. The data required to identify and assess the main effects that the project is likely to have on the environment.
4. A non-technical summary of the information given in 1.-3. above.

14. Environmental assessment is thus about providing relevant information in an understandable form. There is a link with monitoring to ensure that forecasts and promises are maintained. Conditions on a planning consent or separate planning obligations agreements under Section 106 of the 1990 Planning Act may be related to the findings of environmental assessments in order to give them legal effect.

15. Assessment is also about consultation and participation by the regulatory agencies - such as HMIP or the NRA - and the public in the processes of preparing an assessment and making a decision on whether the development should be permitted. The Directive states that

> *"authorities likely to be concerned by the project by reason of their specific environmental responsibilities must be given an opportunity to express their opinion on the request for development consent. The assessment is to be made available to the public, who must be given the opportunity to express an opinion before the project is initiated."*

16. Consent cannot be granted until all this information and comment has been taken into account. They key words are "taken into account". The regulator, in the case of the planning regulations the planning authority, cannot ignore the conclusions of the assessment.

17. In practice the content and presentation of many assessments for complex schemes leave much to be desired. They are frequently over-long or occasionally unacceptably brief. All too often the consultation process is hampered by impenetrable prose and unapproachable graphics. Whilst it may take courage to expose the negative impacts of the project in clear English and straightforward diagrams, in the long run little is to be gained by obfuscation.

National Legislation

18. National legislation is constructed on the foundation of the EC Directive. Section 15 of the 1991 Planning and Compensation Act brings environmental assessment into primary legislation by introducing a new section 71A into the Town and Country Planning Act 1990. Section 71A permits the Secretary of State for the Environment to make regulations to extend the scope of the EC Directive to cover other projects. The Secretary of State has already used these powers to add projects to those listed in the EC Directive. The net is likely to be spread wider to cover more developments - such as operational works by statutory undertakers - for which planning permission is deemed by the General Development Order.

19. A series of Regulations integrates the Directive into UK law. Most of the developments for which the Directive requires environmental assessment also require planning permission. For these projects the

Directive is linked with the planning system by the Town and Country Planning (Assessment of Environmental Effects) Regulations 1988.

20. Separate regulations cover developments which may need EA but do not require planning consent. These cover some land uses which can have significant environmental impact, for example, land drainage and some highways. So the fact that planning consent is not required does not mean that a project will escape assessment.

21. The regulations ensure that environmental assessment is part of the consent process as required by the EC Directive. They complete the framework by listing projects for which EA is mandatory (Schedule 1, reflecting Annexe 1 to the Directive) and those for which it may be required (Schedule 2, reflecting Annexe 2 to the Directive).

Schedule 1.

22. Schedule 1 projects (for which environmental assessment has to be carried out) are all large scale developments likely to have a major impact on the environment. The list includes an oil refinery, an iron or steel works, a motorway, an aerodrome with a runway over 2,100 metres long and a port handling vessels over 1,350 tonnes. Planning applications for these developments must be subject to assessment and accompanied by environmental statements.

Schedule 2.

23. Reservoirs and dams are in schedule 2, which is a list of developments where assessment is required if the project

> *"would be likely to have significant effects on the environment by virtue of factors such as its nature, size or location".*

24. Schedule 2. covers a wide range of developments including agricultural projects, industrial development of various types, mineral extraction, urban development, other infrastructure, waste water treatment works, roads and marinas. In short, virtually every development that is likely to have an environmental impact is listed here. Schedule 2 includes a 'long distance aqueduct', which could be interpreted as a water pipeline or sewer, whether above or below ground.

25. There is some guidance on what is meant by the term "significant". The Secretary of State has said (in DoE Circular 15/88) that he thinks assessment is needed in three circumstances:

1. Major projects which are of more than local importance. There is a link here with the criteria which the Secretary of State's uses in deciding whether to call-in controversial applications for his decision;
2. Occasionally for projects on a smaller scale which are proposed for particularly sensitive or vulnerable locations. The more environmentally sensitive the location, the more likely assessment will be warranted. The Circular mentions National Parks, AONBs, Green Belts, SSSIs, National Nature Reserves, areas of archaeological importance, Special

Protection Areas (EC Birds Directive) and wetlands designated under the Ramsar Convention.

3. In a small number of cases, for projects with unusually complex and potentially adverse environmental effects. Here the processes involved rather than the location is significant. Industrial projects involving emissions which are potentially hazardous to man and nature may well come into this category.

26. The decision about whether the impact is 'significant' lies with the planning authority, or the Secretary of State can be asked for a ruling. It is advisable to talk to the planning authority as early as possible in the process about the need for assessment and the issues which should be addressed. An evaluation of the likely environmental effects of the project, or 'scoping study', should be undertaken at the beginning of the project. It should be remembered that a competent environmental assessment providing objective evidence in support of a planning application will help smooth the path through the planning system. Therefore many applicants volunteer assessments, even where there they may not be essential.

27. Schedule 3 of the Regulations also gives advice on the 'fabric' of the assessment - how an assessment should be put together and presented. It describes the topics which may need to be investigated, but does not and, because each case is different, cannot prescribe how it should be done.

28. Schedule 3 lists "specified information" which has to be provided in every assessment. They were described above under the "foundation" heading and can be summarised as:

1. a description of the project;
2. a description of the mitigating measures;
3. the data required to identify and assess the impact on human beings, flora, fauna, soil, water, air, climate, the landscape; any interaction between the foregoing; material assets, and cultural heritage;
4. A non-technical summary.

29. In addition, Schedule 3 lists further matters on which information may be provided

1. Physical characteristics and land-use requirements during construction and operation.
2. Characteristics of production processes.
3. Type and quantity of residues and emissions.
4. Main alternatives studied and reasons for choosing the development proposed.
5. Direct and indirect effects resulting from use of natural resources and emission of pollutants, creation of nuisances, and the elimination of waste.
6. The forecasting methods used to assess the environmental effects.

7. Any difficulties, such as technical deficiencies or lack of know-how encountered in compiling the specified information.

30. The schedule defines 'effects' very widely to include secondary, cumulative, short-, medium- and long-term, permanent, temporary, positive and negative impacts. Any further information provided must be accompanied by a non-technical summary.

31. It can be seen that, having built the foundations and erected the framework, the law leaves design of the fabric of the assessment very much in the hands of the developer and his or her advisers.

GOOD PRACTICE

32. I have already explained my main conclusion - that environmental assessment should be part of the design process and not tacked on at the end. Environmental assessment identifies the main issues with which the planning authority are likely to be concerned when the application is submitted. These are not necessarily the same issues on which the project's designers and promoters will have been concentrating. Major environmental impacts and issues are best faced at the design stage rather than before a Inquiry Inspector. It may be possible by design to minimise controversial impacts, or it may better to change the site, the process used, or possibly even abandon the project. In any case, early action is preferable to pursuit of lost causes

33. To this main conclusion, I add four extra practice points. First, the assessment must be objective. This is not necessarily the easiest of tasks, but the temptation to use the environmental statement for advocacy should be resisted: it will usually show through, with predictable results. The regulations require an explanation of how the forecasts of impact have been arrived at, and also the requirement that any shortcomings of knowledge or data should be noted.

34. Second, consult the local planning authority, the public and specialist organisations as much as possible. This may also not be easy when dealing with commercial ventures. One of the key features of the regulations and the Directive is that assessment is a process of exploration of likely environmental effects by experts in those fields leading to explanation in understandable terms. Hence the legal requirements for consultation and the non-technical summary.

35. Third, assess alternatives as far as possible. Although the legislation mentions the environmental statements may set out the consideration given to alternatives, where there is controversy, this aspect may become more significant. In the case of reservoirs, alternatives includes not just alternative sites but other ways of achieving the objective, perhaps by water conservation measures or trunk distribution from another source.

36. Fourth, a successful environmental assessment is very often the result of a team effort. The team consists of the client and whatever specialists are required. Co-ordination is vital. The tensions between the

client and his advisers can often show through, and a set of separate reports by individual consultants under one cover is often the worst solution.

37. The general points in these conclusions apply to assessments for dams or reservoirs as much as to other developments. The detailed content of an EA for a reservoir or dam will depend on the environmental context of the locality. Clearly the impact on the aquatic environment must be considered in detail. There will also be major effects on the terrestrial environment and during construction.

38. Finally, a useful checklist of EA best practice can be summarised from advice given by the Institute of Environmental Assessment and others, such as Manchester University, who have studied many environmental statements, good bad and indifferent. A successful assessment should:

1. Describe clearly the development, the processes and the activities involved.
2. Describe the environment as it now is (often referred to as *baseline conditions)* and as it could be expected to develop if the project were not to proceed.
3. Identify the key environmental impacts and evaluate their importance. Methodologies should be explained and the results of consultation with expert bodies and the public outlined.
4. Predict the scale of the impact, quantifying it where possible.
5. Assess the significance of the impacts which remain after mitigation using appropriate national and international quality standards where these exist.
6. Estimate the types and quantities of waste and how it is to be dealt with.
7. Explain the consideration given to alternative sites and the reasons for the final choice. Alternative processes, designs and operating conditions (which should have been considered at an early stage of project planning) should be outlined.
8. Consider all significant impacts for mitigation and put forward measures where practicable. It should be clear how and when the measures will be carried out. Their effectiveness should be evaluated. Where there is uncertainty, monitoring programmes should be proposed.
9. Be clearly laid out with the minimum amount of technical terms.
10. Be an independent objective assessment of environmental impacts, not a best-case statement for the development. Negative impacts should be given equal prominence with positive impacts.
11. Be accompanied by a non-technical summary that must contain sufficient information to explain how the main conclusions were reached. It should briefly explain the project, the baseline conditions, proposed mitigating

measures and their effectiveness, and the remaining impacts. A brief explanation of methodology and an indication of the confidence that can be placed in the data used should also be included.

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Maximising the ecological benefit of a new small reservoir

B.H. ROFE, MA(Cantab), CEng, FICE, FIWEM, Partner, C.G. HOSKINS, BSc(Eng), CEng, MICE, Associate, Rofe Kennard and Lapworth, and P. RICE, BSc, Lecturer, University of Derby

SYNOPSIS. Wetlands of all types, from coastal and inland marshes to small ponds, are habitats under threat from a variety of activities such as drainage, infilling and pollution. Artificially created reservoirs are also wetlands and if designed and managed sensitively can compensate somewhat for the loss of natural wetland habitat. Recent estimates suggest that more than 2,500 small reservoirs exist in Britain with a substantial number being created every year. Collectively they represent a potentially significant addition of wetland habitat and locally each could be a haven for wildlife in an otherwise agricultural or environmentally barren area.

INTRODUCTION

1. Small reservoirs are rarely created for the sole benefit of wildlife interest. In the past many were constructed primarily for agricultural and industrial water supply purposes, but more recently a wider use has been evident, with functions as diverse as fish farming, fishing, fire fighting, amenity use and as landscape features in commercial or industrial developments. Whatever the reason for construction, the opportunity exists to create a small reservoir that also has wildlife and conservation benefits. These may be aimed at specific groups such as waterfowl, fish, amphibians or dragonflies. Alternatively it may be desired to create an attractive environment with general habitat value. Positive measures can also be included to provide a varied habitat, recreate an earlier lost environment or mitigate adverse features. Whatever the objectives, they can be maximised by considering ecological issues at the outset and incorporating them into the design, construction and management procedures.

2. This paper refers essentially to reservoirs with a capacity not exceeding 25,000m^3 and a lower limit of a few thousand cubic metres; such reservoirs typically fall within the scope of the guide on small reservoir creation to be published shortly by CIRIA. Many of the aspects are equally applicable to reservoirs of any size, although the limited

extent of the very small water bodies may not be appropriate or compatible with all of the matters raised.

FEASIBILITY

3. When planning a new small reservoir, the ecological feasibility of the reservoir should be considered, not only in terms of the best location but, more importantly, where not to locate it. Creation of a new water body will necessarily destroy the existing habitat of the site; to achieve maximum ecological benefit it is essential that the existing habitat is not in itself of high conservation or wildlife value. Statutory designations may exist to prevent any form of damaging activity, for example the site may be a Site of Special Scientific Interest (SSSI) or may harbour plant or animal species protected under the Wildlife and Countryside Act 1981. Even if the site is not of designated importance it may still have a present wildlife value that is greater than the proposed replacement. Dams alter drainage patterns, often to the extent that could be detrimental to existing communities of wetland animals and plants downstream. The new reservoir could have an adverse effect on an existing natural wetland, and thus the habitat loss may be greater than the potential gain.

4. It is important that the environmental impact of the new reservoir is fully assessed; this may be a legal requirement in many cases. Constructing a reservoir normally requires planning approval, in which case there may be a statutory requirement to assess the environmental impact of the proposal. (Town and County Planning (Assessment of Environmental Effects) Regulations, 1988). Specific conditions may be attached to the planning consent and early consultations with the Local Planning Authority are useful to identify any issues, environmental or otherwise, which may affect feasibility. If a new reservoir is to be ecologically beneficial the importance of the existing habitat should be fully assessed even if an environmental impact assessment is not a statutory requirement. If the habitat is valuable, then preservation should be considered and the reservoir located elsewhere.

5. In ecological terms it is preferable to locate an new small reservoir near to existing wetland habitat. This not only increases the size of the habitat for wildlife but also provides a close source of flora and fauna for colonisation of the new area. If this is not a possibility then the new reservoir should be located close to other semi-natural habitat such as grassland, hedgerows or scrub, and in quiet, secluded locations to minimise disturbance to wildlife. These measures will enhance the value of the new reservoir for wildlife by strengthening and extending existing habitats rather than creating a separate element in an already fragmented semi-natural landscape.

MAXIMISING ECOLOGICAL BENEFIT

6. There may be specific wildlife requirements that need to be catered for in the design when designing a new small reservoir. If the reservoir is to be used for fishing the focus will be on habitat creation for this purpose. Similarly if the objective is wildfowl shooting then design features that would encourage the breeding of relevant species would be incorporated. Advice on such specific requirements is contained in the CIRIA guide and information produced by specialist organisations should also be consulted e.g. the British Association for Shooting and Conservation (1986), the Game Conservancy (1993) or the Angling Foundation (1981). There are a number of design features that can be incorporated at minimal cost that will improve the ecological value of the new wetland area.

7. It is important that habitat provision is carefully planned to maximise the ecological benefit of a new small reservoir so that the best conditions for the development of ecological zones are created. Different species of plants and animals vary in their habitat requirements and it is important to provide plenty of different habitat features. Ideally there should be space created to allow a succession of vegetation types ranging from open water to adjacent unimproved grassland to nearby scrub and woodland. Indeed, if nature conservation is the primary objective, it is often better to create a slightly smaller water body with surrounding space for associated terrestrial habitats.

8. Variability in the vegetation surrounding the reservoir is not only of ecological benefit but it also greatly improves the visual quality of the location. Habitat features such as varied shoreline, created by islands, spits and bays, steep and shallow banks, not only provide different features for wildlife but also make the reservoir itself more attractive. This type of reservoir becomes more a part of the natural landscape than one with uniform shape as can be seen in Figs. 1 and 2. Such features may be confined to the reservoir perimeter, but there is normally no reason why many of the aspects discussed cannot be continued onto the upstream face of the dam itself provided they are compatible with the reservoir operation and other possible constraints.

DESIGN AND CONSTRUCTION MEASURES

Shoreline

9. One of the most important design aspects of a small reservoir is its shape. A varied shoreline is essential if the reservoir is to be of value to wildlife. A series of irregular and variable bays and spits increase the length of shoreline, creating habitat diversity and providing shelter for plants and invertebrates; both important food sources for birds and fish. Such design is also likely to increase opportunities for breeding wildfowl; small bays increasing

FIG.1 FEATURES OF A WELL DESIGNED RESERVOIR FOR ECOLOGICAL BENEFIT

FIG.2 FEATURES OF A POORLY DESIGNED RESERVOIR FOR ECOLOGICAL BENEFIT

the number of nesting sites available to reducing site lines and thus the size of individuals nesting territory.

10. It is essential that shorelines have a gentle slope if they are to form a base for vegetation establishment and allow access to and from the water by wildlife. This is particularly important if wildfowl breeding is to be encouraged, because ducklings cannot manage steep slopes. As a general rule shallow margins for wildlife should be graded to at least 1V:10H down to a maximum depth of 1.5m. This can be achieved either by grading back the margins, or by depositing overburden. If space and other constraints allow, it is desirable to create some areas of steep bank to increase habitat diversity. These should extend to at least 1.5m above top water level and should attract cliff nesting species.

Islands

11. A further way of increasing the amount of available shoreline is to include islands in the design of a water body. Although islands need only be small to be suitable for wildfowl breeding, they must have low gradient margins to allow access. Ideally they should be an irregular shape (e.g. cross, atoll or horseshoe shaped) and preferably orientated to give some shelter from the prevailing wind. Such islands are only likely to be feasible on larger lakes and can be created by localised excavation or by depositing and sculpturing overburden.

12. On smaller reservoirs it is still possible to encourage wildfowl breeding by providing artificial nesting sites. This can be achieved by substituting floating rafts for islands, a particularly useful technique if water level is likely to fluctuate. A variety of sources contain information on the design and construction of such rafts. (Andrews and Kinsman, 1990, and British Trust for Conservation Volunteers, 1975).

Water depth

13. The maximum water depth will depend on the size of a water body and its purpose. If water storage is the objective, then the reservoir will need to be considerably deeper than would be required for conservation purposes. Variation in water depth, however, does maximise wildlife benefit by providing habitat variation and increasing the range of feeding areas.

14. Most aquatic plants grow best in the shallow water zone where light penetration is good and the water warms quickly in the growing season. Consequently this is the zone richest in invertebrate life and of most value to other wildlife. Shallow ledges should therefore be constructed, ideally with a minimum depth of 50mm grading down to a maximum of 500mm, and should be covered with water for most of the year. These shallow ledges should then grade down

into deeper areas, although water deeper than three metres becomes less valuable to wildlife.

15. Where water is to be abstracted, the reservoir level is likely to fluctuate. Such variations may be regular, seasonal or intermittent and may be slow or rapid. The ecological values of such reservoirs is potentially reduced, but careful design to provide shallows and permanent pools will help plant and wildlife. Vegetation may be more problematical if the drawdowns are substantial and frequent; few plants will thrive under such conditions, especially if the periods of drawdown are extended.

Aquatic vegetation

16. Where a new reservoir is supplied by a river or stream, there will be a ready supply of plant material for natural colonisation. Planting will speed the establishment and assist in the development of an aquatic community and improve the appearance of the new water body. Care must be taken to introduce only those species that are naturally occurring in the local area. Unusual or exotic species should be avoided because they may upset the natural ecological balance and rapidly become dominant in the reservoir.

17. For maximum ecological benefit, a complete planting programme should be planned based on a natural gradation from dry land plants, through marginals and emergents, to submerged and floating species. These zones are based on the position the plants occupy in relation to the water margin and are illustrated in Fig. 3. Species can be selected for planting that have particular food value for fish or wildfowl, or simply provide general habitat value. Information relating to common plant species that can be introduced in each of the vegetation zones can be found in the CIRIA guide together with suggestions for potential sources.

FIG.3 THE MAIN AQUATIC PLANT ZONES

18. Planting of most species is best carried out in spring to allow the plants a full growing season to become established, but rhizomatous species such as common reed should be transplanted in February or March when the rhizomes are about to shoot. If different species are planted in groups, competition leads to the most vigorous species becoming dominant. It is advisable therefore to plant the reservoir margins with separate blocks of species, allowing each to become established and develop a varied waterside flora.

Terrestrial Vegetation

19. Terrestrial vegetation should be planned so that a variety of habitats adjacent to the reservoir are created or extended and structural diversity of vegetation is provided. It is desirable to introduce only native species, as with aquatic vegetation as these also tend to have a greater wildlife value.

20. Wildflower and grass seed mixes are now widely available and enhance the natural value of the reservoir surrounds if introduced upon restoration. It is important to select a combination with appropriate species to suit site conditions; specialist advice can usually be obtained from seed suppliers. If a wildflower sward is the objective, topsoil should be stripped. This is because the higher fertility of topsoil and presence of a seedbank leads to the suppression of the desired wildflowers by more vigorous species. It is often beneficial, however, to create some areas of tall, tussocky vegetation for diversity. This can easily be achieved by mounding uncompacted topsoil and allowing it to revegetate naturally with rough grasses and species such as teasels and thistles.

21. Additional habitat diversity can be obtained by planting a selection of trees and shrubs near the reservoir. Trees and larger shrubs should generally be planted to the north side of the reservoir, leaving the south open but with some shaded areas. Particular planning and care should be exercised in relation to the location and choice of tree species that may be planted near the waters edge. Although the shade provided by carefully sited trees is useful for controlling aquatic weed growth, too much shade could seriously inhibit the growth of many other species that may be useful food plants for other wildlife. Dead and decaying leaves that drop into the water provide detritus upon which many aquatic animals feed, but if too much accumulates then it may smother and inhibit the growth of other vegetation. As a general rule, most trees and larger shrubs should be planted some distance away from the reservoir itself and conifers should never be planted in the vicinity (these may contribute towards acidification of the water). Willows and alder are species that are commonly found overhanging freshwater and could usefully be planted in places at the waters edge away from the dam, to provide visual appeal and an added dimension of habitat variety.

The Embankment

22. Although the design of the dam must focus primarily on safety and function it is possible to incorporate environmentally sympathetic measures. The shape of the embankment can be varied by introducing flatter slopes or a wider crest in places. Crest level can also be varied provided this does not affect any flood and wave retention ability. Tree and shrub planting can also be extended to the embankment itself and to maximise the ecological benefit of a new reservoir, should be considered as an integral part of the design. Provided vegetation is sited with care and certain critical areas are kept clear, it will not only increase the wildlife value of the construction but can also improve the safe functioning of the dam (Hoskins and Rice, 1992, and Hoskins, 1993). Certain water demanding species should generally be avoided, including willows and alder, that may have been planted elsewhere around the reservoir.

VEGETATION MANAGEMENT

23. Once construction is complete, and a new small reservoir is operational, a regular programme of vegetation management will be necessary to provide continuing ecological benefit and visual amenity. Ideally the long-term maintenance requirements should be considered at the design stage and an outline habitat management plan prepared that identifies the major tasks and objectives of management. Details on the practical aspects of management can be obtained from a variety of organisations including the British Trust for Conservation Volunteers, 1975, 1978 and 1980; the Forestry Commission/DOE, 1992, and ADAS/Forestry Commission, 1986, and ADAS, 1990.

24. The aim of aquatic vegetation management should be to maintain the vegetation zonation shown in Fig. 3 and prevent aggressive plants becoming dominant with the loss of less competitive species and diversity. Reservoirs which are overgrown with vegetation provide poor conditions for fish and wildfowl and are also visually less attractive. Unwanted vegetation can be cut and removed either mechanically or by hand, although the spoil should be left at the waters edge to drain so that aquatic invertebrates can return to the water. Herbicide sprays can result in damage to pond life and are best avoided and only used after appropriate advice has been sought.

25. The ecological value of grassland is best maintained by grazing or cutting. If livestock are used, their access to the water should be controlled to avoid trampling or marginal vegetation and fouling of the water. Sheep are preferable to cattle because they are less harmful to the vegetation and the often soft ground adjacent to the reservoir. Cattle should never be allowed to graze embankments because the steep slopes are particularly vulnerable to damage by the animals' hooves.

26. A wildflower sward should be cut at least three times in the year following establishment, with the cuttings removed to reduce nutrient levels. Subsequent management will depend on the effect required and site conditions. Cutting may be followed by grazing for example, or one cut in late summer would produce an open sward suitable for wildflowers, provided the cuttings were removed. Where it is required to maintain tall tussocky vegetation, management will only be necessary every two to four years to prevent scrub encroachment.

27. Newly planted shrubs and trees require careful maintenance to ensure successful establishment. Regular weed control must be carried out to avoid competition and they must be protected adequately from grazers and rabbits. Once fully established, scrub maintenance work should involve mainly cutting back to encourage different stages of growth and improve structural habitat diversity. A rotational system of management is best so that there is always some mature cover available. Trees can be lightly pruned four or five years after establishment to encourage a leading growing shoot. Once they mature, management should focus on preventing their growth shading the water surface too much and ensuring the continuing health of the tree. Trees which become too large, out of balance or unhealthy will require selective lopping and pruning and may ultimately require removal and replacement if they die or become too unhealthy or mis-shapened.

28. The timing of vegetation management is a crucial factor to consider if a reservoir is being managed for wildlife benefit. Wildflower grassland for example should not be cut during the flowering season and any undue disturbance should be avoided during the bird nesting season. Aquatic vegetation management also needs to be carefully planned. Winter management work will reduce the amount of food available to wildfowl at a time when food supplies are already depleted and is likely to disturb dormant animals such as frogs. Similarly work in spring or summer could disturb the breeding and development of amphibians or invertebrates. Ill-timed or inappropriate trimming or pruning of hedges and trees may be detrimental to nesting birds and small mammals. Consequently the timing of any vegetation management should be planned to avoid disturbance at important times of the year, and this aspect is also discussed in the practical guides referred to in section 23.

CONCLUSIONS

29. The ecological benefit of a small reservoir can be maximised by consideration of the appropriate features at an early stage. This should be during the feasibility study for the reservoir itself and not as an add-on feature at a late stage of the reservoir creation. Normally the design can be modified at little or no cost to include features which will do much to improve the ecological benefit of the reservoir creation.

30. Every site is different and the specific requirements and constraints must be identified. Some reservoir usages may preclude or limit certain ecological features, especially if the water level is intended to be varied rapidly or substantially. In other instances, and especially when the water level is essentially constant, a rich and varied habitat can be established which can be directed toward a specific end use for the reservoir. Various features can be incorporated for most, if not all reservoirs, which will be beneficial and maximise the ecological and visual value of the reservoir and surrounding area.

31. The long term maintenance requirements should be considered during the design and an outline habitat management plan prepared which identifies the major tasks and objectives of management. Regular and considered maintenance will then allow the ecological aims to be continued and the visual and other amenity benefits to be maintained.

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Roadford Reservoir — enhanced flows, fisheries and hydroelectric power generation

H. SAMBROOK, BSc, MSc, Principal Fisheries Scientist, and P. GILKES, BSc, CEng, MICE, MIWEM, MBIM, Clean Water Programme Manager, South West Water plc

SYNOPSIS

Roadford Reservoir is a new strategic reservoir which regulates river flows to the River Wolf, a tributary of the River Tamar. Regulation releases are used to generate hydroelectric power at the dam site. Environmentally acceptable rules have been developed to control the overall programme of combined releases in any year and to define specific patterns of release. Specific discharge and velocity criteria are identified in order to progress the development of an optimum operational regime. The enhanced flow programme forms an integral part of the management of the reservoir and its resource. This programme maintains and improves natural recruitment of wild trout and salmon in the River Wolf. The "Non-Fossil Fuel Obligation" has given greater flexibility to the management of the resource and offers real environmental benefits to both the fisheries and the river.

INTRODUCTION

1. Roadford Reservoir is a new strategic reservoir, located in West Devon on the River Wolf (See Figure 1). Construction of the reservoir began in 1986 and was completed in October 1989 when impounding of the river commenced. From the time of first filling, the reservoir took 19 months to reach 99% full. Roadford has a net capacity of 34,500 Ml.

2. Roadford Reservoir is the centrepiece of a complex water resource development. Roadford operates conjunctively with other sources across the county of Devon and links river abstractions for the Tamar, Tavy, Dart, Taw and Torridge with storage from Burrator and Meldon reservoirs in the Plym and Torridge catchments respectively. These sources provide secure supplies of water to three main areas in the South West region, i.e., Plymouth, North Devon and South West Devon.

3. Plymouth is, in part, supplied by water from the River Tamar either from the unsupported natural flows or by supported releases from the reservoir to the River Wolf. The main abstraction point on the River Tamar is located approximately one kilometre upstream of the tidal limit at Gunnislake. Water is pumped to water treatment works at Crownhill and Roborough (the latter is to be constructed), with maximum combined treatment capacities of 145Ml/d. From these works the water can be

Figure1: Roadford Reservoir and the Tamar Catchment

distributed throughout south-west Devon via the South Hams Trunk main. Water for North Devon is abstracted directly from the reservoir and transferred via pipeline to a new water treatment works at Northcombe (existing capacity of 40Ml/d; phase II maximum capacity of 80Ml/d), from where it gravitates throughout much of the northern distribution network.

4. The concept of the Roadford Strategy is that abstraction is permitted when there is sufficient water in the rivers, yet under dry periods and receding low river flows, river abstraction is reduced with greater supplies obtained from Roadford Reservoir (Lawson et al, 1991). Potentially, the scheme could have significant impact on the biota and environment of the six named river catchments and three reservoirs, but since 1990 sophisticated operating rules have continually been developed and implemented to avoid this occurring. These rules are evolving, based partially on detailed knowledge of the relationships between such factors as river flows, water quality, fisheries/ fish behaviour and aquatic life in the rivers and river corridors.

5. All six river catchments support important sport and commercial fisheries for salmon, sea trout and brown trout. The extent of these fisheries represents an important financial contribution to the local economies and directly reflects the high quality of the riverine environments in the South West of England. As a result, throughout the promotion and development of the Roadford Scheme, fisheries and environmental aspects have always been considered as factors of significant importance. At the Public Inquiry (1978) undertakings were given to fishery owners and outside interests which still have to be honoured as the present scheme evolves. South West Water's commitment to working in harmony with the environment is partially reflected in the financial commitment to the environmental studies associated with Roadford. To date, an estimated 6% of the total scheme budget has been allocated to environmental and fisheries related work (Gilkes et al, 1991). The financial return on this investment will be long term as refined operating rules are implemented and the optimum balance obtained between environmental impact/benefit, sustainable resources, drought reliable yield and operating costs.

6. Environmental principles are being applied to the operation of this major strategic resource and benefits are already identified. By means of one example only, this paper describes the pathway by which flow requirements to support fish have been used as the basis for the control of all augmentation releases to the River Wolf. Rules have been developed and are continually being refined to promote an environmentally acceptable strategy that supports hydroelectric power generation. Such an approach has been implemented through a process which involved the identification of specific flow criteria, development of appropriate installations to control flows to set patterns of release and finally the integration of an enhanced flow programme based on seasonally variable flow requirements. (Note: This approach has been

adopted and forms an integral part of the management of the water resource, where supplies to customers is the first priority.)

AUGMENTATION RELEASES AND FLOW CONTROL UTILIZING HYDROELECTRIC POWER GENERATION

7. One major aspect of any impoundment is the impact on the natural flow regimes in the downstream river reaches. During the promotion of the Scheme in 1978, it was acknowledged that, with the exception of the contributory flows of the lower catchment, the future flow regime in the River Wolf would be limited to the fixed compensation flow (9 Ml/d) and regulation releases for supply purposes (maximum 172 Ml/d). These flows could be supplemented in any year by the volume of spill water.

8. In the ten year period following the first Public Inquiry the potential to utilise regulation releases for HEP generation at Roadford was investigated and promoted. In addition to the compensation, supply and spill volumes, the use of "spare" storage was considered. (The "spare" water represents the surplus supply water available in the early years of the Scheme when design yield is greater than demand). It was intended to utilize the majority of the water in order to maximize revenue, reduce the payback period. and offset operating costs. In order to achieve these objectives, a strategy was required to ensure operation within environmentally responsible constraints.

9. In the assessment of HEP generation and operation at Roadford, detailed hydrological and operational modelling was undertaken. This study resulted in the derivation of a reservoir control curve which ensured the provision of reliable supplies through the worst drought sequence. With time, and higher demands, the control curve will gradually approach the top water level. This reduction in spare water will be offset by increased volumes of supply releases.

10. The hydraulic aspects associated with any of the releases are of particular significance to the biota and river environment. Environmental and fisheries concerns were stated regarding any releases which involved extreme rates of change from 9 Ml/d to maximum flows or vice versa, together with frequent and pulsed releases. A preference was specified for all regulation releases to mimic natural spate events recorded at Roadford, as closely as possible. Also that supply releases should follow a steady block pattern.

11. Natural spate events in the River Wolf were distinguished by a rapid rising limb and a slower gradual recession limb to the hydrograph. Such events have been partially responsible for the evolution of channel morphology, physical habitat, the river corridor and the diversity of the biota. Historic records from the Roadford gauging station were used to define the characteristics of the "classic" spate and to derive important velocity and stage height information. Such information was essential in assessing the impact of any releases and to set threshold levels for patterns of releases in the early scheme years.

12. The "classic" hydrograph showed a natural rise from 9 Ml/d to a peak in excess of 320 Ml/d in less than 2 hours, while the recession took approximately 2 days. This generic hydrograph was mimicked and subsequently modified to reflect further environmental refinements and to overcome mechanical/electrical constraints of the control system. The hydraulic conditions were built into the contract specifications for the supply of the turbines and control equipment as part of the overall electrical and mechanical contract. A fundamental part of the contract was the computer based software control of releases via the turbines.

13. Figure 2 shows the current curves used to control the rate of rise and recession of the release flows to the River Wolf. The increase to the maximum discharge of 320 Ml/d from the base flow of 9 Ml/d occurs over a period of approximately 3.5 hours. Reduction in releases from 320 Ml/d to 9 Ml/d takes in excess of 40 hours. Constraints of the control system prevent the hydrograph curves being reproduced precisely. Releases involve numerous small "stepped" changes which are "smoothed" by the relatively short time intervals. The releases to the river are effected by software control of the turbines and valves. The "PLC" controls all releases of water to the predetermined pattern. However, changes can be made to the mimicked hydrograph by simple changes to selected constants. This facility offers greater operational flexibility and obviates the need for additional programming.

HEP GENERATION PLANT AND NON FOSSIL FUEL OBLIGATION

14. Francis turbines were selected and installed at Roadford based on a feasibility study into the performance, cost, revenue and suitability for the flow control system. The plant consists of three Francis turbines which at peak output generate approximately one megawatt. The turbines are sized to meet the full range of flows from 9 Ml/d to 320 Ml/d, with all three turbines operating to give maximum output.

The three turbines operate in the following flow ranges and have outputs of:

T3	6 -15	Ml/d	(40 kW)
T2	18-65	Ml/d	(180 kW)
T1	80-240	Ml/d	(730 kW)

The original assessment of the HEP generation plant was based on compensation flows and regulation releases for supply purposes. At the time the electricity tariff structure and conditions were extremely complicated. Different tariffs prevailed depending upon the time of day, the day of the week and month of the year. Compliance with these operating conditions would have resulted in a "tariff chasing" philosophy in order to maximise revenues. However, the privatisation Bill for the Electricity Supply Industry introduced the Non-Fossil Fuel Obligation ("NFFO"). This obligation enabled renewable energy sources and techniques to be developed and

a. RISING HYDROGRAPH LIMB

b. RECEDING HYDROGRAPH LIMB

FIGURE 2: CONTROL CURVES FOR ROADFORD RELEASES

promoted. A successful NFFO application was gained resulting in a flat rate tariff for the sale of hydro electricity. The new single high tariff rate, together with the introduction of the enhanced flow programme to complement the HEP generation strategy have reduced the pay back period to less than 6 years.

15. The Non-Fossil Fuel Obligation has given greater operational flexibility and is a fundamental part of the success of the enhanced flow programme. NFFO is considered synonymous with greater environmental benefits, not only in terms of renewable energy but also with respect to sustainable fisheries and the riverine environment.

FISHERIES NEEDS AND FLOW REQUIREMENTS: BASIS TO THE ENHANCED FLOW PROGRAMME

16. The aim of the enhanced flow programme is to maintain and improve natural recruitment of wild trout and salmon population in the River Wolf and other waters located downstream of Roadford Reservoir. In order to achieve this aim in any year specific discharges must be available and guaranteed throughout the year to enable completion of the natural life cycle of the fish. Specific discharge and velocity criteria have been identified which form the basis of an environmentally acceptable operational strategy.

17. Specific flow requirements have been identified for the various life cycle stages of salmon and trout. These have been based on extensive field studies in the Tamar catchment, together with literature reviews.

18. The operation of Roadford has affected the flow regime in the receiving water courses but of equal importance could be the water quality and temperature aspects. Salmonids require clean, well oxygenated and high quality water to survive. Sophisticated destratification equipment, draw off facilities, water quality instrumentation and control rules at Roadford ensure that the best available water is released to the River Wolf. This river is a designated water for purposes of the EC Freshwater Fish Directives and also has a River Quality Objective of 1B. All releases have complied with the standards defined in the legislation and set down by the regulatory bodies.

19. The enhanced flow programme has been based on a knowledge of the instream requirements of salmonids. The interactions between discharge, velocity, substrate composition, sediment movement and physical/chemical habitats and the resulting effects on salmonids, both temporarily and specially, are complex. Such factors can influence the choice of spawning sites, and upstream distribution of the adult fish, survival of the eggs and early stages (alevins) in the river gravels, emergence and survival of the older age classes, together with the resulting year class strengths.

20. The following points, as means of examples only, represent a selection of important influencing factors that underpin the enhanced flow programme for the River Wolf and are extensively linked to the life cycle of the fish:

(a) Spawning fish usually select a site where the gravels contain low silt content and well oxygenated high intra-gravel flows. Salmonid fish deposit eggs in shallow gravel excavations known as redds in the November-December period. Redd excavation results in coarsening of the gravels as fine materials are transported downstream. A minimum current is necessary to initiate motion of the fines. In addition, a minimum velocity is required to enable the adults to orientate to the spawning bed and excavate the redd effectively. Jones and King (1950) observed salmon spawning in depths of 0.15-0.60 m, with optimal surface velocities of 0.3-0.4 m/sec. Spawning was not observed in velocities of less than 0.08 m/sec. Brayshaw (1966) showed that salmon spawn at depths of 0.30-0.80 m and at velocities of 0.44-0.97 m/sec. Other researchers have recorded preferred depths between 0.21-0.49 m (ranges:0.09-0.76 m) and preferred midwater velocities between 0.31-0.55 m/sec (ranges 0.15-0.90 m/sec) (Beland et al 1982). At velocities greater than 3.66 m/sec spawning is inhibited as salmon are unable to maintain sustained swimming effort. Reiser (1976) found that brown trout utilise velocities of 0.22-0.38 m/sec and water depths of 0.14-0.28 m. Variations in depth and velocity criteria are expressions of the size of the fish, larger fish being more able to survive higher velocities due to their overall greater swimming speed. Crisp (1985) suggested that salmonids generally avoid spawning in water shallower than their own body depth and at velocities less than 0.2 m/sec.

(b) Salmonids are unable to distinguish gravels which will dry out during the incubation period (December-March period). Trout are more vulnerable to this problem as the redds are shallower and are cut in shallower water. Depth of salmonid redds are primarily a function of the size of the fish. Depths of 0.15-0.30 m and 0.08-0.22 m have been recorded for salmon and trout respectively.

(c) Permanent and temporary restriction of the stream bed available for spawning increases the overall densities of adults on confined spawning areas, resulting in overcrowding, overcutting of redds and under production of progeny.

(d) The structure and composition of the bed material will affect the successful development of the eggs. Bed stability and intra-gravel flow, which control oxygen supply and removal of toxic metabolic waste, influence survival. Water temperature is also an important factor in determining the rates of development of eggs and alevins and subsequently the residence time in the gravel. Gravel movement and disturbance by shifting debris are known to cause washout of the eggs and physical damage from crushing and smothering by redisposition of upstream material. Along many rivers, substantial transport of

channel bed sediment has been found to begin at 60% of the most probable annual flood which recurs on average once every 1.5 years ($Q_{1.5}$). On the River Wolf, 60% of $Q_{1.5}$ is 11.64m^3/s, well above the proposed maximum supply release. Maximum supply releases of 3.7m^3/s are unlikely to cause substantial bed degradation. Releases greater than 3.7m^3/s, of sediment free water, could cause local channel degradation but limited to approximately a 500m reach downstream of the dam. Substantial degradation has not been recorded as 30% of the substrate sediments are coarser than 5cm. The natural armour layer of coarse stones prohibits erosion.

The maximum rate of rise of supply release is less than 1.0 m^3/s in any 30 minute period. The channel width along a 3 km reach downstream of the dam is 6-8 metres. Thus, the proposed change of Q/W is 0.166 m^3/s in any 30minute period. Research on similar rivers shows that Q/W changes of 0.232 m^3/s in a 30 minute period does not cause any adverse effects to fish or macroinvertebrates. Major drifting of invertebrates and transport of fines has been observed with Q/W change greater than 0.4 m^3/s. Where the pre- release unit discharge (Q/W) is less than 0.1 m^3/s the rate of rise will increase downstream as the release "wave" front steepens. The wave front may steepen for approximately 5km downstream of the dam before declining, i.e. within the confines of the River Wolf. Commonly the rate of water level rise increases by 2 and 2.5 times. The impact of such rate of rise is a factor for consideration in defining the basic pattern of release.

(e) Oxygen consumption by salmonid eggs varies during development between December and April. Total consumption rates of individual eggs and critical concentrates increase during embryonic development and then decrease sharply after hatching. Critical dissolved oxygen concentrations for early eggs are 0.76 mg/l; eyed-ova 3.1 mg/l, pre-hatch 5-8 mg/l and hatching 7-10 mg/l.

(f) Oxygen supply to the embryos in the gravel is determined by the velocity of intra gravel water and the oxygen content of the water. Siltation and low water velocities will inhibit oxygen supply and removal of metabolic wastes. Sublethal effects of impaired oxygen supply include morphological changes, reduced size at hatching, premature hatching and reduced growth rates.

(g) Water movement through gravels can be affected by particle size, permeability and apparent velocity. The proportion of fine particles greater than 20% can have a major effect on egg survival. This factor influenced the "flushing" releases in October, especially in the early years of impounding and immediately post construction. Permeability in trout redds range from 160-6000 cm/hr. Trout are able to tolerate lower permeabilities than salmon. Minimum apparent velocities of

4.5-12.0 cm/hr have been recorded for salmon.

(h) Flow and substrata modification can have a direct impact on salmonid stocks by influencing regulating processes of early life stages of free swimming fry. As territoriality develops amongst the fry, these young fish migrate downstream to take up residence on gravel areas of 2-10 cm depth of water. As the fish grow, salmonids require larger bed material, greater water depth and more diverse habitat. Environmental pressures caused by changes in wetted area, depth and velocity will increase inter and intra-specific competition. This will increase mortality/emigration and reduce production of the stream. Deep runs and pools are utilized by larger juvenile and adult trout as these features provide cover and reduced velocities.

(i) Management of stream discharge and enhancement of specific habitat features can be used to optimize salmonid production. Greatest densities of salmon fry and parr have been recorded in the summer months at average velocities of 0.50-0.65 m/sec.

(j) Benthic macroinvertebrate populations show considerable spatial and temporal variation in abundance, diversity and composition. Like fish, invertebrates inhabiting flowing water are adapted morphologically, behaviourally and physiologically to their environment. The physical channel characteristics such as width, depth, gradient, roughness, bed type. hydraulics, together with water velocity all influence the invertebrate community. Substrate stability and heterogeneity of substrate particle size are critical in providing varied microhabitats which are able to support an abundant and diverse fauna. Both diversity and abundance of invertebrates are greatest in areas of medium water velocity 0.20-0.75 m/sec, over cobble substrate 6-25 cm diameter and at depths of 0.15-0.40 m.

21. Translation of such knowledge on fish needs, together with an appreciation of the complexities of the influencing factors, has progressed into flow control rules based on a combination of educated predictions and detailed field investigations. Routine monitoring stations linked to annual reviews have enabled the programmes to evolve and refinements made to improve salmonid production. In time, a definitive programme will be devised to attain an optimum balance between water resource usage and fisheries benefit.

ENHANCED FLOW PROGRAMME AND HYDROELECTRIC POWER GENERATION

22. Figure 3 shows a typical pattern of augmentation releases used to generate hydroelectric power in any year. The enhanced flow element is an important part of the integrated HEP and fisheries management programme.

The enhanced flow regime for trout and salmon production is comprised of

Fig. 3. Roadford Reservoir: Enhanced flows and hydroelectric power generation (period – 15 Oct. 1993 to 14 Oct. 1994)

the following seasonal variable releases:

(a) Month: October

Two artificial freshets are released outside the fishing season, i.e. after 15 October. These releases are used to flush fine sediments from the surface layer of the spawning gravels, hence reducing the risk of compaction and consolidation. Releases involve a steady build-up and should not be markedly stepped. Releases follow the basic shape of a natural hydrograph. The increase in the maximum discharge of 320 Ml/d from the base flow of 9 Ml/d occurs over a period of not less than 3.5 hours. The maximum duration of the peak can be up to 48 hours. The minimum discharge between the freshets is 45 Ml/d. The maximum rate of increase is 0.75 m^3/s in any 30 minute period. The time increment for the rate of rise must not be reduced to less than 30 minutes. Reduction in releases from 320 Ml/d to 9 Ml/d takes not less than 40 hours. On completion of the two large freshets, the enhanced flows for the remainder of the month is a minimum of 45 Ml/d.

(b) Months: November and December

The discharge required to promote trout spawning in the upper reaches of the River Wolf, i.e. upstream of the Kellacott confluence, has been derived from an optimal spawning velocity (mid-range) for this species i.e. 0.30 m/sec. This velocity equates to a discharge of 45 Ml/d. Hence a minimum discharge of 45 Ml/d is released throughout this period.

Several artificial releases are made in November and December to stimulate adult salmon and trout to migrate into and throughout the River Wolf. All these freshets are timed to coincide with high flow and spate conditions prevailing in the adjacent rivers. Radio telemetry studies of salmon behaviour show that the majority of salmonids migrate on the receding limb of a spate. Continuous flow records from three primary gauging stations are used to ensure that the freshets are released to coincide with the appropriate phase of the spate.

Any releases follow the form of a natural hydrograph. The increase in discharge to the maximum release of 130 Ml/d from a base flow of 45 Ml/d occurs over a period of not less than one hour. The maximum duration of the peak is 15 hours. The maximum rate of increase is 0.75 m^3/s in any 30 minute period. The time increment for the rate of rise must not be reduced to less than 30 minutes. Reduction in releases from 130 Ml/d to 45 Ml/d takes approximately 7 hours.

(c) Months: January to March

This period coincides with the incubation and development stages of the eggs and alevins within the river gravels. Throughout these months a minimum discharge of 25 Ml/d is released. This flow maintains the wetted area of the stream and prevents stranding of the salmonid redds

above the water level.

(d) Months: April and May
This period coincides with the emergence of salmon and trout fry from the gravel to become free-swimming fish in the stream. These young fish establish territories and become distributed over the nursery grounds. A minimum discharge of 30 Ml/d is released over this period.

(e) Months: June and September
As in all months, the compensation flow of 9 Ml/d must be released at all times. This flow remains as the base flow during the summer period. Superimposed on this flow are regulation releases for supported abstractions at Gunnislake.
(Note: the River Wolf and the fish stocks benefit from the compensation flow of 9 Ml/d in dry and drought years. The compensation flow is three times the Q_{95} and represents total flow recovery in drought years when, historically, the river resorted to intra-gravel flows only).

(f) Months: mid-September to mid-October
Artificial releases are made to attract additional adult trout into the River Wolf. Freshets are synchronised with receding waters of natural spate events and targeted at the peak migration times for sea trout and brown trout.
The pattern of releases is the same as that described for November/December. The only exception being that the minimum discharge is 30 Ml/d as opposed to 45 Ml/d.

23. In addition to the enhanced flows, spare and "saved" spill waters can be released specifically for HEP generation in the period January to March. Such releases are dependent on reservoir storage. Maximum daily discharge levels of 100 Ml/d and 50 Ml/d have been set for the first and second six-week periods respectively. Such releases have minimal impact on the biota in the river. Control of the rising and receding limbs of the hydrograph are the same as described earlier. The majority of the releases are in steady block format, and avoid regular fluctuations in release volumes.

24. The maximum supply release to support abstraction at Gunnislake is 172 Ml/d. All releases must comply with the controls described and the water is released via the turbines. Based on predicted demands for supply purposes, voluntary restrictions to regulations releases can be identified and imposed in the April-October period of any year.

MONITORING AND RESULTS

25. The benefits of the enhanced flow programme to the wild salmonid stocks are monitored by annual electro fishing surveys. These surveys record the densities of juvenile salmon and trout stocks in the River Wolf and adjacent rivers. Since the implementation of the first programme, spawning

and recruitment of salmon in the River Wolf has improved, in particular more consistent density levels have been recorded each year. Comparison of average salmon fry densities reveal that the densities post-impoundment are double those densities recorded pre-impoundment.

CONCLUSIONS

26. The potential of the enhanced flow programme and environmental approach are recognised and the full benefits will be achieved over coming years. In time, an optimum balance will be attained between resources and fisheries production. Other direct benefits include continued commitment to the Public Inquiry undertakings, resolution of the mitigation programmes, optimization of the assets usage and improved financial returns.

27. Numerous operational aspects of the Roadford Scheme have been developed based on programmes of comprehensive investigations and predictions of environmentally acceptable flow conditions. Promotion and implementation of operating rules have been and will continue to prove a real challenge. Working in harmony with the environment and in conjunction with other catchment users ensures that the principle of sustainability forms an integral part of any operating strategy. South West Water promotes a responsible and holistic approach to the planning and management of water resources. As a result, major initiatives are taken to develop sustainable strategies which incorporate legislative and economic aspects, comprehensive environmental assessments/investigations and thorough consultation. Whilst many of the initial operating rules are still evolving, the underlying principles have sound environmental bases which set the foundations for all future commitments. The development of the Roadford strategy involves numerous positive and novel aspects that compliment and support the philosophy of sustainability, hence ensuring the needs of the present but without compromising future requirements and uses.

28. Finally, the Non Fossil Fuel Obligation generates one form of environmental benefit, but the instigation of the enhanced flow programme demonstrates other substantial environmental benefits. However, such benefits could be compromised by the withdrawal of the flat rate tariff in 1998. Potential conflicts between small hydropower generators and the regulatory bodies could ensue.

29. Consideration must be given to a guaranteed extension of NFFO to those installations which can prove real environmental benefits to the rivers. A more open and long term commitment to NFFO would compliment the development of sustainable water resources within a balanced environmental framework.

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Roadford Lake — leisure development

E. STACEY, Leisure Services Manager, South West Water plc

SYNOPSIS. The construction of a dam at Roadford in West Devon presented inland water-based opportunities of a scale previously unknown in the South West of England. There are increasing demands for recreational activities - understandably so, given the South West's position as the country's premier tourist area. Faced with the challenge of integrating operational, conservation and recreation interests, South West Water adopted an incremental approach to development to ensure a proper balance between competing demands on the site.

INTRODUCTION

1. South West Water Plc manages some of the most precious environmental resources in Devon, Cornwall and West Somerset. Across the region 34 reservoir sites covering 5,000 hectares of land and water provide outstanding conservation and recreation opportunities for residents and tourists. Its subsidiary company, South West Water Services Limited, has obligations under the Water Act 1989 Code of Practice on Conservation, Access and Recreation (ref. 1) prepared by the Department of the Environment to make adequate provision for public use of land and water in its ownership, as follows:-

- to further the conservation and enhancement of natural beauty and the conservation of flora, fauna and geological or physiographical features of special interests;
- to have regard to the desirability of protecting and conserving buildings, sites and other objects of archaeological, architectural, or historic interest;
- to have regard to the desirability of preserving for the public freedom of access to places of natural beauty, and to buildings, sites and other objects of archaeological, architectural or historic interest; and,
- to ensure that water and land associated with water which is under the control of the undertakers is made available for recreational purposes and in the best manner, and in so doing, that the needs of persons who are chronically sick or disabled are taken into account.

These duties have to be balanced with operational imperatives.

2. With almost 20 years' experience in integrated land management in sensitive locations, South West Water looked to fulfil its obligations by developing facilities at Roadford in an environmentally-sustainable way.

Nature Conservation Zone
Westweek
Restricted Use
Accommodation and Water Activities Zone
Self Catering and/or Serviced Accommodation
Access Point
Watersports Centre
Low Key Activity & Accommodation
Visitor Focal Point
Dam
Access Points

BACKGROUND

3. Three Public Inquiries were held before permission was finally granted in 1986 for construction of the Dam. The growing urgency for this resource led the former South West Water Authority to give undertakings that placed tight restrictions on the amount of future public activity. In the face of opposition to increased public use of the area, it was agreed that activity would be initially low-key and informal, with an incremental approach to development. Activities were to be introduced subject to demand, and subject to consultation with local communities and farming interests.

PREPARATORY WORK

4. Comprehensive environmental studies of the river system downstream of the proposed dam commenced in 1985. Detailed studies of the flora and fauna of the reservoir basin followed from 1986.

5. South West Water's landholding at Roadford amounts to 663 hectares of which 304 hectares would be above top water level. In addition to land required for operational purposes there were three main blocks which could be used for amenity purposes: 53 hectares to the south of the dam, 154 hectares to the east of the dam and 60 hectares to the west of the dam.

6. Landscape architects Colvin & Moggridge were engaged at an early stage. The primary objective was to restore the original pattern of landscape in which woodland and the bulk of hedgerows were in the valley bottoms and promontories were comparatively bare. Above all, the purpose was to ensure creation of a meeting between the landscape pattern above top water level and the water, which would look as if both had evolved together (as in Devon estuaries). Landscape plans ensured that many original features such as hedgebanks and mature trees would be retained with new hedgebanks giving the impression of long established landscape features.

7. Roadford had become a great attraction to bird life well before the valley was flooded. The secluded nature of the area with its rich fabric of fields, hedgerows and woods offered ideal conditions. The introduction of a vast expanse of water of varying depths was expected to provide a food source and developing habitat for a much wider range of bird species. The landscaping layout was designed to offer a changing habitat with screening to the most sensitive areas and minimisation of visual disturbance by walkers around the lake. It was acknowledged that in certain areas waterborne activity might not be permitted either permanently or on a time-zoned arrangement.

8. Initial studies of fauna within the reservoir basin and surrounding land led to projects to relocate dormice, otters, badgers, bats and barn owls.

9. Six farmhouses, several in excess of 200 years old, were left empty during the construction period and had been colonised by various species of wildlife. Measures were taken to ensure that protected species such as bats and owls would be undisturbed by any renovation work.

10. Flora studies highlighted many scarce species, particularly plants that only survive on unimproved grassland. The investigations resulted in a pioneering exercise to transplant areas of acid marshland from the land to be flooded to suitable locations above the projected shoreline.

11. Archaeological investigations concentrating on farmsteads to be flooded continued into 1990 until rising water prevented further work. A wealth of information was produced on what was described by the Devon County Archaeologist as an "unexceptional" valley - remarkable because of the lack of influence by the outside world over centuries. A detailed picture of the Wolf Valley emerged, reaching back 4,500 years.

12. Highlights of the archaeological project included:

- discovery of flint tools used by farmers 4,000 years before;
- discovery and excavation of the lost medieval village of Hennard Mill, with well-presented remains of cottages, mills and streets;
- discovery and recording of early industrial sites including a brickworks and the first cloth mill to be excavated in the South West.

CONSULTATION

13. A consultative forum - the Roadford Water and Land Use Consultative Group - was formed in 1987 as a vehicle for discussion and agreement on appropriate levels of public provision. This was believed to be the best way forward, in view of the undertakings given at Public Inquiry. Representatives from Devon County Council, two district and five parish councils together with English Nature, the Countryside Commission, statutory and voluntary sports and conservation bodies and those representing disabled people and other special interest groups met with South West Water managers and Board members. Initially meetings were frequent and progress was slow. Each representative (from some 30 organisations) had legitimate suggestions for future use but quite often conflicting interests emerged.

14. This consultation process was a necessary stage in the development of the Lake. It was clear that without local support, there could be insurmountable planning difficulties and resistance to change. Equally, only with the backing of conservation bodies could active recreations be introduced at a site with such obvious environmental significance.

15. As common aims developed within the Group, it became possible to devolve discussion on technical aspects to small sub-groups and in this way it was possible to reach consensus on every aspect.

AGREED DEVELOPMENT FRAMEWORK

16. By 1990, it became apparent that further progress would depend on the production of an agreed strategic document that could be adopted by the two local planning authorities and then translated into action.

17. The PIEDA Consultancy was engaged to produce this key document - the Development Framework (ref. 2) - drawing together the threads of the consultations and bringing their own considerable experience in leisure and tourism development.

18. The Development Framework was adopted by the Group and accepted by planning authorities after months of careful discussion, further consultation and a series of public presentations. The value of this document cannot be understated as it was to provide a reference point for planning committees as individual aspects of the development were progressed.

19. The Framework set the scene in terms of resources, planning policies, market potential and aims. It recommended division of the Lake into three broad zones. Whilst not denying the conservation value of the whole site, the northern reaches with their obvious value to wildlife were designated a Nature Conservation Zone. Two further zones were established - the Day Visitor Activity Zone and the Accommodation and Water Activities Zone.

20. The aim of the Framework was to:

safeguard the amenity of local residents through concentration of activity in the most accessible part of the site;

provide a simple and workable system of site, access and visitor management;

achieve optimal development of the reservoir which substantially retains its rural character;

enable phased development in accordance with planning requirements and market constraints;

accommodate a range of complementary yet distinct activities meeting the requirements of different markets, notably

- car-borne recreational trips
- water-based activity, eg fishing and watersports
- nature conservation-based trips
- country-based activity holidays

DEVELOPMENT OF THE ZONES

Nature Conservation Zone

21. Careful attention had already been paid to landscaping around the Lake with a view to recreating habitat lost as a result of the dam construction. Over 150,000 broadleaved trees were planted in a three year period from 1989 and three existing conifer plantations are now managed to favour broadleaved species.

22. The two northern arms of the lake were highlighted as being of environmental importance and particularly attractive to bird life, offering a haven to overwintering wildfowl and it was recommended they be given special conservation status.

23. The "Westweek Arm" became a total exclusion area, with access only for staff engaged on essential conservation work. The parallel, "Southweek Arm", bounded on both sides by public footpaths, needed careful management measures from an early stage to ensure that the majority of visitors to that area were dedicated walkers or bird watchers. As well as the bird interest at Roadford, the surrounding countryside has a wealth of life within it ranging from butterflies to badgers and deer. It was recommended that Nature trails be developed along the footpaths at some stage to give visitors the opportunity to observe the natural history.

24. Islands were created in the northern reaches of the Lake, providing refuges for nesting birds in drawdown conditions.

25. An early priority was the employment of an experienced countryside ranger to manage the site and in particular this zone with nature conservation as a priority. In view of the size and importance of the site, the ranger was required to live on site.

26. A conscious decision was made to provide only limited car parking in the Nature Conservation Zone and to give information designed not to encourage informal general public visits to this part of the Lake. During Dam construction, publicity had been given to a new viaduct built across the Southweek Arm. Once the water level rose, however, the existence of the viaduct was no longer promoted. As well as assisting in conservation, this also satisfied local residents who had objected to increased traffic in this area.

27. Two farmhouses with outbuildings and barns owned by South West Water situated within the Nature Conservation zone had been derelict for many years. The local community was fully supportive of the recommendation to develop high quality self-catering accommodation aimed at "green" holiday makers, anglers and birdwatchers.

28. The Nature Conservation Zone can be summarised as a quiet area, with little recreation and with a presumption against development.

Day Visitor Activity Zone

29. Using a wealth of data from other South West Water recreation sites in the region, it was possible to predict 250,000 - 300,000 annual visits to Roadford within a three year period. It was believed that over 90% of these visitors would be unlikely to take part in organised recreations but would have certain expectations from an informal visit to the countryside. In order to contain these general public visitors in an area of least environmental sensitivity, creation of a Day Visitor Activity Zone - ie a "honeypot" area - was an essential early requirement.

30. Forward planning was imperative, since the A30(T) realignment scheduled for the Autumn of 1993 would bring that trunk road within 1.5km of the Roadford Dam, giving easy access for the majority of residents of Devon and East and North Cornwall. It was decided that the most accessible area, ie the southeast corner of the Lake, adjacent to the Dam, was most appropriate.

31. Potential problems that had to be addressed in order to allay fears of the local community were

- increased disturbance
- traffic congestion and damage to verges
- indiscriminate access to land

32. The Framework therefore recommended early development of an area to which all informal visitors would be signposted. Within that zone, facilities could include:

- visitor car parks and viewpoints
- walks
- children's play facilities
- picnic facilities

- shop and tea room
- interpretation centre focusing on the natural and social history of the valley.

33. In 1990 a derelict cottage was rebuilt within this zone to house the resident ranger. At the same time public toilets and a 120 space car park were constructed at the Dam, with six further small satellite car parks strategically placed around the Lake. Interestingly, the size of the Dam car park was doubled to accommodate the wishes of the local community.

34. Surfaced waymarked footpaths (including one for wheelchairs) were constructed in 1991, together with a rustic children's play area and a gift shop and tea room. The latter incorporated artefacts obtained during archaeological investigations.

35. With each phase, local employment was created and this provided some consolation to local people who had been faced with the loss of land and the even greater challenge of having the largest expanse of water in the South West on their doorstep.

36. As an example, the shop and tea room was designed and built by local architects and builders and provides employment for eight local people on a full-time or part-time basis. The gifts sold are based on a natural history theme and are mostly crafted in the Westcountry. Refreshments are baked on the premises or sourced locally. The result is a focal point, popular with the local community and a venue for birthday, christening and even wedding parties.

37. Special events are now held for day visitors - 'Music over the Water', Conservation Days and charity fund raising events. In 1993 over 3,000 people attended one special event. A review of that event with members of the Roadford Water and Land Use Group illustrated local support for what had been a well-run day, with no inconvenience or detriment to the local community. Their fears of over-development and unnecessary intrusion have largely been allayed.

38. Planned future developments include an Interpretation Centre focusing upon the natural and social history of the valley, providing a multi-purpose room for meetings, education and community use including audio-visual facilities together with a tourist information point. It is hoped that the project can incorporate development of a reconstruction of Hennard Mill as the principal focal point (see paragraph 11).

Accommodation and Water Activity Zone

39. As might have been anticipated, development of activities and accommodation generated the most debate and was the most difficult area in which to achieve majority agreement. However, a satisfactory formula was reached and a range of quiet activities agreed.

40. The Lake was stocked with 150,000 fingerling brown trout from 1991 and opened as a wild brown trout bank fishery in 1993, to wide acclaim. The 'natural' fishing is likened to that experienced on Scottish lochs. In resisting pressure to develop a standard reservoir rainbow fishery, we now have a unique product that has attracted a completely new market.

41. Fishing permits, weighing-in facilities, clubroom, refreshments and changing rooms/toilets have been developed on the west side of the Lake, across the water from the Day Visitor zone. The facilities have been developed to service both anglers and watersports enthusiasts, around a set of farm buildings - Lower Goodacre - renovated with the assistance of grant aid from the Rural Development Commission. The Commission were particularly interested in this project because of its impact on tourism-related businesses in the area and the direct employment opportunities it created. This was the first occasion on which the Rural Development Commission had grant-aided a Plc.

42. South West Water has its own training centres recognised by governing bodies of sport such as the Royal Yachting Association, teaching activities such as dinghy sailing, windsurfing and canoeing. At Roadford it was agreed that such a Centre would be set up for quiet watersports. Powerboating or waterskiing were not favoured both for operational reasons and noise pollution, and because of potential disturbance. The Centre opened on 1 October 1993 and will offer hire and launch facilities for day visitors and club members as well as tuition. Competitive rowing and sub aqua diving may also be permitted, subject to demand.

43. A Code of Conduct was an early essential element to ensure that all users of the water respect fellow users, wildlife and operational interests. This Code is issued to all water users and is designed to anticipate and prevent potential conflicts.

44. The integrated facility for anglers and sailors was another initiative arising out of consultation and aims to foster good relationships between all water users.

45. Pressure continues to grow for accommodation of other sports and fringe activities such as waterskiing and cycling. These are not considered appropriate in the case of the former, or financially viable in the case of the latter at the present time. However, should demand increase, the mechanism now exists for resolution through the Water and Land Use Consultative Group.

46. Alongside water-based activities the Development Framework highlighted the opportunity for provision of accommodation in the vicinity adjacent to the angling and watersports centre. Higher Goodacre Farm offers an accessible, yet relatively secluded location capable of providing serviced accommodation in the form of a country house hotel of say 50 to 100 rooms and/or a self catering complex of 150 to 200 units.

47. This proposal was recognised locally as being of substantial benefit in offering employment opportunities but met with some resistance initially as possibly the "thin end of the wedge". In the current climate the development has not been progressed but it is anticipated that a great deal of discussion and consultation will be necessary when the project is taken forward.

SUMMARY

48. The constraints imposed by Public Inquiry undertakings relating to public access appeared at the outset to be daunting, but in fact provided an unprecedented opportunity.

49. The necessity to keep activities initially low key and informal enabled South West Water to adopt a sensitive incremental approach to development. Over a relatively long timescale it was possible to carry out detailed consultations with all interested parties and market research with potential customers. In this way feedback was obtained at every stage and any problems ironed out before development took place.

50. In introducing facilities and activities gradually, proper consideration was afforded to landscape and wildlife considerations, and the natural and cultural history of the valley have been recorded in detail.

51. Local communities have supported introduction of high quality facilities which bring further regeneration to an area that has suffered economically with the decline in agriculture.

52. Roadford Lake is now a haven for wildlife and a regional tourist resource, and will give local small business operators an opportunity to gain added value from its success.

53. Future developments will be in line with an agreed Development Framework.

54. South West Water believes that its sensitive approach to development ensures a proper balance between competing demands on this site. The temptation to over-develop has been avoided but a varied range of facilities guarantees Roadford's rightful place as one of the West of England's premier visitor attractions.

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Reservoir, environment and private sector

A.G. YENER, BS, Civil Engineer, and H.S. YENER, BS, MSc, DIC, PGDIP, Civil Geotechnical Engineer, Sir Alexander Gibb and Partners Ltd

SYNOPSIS. "Reservoir Environmental Engineering" is an increasingly important interdisciplinary art and science. Prevailing contribution of the private sector to this field should meet the challenging targets on both research and development fronts. Various aspects of this subject is studied considering the case studies of Turkey and her South-Eastern Anatolian Project - GAP.

INTRODUCTION

1. Today's world is experiencing unprecedented water shortages due to the uneven distribution of available water in time. Reservoirs are the obvious solution to this problem, especially when the total amount of water is sufficient to cover the needs of the dry periods. Even if it is not, reservoirs will be capable of mitigating the adverse effects of the drought. In fact, human kind have not been able to find any reasonable alternative to reservoirs so far. Consequently it is now the engineers task to make the most out of the reservoirs while minimizing the problems associated with them. This is called "Reservoir Engineering" which is an interdisciplinary field formed by tens of different branches of science.

2. Although in many of these sciences a wealth of experience and qualified personnel has been available for decades, few of them can be considered to be new. The remarkable example for these new sciences is environmental engineering and there is a significant lack of research and experience in "Environmental Reservoir Engineering". The scientific and practical work in the field is going to make its invaluable contribution when solving the environmental problems which may be exemplified as: Relocation of people, changing ice regime, water-related diseases, water quality deterioration, reservoir induced seismicity, reservoir safety, flora and fauna destruction, inundation of historic monuments, unfavourable climate changes, adverse effects on regional microbiology etc..

GAP

3. To apply some environmental reservoir engineering principles South-Eastern Anatolian Project of Turkey (GAP) is going to be examined. GAP is one of the world's biggest regional development projects with 22 major reservoirs which will be fed by Fırat (Euphrates) and Dicle (Tigris) rivers draining an area of 70000 km² and discharging 50 km³ of water annually on the average. When completed, energy output of the 19 hydropower plants will equal

7.5 GW and 17000 km² land will be irrigated. Project incorporates some of the world's largest man-made reservoirs as well as dams with largest embankment volumes and two of the worlds longest irrigation tunnels. It is calculated that the annual revenue only from irrigation and energy production will be well in excess of £2 billion. Currently, roughly a third of the project is completed, another third is under construction and the rest awaits finance.

Fig. 1. Turkey and GAP

4. Considering that the required capital investment will be around £20 billion and the project life on which the economical analyses are based is 50 years, project can be qualified as highly profitable (DSİ, 1993). Additionally, GAP will have a tremendous impact on two crucial issues:

1- GAP will make Turkey the determinative power on water-sharing in the region - Middle East.
2- Economic conditions in Eastern Turkey will be improved mitigating secessionist pressures.

5. Consequently there is every reason to believe that before the end of the millennium all of the major components of GAP will be up and running and by 2010 all the details of this colossal project will have been completed. In the context of this paper, using GAP as a case study two main areas in environmental reservoir engineering will be discussed:

1- Research and experience needs
2- Role of the private sector

ENVIRONMENTAL IMPACT ASSESSMENT IN TURKEY

6. In all major human activities environment is affected. If such effects are adverse they will result in "environmental deterioration"; on the other hand if they are favourable the result will be "environmental development". The general

trend until the eighties was noticing the environmental catastrophes after they happened. The fact that the damage to the environment cannot be easily incorporated in the financial analyses, made it extremely difficult to take such factors into account during the planning stage, especially in the absence of well-established environmental techniques. However today both the environmental science and the computers allow us both to plan, monitor and remedy the environmental problems reasonably accurately. Such a capability will certainly find extensive application not only in GAP but also in all of the water resources projects in the Middle East, where an environmental mistake is likely to cause disastrous consequences on already limited water and arable land resources of the region (Canter & Vieux, 1993).

Fig. 2. Fırat (Euphrates), Dicle(Tigris) and Middle East

7. Unfortunately Turkey does not have the experience in various areas of environmental reservoir engineering to fully address the burning issues in GAP. A complete environmental monitoring system, a reliable inventory of the environmental assets and last but not least a proven track record in environmental impact assessment are the few examples of the capabilities, yet to be developed in Turkey.

PUBLIC SECTOR VERSUS PRIVATE SECTOR

8. State Water Works, Ministry of Forestry, Ministry of Health, Ministry of Agriculture, Nuclear Energy Agency, Turkish Scientific and Technical Research Agency, universities, municipalities and many other public sector entities are working on various aspects of environmental science. However except for State Water Works and the universities they generally concentrate on their individual specialities. Furthermore almost all of them suffer from common problems like experienced personnel at all levels, lack of coordination, inadequate legislation and finance.

9. If the complete and generally expensive environmental measures are taken before an activity, usually the best result will be conserving what is already there. So although significant money from the sponsor's purse flows into the environmental pot, the results will be hardly visible. In fact this invisibility in itself is an environmental success but considering the often long-term environmental impacts of many projects, it is politically more profitable to allocate the resources to other revenue-bringing investments instead of environmental protection.

10. This dilemma can only be overcome by tough legislative measures against all political and financial temptations. After all the state's first and foremost obligation is professionally designing applicable laws based on reasonable consensus and then reinforcing them without compromise. In other words the state is there to set the rules and set them the same for everyone, supervise whether they are observed and intervene whenever and wherever they are not. However the state should certainly not have an obligation to undertake derelict land environmental audits, feasibility studies etc..

GAP AND THE ENVIRONMENT

11. At the present South-Eastern Turkey is a typical example of a dry climate. Average annual rainfall is about 400 mm, summer months being almost totally dry. Mean annual temperature is 18 °C rising up to 50 °C during hot seasons. Construction of 22 reservoirs with very large surface areas will create the major impact on the climate. Average humidity of currently around 40% will dramatically increase and this will lead to major changes in the flora and fauna. The groundwater table which has been severely lowered due to over-exploitation will be automatically recharged to much higher levels causing substantial changes to underground biosphere.

12. This part of the world is the genetic origin of many species like grasshopper types orthoptera, gampsocleis acutipennis, phytodrymadusa expugnata, phytodrymadusa haccarica etc.. Many of the mammal species have already been seriously endangered because of the activities of the human kind

(Şişli, 1990).

13. The flora side of the story is even more interesting. In spite of the significant droughts in the region, 35% of Turkey's plant species grow only there. Almost half of the daisy and rose types in Turkey for example, can only be found in the region. There is an extensive accumulation of knowledge on herbal medicine which is now under the threat of extinction because of rapid urbanization.

TURKEY AND PRIVATE SECTOR

14. The highly fruitful economical transformation which Turkey has been undergoing over the last ten years resulted in a boom both in the infrastructure and in the superstructure of the country. Environmental policies followed by various governments in this period were remarkable in that, the actions taken by these governments have been far more radical and comprehensive than the already very high public expectations. The last two links in the chain has just been unveiled by the prime minister herself who came to power only in June 1993: The new taxation-expenditure system to reduce significantly the extent of the state's involvement and the colossal £20 billion privatisation programme. It is debatable how long all this will take to totally settle, however it is certain that Turkey is determined to have a free market economy and has all the intentions to be an integral part of the Western capital markets.

TURKEY AND INTERNATIONAL CONSULTANTS

15. For many decades Turkey has been enjoying the benefits of employing international engineering consultants in a variety of projects, many of whom are British. Together with the booming infrastructure investments and the country's credibility in the international financial markets, Turkey has become a sought-after client for major consultancy companies. As a result of the collaboration between the public sector, the private sector and the local engineering firms of Turkey and the international practices, enormous benefits for all parties are likely to follow. Perhaps more importantly, all these parties are now aware of this fact which certainly was not the case in the past. Finally, even more encouraging trend among many multinational companies of different areas is, opening to the Central Asian markets from their Turkish bases.

16. Neither the reservoir engineering nor the environmental reservoir engineering are the exceptions to these basic facts about Turkey...

INTERNATIONAL INPUT AND RESEARCH NEEDS

17. The first and foremost need is a comprehensive inventory of all flora, fauna, water and soil resources and reserves in the region. This will require extensive input from many disciplines ranging from civil engineering to biology. The end product of such a study will be a well-established data-base which will allow researchers and engineers to gain access to all information they may practically need.

18. Secondly, an advanced monitoring system is required. This should be carried out on as many parameters as possible, from the pollution parameters to sociological changes in the region. The results of the monitoring exercises

will form an integral part of the data-base for future references and in the determination of various statistical relations.

19. Finally a comprehensive mathematical model which can then be applied on a computer is very much needed. Such a model will run not only using the physical and analytical rules of sciences but also it should be capable of utilizing the data accumulated in the above-mentioned data-base. Another important feature of the model may be incorporating an environmental expert system.

20. For the authors, the ultimate mathematical model for the environment must be "self calibrating" based on the inflow of new data from monitoring.

CONCLUSION

21. There are general development needs in the various fields of environmental engineering on both theoretical and practical fronts. Moreover there are country-specific and even project- specific development needs. Many of these are readily observable in Turkey and in GAP. The international engineering consultancies have the important responsibility of improving the state-of-the-art and they are expected to come up with increasingly simpler and cheaper alternatives. This fact becomes even more significant with the decreasing role of the state in the economy towards the end of the millennium.

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Concrete dams, stability and the role of internal drainage. Tai Tam Tuk — a case study

D. GALLACHER, MSc, DIC, CEng, FICE, FIWEM, MConsE, Senior Partner, EurIng D. MORRIS, MSc, CEng, FICE, MASCE, FGS, Consultant, R.J. MANN, BA, MICE, MIWEM, Senior Engineer, Robert H Cuthbertson & Partners, and K.L. CHAN, BSc, MSc, CEng, MICE, MHKIE, Engineer, Hong Kong Water Supplies Department

Synopsis

The stability of Tai Tam Tuk Dam, a mass concrete gravity dam in Hong Kong constructed in 1917, was found to be inadequate under extreme loadings using contemporary design standards. A study of pore pressures within the dam and foundations, followed by installation of trial drainage within the dam body and foundations to reduce pore pressures, are described. Results indicate that significant reduction in pore pressures and uplift, with a corresponding improvement in stability, can be achieved by internal drainage.

Introduction

1. Tai Tam Tuk Dam is a masonry faced mass concrete gravity structure, some 49 m high and 364 m long. It was constructed in 1917 in the eastern part of Hong Kong Island as part of the Water Supply Department's facilities.

2. During early Independent Inspections, the stability of the structure under extreme loadings was questioned and a number of proposals for remedial measures, some of which were implemented, were put forward. As part of the present Fourth Independent Reservoir Inspection Programme, piezometers were installed in the dam body and the foundations to measure pore pressures. The stability of the dam was analysed and was found to be less than desirable under extreme conditions, and a trial drainage scheme prepared and undertaken to relieve the unacceptably high values measured. The seismic hazard of the site examined and revised values for Peak Ground Motions were derived. A pseudo-dynamic analysis considering the response between the dam, foundation and reservoir, using the new derived seismic parameters, was carried out as part of the study.

3. This paper describes the results of the measurements and the various studies and trial drainage scheme.

Background

4. The dam and foundations are described in detail in the paper by Jaffé (1918-19)[1] who was the Engineer for the construction. Figures 1 and 2 are based on that paper and show the dam as a monolithic concrete structure with no vertical movement joints, and containing a rich zone towards the upstream face.

5. Prior to 1974, no significant problems related to the dam were recorded, although in the First Independent Inspection of that year (IIR1), the stability of the structure under extreme loads was questioned and improvement works using post-tensioning were studied and recommended, but were not carried out.

Figure 1 Downstream elevation of Tai Tam Tuk Dam

Figure 2 Cross-section of dam showing different zones of concrete

6. Two vertical boreholes were drilled through the dam in 1977, in conjunction with Binnie & Partners' 1978[2] Report on Post-Tensioning. From these, the foundation is described as "fresh to slightly weathered volcanic rock consisting of welded tuffs". Further boreholes were drilled in the dam and foundation Left of the Valve tower in 1988 by Bachy Soletanche Group[3], for Site Investigations in connection with trial drainage. Logs indicated extensive zones of honeycombing in the dam hearting.

7. In 1980, under IIR2, recommendations to install siphons and lower the overflow level were made and the work executed. At about the same time, the question of uplift pressures within the dam and foundation was considered and a figure of some 66% of the full uplift pressure was adopted as representing reality. On the basis of the lower water level and the reduced uplift, stability was considered to be satisfactory.

8. In 1987, under IIR3, stability was again assessed and was found to be marginal under extreme loading conditions, with low Factors of Safety and high tensile stresses at the upstream face. In 1990, multi-point piezometers were installed within the body of the dam and foundations, to monitor the actual situation and test the

feasibility of reducing uplift by drainage and a trial filling of the reservoir was carried out.

9. In 1993 a trial drainage scheme was undertaken from the outlet culvert and showed that a significant reduction in pore pressures and uplift could be achieved. The results from the monitoring programme were incorporated into stability analyses of the dam and showed that an improvement in stability would be obtained by the use of reasonably efficient drainage. This paper describes the analyses carried out along with the course of action proposed to improve the stability of the whole dam to accepted modern levels.

10. Four sections of the dam were studied, and the results of the two highest sections, the Highest Outlet Section and the Highest Overflow Section are given below.

Multi-Point Piezometers

11. In 1988, three multi-point piezometers each with nine individual sensors were installed adjacent to the outlet section of the dam. During the drilling, cores were obtained from the boreholes and showed areas of honeycombing distributed throughout the full height of the dam tending to be more frequent towards the top of the structure. Typically, the areas of honeycombing were between 0.1 and 0.3 m thick with a maximum thickness of 0.6 m. The piezometers were arranged to include relatively severely honeycombed and relatively honeycomb-free zones.

12. The results showed a good correlation with fluctuations in reservoir water level, and response was rapid -generally, within a 24-hour timescale. The response of selected piezometers is shown in Figure 3. The consistent nature of the various plots allowed extrapolation to probable piezometric values at Top Water Level. The plots from each piezometer array showed a consistent pattern, giving confidence that a representative situation had been measured. The piezometer readings did not appear to be affected by the quality of the concrete, with the sensors in both the honeycombed and un-honeycombed areas responding in a generally similar manner and speed.

Figure 3 Selected piezometer readings before and after drainage

13. Based on the results, the phreatic surface within each cross-section was determined showing definite non-linear distribution of pore pressure across the section. The results of selected piezometers are shown in Figure 3 along with the appropriate reservoir level.

Trial Drainage

14. In 1993, a trial drainage scheme, which did not require vertical drilling from the public road across the dam crest (which would have presented difficulties in terms of access and connecting drainage at lower levels) was developed and implemented. Angled holes were drilled into the Highest Abutment section from the Left outlet culvert, draining directly into it. The arrangement of drains is shown in Figure 4.

Figure 4 **Downstream elevation showing trial drainage**

15. An assessment of the results to date indicates that the distribution of pore pressure across the section, as indicated by the piezometers, and defined by the sketch below, has been reduced to some 25% of its pre-drainage value.

REDUCTION IN UPLIFT BY INTERNAL DRAINAGE

16. In the analyses of the outlet section this reduction has been retained, while in the overflow section, a reduction in uplift of 30% at the intermediate point, has been assumed, to allow for any reduced efficiency in the different layout of drains necessary in this section to take account of the constraints of the location.

Seismic Hazard

17. The tectonic setting of Hong Kong was investigated along with the historical seismicity of the region based on published data. This led to the conclusion that it was possible that earthquakes of significant magnitude, up to Magnitude 6, could occur close to the Territory. Using the Cornell Method, the Peak Ground Acceleration (PGA) was derived for both the 50 and 84 percentile values of the Operating Basis Earthquake (OBE) and Safety Evaluation Earthquake (SEE) events.

18. A probabilistic approach in which design values for the peak ground motion parameters are derived on the basis of accepted risk, ie., the probability of exceedance of some value in the lifetime of the structure was used. In the case of the Safety Evaluation Earthquake the 84 percentile value of the PGA, approximating to the mean plus one Standard Deviation, represents a reasonable maximum value for this parameter allowing for normal errors.

19. In line with other projects in seismically active areas worldwide, probabilities of exceedance of 0.1 or 0.2 in the case of the Safety Evaluation Event ie., no catastrophic failure, and 0.4 in the case of the operating basis event, ie., no damage or loss of function, were used to establish the Peak Ground Acceleration values for design along with the Elastic Response Spectra. Return Periods of 1000 years were considered acceptable and appropriate for the SEE events in the light of the length of accurate record available without excessive extrapolation to longer periods. The results are given below:

Case	Probability of Exceedance (q)	Return Period (years)	50 percentile values acceleration (cm/s^2)	50 percentile values velocity (cm/s)	84 percentile values acceleration (cm/s^2)	84 percentile values velocity (cm/s)
OBE	0.4	200	59	3.97	98	6.60
SEE	0.2	500	78	5.25	147	9.90
SEE	0.1	1000	98	6.60	167	11.22

20. For the purposes of the analyses, the seismic acceleration used was that for the SEE with a 10% probability of exceedence in the 100 year lifetime of the structure.

Stability Analyses

Static Analyses

21. A static analysis was carried out using a simplified two-dimensional, beam analogy model, chosen because of its simplicity and almost universal application to gravity dams, with computation of all forces acting on, or within, a unit width section of the dam. Loads and moments were assessed at various selected elevations, and classified as stabilising or de-stabilising in accordance with the following scheme:

Stabilising	Destabilising
Weight of dam body	Water pressure horizontally against upstream face
Weight of water acting vertically on slope of upstream face	Uplift within dam body and foundation contact zone
Tail water pressure	At-rest Pressure of backfill horizontally against upstream face
Surcharge from bridge &c on top of dam	EARTHQUAKE loadings
Weight of tail water on slope of downstream face	
Weight of backfill acting vertically on slope of upstream and downstream faces	
At-rest Pressure of backfill horizontally against downstream face	
(For simplicity, the weight of overflowing water not taken into account)	

22. Factors of Safety against overturning, and the line of thrust, were determined by static computation, and stresses normal or parallel to horizontal planes at selected elevations, were assessed, assuming the dam to act as a vertical elastic beam of varying thickness.

Pseudo Dynamic Analyses

23. The Pseudo Dynamic Method of Analyses developed by Fenves & Chopra (1986)[4] was used. The method assesses equivalent lateral forces caused by the dynamic response of the dam-foundation-water system to earthquake excitation, setting up a dynamic response in the dam, foundation, and the impounded water. The dynamic response comprises vibrations of the dam in the Fundamental Mode (corresponding to the natural frequency of the dam) and in an aggregate of higher frequency modes.

24. The dynamic response is expressed as equivalent lateral forces comprising both a hydro-dynamic component (water shock) and a component resulting from peak dynamic deflection of the dam and foundation. The effect of the impounded water, reservoir bed and backfill, in providing a degree of damping, is allowed for.

25. The model is fundamentally different from the static-based model used previously, in that the effect of dynamic elastic response of the overall system is taken into account. The model is described as "Simplified" in that it is only a two-dimensional assessment, and a number of factors are simplified in the procedure. The

dynamic analysis provides a more realistic representation of the dam behaviour under seismic loading than the static analysis used previously.

26. In the static-based model, ground acceleration was expressed as a fixed value regardless of the dynamic response of the dam. In the pseudo-dynamic model, the ground acceleration used is a function of the dynamic properties (fundamental frequency) of the dam.

Sections and Load Cases Analysed

27. Two of the four cross-sections of the dam analysed are presented viz:

1 Highest outlet section, i.e. adjacent to the outlet culverts.

2 Highest overflow section.

28. Four loading conditions were considered for each of the sections:

(a) Reservoir full - existing pore pressures- (undrained) no earthquake

(b) Reservoir full - existing pore pressures- (undrained) earthquake loadings, (0.06, 0.1 and 0.17g)

(c) Reservoir full - reduced pore pressures- (drained) no earthquake

(d) Reservoir full - reduced pore pressures- (drained) earthquake loadings (0.06, 0.1 and 0.17g)

29. The combination of extreme earthquake and PMF flood is considered too remote a possibility to warrant consideration.

Results of Stability Analyses

30. Typical results of the analyses for the highest cross-sections and the loading conditions described above are summarised in Figure 5 and Tables X and Y. Factors of Safety (Overturning) at the base of the dam are plotted against Peak Ground Acceleration (PGA) and return period of earthquake (50 percentile and 84 percentile) in Figure 5 for the four cross-sections of the dam analysed. The corresponding Maximum Tensile Stresses at the four cross-sections are given in Figure 6.

General

31. The Factor of Safety against Overturning decreases with Peak Ground Acceleration (Figure 5). Similarly, the Maximum Tensile Stress increase with Peak Ground Acceleration (Figure 6).

32. The Factor of Safety against overturning is significantly increased by drainage for all load cases (Figure 5), and the principal tensile stress at the upstream face of the dam is decreased (Figure 6).

33. The results of the stability analyses for the four cross-sections of the dam have been summarised to show the Factor of Safety against overturning and the tensile stress at the upstream face for the three principal loading conditions, i.e. Reservoir Full, Reservoir Full plus OBE Earthquake, and Reservoir Full plus SEE Earthquake. For each earthquake loading condition both the 84 and 50 percentile values were considered.

Figure 5 Results of stability calculations for Highest Outlet and Highest Overflow sections, before and after drainage

Figure 6 Factor of Safety (Overturning) and Maximum tensile stress plotted against earthquake PGA, before and after drainage

34. Criteria for acceptance, in terms of stability, took account of both the Factor of Safety and Tensile Stress for all loading conditions. The required minimum Factors of Safety considered were as follows:

Reservoir Full (a and c above):	1.50
Reservoir Full plus OBE Earthquake:	1.25
Reservoir Full plus SEE Earthquake:	1.00

35. Tensile stress should be considered in conjunction with other factors, particularly the Factor of Safety against overturning and the extent of the section subject to tensile stress. The acceptable maximum tensile stress was determined from data obtained from tests on core samples from the dam and from a consideration of the brief duration of the seismic loading.

Highest Outlet Section

36. The stability results for the Highest Outlet Section for the undrained and drained analyses, are given in Table X. The undrained analysis adopts pore pressures measured by the multi-point piezometer system local to the outlet works. The drained analysis uses pore pressures based on the preliminary results obtained from the trial drainage scheme.

HIGHEST OUTLET SECTION

LOAD CASE	Res Full, Static		Full, O.B.E 84%ile (50%ile)		Full, S.E.E 84%ile (50%ile)		COMMENT
	FoS	Tension	FoS	Tension	FoS	Tension	
IIR4 Undrained	*1.27*	*0.13*	1.01 *(1.10)*	0.52 *(0.36)*	0.87 *(1.01)*	0.83 *(0.52)*	DRAINAGE REQUIRED
IIR4 Drained	1.66	0.01	1.24 (1.38)	0.44 (0.26)	1.03 (1.24)	0.78 (0.44)	SATISFACTORY

Notes: Shaded areas denote unacceptable values; italics indicate borderline values.
Values of Factor of Safety stated are at the base of the dam

Highest Overflow Section

37. The stability results are given in Table Y. The undrained analysis adopts pore pressures based on those measured by the multi-point piezometer system local to the outlet works. The drained analysis uses pore pressures assumed to be obtainable by drainage on the basis of the preliminary results obtained from the trial drainage scheme.

HIGHEST OVERFLOW SECTION

LOAD CASE	Res Full, Static		Full, O.B.E 84%ile (50%ile)		Full, S.E.E 84%ile (50%ile)		COMMENT
	FoS	Tension	FoS	Tension	FoS	Tension	
IIR4 Undrained	*1.22*	*0.16*	1.02 *(1.10)*	0.54 *(0.38)*	0.91 *(1.02)*	0.83 *(0.54)*	DRAINAGE REQUIRED
IIR4 Drained -	1.51	0.03	1.22 (1.33)	0.46 (0.28)	1.06 (1.22)	0.76 (0.46)	SATISFACTORY

Notes: Shaded areas denote unacceptable values; italics indicates borderline values.
Values of Factor of Safety stated are at the base of the dam

Conclusions

38. Very significant improvement in stability is achieved by drainage as shown by the IIR4 (drained) analyses for all loading conditions. The estimated tensile stress at the upstream face under the Reservoir Full condition is 0.13 N/mm^2 which is acceptable as it occurs in the area of good quality masonry facing and the zone of richer concrete. Tensile stresses at the upstream face are increased under earthquake loadings up to estimated maximum values of 0.52 and 0.83 N/mm^2 for the OBE and SEE respectively. These relatively high tensile values are acceptable as transient stresses in conjunction with the corresponding satisfactory Factors of Safety against overturning.

39. As for the highest Outlet Section, very significant improvement in stability is achieved by drainage for all loading conditions. The estimated tensile stress at the upstream face under the Reservoir Full condition is 0.15 N/mm^2 which is acceptable. Tensile stresses at the upstream face are increased under earthquake loadings up to estimated maximum values of 0.54 and 0.83 N/mm^2 for the OBE and SEE respectively and, as abnormal transient stresses in conjunction with the corresponding satisfactory Factors of Safety against overturning, are acceptable.

40. The above results for the IIR4 (drained) case show that an acceptable solution in terms of stability can be achieved in this section of the dam by drainage on the basis of the preliminary results from the trial drainage scheme at the outlet section, if there are similar pore pressures in the overflow section of the dam as there were at the outlet section.

41. Three Stages are proposed for the Drainage Stabilization Works:

- Stage 1 The installation of piezometers within the dam and the foundation to measure pore pressures in relation to reservoir water level in advance of drainage works.

  This will also allow the effects of drainage to be monitored and the scope of the drainage works to be modified to provide an efficient and economic solution.
- Stage 2 The drilling of primary drainage holes in the body of the dam and its foundation, and the drilling of shallow drainage holes through the masonry facing on the lower part of the downstream face of the dam to relieve any pore pressures.
- Stage 3 The drilling of secondary drainage holes in the body of the dam and its foundation to provide further relief of pore pressures where adequate reduction is not achieved in Stage 2.

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Val de la Mare Dam, Jersey: Instrumentation, monitoring and stability analysis of an ASR damaged dam

P. HORSWILL, MA, MSc, DIC, CGeol, FGS, Associate and Principal Geologist, Montgomery Watson, H.N. SNOWDEN, BSc(Eng), CEng, MIWEM, Senior Engineer, Jersey New Waterworks Company Ltd, and R.C. WEEKS, DLC, CEng, MICE, Geotechnical Instruments (UK) Ltd

SYNOPSIS. Val de la Mare dam is a mass concrete gravity structure constructed in 1957-62. Alkali-silica reactivity (ASR) was diagnosed in 1971 with damage limited to specific but numerous lifts. Internal uplift pressures, measured with electric piezometers in the most damaged block were sufficiently high to warrant the installation of post-stressing tendons to restore factors of safety. Extensive piezometer instrumentation has, since 1981, monitored internal pressures elsewhere in the dam, but suffered from corrosion. Recently a more reliable stainless-steel retrievable electric piezometer system has been installed. Stability analyses using the recent uplift data have given satisfactory results.

BRIEF HISTORY OF DAM

1. Val de la Mare dam is a mass concrete gravity dam, 30m high, constructed between 1957 and 1962. Unusual behaviour in 1971 led to the diagnosis of the first major case of alkali-silica reactivity (ASR) in the British Isles (Coombes, Cole & Clarke, 1975). Opaline lenses in the coarse aggregate supply quarry along with some deliveries of high alkali cement were shown to be the cause, limiting damage to specific, but numerous, lifts as shown in Fig. 1.

2. Construction took place using direct labour, with coarse aggregate from a local quarry and from beach gravel, fine beach aggregate, and cement imported from the UK. The dam was constructed in twenty-six 6.7m wide monoliths (blocks) in lifts approximately 1.22m high as shown in Fig. 2. The upstream face is vertical while the downstream face slopes at 1 on 0.7. The dam is straight in profile and stands a maximum of 23m above, the lowest ground level.

ASR studies

3. In January 1971 it was noticed that the hand-rails on the bridge were no longer aligned, blocks 6, 8, 10 and 12 being displaced upstream at the crest by a maximum of about 12mm. The downstream face of the dam showed damp patches with random hairline cracking. Concrete cores were taken from blocks 5 and 6 and analysed for residual alkali and expansivity. It was shown that the majority of problems encountered with alkali-silica reactivity were in the

concrete that was placed in the critical three-month period of June to August 1960 (Sherwood, Marriott & Smith, 1975; Cole & Horswill, 1988).

4. The cement works supplying during the critical period were situated south of the Thames and subsequently closed. Emission controls had led to an increased cement alkali content. A detailed geological survey of the quarry found seams of chalcedony and associated opaline silica and an examination of air survey records confirmed the quarry face included these during the critical period.

Fig. 1 Downstream elevation of dam (walkway omitted)

Sonic velocity testing

5. Sonic testing of the structure started in 1972 (Sherwood et al, 1975) and has been repeated periodically. A summary is given in Table 1.

Table 1. Sonic velocity measurements, 1972-93

Category	Year						
	1972	1973	1976	1979	1983	1988	1993
	m/millisecond						
Average sonic velocity for all 124 lifts surveyed	4.47	4.46	4.57	4.35	4.48	4.54	4.61
Average sonic velocity for 47 lifts cast June to August 1960	4.32	4.31	4.43	4.13	4.31	4.12	4.47
Average sonic velocity for remaining 77 lifts	4.56	4.57	4.66	4.48	4.58	4.65	4.69

Fig. 3 shows a time plot for lifts 2, 3 and 5 in block 6 (the worst affected block) compared with unaffected concrete.

Remedial works and monitoring

6. Consultations were held with international experts and Montgomery Watson reported on remedial works. Monitoring, drainage and anchoring of block 6 by three "Macalloy" high tensile anchors of about 850kN each were carried out (Coombes et al, 1975). By 1977, significant drain flows were occurring in blocks 10 and 12, with deterioration of blocks 7 and 8 and increased surface cracking in block 6. In May 1977 it was found that anchor No 3 had suffered brittle fracture, perhaps due to a minor seismic event (Cole & Horswill, 1988).

Fig. 2 Typical section of central part of dam (blocks 12 – 14)

7. By 1980, drain flows remained locally high and some remedial underwater sealing of joints was successfully carried out. In 1981, forty-four vibrating wire piezometers were installed in cored diamond drilled holes and a seismometer was set up nearby (Thébault & Walker, 1986). In 1982 the loads in the remaining two tendons were relaxed from 870kN to 830kN, the increasing loading being attributable to ASR expansion in the intervening seven years (Cole & Horswill, 1988).

8. By 1984, a nearly stable ASR condition was evident and dam stability was adequate even with a seismic horizontal acceleration of 0.12g. The crest to gallery drains were cleared of build-ups of calcareous deposits in 1985 and 1986.

Corrosion problems with original instruments

9. Early readings from the vibrating wire piezometers in 1981-82 were judged to be unreliable because of calibration and creep problems. After discussions with the instrument manufacturer and the supplier, all the piezometers were removed from the dam in February 1983 to check their calibration constants and to try to eliminate creep effects by means of cyclic loading. The piezometers were subjected to pressure load cycling on site, the number of load cycles varying between 400 and 1000 to obtain stability of the readings. They were then carefully reinstalled in a weak grout, the opportunity being taken to insert a small twin-tube hydraulic piezometer alongside each one for checking purposes should the need arise.

Fig. 3 Sonic velocities in concrete versus time

10. With few exceptions, the re-installed piezometers appeared at first to give reliable readings but after about three years it was apparent that more than a third of the instruments were either behaving unexpectedly, becoming difficult to read or drifting to give eventually improbably low readings, indicating for example high negative pressures at the tip. Validation checks carried out in October 1986 involved de-airing the hydraulic tips and then taking readings simultaneously on both the hydraulic tips and the vibrating wire instruments. Three instruments were also removed from the dam for closer examination.

11. The principal finding of the examination was that the instruments had been subject to corrosive attack of a crucial soldered joint by the highly alkaline pore water in the concrete. The pH of a sample of fluid recovered with one of the instruments was 13.0, a value which is normal for concrete containing alkali metal ions and consistent with the observed ASR effects. The foremost cause of the instrument drift appeared to have corrosion to the lead-tin solder securing the pressure sensing diaphragm to the body of the instrument by direct reaction with the alkaline fluids.

12. Ironically, the original vibrating wire piezometers installed as part of the

ASR studies in 1974, which were from a different manufacturer, continued to perform perfectly satisfactorily. By 1990, only about a third of the 1981 piezometers were giving what appeared to be reliable readings and replacement was required.

NEW INSTRUMENTATION AND MONITORING REGIME

Replacement instrumentation

13. It was decided to replace all forty-four 1981 piezometers using the same drillholes and locations as before (Figs. 4 and 5). The type of piezometer would be essentially the same (i.e. vibrating wire electric transducer) but with

Fig. 4 Distribution of vibrating–wire piezometers

Fig. 5 Location of vibrating–wire piezometers in blocks 13 – 14

all vulnerable components of the transducer now manufactured from stainless steel, including the connection between the pressure sensing diaphragm and the body of the instrument, which would be electron-beam welded.

14. The corrosion resistance of the chosen transducer (Geokon Model 4500S) was tested in the laboratory by immersing instruments in a highly alkaline

Fig. 6 Details of retrievable vibrating wire piezometer assembly installed in concrete dam

solution at 60° Celsius for 105 days with no discernible effect on their performance. This gave some reassurance for satisfactory long-term performance. However, it was also considered prudent to incorporate into the design means of easy retrieval of the piezometers, should any problems arise in service. Previously retrieval had been time-consuming involving the use of high pressure water jetting to remove the weak bentonite : cement grout.

15. A retrievable mechanical packer system was devised to hold each transducer at the required level within 38mm rigid plastic casing grouted into the existing drill holes in the dam (Fig. 6). It consists of a bottom assembly, incorporating a 130mm long x 36mm diameter "Neoprene" rubber packer, connected to a head assembly by means of two nesting stainless steel tubes up to 10 metres long. A tightening nut (Nut A) operated in the head assembly at the dam face, expands the packer inside the plastic casing by reducing the distance between Plate 1 and Plate 2, so making a watertight seal against the plastic casing. The latter is perforated near its base allowing access for the pore water in the concrete to the transducer. Cable glands and disc springs (to combat creep in the rubber packer) complete the assembly. The packer system proved to be easy to install. Both the packer and the cabling are so arranged that retrieval of any transducer can be readily achieved by simply reversing the installation procedure. The system was designed and manufactured by Geotechnical Instruments Limited of Leamington Spa.

16. Cabling was run in galvanised and painted 20 and 25mm diameter conduit to the nearby pumphouse building where the data logger and interrogator units are located. Facilities for remote interrogation by telephone landline using a modem were also incorporated. New cabling and readout arrangements were also made for the remaining tendon load cells in block 6 and two old (1974) piezometers in the same block were replaced with non-retrievable transducers. Elaborate measures were taken to protect the system against lightning strikes, which have caused considerable damage here to instruments in the past. Over-voltage protection devices were incorporated in each transducer and at the data logger; gas discharge tubes and current limiting resistors were fitted at the head of each drill hole; and a high quality ground earth was installed for all electrical equipment and conduits.

17. The present instrumentation array therefore consists of forty-four electric piezometers in seven monoliths, covering both the ASR damaged concrete and some unaffected parts of the dam, and the two load cells in block 6. Barometric pressure (used for correction of readings from the unvented transducers) and reservoir water level are also measured using transducers.

Monitoring

18. The new readout arrangement has greatly simplified the monitoring of the piezometers and load cells, which are now read daily by the Jersey New Waterworks Company using the telephone link from their Head Office. Weekly site inspections are, however, made to examine crest movement gauges between each block and to observe the internal relief drain flows. Drain readings are taken monthly unless unusual flows occur. Every six months, a

complete inspection is made by the Supervising Engineer (the owner operates the Reservoirs Act 1975 as if it applied in Jersey) who records the external and internal condition of all exposed concrete, in particular the precise extent of known cracks and minor seepages. The inspection extends only to reservoir level on the upstream face.

Sonic velocity and seismic tomography surveys

19. Sonic velocity surveys are conducted about every five years using (latterly) a power-operated cradle suspended from the crest of the dam. The last two surveys have been augmented by seismic tomography measurements in seven selected blocks to try to obtain a full understanding of the distribution of sonic velocities in the dam (Smith & Dyer, 1990). The method generates a large number of ray paths through a section of the dam and then, by applying computer based tomographic algorithms to invert time travel data, produces a velocity image of the dam section. Hydrophones (below water) or accelerometers (in air) at 600mm centres were used as receivers on the upstream face and elsewhere for sonic pulses generated at 600mm centres on the downstream face. The results to date correlate quite well with the sonic velocity data, but indicate that the velocity distribution through the dam does not directly reflect the horizontal configuration of lifts used in construction. In the blocks affected by ASR, there is a significant variation in velocity along the horizontal travel paths used for the sonic velocity measurements in the individual lifts.

STABILITY ANALYSES

20. The designer of the dam in the 1950s followed convention for that period in assuming internal and foundation uplift pressures of 50% of reservoir head at the upstream face decreasing linearly to zero at the downstream face. Applying the middle third rule to the resultant forces led to the adoption of a section with a vertical upstream face and a downstream slope of 0.7 (horizontal) on 1 (vertical). No consideration was given to seismic loading at that time.

21. After the diagnosis of ASR, there was concern that the expansive cracking of the concrete would lead to higher internal pore pressures than had been allowed for in the design thereby resulting in an unacceptable reduction in stability factors. Measures were taken in 1974 to ensure some control over internal uplift pressures by installing 76mm diameter relief drain holes drilled upwards and downwards from the existing drainage gallery near the upstream face in the central blocks of the dam (blocks 10 - 20). The drains were at 3.35m centres (Fig. 2).

22. Further calculations have been carried out recently (1993) to check the stability of the dam under a variety of loading conditions (i.e. normal, unusual and extreme conditions, including earthquake loadings). They addressed both sliding and overturning (moment) modes using two dimensional classical methods for static analysis and simple pseudostatic analysis to cater for earthquake accelerations and hydrodynamic loads. The effects of ice loadings

have been examined but silt and wave loadings were regarded as negligible in this instance. Conservative parameters have been adopted for concrete strength ($c' = MPa$, $\phi' = 35°$) and the possible beneficial effects of vertical and horizontal keyways in the dam have been ignored.

23. Seismic loadings are derived from the peak ground accelerations (PGA) which are calculated to occur in Maximum Credible Earthquake (MCE) and Design Basis Earthquake (DBE) conditions, namely 0.2g and 0.1g horizontally and 0.1g and 0.05g vertically respectively and acting in concert. These values were obtained by analysis of the existing seismic data for the area and are comparable with recent UK guidelines (Charles, Abbiss, Gosschalk & Hinks, 1991). The same guidelines were used to derive hydrodynamic loadings. The dam classifies as Category II using Charles et al (1991); pseudostatic analysis only for earthquake loading is judged to be appropriate in this case.

24. Uplift pressures were varied to include a number of assumed theoretical distributions (Moffat, 1975; Novak, Moffat, Nalluri & Narayanan, 1990) and also to incorporate projected uplift pressures based on those measured in the new piezometers. Analyses using assumed theoretical 100% - 0% internal uplift pressures showed that the lowest static factor of safety against overturning under maximum flood conditions was 1.55 for those blocks with operational internal drainage and 1.31 for those without. This is considered satisfactory. Under seismic loading, adequate factors of safety were just achieved with operational drainage but were slightly less than unity without. Factors of safety against sliding were always high in both static and seismic conditions. The inclusion of ice loading (assumed to be 58 kN/m for ice 400mm thick) not unexpectedly gave factors of safety less than unity near the crest, which is not reinforced.

25. The piezometers showed, however, that actual uplift pressures measured in the dam are always less than 100% - 0% design uplift line, usually by a considerable margin, and that in only three instances has it exceeded the 50% - 0% design uplift line with the reservoir at top water level (April - May 1993). one piezometer has subsequently fallen below the design line apparently having recovered only very slowly from excess pressures developed during installation, and another is only about 1.5m above the design line. Only one, in block 20, remains significantly above the 50% - 0% line. All three piezometers in question are located on lift joints only about 1.2m from the upstream face, where such an occurrence is not unexpected.

26. Analyses carried out using uplift pressures projected from measured piezometric pressures in the seven blocks concerned gave a satisfactory minimum static factor of safety of 1.63 against overturning under design flood conditions and not less than 1.40 under ice loading. During the extreme conditions of seismic loading, the minimum factor of safety against overturning remained in excess of unity. Factors of safety against sliding were high in all cases.

27. Resultant load loci fell within the middle third for all static cases without ice load but some fell outside when ice loading was introduced. Similarly,

under seismic loading, resultant loci almost invariably fell outside the middle third indicating the likelihood of tensile stresses being set up in the upstream face under these extreme conditions. The magnitudes of these seismically induced transient tensile forces at the upstream face were however very small (calculated not to exceed 0.08 MPa) and well below the likely minimum operating tensile strength of the concrete, even when damaged by ASR (judged to be not less than 0.3 MPa based on Brazil tensile strength tests on concrete taken from the diamond drill instrumentation holes). It was concluded from these analyses that there was no risk of cracks propagating downstream from the upstream face even during extreme loading conditions, except in the special case of ice loading at the crest.

FUTURE REMEDIAL ACTION

28. The studies carried out after 1971, and in particular the installation of piezometers on selected lift joints after 1981, have shown that uplift pressures have only rarely exceeded those assumed in the original design. Stability analyses carried out with the actual pore pressure distributions in the dam do not give cause for concern under normal loading conditions.

29. However, under the unusual loading condition with thick ice on the reservoir surface, some potential instability is possible near the crest of the dam when the reservoir is full. Excessive upstream tensile stresses may develop under these conditions leading to the progressive downstream migration of horizontal cracks and hence higher overall uplift pressures.

30. Effective internal drainage therefore remains essential if the current relatively low internal piezometric pressures are to be maintained. Regular CCTV inspection of the relief drains is carried out by The Jersey New Waterworks Co and any visible build-up of carbonates which could impair their function is removed by high pressure jetting and scaling. Gas traps have also been installed at the base of the above-gallery drains to try to reduce the rate of build-up of carbonates by depleting the carbon dioxide content of their internal atmosphere.

ASR prognosis

31. It is clear from various lines of evidence that expansive cracking due to ASR in the dam has now virtually ceased. Visual inspections show no significant changes in external appearance; the loads carried by the anchors in block 6 (which increased annually by between 4.0 and 2.5kN for some 13 years) are now effectively constant except for seasonal and diurnal fluctuations as shown in Fig. 7; needle gauges at the crest between all the blocks show no significant movement; and there has been no long term decline in seismic velocities, even in the blocks most damaged by ASR, which might indicate continuing internal cracking (Fig. 3). It is therefore not expected that there will be any major changes in internal pore pressures due to ongoing ASR cracking and future uplift conditions are predicted to remain more or less constant provided the drains are kept in good condition.

Possible future remedial action

32. Remedial action will probably be limited to:-

(a) taking measure to ensure that there can be no build up of ice against the crest of the dam. The climate in Jersey is maritime and sustained freezing weather is rare. An air bubbler line operating close to the upstream face would probably suffice.

Fig. 7 Load on anchor 1 in block 6 (1975 – 1994)

(b) improving internal drainage within the ASR damaged parts of block 20 to control the local high piezometric pressures which have been observed there. Drainage can be achieved by drilling diagonally from the gallery in the adjacent block.

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Recent investigations at Wimbleball Dam, Somerset

P. PHILLIPS, BSc, CEng, MEng, MICE, Technical Director, Babtie Shaw & Morton, and EurIng F. SHANNON, BSc, MSc, CEng, MIEI, MICE, Principal Engineer, Babtie Geotechnical

SYNOPSIS. The concrete surface of Wimbleball Dam has suffered from spalling. Relief well flows beneath the foundation have tended to increase significantly in certain areas. This paper describes recent investigations into these problems.

INTRODUCTION

1. Wimbleball Dam, which forms a water supply reservoir for South West Water and Wessex Water, is located in north Somerset on the edge of Exmoor National Park. It is a mass concrete buttress dam approximately 300 metres long with a maximum height of nearly 60 metres. Construction started in 1974 by Bovis Civil Engineering with impounding of the River Haddow taking place some three years later. Since then there have been investigations by Rofe Kennard and Lapworth, Babtie Shaw and Morton and others into concrete ageing defects and increasing drainage trends beneath the dam.

CONCRETE SPALLING - EARLY INVESTIGATIONS

2. Towards the end of the construction of the dam, Rofe Kennard and Lapworth observed small "pop-outs"(minor spalling) on the inside concrete faces of the dam parapet walls. These were about 10mm in diameter with soft dark aggregate exposed and a rust coloured stain below. During the next few years spalling was noted at an increasing frequency on almost all concrete areas of the dam above normal water level.

3. A chemical analysis of the popout material was undertaken by Edgar Morton and Partners, which confirmed that the black aggregate is a pyrites-rich shale or mudstone and that the spalling was due to the oxidation of pyrites(iron sulphide) in the presence of oxygen and water, ie the normal external environment. The reaction uses the calcium in the cement matrix to produce ferric hydroxide and gypsum which have about double the original volume. This expansion, combined with a weakening of the concrete produced by the gypsum, causes spalling. The rust coloured stain is caused by the ferric hydroxide in solution. The possible reactions

Figure 1 Downstream Face of Dam

are given in equations 1, 2 and 3 below.

4. The incidence of "black spots" or "pop-outs" was only one spot in every four square metres and the upstream face of the dam seemed to be clear of pop-outs. The problem appeared to be more severe in concrete poured in the summer period. It was considered that pulverised fuel ash present in the mix could act as a catalyst for the reaction which could increase the rate of pyrites oxidation, but not its overall extent. It was thought that the problem was not likely to get worse but that the situation should be monitored and that the quarry from which the aggregate had originated should be identified, in order to assess the likely amount of pyrites- bearing aggregate.

5. In order to trace the supply quarry, Dr G D Nicholls of the University of Manchester was asked to analyse cores taken from various parts of the dam. These included samples from both "spotted" (popouts) and "unspotted" concrete, which were found to contain a higher than expected proportion of mudstone/siltstone. Pyrites was found but not apparently in sufficient quantities to cause the reaction. The sample analysis did not match that from samples taken from the three sources of aggregate approved for supply to the dam. A fourth quarry, near Bampton, was checked and found to have in some instances an iron sulphide content of 5%, enough to cause the spalling found at Wimbleball.

6. An estimate was made that 10% to 20% of the aggregate had originated from this quarry. It was further estimated that 10% of the aggregate was reactive, suggesting that 1 to 2 percent of the aggregate in the dam would be reactive. This could represent about 10 reactive pieces within 50mm of the surface in each square metre of concrete surface. This possible incidence is considerably greater than the one per four square metres observed earlier.

RECENT INVESTIGATIONS

7. Following an Inspection of the reservoir in 1989, South West Water appointed Babtie Shaw and Morton to undertake a further appraisal of the concrete spalling. Firstly a visual assessment was made of the concrete deterioration which indicated that nearly all visible parts of the dam were suffering spalling to some extent, even painted internal surfaces within the

pumphouse and the inside of the valve tower. However there was almost no spalling below the normal water level on the upstream face. Spalling was observed to be of two principal types, one with rust stains, the other without. The former appear to be caused by pyrites reaction and the latter due to frost or moisture expansion of a very weak and friable aggregate which was exposed in each case. The spalls range in size from as little as 100sqmm in area by 5mm deep to 15000sqmm by 20mm deep or more. Some repairs had been carried out, one of which fell away during inspection revealing a still reactive aggregate particle beneath.

8. A comparison was made with the extent of spalling recorded in 1982 which indicated that on the valve tower in particular the incidence of spalling appeared to have increased about twentyfold. A rough approximation, by extrapolation, gives a total of 35000 spalls in the unreinforced elements of the dam and 2000 in the reinforced elements. The former figure seems high but is really only of cosmetic importance, since the mass concrete is essentially structurally unaffected. A limited covermeter survey was undertaken on the more easily accessible reinforced elements of the dam. The results showed that the areas with the least cover were the valve tower (30 to 45mm) and the valve tower bridge (16 to 25mm).

9. A literature search was conducted for previous experience of pyrites oxidation in concrete and research establishments were contacted. Various formulae for the reaction are given in the literature; those which seem most appropriate at Wimbleball are given in equations 1 to 3 below.

$$2FeS_2(\text{pyrites}) + 7O_2 + 2H_2O = 2FeSO_4 + 2H_2SO_4 \qquad (1)$$

$$FeSO_4 + Ca(OH)_2(\text{in mortar}) + 2H_2O = CaSO_4 2H_2O + Fe(OH)_2 \qquad (2)$$

$$2Fe(OH)_2 + H_2O = 1/2\ O_2 + 2Fe(OH)_3(\text{rust coloured stain}) \qquad (3)$$

The calcium hydroxide in equation 2 is derived from the mortar and is converted to gypsum. The iron hydroxide produces the rust coloured stain.

10. It was recommended that as the mass concrete dam is considered safe only the reinforced concrete valve tower and access bridge need be protected. The work would involve breaking out all patches of soft or reactive aggregate near the surface and patching with a cementitious mortar, then coating the whole surface to reduce the quantity of oxygenated water penetrating the surface in order to slow down the rate of deterioration of unexposed reactive aggregate. It was also recommended that the dam face should be fenced off to inhibit public access in case of injury from spalling concrete falling from the dam.

REMEDIAL WORKS AND TRIAL PANELS

11. A contract valued at £48000 was let to Makers Industrial Ltd in May

1990 for repair work to the valve tower and access bridge. The contract lasted for nearly three months. Various repair systems were investigated, but in view of the need to expedite the work it was decided to adopt the Renderoc repair system from Fosroc International Ltd, which had been used on Haweswater Dam in Cumbria after successful trials. In order to assess the performance of other materials which may be used in any future repair work three trial panels were set up on each of two concrete buttresses. The systems used for the trial panels were from Sealocrete Ltd, Vandex (UK) Ltd and Sika Ltd. Several additional smaller panels were set up to check colour matching of the Renderoc system. South West Water maintain annual photographic records to a specification of camera type, location and weather conditions, so that changes can be monitored.

Surface preparation

12. The concrete surfaces of the valve tower and access bridge were cleaned by high pressure water washing to one metre below top water level. Exposed and spalling pieces of soft flaky aggregate, 20mm in size, were removed by use of a pointed hand- held percussion breaker. Smaller aggregate, about 10mm in size or less, was removed by drilling out with a 12mm dia drill bit. The number of the larger repairs averaged 7/sq metre on the valve tower and 3/sq metre on the access bridge. On the trial panels the preparation was similar except that no smaller spalls were treated.

Filling and coating

13. The larger holes were filled with Renderoc HB (high build) mortar and the smaller holes with Renderoc S (high strength mortar). The surfaces of the valve tower and the access bridge above water level were coated with 3 mm thickness of Renderoc FC (fairing coat), followed by a protective coat of elastomeric Nitocote Dekguard Clear on a Dekguard primer. Where, in a very few cases, reinforcement had to be exposed, the steel was wire brushed and primed prior to rendering.

UNDERDRAINAGE AND HYDROGEOLOGICAL CONSIDERATIONS

HISTORY OF MONITORING

14. Relief wells were installed, during construction, beneath the foundation of the dam to collect flows from downstream of the grout curtain and pipe them into a series of collecting chambers. These flows have been monitored since installation together with other instrumentation built into the dam. Concern was expressed during first impounding at increasing flow rates in some of the relief wells located on the southern flank. Subsequent flow measurements taken from the south relief wells show a steady increase from 1.5 litres/second to 8 litres/second between 1982 and 1989. Figure 2 shows the total drainage flow from the south side and north side plotted against reservoir level and rainfall. Measurements of total flow at buttress 13 are also indicated as these appear to constitute

most of the south side flow but they are considered to be somewhat inaccurate, due to poorly calibrated V notches. Measurement accuracy improved with the installation of a Polysonics flow meter in November 1992.

Figure 2 Relief Well Flows (0 Potentially Incorrect)

15. The Inspecting Engineer's report (1989) also expressed concern that the alteration in trend of relief well flows may affect hydrostatic pressure conditions beneath the dam with consequential changes to the stability of the structure. It recommended that piezometers should be installed at foundation level beneath buttresses 12-14 in order to monitor uplift pressures. The installation of these piezometers was carried out during January 1991 enabling hydrostatic pressures in the foundation to be measured and flow nets to be constructed. Uplift pressures at selected buttresses were checked with respect to the overall stability of the dam and gave no cause for concern.

16. An analysis of the construction records and drawings, the site investigation information, foundation rock levels, relief well configurations, drainage layout, curtain and blanket grouting was undertaken by Babtie Shaw and Morton to determine whether reasons for the increase in relief well flows could be found. In addition, reports and published papers on the project were reviewed.

GEOLOGICAL CONDITIONS

17. The bedrock encountered beneath the dam is reported by Bass and

Isherwood to have consisted of sandstones, siltstones and interbedded mudstones of Upper Devonian age, folded, sheared and faulted. The dip of the strata varied from 4° - 62° in the borehole cores. Many of the joints and fissures were observed to contain clay infill (up to 41cm thick). The zones of clay infill were noted to persist to "considerable depths".

Figure 3. Plan of Dam Showing Principal Shear Zones

18. Bass and Isherwood's paper states that "The most significant aspects affecting the foundation level of the dam were the shear zones". The distribution of the principle shear zones within the exposed foundation is shown in figure 3 (based on figure 3 of the published paper). It can be seen that there are two shear zones at the upstream toe of buttresses 11 and 13 trending towards the downstream toe of buttress 12. These shear zones generally comprise fractured and shattered rock within a matrix of soft red clay. Occasionally a shear zone was reported as "rock, shattered and crumbled to a gravel". A major shear zone in the vicinity of buttress 12 is reported as between 3-5 metres wide. A seismic survey carried out during construction indicated that poor quality rock was deeper on the south flank of the site than elsewhere and it was postulated by Knill(1976) that the south flank appeared to be underlain by a zone of fractured and sheared rock lying between two faults separated by about 150m.

RELIEF WELL DRAINAGE

19. The variation in relief well flow rate shows a direct correlation with reservoir level with little time lag. During periods of constant high reservoir level the flows from the south relief wells continued to increase to their highest rate of flow in excess of 8 litres/second in 1989-90. The discharge rate would therefore appear to be influenced by the length of time high water levels are maintained, increasing at an average rate of 0.2 litres/second per month of constant high water level. At the end of 1990

the effect of lower reservoir levels than normal in the previous two years has depressed the relief well flows but since early 1991 the flow rate increased at almost the same rate as before, to reach the same peak of 8 l/s in the spring of 1993.

GROUT CURTAIN ANALYSIS

20. As first impounding progressed, the southern relief well flows responded by increasing to reach 10 litres/second. In addition, a group of springs developed 200m downstream of the dam above stream level. The inference from chemical analysis was that 80% of the springs' flow originated from the reservoir. Remedial measures were instigated in the form of an additional programme of grouting. This was carried out within an area from buttress 16 southwards for approximately 110m. The zone of influence around each drillhole was increased by raising the grouting pressures by 25%. The grout consumption under buttress 16 was reported by Bruce and George to be "very high", primarily in the upper stages. Grout connections into relief wells and to the tailbay of the dam were recorded. The distance of such grout travel is surprising given that this additional grouting was carried out to reinforce the existing grout curtain. During these additional grouting operations relief well flows dropped by over 50% despite a rise in reservoir level. It was postulated by Bruce and George that the pattern of grout acceptance in the later stages of the treatment indicated that residual flow was occurring through small fissures (less that 160 μm) or in slightly larger fissures at a greater depth or distance from the dam.

21. The grouting records during construction for Buttress numbers 12, 13 and 14 were assessed by Babtie Shaw and Morton. By summating the mass of cement used under each buttress and dividing either by the total associated lengths of drill-holes or area of curtain beneath, the mean total, lineal, and areal grout take has been calculated for individual buttresses. Areal grout take is the mass of grout used per square metre of grout curtain in elevation, and gives a measure of grout consumption within the rock mass. The results are tabulated in Table 1.

Buttress No	Total Area M2	Total B/H Length	Total Grout kg	Lineal Grout Take kg/m	Areal Grout Take kg/sm
12	532	812	31193	39	59
13	602	1180	29872	25	50
14	602	1242	48313	39	80

Table 1. Grout Curtain Analysis.

From the table it can be seen that buttress 13 has a lower lineal and areal

grout consumption than the two adjacent buttresses. This could be due to the nature of the fissures within the rock mass or premature curtailment of grouting operations. The character of the shear zone running parallel to the upstream toe of buttress 13 and coincident with the plane of the grout curtain may have reduced the capacity of the rock mass to accept grout.

BLANKET GROUTING ANALYSIS

22. An assessment of the blanket grouting records for buttress numbers 7-16 has been similarly carried out to establish whether any corresponding trend in grout take exists. Grout takes were calculated for each southern buttress and are graphed in Figure 4.

Figure 4 Blanket Grouting Analysis

23. The grout takes were fairly consistent over buttresses 7-11 at between 28-35 kg/m^2. At buttress 12, however, the areal grout take is doubled at 64 kg/m^2 with that for buttress 13 showing a similar grout consumption at 66 kg/m^2. It can be seen from figure 3 that buttress 12 is located over a junction of principle shear zones. From buttress 13 southwards, the trend in grout consumption is generally upwards reaching 119 kg/m^2 and 237 kg/m^2 at buttresses 14 and 16 respectively.

24. The record drawings show that some of the blanket grouting for buttress 11 and 12 was carried out beyond the upstream toe of the dam whereas at buttress 13 only the area contained beneath the foundation was grouted. This may have had a long term effect on the performance of the relief wells in the vicinity of buttress 13 allowing them to respond quickly due to the intensity of blanket grouting around the upstream toe. It is possible that groundwater movement since impounding may have eroded some of the clay infill from the shear zones or indeed some of the grout contained within fissures, causing a progressive increase in relief well flows at the upstream end of buttress 13.

FLOW NETS

25. Flow nets were constructed for two buttress locations - No. 9 located within the deep section of the valley where existing piezometer information is available, and No. 13 selected for the high flowing relief wells there, and where recent piezometric results are available for analysis. A back analysis of measured relief well flows at Buttress 9 using the flow net shown in figure 5 indicates a rock mass permeability of approximately 1.5 x 10-6 m/s (in line with expected values).

26. A back analysis of measured relief well flows at Buttress 13 in conjunction with the normal flow net for a full depth grout curtain indicates a rock mass permeability in the order of 1 x 10^{-5} m/s for flow rates prior to 1982. This flow net (not shown) was modified to model a grout curtain curtailed in depth to one third of its actual depth. This model represents either a "window" in the grout curtain or erosion of grout at depth. Using the flow net applicable to a curtailed grout curtain of one third depth, a fourfold increase in relief well flow results. This four-fold level of theoretical increase corresponds approximately to the actual increase in total relief well flow between 1982 and 1989. A four-fold increase in relief well flow also results from calculations where the rock mass permeability is increased by a factor of 4 assuming erosion of fine particles from fissures or shear zones. Of these two options a reduction of grout durability is considered more likely. However the recent trend has been for the flows at buttress 13 to stabilise.

Figure 5. Flow net at Buttress 9.

"MONITOR" COMPUTER PROGRAM

27. In order to improve the storage and monitoring of the dam records, which include regular readings of relief well flows, groundwater, pendulums, crest position etc., a database program was commissioned by South West Water. The program is designed to accept regular data input and to automatically graph a selection of readings or groups of readings to allow trends to be monitored. Graphs are set up in the form recommended in the "Guide to the Safety of Embankment Dams". Consideration was given to the inclusion of a trigger line to highlight excessive flows and

pressures but this was thought to be inadvisable in a situation where interpretation of the data is complex. Instead, the data is regularly monitored by South West Water. The program has links to a program called "Surfer" which allows for three dimensional representation of ground water levels although no practical use has yet been made of this facility.

CONCLUSION

28. Concrete surfaces have been protected to reduce the chance of reinforcement corrosion. Spalling on the mass concrete is of no structural importance and may reduce in time. Monitoring will indicate if or when a further repair programme is necessary. The flows at buttress 13 appear to have stabilised, and since the introduction of a Polysonics recorder, can be considered accurate. However the total south side flow still appears to be increasing and further monitoring and investigation will be required to establish the reasons for the increase.

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Introduction to the CIRIA engineering guide to concrete and masonry dam structures in the UK

R.A. READER, BSc, CEng, FICE, FIWEM, Partner, C.L. OWENS, BSc, CEng, FICE, FIWEM, Associate, and M.F. KENNARD, BSc, CEng, FICE, FIWEM, FASCE, Consultant, Rofe Kennard and Lapworth

SYNOPSIS. Between February 1993 and February 1994 the authors put together the material being published by CIRIA as "An Engineering Guide to the Safety of Concrete and Masonry Dam Structures in the UK". The material was derived from dam owners, panel engineers, members of the steering group for the project and from published sources, as well as from the authors' own sources of information and experience. A visit to other European countries, sponsored by the Department of Trade and Industry, widened the background to the Guide. This paper endeavours to introduce the Guide, explain its aspirations and highlight a few points that have emerged during its compilation.

SCOPE

1. This Guide sets out to give comprehensive advice on matters related to the safety of concrete and masonry dams and associated dam structures in the United Kingdom. It is intended as a companion volume to the BRE Guide to the Safety of Embankment Dams and is arranged in a similar format for the convenience of those who deal with both kinds of dam. The Institution of Civil Engineers 'Floods and Reservoir Safety: an Engineering Guide' and the BRE 'Guide to Seismic Risk to Dams in the UK' are also complementary to this Guide.

The Guide is divided into sections:

Historical background

2. The Guide includes an historical background to the development of the various types of concrete and masonry dams, giving examples of typical designs.

3. During the 19th Century the art of masonry dam construction, which later led to concrete dams, made important advances. In 1853, M de Sazilly, a French engineer, advocated that stresses within a dam be held to specific limits and the cross-section be such to preclude sliding. He did not recognise the concept of keeping the resultant of all the forces within the middle third of each horizontal section to avoid tensile forces. This

concept was set out by Professor W J M Rankine of Glasgow, about 25 years later and Professor Unwin, when investigating the failure of Bouzey dam in France, concluded that this not only applied at foundation level, but at any horizontal section above (Unwin, 1895). These views were important stepping stones towards successfully engineered designs of gravity dams. The 50m high Furens dam in France, built 1861–65, was the first masonry dam built with a rationally designed cross-section.

4. Engineered masonry and concrete dams have been built in the UK since the last quarter of the nineteenth century, although many substantial masonry weirs had been completed earlier. Abbeystead dam, Lancashire, completed in 1881 was designed by J L Mansergh and is a mass concrete masonry-faced structure built to a curve or arch. This remarkable structure predates all the masonry dams and is the first mass concrete arch-gravity dam exceeding 15m in height above foundation level built in Great Britain.

5. Vyrnwy dam in Wales, designed by T Hawksley & G F Deacon, completed in 1890, was the first gravity dam where the importance of uplift pressures at foundation level was recognised, and drains were incorporated on the rock foundation. The 33m high Haweswater dam for Manchester's water supply, completed in 1941, was the first buttress dam built in Great Britain and was the forerunner of several others built in Scotland after 1945.

6. The post war Scottish hydro-electric development gave scope for the refinement of the buttress dam to achieve maximum economy. Examples include Sloy and Giorra completed in 1951 and 1959 respectively, Errochty, completed in 1957, and Shira completed in 1959. Arguably the most elegant of the hydro-buttress dam designs was Cruachan completed in 1964. The highest buttress dam in the UK to date is Clywedog, Wales, standing 72m above foundation level, and the most recent, Wimbleball, Somerset, completed in 1979.

7. The section ends with glossary of dam types with photographs and references for each.

Key component materials

8. Consideration of the safety of any dam, but particularly a concrete or masonry dam, must include an assessment of the foundation which is an integral part of the structure. The Guide reviews the materials used in concrete and masonry dams and the concrete mixes and placing methods adopted before modern practice developed.

Design background

9. The Guide gives a background to the design and construction of dams, describing the development of design methods and the features that occur in older structures. It is not intended as a guide to the design of a new dam.

10. The original designs of concrete and masonry dams fundamentally affect not only their safety, but also their potential for deterioration; many inadequacies can be traced to defects inherent in the original design.

11. Uplift is treated in some detail as it is the most significant factor in the design of modern gravity dam profiles; significant because it is the only factor that can be altered by careful design.

12. Concerns about the seismic safety of concrete dams have been growing during recent years. However, no *failure* of a concrete dam has resulted from earthquake excitation; in fact the only complete collapses of concrete dams have been due to failures in the foundation rock supporting the dam.

Construction

13. The Guide describes construction methods and tries to highlight those features that differ from modern methods or are not usually evident from an inspection of the structure or the record drawings.

Ancillary structures gates and valves

14. A section on ancillary structures also embraces those on embankment dams by describing typical layouts and arrangements of pipes, valves and gates and dealing with electrical and mechanical equipment, its testing and maintenance.

15. In Britain the likelihood that a concrete or masonry dam (or earth dam with a valve shaft) has provision for a bulkhead gate of some design seems to increase with latitude: in Scottish dams they are almost standard whereas dams in the south of England rarely have such a facility.

Deterioration and remedial works

16. All dams deteriorate slowly through the normal wear and tear associated with any structure in a harsh environment. But normal wear and tear can be exacerbated by poor design, poor maintenance, or lack of surveillance to detect the early signs of deterioration. Without regular planned surveillance and regular maintenance, deterioration can go unnoticed and dangerous conditions may arise which could be avoided. For example, maintenance of the drainage systems by flushing and the study of observed pressures and flows can prevent further deterioration of concrete by controlling the quantity and extent of seepage.

17. In recent years many dam structures and ancillary works have been examined for deterioration of their concrete and many have been diagnosed as having ASR. The Guide sets out to give practical advice on dam maintenance by describing how dams deteriorate, how they can be investigated, instrumented, monitored, subjected to planned surveillance, and rectified.

18. On many dams vertical contraction joints have become seriously blocked through buildup of deposits so that movement can no longer

occur. In straight dams this may not lead to serious defects, but in Mullardoch, where two straight sections met at an angle, the resulting compression caused downstream movement of the apex block and cracking on horizontal joints.

19. Cracks in concrete remote from design joints often give cause for concern and the Guide sets out to give guidance on their effect on safety and methods of repair.

Surveillance, instrumentation and monitoring

20. Effective surveillance is essential for reservoir safety. Surveillance is a term used to describe visual observation, while monitoring describes the reading and interpretation of instruments. Instrumentation is often installed to monitor effectively a phenomenon noticed initially during routine surveillance.

21. Surveillance should comprise a planned range of activities carried out by suitable (but possibly several) personnel. The programme should be based upon the periodic Statutory visits by Inspecting and Supervising Engineers and discussed with them, and on the Statutory requirement to keep records. It should be remembered that many signs of deterioration have been first observed by reservoir keepers, members of the undertaker's staff or the general public, who have reported anomalous behaviour immediately.

22. Since the 1950s many dams have been instrumented extensively during construction. Older dams also had instruments, particularly pendulums, installed mainly to monitor first impounding. Many structures have had instrumentation of various types installed since construction to monitor some phenomenon highlighted during inspection.

Investigation

23. Investigation of a dam is generally undertaken only when observation of surface features or a review of monitoring records gives cause for concern. Investigations usually include the installation of additional instrumentation and more intensive monitoring. As a full site investigation involving the use of boreholes, diamond drilling and coring, trial pits and new instrumentation is relatively expensive, the Guide gives advice so that the objectives are clearly defined and the work carefully planned.

24. Before an investigation is carried out, a full study of existing information should be made using existing drawings, surveys, site investigation logs and reports, construction photographs, instrumentation data and any relevant papers and reports.

Appendices

25. The Guide contains appendices to give more detail on particular aspects of reservoir safety, a list of useful references covering text books

and published papers, tables of some typical parameters for guidance and figures and photographs to illustrate new and old construction techniques. A surveillance check list is included.

26. The Guide does not attempt to cover embankment or tailings dams, (other than their appurtenant concrete or masonry structures), reinforced concrete or masonry service reservoirs, the prediction of floods or overflow design, hazards to downstream areas or dam break analysis. Information on the design and construction of new dams is purposely omitted as being inappropriate in this Guide.

USE OF THE GUIDE

27. The 1975 Reservoirs Act requires owners to keep their reservoirs under more constant supervision by the continuous appointment of a Supervising Engineer from a panel of qualified engineers in addition to the periodic inspections by Inspecting Engineers. It is to this panel of engineers that the Guide is primarily addressed.

28. The Guide seeks to emphasise the importance of observation and regular surveillance by perceptive and curious engineers. No two dams are the same and a common system is inappropriate. Common-sense and a vigilant approach by open-eyed and suitably qualified owners and engineers should keep the UK stock of dams in good order, and safe.

29. Most masonry and concrete dams are categorized as high-hazard, low-risk structures; that is, the consequences of them failing are very great, but the probability of a failure is remote. Large dams in the north of England probably present some of the largest man-made hazards to life and property in the UK.

30. The public do not perhaps perceive dams as a high hazard because failures in the UK have been few. But catastrophic failures at Malpasset (France) in 1959, at Vaiont (Italy) in 1963, Teton (USA) in 1976 and at Macchu II (India) 1979 focused attention on dam safety in those countries and had an effect worldwide on dam engineering, largely through the work of the International Commission on Large Dams (ICOLD).

COMPILING THE GUIDE

31. The century covering the construction of concrete and masonry dams in the UK, from 1880 till 1980, was a classic century for the publication of technical papers. Almost every major concrete or masonry dam in the UK has had one or more papers published on its construction or its problems. These contain a wealth of information for the present day engineer. They not only give details of the dam and its original ancillaries but also give a flavour of the concerns that were then uppermost in engineer's minds. Some describe construction methods and construction sequences in considerable detail. The list of references given in the Guide is but a sample of the papers that have been published. It is tempting to

suggest that the relevant published papers should be bound in to every dam's Form of Record under the Act.

32. This wealth of papers describes a huge variety of dam designs. Whereas the BRE Embankment Dams Guide could use three typical cross–sections to describe the vast majority of embankment dams almost every concrete or masonry dam has a cross–section in which certain features pertinent to its safety are unique. Indeed, many of the eminent dam engineers on the steering group for this project were adept at 'spotting the dam' from its unlabelled cross–section! This has made the production of a Guide to the safety of a 'standard' dam difficult; there are so many exceptions to every potential rule. For this reason we have emphasised the need to return to the basic information available about the dam and analyse it with an open mind. There are very few standard features.

33. Information for the section on deterioration and remedial work started from a questionnaire to dam owners to identify those features that had caused problems in the past or were causing disquiet currently. This information was amplified by visits to dam owners to discuss their problems, and, of course, the experiences of the Steering Group appointed by CIRIA to comment on drafts of the Guide.

34. The extremely helpful replies to the questionnaire enabled the authors to focus attention on those matters that were causing most concern. Uppermost in many dam owners' minds was a factor now commonly called 'ageing'. An ICOLD Bulletin on this subject is currently being prepared and the authors were able to inspect a draft of the conclusions made after a study of ageing case histories around the world. A conclusion of this study was that there were relatively few modes of ageing in the concrete and masonry dam structures themselves:

- chemical reactions resulting in swelling
- shrinkage, creep and reaction leading to contraction
- loss of strength under repeated loadings
- degradation due to chemical reactions
- poor freeze/thaw resistance

35. The last two are usually discernible on the surface of the structure and are exacerbated by seepage or dampness. But the first three are usually only discerned by a close analysis of deformation readings taken over an extended period. The level of monitoring to make such an analysis possible is rarely present on UK dams – either the instrumentation is lacking or the frequency of reading insufficient.

VISIT TO OTHER EUROPEAN COUNTRIES

36. The DTI sponsored visit to six other European countries allowed the authors to set UK safety practice in a wider context. The conclusions on safety legislation have been presented to BDS previously.

37. The legislation in most of the countries visited is more recent than that in Britain. It tends to be more detailed, to deal more fully with design and construction and to set up more centralised authorities to keep records and carry out technical, as well as administrative, counterchecks on safety during operation. It deals with emergency procedures and tries to define hazard classifications. It sometimes specifies links with academic centres but does not set out any arrangements for the qualification of dam safety personnel. The legislation is followed quite closely by the main owners of the large dams for which it was formulated: it is largely ignored by the owners of small dams.

38. Britain's legislation has unique features: it applies to a smaller size of reservoir than anywhere else, it sets up a system for appointing qualified dam safety engineers and it is adhered to by virtually every dam owner.

39. British legislation is a framework for a 'minimum programme' of safety that is universally applied and exceeded in practice by most owners and certainly exceeded by virtually all recent construction.

INSTRUMENTATION

40. Many continental dams are instrumented to a degree not seen normally (or ever) in the UK. The larger dams with sophisticated arch designs often contain over a thousand items of instrumentation, frequently read by a small, dedicated team, and plotted by computerised methods. The computers are programmed with a predicted range of readings for each instrument, derived from calculation or past patterns, and instrument readings falling outside those boundaries are immediately highlighted. These systems are extremely impressive but more suited to large, highly-stressed arch dams rather than the more workaday structures usual in the UK. But some important lessons can be learnt from the clearcut approach that is taken to safety on these impressive structures.

41. In the Guide the authors have tried to set instrumentation in a clear framework by looking closely at its purposes:

- to monitor construction
- to monitor first filling
- to monitor operation

Monitoring during operation has two functions:

- monitoring for risk scenarios
- rapid drawdown and refilling

- earthquake
- overtopping or high water level due to floods
- monitoring gradual deterioration

42. Concrete and masonry dams are relatively rigid structures on a relatively rigid foundation. In monitoring gradual deterioration of the structure it must be assumed that nearly everything affects nearly everything else. Changes in reservoir water level will certainly affect the seepage flow and the deformation of the dam, but so will creep and shrinkage of the concrete, the ground water level in the hillsides, the time since the pressure relief drains were last cleared – etc. But the extensive instrumentation in many continental dams shows that the greatest factors affecting all instrumentation readings are the temperature of the structure and the air around it. Many examples were quoted where the cyclic variation in an instrument from thermal effects was several times greater than that caused by the factor the instrument was designed to monitor.

43. This phenomenon has been remarked upon in the UK. In the North of Scotland Hydro–electric Board the influence of ambient temperature on instrument readings was shown to be a major factor in their interpretation (Curtis, 1975). More recently Scottish Hydro–electric has published typical results of its dam monitoring showing similar effects (Beak, 1992). Thermal effects vary not only annually with the seasons, but daily. The heavily instrumented dams in Portugal and Austria showed thermal effects clearly on instruments that could be read automatically many times daily. The band that showed the range of normal readings oscillated daily and moved annually over a range far greater than that generated by a large alteration in reservoir level. And because lower reservoir levels allowed the upstream face of the dam also to be affected by ambient temperatures, the thermal effects were then even greater. Thermal stresses and strains affected not only measurements of crest or monolith deformation, they also sometimes affected seepage flows and uplift, presumably by altering the crack widths in the structure and its foundations.

44. The conclusion must be that many instruments on concrete and masonry dams are so greatly affected by thermal stresses that their readings are only of use to detect deterioration if:

- they are modified by temperature readings taken both inside and outside the dam, or
- they are only read when temperature conditions conform to some standard situation so that thermal effects can be largely disregarded.

45. In the UK our masonry and concrete dams generally have few instruments. These are often read infrequently. But the gradual change in

those readings is likely to be the only warning sign of the insidious onset of dam ageing; and if ageing is not diagnosed soon enough, expensive remedial works can follow.

46. So the Guide recommends that rarely-read instrumentation should be read at times when a standard ambience applies. In this way some attempt can be made to take out the 'unknowns' from the performance of the instrument and obtain a sequence of readings that will isolate the small deformations that signify ageing and deterioration of the structure.

CONCLUSION

47. A well-designed concrete or masonry dam with adequate specifications, construction control, and subsequent surveillance, inspection and maintenance should continue as a safe structure. But deterioration of the concrete and internal drainage can lead to a reduction in the factors of safety because some of the original design assumptions no longer apply. It cannot be assumed that a gravity dam that is safe on first filling will always remain in an adequately safe condition.

48. Maintenance must uphold not only the engineering principles of the original design but also take account of new engineering requirements (e.g. updated flood estimates), as well as the effects of gradual deterioration (ageing), seismic forces, legislation and other procedural matters.

49. The authors hope that this Guide will complement the ICE Floods Guide, the BRE Seismic Guide, the BRE Embankment Dams Guide and CIRIA's Small Embankment Dams Guide to form a quintet of volumes designed to assist dam Engineers in their task of keeping our dams under constant review and safe.

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Pitlochry Dam — the use of post-tensioned ground anchors to increase stability

N.M. SANDILANDS, BSc, CEng, MICE, Principal Engineer, Civil Works, A.T. CAMERON, BSc, CEng, MICE, Project Engineer, and W.M. BRYCE, BSc, CEng, MICE, Civil Engineer, Scottish Hydro-Electric plc, Engineering Department

SYNOPSIS. Pitlochry Dam in the Tummel Valley Scheme was completed in 1951. Design checks in 1990 using current guidance of flood assessment indicated that stability was not satisfactory for extreme load cases. A decision was taken to install post-tensioned ground anchors.

This paper describes the project from that stage until commencement of anchor installation on site in October 1993. The stability analysis, site investigation, design of anchors and contract arrangements are all covered. In view of the status of the site as a major tourist attraction, particular consideration was also given to minimising the impact of the works on the public.

GENERAL DESCRIPTION OF DAM

1. Pitlochry Dam is situated on Loch Faskally in the Tummel Valley Hydro-Electric Scheme. Loch Faskally is a man-made reservoir which was formed by the construction of Pitlochry Dam. The dam consists of concrete gravity sections on the north and south sides with downstream earth embankments, the power station and the flood gate section. The crest length of the dam is 162m and the height above foundations is 20m.

2. The power station has two 7.5MW Kaplan turbines, giving an installed capacity of 15MW at an operating head of 15m. Hydro-Electric operate a visitor centre at the power station. There is a salmon ladder allowing fish to pass upstream into the reservoir. This is equipped with a viewing chamber and is a major tourist attraction. The flood gate section consists of two 27m long by 4.8m high drum gates. The gates are float controlled and lower automatically during a flood. The total capacity of the two gates is $1200m^3/s$. Three control towers give access to the floatation chambers and house the control equipment. A ground sluice gate is located in the central tower. This is used to scour silt, to control reservoir levels and to increase spill capacity.

3. The dam foundations are on sound rock. There is a cut off trench 1.5m deep with a grout curtain on the upstream edge. Concrete cut off walls extend through the permeable overlying deposits for a distance of 290m on the north side and 23m on the south side. A layout of the dam is shown in Figure 1.

 Reservoir safety and environment. Thomas Telford, London, 1994

FIG 1 - PITLOCHRY DAM GENERAL ARRANGEMENT

RESERVOIR MONITORING

4. In accordance with the requirements of the Reservoirs Act 1975, the dam is inspected by an independent qualified engineer at ten-year intervals. The Supervising Engineer inspects Pitlochry Dam four times each year. Design standards for dams have changed significantly since the dam was constructed in 1951. An inspection of the reservoir was carried out in 1990 by Mr John Cowie of Mott Macdonald Power. A

reassessment of stability was carried out at this time using recalculated flood levels. Flood levels were based on the Flood and Reservoir Safety Guide (Ref.1).

5. The assessment indicated that factors of safety against overturning and tensile stresses in the upstream face were not satisfactory. In view of the good condition of the dam and the competent rock on which it was founded a decision was taken to install a post-tensioning system.

STABILITY ANALYSIS

6. Once the decision was taken to proceed with post tensioning a full stability analysis was carried out on every section of the dam to determine which sections required post tensioning and the anchor loadings which would be required. In addition to the flood cases considered in the earlier analysis seismic loading was also assessed. Seismic loadings were assessed in accordance with "An Engineering Guide to Seismic Risk" (Ref.2).

7. Four loading cases were considered:

a) Probable Maximum Flood (PMF).
b) 1 in 10,000 year flood.
c) 1 in 150 year flood combined with failure of one flood gate.
d) Seismic loading.

In all cases voids within the structure were included in the calculation and for certain overtopping conditions they were assumed to be flooded. The beneficial effect of any overspilling water on the crest and downstream face was neglected.

8. It was assumed that uplift pressure reduced linearly from the upstream to the downstream faces (Figure 2). This initial assumption

FIG 2
UPLIFT PRESSURE

was simplified and conservative and ignored the effect of the grout curtain. The downstream embankment on the gravity sections were ignored. It was assumed that these would be washed away during an extreme flood.

9. Three modes of failure were investigated.
   a) Overturning
   b) Sliding
   c) Tensile stresses

10. For case (a) the point of rotation was taken at the downstream toe. The Factor of Safety was calculated as the ratio of restoring moments to disturbing moments.

11. For case (b) the resistance in the plane being considered was calculated taking into account friction and shear stress in the concrete. The friction component was based on the vertical stress on the plane with a coefficient of 0.6 The shear stress assumed 20 grade concrete at a construction joint. An allowable stress of $0.3N/mm^2$ was adopted with a factor of 0.5 to allow for stress concentrations. During construction horizontal lift joints were stepped to avoid a single plane of weakness and so the approach taken is clearly conservative.

12. In case (c) vertical stresses at both upstream and downstream faces were derived from the direct forces and moments. It was assumed that the structure behaved elastically and that plane sections remained plane.

13. For assessment of the seismic loading the dam was classified in terms of reservoir capacity, height, evacuation requirements and potential downstream damage. This placed the dam in category III. Examination of the site location placed the dam in zone B. A peak ground acceleration (PGA) of 0.20g was adopted. Horizontal acceleration was taken as full PGA and vertical acceleration as two-thirds PGA. Two load cases were examined to cater for the possibility of simultaneous horizontal and vertical accelerations. These were maximum horizontal acceleration with 40% vertical, and maximum vertical with 40% horizontal. Calculations were based on a pseudo-static analysis only. It was assumed that the seismic load would occur with the reservoir full but not spilling.

RESULTS AND INTERPRETATION

14. Acceptance criteria were agreed with the Qualified Engineer. These values were derived from code recommendations, theory and previous experience. Values are shown in Table 1. Summarised results of the analysis are shown in Table 2. These show that three sections failed to meet the agreed criteria. These sections were the north and south gravity sections and the north control tower.

15. The failure of the control tower was marginal and installation of anchors in this section would be very difficult and expensive. The design assumptions were therefore re-examined. Since the assumption on uplift pressures was conservative it was decided to install piezometers during the site investigation to justify the adoption of a modified uplift pressure.

16. For the other two sections it was decided that remedial works would be required.

SITE INVESTIGATION

17. A site investigation was planned for the winter of 1992-93. The prime objective of this investigation was to confirm foundation levels and establish the underlying rock quality. Hydro-Electric retain a consultant geologist and he was appointed to advise on rock testing and interpretation of results.

18. Eight boreholes were sunk - four each in the north and south gravity sections. The boreholes varied between a minimum of 5.5m and a maximum of 14.5m apart and were all located on the centreline of the dam. All boreholes were cored throughout with full core recovery. Borehole depths varied from 26m to 44m.

19. All boreholes showed fresh and unweathered rock at the junction between rock and concrete. Rock was generally a hard, fine-grained mica-schist. There were veins of white quartz at irregular intervals. In two of the boreholes a dense, fine-grained, hard igneous rock occurred as a parallel sided intrusion lying within the cleavage plane.

20. Logging, sampling and testing were carried out in accordance with BS5930 : 1981 (Ref.3) and a colour photographic record of the cores was taken. Nine different laboratory tests were carried out on sixteen samples. These included rock quality designation index, unconfined compression strength, point load test, moisture content, bulk density, porosity, sulphate content, slake durability and REDOX potential. Test results are summarised in Table 3.

21. Cores of the dam concrete were also tested to establish concrete properties. During construction two classes of concrete were used. Facing concrete had a higher cement content for increased durability and lower porosity. Hearting concrete had a lower cement content to reduce heat of hydration and hence thermal cracking. Core samples were taken from both zones. Samples were tested for compressive strength, density, indirect tensile strength and thermal expansion. Results are summarised in Table 4.

22. A survey of joints in exposed rock cuttings near the dam was also carried out and a stereographic projection of joints and cleavage planes was drawn up.

23. From the borehole core information laboratory tests and joint plane survey it was concluded that the dominant rock type did not vary significantly over the site and that the arrangement of joint planes was such as to make anchorage failure by pull-out extremely unlikely.

PIEZOMETERS

24. During the site investigation two vibrating wire piezometers were installed in the boreholes adjacent to the north tower. These were attached to a data logger which automatically recorded readings at two-hour intervals. Readings were taken over a three month period.

25. The results were plotted in conjunction with upstream and downstream water levels. This confirmed that water pressures on the downstream side were reduced by the grout curtain as anticipated. A revised uplift pressure distribution is shown in Figure 3. The stability of the north tower was assessed using the revised uplift pressure and was found to be satisfactory. The uplift pressure was not revised for any other part of the structure.

FIG 3
REVISED UPLIFT PRESSURE

ANCHOR DESIGN

26. A preliminary layout of anchors was determined to provide the restoring forces calculated from the stability analysis. This layout comprised 17 No. 220 tonne anchors on the north side and 21 No. 195 tonne anchors on the south side.

27. Anchors were designed in accordance with BS8081 Ground Anchorages - Ref. 4. The depth of anchorage required was determined from the "weight of cone" analogy using a 60° cone. The fixed anchor design assumed that the load capacity was dependent on grout bond to the borehole shaft (Type A). Based on the UCS results, a bond stress of 1N/mm^2 was adopted.

28. In view of the dependence on proprietory products a performance specification was prepared with the final anchor design and layout to be undertaken by the contractors. A maximum bearing stress of 10N/mm^2 under the anchor bearing plate was specified. This was necessary to avoid the need for additional reinforcement and hence excessive concrete excavation on a highly restricted site.

29. A double corrosion protection system was specified. This utilises two PVC barriers with a corrosion inhibiting grease. The contract included for a preliminary test anchor to prove the design assumptions. This anchor was to be installed at a site close to the dam and tested to 80% of ultimate load over a series of eight load cycles.

OTHER INVESTIGATIONS

30. To complete the design study three further investigations were carried out:

1 Check on reinforced concrete element within the towers.
2 Check on failure of dam crest parapets.
3 Check on operation of flood gates at extreme water levels.

31. The main stability analysis assessed the various sections of the dam as gravity structures. Checks were also carried out on the relatively thin walls and floors of the towers. For most of these reinforced concrete elements the most extreme load condition occurred at a water level just before the openings in towers were overtopped and they would fill with water. All these elements were found to be satisfactory.

32. The calculated PMF level assumed that parapets on the dam crest would fail. A higher water level would occur if these remained intact. Calculations were carried out to determine the level at which failure of the parapets was anticipated. This confirmed that the water level immediately before failure of the parapets was not critical. Flooding of the drum gate control towers would occur at a level significantly below PMF. An investigation was carried out to determine the effects of this on gate operation.

33. It was concluded that with the existing arrangement the floatation chambers would fill with water causing the drum gate to rise suddenly and a potentially catastrohpic rise in water level. Waterproof doors are to be installed in the control towers to avoid this situation arising.

PUBLIC RELATIONS

34. Pitlochry Dam is a popular tourist attraction. There is an observation chamber to allow visitors to view salmon passing through the fish ladder and there is a visitor centre and exhibition at the power station. In excess of 150,000 visitors cross the dam each year. In view of this the works were carefully planned to avoid the tourist season and minimise disruption to the public. The site investigation was therefore programmed for November and December 1992. Installation of anchors was planned for the period October 1993 to March 1994.

35. During the site investigation the dam was closed from Monday to Friday throughout the contract period. Heavy lifting causing closure of the access paths was carried out early in the morning. Drilling rigs were removed each weekend and the access routes cleaned and reinstated to allow public access across the dam. The more major works during anchor installation precluded this approach. For public safety the dam is closed completely during these works.

36. Extensive consultations took place with interested parties prior to commencement of both phases of the work. Notices were also placed in the local press advising of the closures. A brochure was prepared giving information on the works and details of alternative access routes and copies were issued locally. Notice boards were also erected at both ends of the site giving similar information.

37. Loch Faskally and the River Tummel are important waters for fishing. Special anti-pollution measures were specified for both the site investigation and the anchor installation. Settling tanks were required for drilling arisings to avoid contaminated water being discharged. Drip and absorbent pads were also required for plant items to avoid oil spillage.

ANCHOR INSTALLATION WORKS

38. Tenders were issued for the anchor installation in August 1993. A contract was placed with Morrison Construction with Bachy as specialist subcontractor for anchor installation.

39. Work commenced on site on 18 October 1993. The test anchor was installed by 22 October and testing commenced on 29 October. The behaviour of the test anchor was consistent with the predictions. At the time of submitting the paper excavation of anchor pits was just commencing.

Table 1. Acceptance Criteria

Load Case	Overturning FoS	Tension (N/mm^2)
PMF	1.00	-0.07
1:10,000yr	1.05	-0.04
Seismic	1.00	-0.07

Table 2. Results for Right Hand Gravity Section

	Overturning FoS	Tension (N/mm^2)
Before anchoring	0.76	-0.43
After anchoring	1.43	-0.06

Table 3. Typical Rock Properties

Unconfined Compressive Strength	30 MN/m^2
Moisture Content	0.3%
Bulk Density	2.8 Mg/m^3
Stone Durability	98.5%

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Maximum reservoir water levels

C.W. ANDERSON, Senior Lecturer, Department of Probability & Statistics, University of Sheffield, I.J. DWYER, Mathematical Modeller, Institute of Hydrology, Wallingford, S. NADARAJAH, Research Student, Department of Probability & Statistics, University of Sheffield, D.W. REED, Flood & Storm Hazard Section Head, Institute of Hydrology, Wallingford, J.A. TAWN, Senior Lecturer, Department of Mathematics, University of Lancaster

SYNOPSIS. The paper presents the first end-results of the DOE-funded Joint Probability Studies for Reservoir Flood Safety. The methodology combines new techniques for the statistical analysis of multivariate extremes with the use of composite mathematical models of a catchment-reservoir system. Submodels describe precipitation form, snowmelt, catchment runoff, reservoir routing, wave generation and wave run-up. As an example of results, the sensitivity of the T-year maximum water level at an impounding reservoir is explored with respect to such factors as reservoir size, operating policy, and lake orientation. The paper closes with an illustration of the influence that dependence between extremes of wind and rainfall can have on safety assessments for dams exposed to wave attack.

INTRODUCTION

1. The principal aim of the Joint Probability Studies for Reservoir Flood Safety is to appraise the present rules for combining design inputs in reservoir flood safety assessment. These are largely set by Table 1 of the ICE guide to Floods and Reservoir Safety (ICE, 1989). The general approach being taken in this DOE-funded research is introduced by Reed & Anderson (1992), and some of the statistical issues are amplified by Anderson & Nadarajah (1993). After introducing the study catchment, the current paper describes the model being used to simulate reservoir water level, and presents the first end-results from the project.

BLACK ESK RESERVOIR

The dam

2. Black Esk reservoir impounds one of the main tributaries of the river Esk, in south-west Scotland (Fig. 1). It is a water-supply reservoir, owned by Dumfries & Galloway Regional Council. Completed in 1962, the dam is a 23 m high earthen embankment with a crest length of 291 m. Protection against wave surcharge is provided by a 1.07 m wave-wall, with crest some 1.4 m above the top of the rolled-fill core and a total of 3.5 m above the spillweir crest. Slope protection on the 1:4 upstream face comprises squared hand-set pitching.

3. Overflow from the reservoir discharges through a

Fig. 1 Black Esk reservoir

dropshaft spillway with a rated capacity of about 170 m^3s^{-1} at a water level of 1.6 m above the bellmouth spillweir. Model tests indicate that the spillway begins to "gorge" thereafter.

4. At spillweir crest level, the reservoir lake extends over 0.33 km^2. The dam is exposed to wave attack principally from the north, with fetch lengths of up to 1400 m.

The catchment

5. Excluding the lake, the natural drainage area is some 18.0 km^2. The catchment is almost entirely forested, and there are no catchwaters. Although relatively close to the Solway Firth, the altitude range from 247.5 m at the reservoir spillweir to 536 m at Jock's Shoulder presents a moderately severe climate, with frequent snowfall in winter and spring.

Eskdalemuir Observatory

6. The Met. Office maintains a premier climate observatory at Eskdalemuir, with detailed records commencing in 1911. The observatory is 5 km north-east of the catchment centroid and lies at an altitude of about 250 m. Save for typical lapse rates with altitude, meteorological data from the station can be taken to be typical of the Black Esk catchment. For the 1941-1970 standard period, the estimated average annual rainfall for the catchment is 1700 mm, compared to a gauged value of 1506 mm at the observatory. The rainfall regime at Eskdalemuir is characterized by a marked annual cycle, with typically rather higher monthly falls in August to January than in February to July.

7. Precipitation totals, mean wind speeds, mean wind directions, and mean air temperatures were provided at an hourly interval for the period 1970-1989. The air temperature data were generally of excellent quality. The precipitation data required extensive checks but, after apportioning or estimating aggregated or missing observations, it was possible to derive a continuous series for the 17-year period 1970-1986. However, the wind speed data were generally less satisfactory, due to the introduction of a biased sensing and processing

system for wind speed data in March 1981, and to the progressive encroachment of Eskdalemuir Forest on the Observatory. Both factors act to reduce the wind speeds recorded in later years.

Design flood calculations

8. The consequences of overtopping are such that the Probable Maximum Flood is the appropriate design standard for Black Esk reservoir, according to Table 1 of the ICE guide. Conventional calculations using the Micro-FSR computer package indicate that the dropshaft spillway is sized to meet this standard.

9. Estimation of lesser floods using the Flood Studies Report (FSR) rainfall-runoff method (NERC, 1975) - as updated by Flood Studies Supplementary Report No. 16 (Institute of Hydrology, 1985) - indicates, for example, a 1000-year peak discharge of 57 m^3s^{-1}, corresponding to a stillwater level of 0.75 m above the spillweir crest. The method assumes an 11.5-hour design storm with a bell-shaped temporal profile; the maximum water level is indicated to occur about 1.5 hours before the end of the storm.

SUMMARY OF MODEL

General requirement

10. It is known that, in addition to extreme rainfall, factors such as snowfall, soil moisture and reservoir fullness *prior* to the event, and air temperature and wind speed *during* the event, can influence the size of the flood entering the reservoir; air temperature and wind speed are relevant through their effect on snowmelt rate. In addition, wind speed and direction, concurrent with the passage of the flood through the reservoir, can influence the total water height running up against the dam.

11. In order to investigate the sensitivity of maximum water level to these various input variables and initial conditions (Fig. 2), it is necessary to construct a model capable of representing all these factors. The model comprises a number of submodels of precipitation form (i.e. rain or snow), snow-melt, catchment runoff, reservoir routing, wave formation and wave run-up. The aim was to choose reasonable but, where possible, simple submodels.

Precipitation form

12. Snowfall may be retained on a catchment for many hours, days or weeks, and it is therefore important to distinguish snow and rain if the joint dependence of flood runoff on precipitation and air temperature is to be studied. The adopted model assumes that precipitation falls as snow when the air temperature is below 0.0 °C, as rain when the air temperature is above 2.2 °C, and as a mixture of the two at intermediate temperatures; in the last case, the proportion occurring as rain is taken to be the ratio of the air temperature to 2.2 °C.

13. Whereas precipitation occurring as rain is input to the catchment runoff model directly, that occurring as snow enters

Fig. 2 Factors relevant to reservoir flood safety

the snowpack, on which the snowmelt submodel operates.

14. Air temperature is influenced by altitude, generally decreasing at higher altitude. As an adjoint to the precipitation form and snowmelt submodels, a lapse rate of 0.9°C per 100 m is assumed and the catchment zoned into 100 m altitude bands, so that spatial variations in snowpack depth and snowmelt are represented.

Snowmelt

15. Although air temperature variations are largely responsible for the onset of snowmelt, the rate of melt is strongly influenced by other factors, notably wind speed. (See Reed & Field, 1992, for a review of snowmelt processes.) The snowmelt formula adopted is a temperature excess model, modified by wind speed:

$$M = (1.0 + 0.5\ U)\ T \qquad (1)$$

where M is the melt rate in mm h^{-1}, U the surface wind speed in m s^{-1}, and T the air temperature in °C. Surface wind speed typically increases with altitude and a wind speed growth rate of 0.64 m s^{-1} per 100 m is assumed when assessing the snowmelt contribution from each altitude zone. The aggregate snowmelt augments any rainfall entering the catchment runoff model.

Catchment runoff

16. Because antecedent wetness conditions are known to be relevant to flood formation, the catchment runoff model must be capable of simulating soil moisture and runoff continuously, as opposed to just for isolated storm events. Of models meeting this requirement, a relatively simple probability-distributed model (e.g. Moore, 1985) is adopted. This represents spatial variations in soil moisture implicitly, by a statistical distribution, rather than explicitly, by a grid-based distributed model (e.g. Chander & Fattorelli, 1991; Moore &

Fig. 3 Catchment runoff model

Bell, 1991). The structure of the catchment runoff model is outlined in Fig. 3.

17. The soil moisture store has two parameters: a maximum store capacity, c_{max}, expressed in mm, and a distribution shape parameter, n, where the cumulative distribution of soil moisture capacity across the catchment is given by the generalized Pareto distribution:

$$F(c) = Pr(C \leq c) = 1 - (1 - c/c_{max})^n \ . \qquad (2)$$

Thus the maximum soil moisture content averaged across the catchment is given by:

$$S_{max} = \int_0^{c_{max}} c f(c) dc = c_{max}/(n+1) \ , \text{ where } f(c) = dF/dc \ . \qquad (3)$$

18. Soil moisture, S, is augmented by rain and snowmelt, and depleted by evapotranspiration, e, and drainage, d. Any rain or snowmelt in excess of the soil moisture capacity produces direct runoff, q_d, which is passed through a runoff-routing element to yield response runoff, q_r. The sum of response runoff and baseflow, q_b, forms the total runoff, q. In most applications of the probability-distributed model, a groundwater store intervenes between drainage from the soil store and baseflow discharge to the river. However, where soils are generally shallow and the water table is seldom far from the surface, and where baseflow forms a relatively minor component of total runoff, the groundwater store can be omitted and the soil moisture store allowed to drain directly to baseflow, so that q_b=d.

19. The rate of drainage from the soil moisture store is in proportion to content, i.e.:

$$d = \alpha_s S \ , \qquad (4)$$

where $\log_e 2/\alpha_s$ defines the half-life of the soil moisture recession under drainage alone.

20. Evapotranspiration from the soil store is modelled by:

$$e = e_p \ [1 - \exp(-6.68 \ S/S_{max})] \ , \qquad (5)$$

where e_p denotes potential evaporation. The term in square

brackets is sometimes called the "rate function"; its purpose is to limit the rate of evapotranspiration when the soil moisture falls well short of its maximum value, S_{max}. It is a relatively simple model, and eqn. 5 is not especially tailored to the heavily forested Black Esk catchment.

21. Although other factors are known to affect the potential evaporation, it suffices in many applications to represent e_p by a sine wave according to time of year:

$$e_p = a\ [1 + \sin(2\pi J/365 - \theta)]\ , \qquad (6)$$

where J is the ordinal day. The amplitude and phase parameters, a and θ, are determined by reference to long-term average values of e_p estimated for a nearby climate station.

22. The final element of the catchment runoff model relates to runoff routing. In this application, a cascade of two identical "linear reservoirs" was adopted. The rate of discharge from each is in proportion to content. If the constant of proportionality is denoted by α_r, the mean lag time of the cascade is $2/\alpha_r$.

Reservoir routing

23. Runoff from the catchment passes into the (real) reservoir and is either impounded (if the reservoir is drawn down) or stored temporarily, while finding its way over the spillweir. The usual method of "level-pool" routing is applied, the ROUTER implementation of Reed & Field (1992) being preferred. The storage characteristics of the reservoir are defined in terms of the lake area at spillweir level, and the area growth rate thereabove. The discharge characteristics are required either in equation form or as a series of tabulated values; the latter often derive from scale-model testing.

24. ROUTER allows rain falling on the reservoir lake area to be treated explicitly, so that it is not unrealistically diminished by infiltration or delayed by routing.

25. When carrying out long-term simulation of reservoir water levels, it is necessary to know the mean draw-off to supply. In special cases it might be necessary to represent reservoir operating policy in greater detail.

Wave formation

26. The ICE guide to Floods and Reservoir Safety (ICE, 1989) recommends use of the SMB/Saville method to estimate significant wave height due to wind action. The method has two particular features. First, in design use, the effective fetch (i.e. the straight-line distance over which the wind can act to generate waves) is calculated as an average over a 90° arc centred on the longest fetch-line to the dam. Second, in simulation use, the wave train is assumed to form in the direction of the wind. However, the method has fallen from favour among hydraulicians (e.g. Herbert, Lovenbury, Allsop & Reader, 1993), who have sought to explain observations to the contrary, and to confirm suspicions that the SMB/Saville method systematically underestimates wave formation for some shoreline configurations.

27. The Donelan/JONSWAP method is therefore used as an alternative. In Donelan's method, the wave train is assumed to

form, not in the direction of the wind, but in the "predominant wave direction". This is defined as the direction that produces the largest peak wave period, T_p. Using the standard JONSWAP (Hasselmann *et al.*, 1973) formula, this is achieved for the ϕ that maximizes $F_\phi^{0.3}\cos^{0.4}(\theta_w - \phi)$, where F_ϕ is the longest fetch to the dam in the direction ϕ, and θ_w is the incident wind direction. Having carried out the maximization, the relevant peak wave period is:

$$T_p = 0.0712\ F_\phi^{0.3}\cos^{0.4}(\theta_w - \phi)\ . \qquad (7)$$

Effective fetch thus calculated for each wind direction at Black Esk is included in Appendix 1. Herbert *et al.* (1993) advise on effective fetch calculations for bowed lakes.

28. The significant wave height, H_s, is then estimated from:

$$H_s = 0.00178\ U\ F_\phi^{0.5}\cos(\theta_w - \phi)\ g^{-0.5}\ , \qquad (8)$$

where U is wind speed in m s^{-1}, F_ϕ is in m, and g is the gravitational constant (9.81 m s^{-2}).

Wave run-up

29. The ratio of significant wave run-up, R_{us}, to H_s is estimated using empirical relationships given by CIRIA/CUR (1991, beware misprint in their eqn. 5.15):

$$R_{us}/H_s = \begin{cases} 1.35\ Ir_p & \text{for } 0 < Ir_p \leq 2 \\ 3.0 - 0.15\ Ir_p & \text{for } 2 < Ir_p \end{cases} \qquad (9)$$

where

$$Ir_p = s_p^{0.5}\tan(\alpha) \qquad (10)$$

is the Iribarren number. Here α denotes the slope angle of the upstream face of the dam, and s_p is the effective wave steepness. The latter is calculated from:

$$s_p = 2\pi H_s\ /\ (g\ T_p^2)\ . \qquad (11)$$

30. According to CIRIA/CUR (1991), the phenomenon of *greater* run-up for oblique attack is absent for short-crested waves running up a 1:4 smooth slope. The model therefore assumes modest reduction factors of 0.975 to 0.8 for oblique attack. A further reduction factor of 0.9 is applied to allow for the particular pitched slope protection used at Black Esk.

Catchment runoff parameter values

31. As is the case for most UK reservoirs, no detailed runoff data are available for Black Esk reservoir, nor is any tributary formally gauged. Consequently there is little information by which to calibrate the catchment runoff model. In contrast to the simpler unit hydrograph/losses model, there is only limited practical experience of relating parameters of the probability-distributed model to physical characteristics of catchments.

32. As an alternative, the catchment runoff model was calibrated (Bonvoisin & Boorman, 1992) for an experimental site in mid-Wales (the 10.5 km^2 Wye at Cefn Brwyn) and transplanted to Black Esk. The probability-distributed model expresses runoff in units of mm h^{-1}; consequently, it is a simple matter to convert these to reservoir inflows in m^3s^{-1} using the catchment area appropriate to Black Esk (i.e. 18.0 km^2). In marked contrast to Black Esk, the Wye catchment is largely unforested.

33. Because the goal of the research is to explore joint probability aspects of flood formation (i.e. to examine sensitivities to multiple inputs), rather than to estimate specific design floods for the particular reservoir, the substitution seems reasonable.

34. Appendix 1 summarizes the parameter values and characteristics for the Black Esk catchment-reservoir system. The sensitivity of the maximum reservoir water level to the orientation of the lake, and to the average draw-off, is explored later by varying particular characteristics.

SUMMARY OF SIMULATION PROCEDURE

35. The analysis of environmental extremes provides a rich source of multivariate problems in which considerable progress has been made in recent years (e.g. Coles & Tawn, 1994). The selection of events for analysis - sometimes referred to as "declustering" - is in itself a complication, since the relative importance of the different variables is intimately linked to the particular system being analysed. The assessment and treatment of *dependence* in the extremes of different variables form another crucial topic described in the main study report (Reed, Anderson, Tawn, Nadarajah & Dwyer, 1994).

36. Fundamental to the simulation of maximum reservoir water levels is the generation of periods of wild weather, loosely termed storm events. In the application outlined by Reed & Anderson (1992), extremal models are used to generate values of the *primary* (rain, snowmelt and wind speed) and *secondary* (catchment wetness, reservoir fullness, wind direction and storm profile) *input variables*, which collectively define the storm event. The catchment-reservoir model previously described is then used to evaluate the *output variable*, which corresponds to the maximum reservoir water level.

37. For the primary input variables, the generation of storm events takes explicit account of the trivariate dependence through general extremal models. Because of their prime importance, the generation of extreme values of a primary variable derives from a marginal (i.e. univariate) distribution that exploits regional or longer-term data. That the models of dependence cannot likewise be insulated from peculiarities of the 17-year Eskdalemuir record, is a limitation of the procedure, discussed later.

38. Values of the secondary variables are generated conditional on those of the primary variables. In the case of the secondary variables, the generation is based on the empirical distribution of antecedent values for storms drawn from continuous simulation of the catchment-reservoir system over the specific study period (i.e. 1970-1986).

39. The declustering analysis defined 1855 storms in the study period. The basic approach to estimating the frequency distribution of the output variable is to generate storm events at the designated rate (i.e. about 109 per year) and to carry out an extreme value analysis of the resultant maximum water levels. However, "importance sampling" (Hammersley &

Handscomb, 1964) is used to economize on the number of event simulations needed, while still defining the required segment of the frequency relationship.

40. Analyses to date define a storm as an extreme 11-hour rainfall total, an extreme 11-hour snowmelt total, or an extreme 1-hour mean wind speed. The FSR rainfall-runoff design procedure supports the choice of an 11 hour "design duration" for Black Esk. Although it is known that upland reservoirs sometimes respond to extreme winds over shorter periods (Herbert *et al.*, 1993), the choice of one hour is judged reasonable given the unavailability of long-term wind speed data at finer time resolution.

RESULTS

41. Some example results from the simulation procedure are shown in Figs. 4 to 7. These are intended to be illustrative rather than definitive; it should be recalled that the catchment runoff model employed has not been explicitly calibrated to Black Esk, and that the Donelan-JONSWAP method has been used to estimate wave heights.

42. The solid frequency curve in Fig. 4 estimates the T-year maximum stillwater level for the basic configuration at Black Esk (i.e. draw-off rate of 30 Ml day^{-1}, 0.33 km^2 lake area, longest fetch from north). The broken line shows the frequency relationship assuming zero draw-off, i.e. if the reservoir is deliberately kept full. It is seen that the change in policy has little effect on maximum stillwater levels for return periods typical in reservoir flood safety assessment. Also shown are results for a *doubled* reservoir (0.66 km^2 lake area and 8% growth per metre rise). The maximum water levels are lower than for the basic configuration because of the greater attenuating effect of the larger lake.

43. Figure 5 differs in that the *total* water level is now shown, i.e. including wave run-up. Wind-induced waves are seen to contribute significantly, even though the dam is not especially exposed to the prevailing wind. It is interesting that the maximum water levels are now *greater* for the doubled reservoir, the higher waves (from the $\sqrt{2}$-longer fetches) outweighing the attenuating effect of the larger lake.

44. Figure 6 illustrates the effect of rotating the reservoir through 120° so that the dam is fully exposed to the south-west, the direction from which high winds are most frequent at Eskdalemuir. The maximum water levels are seen to be rather sensitive to the re-orientation, the return period of a given water level being approximately halved.

45. Finally, Fig. 7 illustrates the sensitivity of the basic configuration to the degree of dependence in the extremes of wind speed and rainfall. Were the true degree of dependence a little more marked - e.g. equivalent to 2.79 independent variables rather than the 2.91 independent variables observed in the 1970-1986 period at Eskdalemuir - estimated T-year maximum water levels would be higher.

Fig. 4 Frequency distribution of maximum stillwater level

Fig. 5 Frequency distribution of maximum water level (inc. wave run-up)

Fig. 6 Sensitivity of maximum water level frequency (inc. wave run-up) to reservoir orientation

Fig. 7 Sensitivity of maximum water level frequency (inc. wave run-up) to dependence in input factors

CONCLUDING REMARKS

46. The Joint Probability Studies are exploring the use of multivariate extreme value analysis techniques to review design inputs for reservoir flood safety appraisal. The paper presents some preliminary results from the project. Estimation of the degree of dependence between the environmental extremes is relevant to assessing T-year maximum reservoir water levels, particularly for dams exposed to wave attack.

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APPENDIX 1 CATCHMENT CHARACTERISTICS AND MODEL PARAMETERS

Case study

Black Esk reservoir, south-west Scotland

[Not all values are definitive; see text for qualifications]

Catchment sub-areas and mean altitudes

Altitude range (m)	Mean altitude (m)	Prop^n of catch^t area (exc. lake)
< 300	272	.26
300 to 400	350	.66
400 to 500	450	.07
> 500	518	.01

Altitudinal variations

Air temperature lapse rate: 0.9°C per 100 m

Wind speed growth rate: 0.64 m s^{-1} per 100 m

Catchment runoff model parameters

Maximum soil moisture storage capacity, c_{max}: 200 mm

Shape parameter in dist^n of soil store capacities, n: 1.1

∴ Mean max. soil moisture content across catch^t, S_{max}: 95.2 mm

Soil store drainage coefficient, α_s: 0.0041 h^{-1}, corresponding to a recession half-life of 7 days

Runoff-routing drainage coeff^t, α_r: 0.333 h^{-1} for each linear reservoir, correspond^g to a lag time of 2 h (for the pair)

Reservoir lake

Spillweir level: 247.5 m AOD

Area at spillweir level: 0.33 km^2

Area growth rate above spillweir level: 0.027 km^2/m, or 8%/m

Reservoir use

Mean drawoff rate: 30 Ml day^{-1}

Spillway level-discharge relation

Level (m)	Discharge (m^3s^{-1})	Level (m)	Discharge (m^3s^{-1})	Level (m)	Discharge (m^3s^{-1})
0.000	0.0	0.613	42.5	1.622	167.1
0.122	5.7	0.744	56.6	1.667	169.9
0.204	9.9	0.991	85.0	1.743	172.8
0.274	14.2	1.207	113.3	1.875	174.2
0.358	19.8	1.408	141.6	2.134	175.7
0.469	28.3	1.554	158.6	2.438	177.9

Fetch lengths to dam

Incident wind direction, θ_w	Predominant wave direction, ϕ	Effective fetch length, F_ϕ
°	°	m
320 to 60	360	1335
70 & 80	10	943
240	250	246
250	260	252
260	270	268
270	300	349
280 & 290	330	670
300 & 310	350	1131

The construction and performance of a wedge block spillway at Brushes Clough Reservoir

R. BAKER, BSc, PhD, CEng, MICE, MIWEM, Hydraulics Lecturer, University of Salford, Department of Civil Engineering and Construction, and K. GARDINER, BSc, CEng, MICE, Senior Engineer, North West Water Engineering

SYNOPSIS. Wedge shaped pre-cast concrete blocks use the hydrodynamics of the flow to enhance their stability and allow a low cost lining for dam spillways. North West Water (NWW) have utilized this technology on a spillway at Brushes Clough Reservoir, the first of its type in the United Kingdom. The procedures adopted to design and construct the spillway are described in the paper along with details of test equipment installed by CIRIA to monitor the performance of the blocks.

INTRODUCTION

1. Wedge shaped blocks form a stepped spillway surface. As the flow passes over each step it separates from the block and re-attaches to the surface part-way along the next block downstream. In the separation zone in the lee of the step the water rotates in the vertical plane generating a low pressure core. A series of holes or slots through the block connect this low pressure zone to an underdrain which evacuates seepage flows and effectively sucks the block onto the embankment, Hewlett & Baker (1992).

2. The principle of the wedge block was conceived in Russia in the 1960's and has been in use on some Russian dams since 1976. In 1988 the Construction Industry Research and Information Association (CIRIA) embarked upon a literature review of Russian laboratory and prototype data, Bramley et. al. (1989), and large scale model testing, Baker (1991), to bring the technology to the West. This work resulted in the publication of a design guide for the construction of stepped block spillways, Hewlett et. al. (1993).

BRUSHES CLOUGH RESERVOIR

3. Brushes Clough reservoir is located in the hills above the town of Shaw, near Oldham. The earth embankment with a puddle clay core was constructed between 1856 and 1860. The reservoir has a catchment area of 1.2km^2 generating a PMF inflow of 31.4 m^3/s attenuated to give a peak outflow of 10.9m^3/s, which corresponded to a head over the existing masonry weir of 5.06m. Adding the wave surcharge, the freeboard required to pass the PMF was 5.66m. The freeboard available was 1.24m. At TWL, before the recent works were carried out, the reservoir had an area of 1.5ha and a capacity of

Reservoir safety and environment. Thomas Telford, London, 1994

93Ml, with a maximum water depth of approximately 13m.

4. The Inspecting Engineer's Report, in 1988, recommended that the overflow capacity be increased to meet the requirements of a Category A reservoir and that extensive remedial works be carried out to the valve, drawoff and scour arrangements. Rather than carry out the remedial works it was decided that the reservoir should be taken out of the remit of the Reservoirs Act by reducing its capacity to below 25Ml. To achieve this, the crest level of the earth dam was lowered by 9m and the spoil deposited in the reservoir basin. A new overflow was constructed across the new crest and down the 1 in 3 downstream face of the embankment to pass floods up to the 1 in 50 year event (3.66 m^3/s). Floods with a longer return period will flow over the entire width of the embankment crest.

ORIGINAL SPILLWAY DESIGN

5. The design of the 35m wide breach in the embankment and the new overflow works was completed in December 1991. The base of the sloping channel consisted of 1m high steps formed from articulating reinforced concrete units each provided with a large shear key into the soft foundation material to prevent sliding, Fig 1.

Fig. 1 Profile of the original design with the wedge block alternative shown as an inset at the same scale.

THE WEDGE BLOCK ALTERNATIVE

6. In January 1992 the draft version of CIRIA's wedge block design guide was published, Hewlett et al. (1993), and NWW Engineering recognized that this form of construction was likely to be cheaper than the original, conventional, design. An alternative design was prepared using wedge blocks, Fig 2, and the tendering contractors were asked to submit a price for both the original and the new designs. All other works, including the stilling basin and the crest channel, were common to both options.

Fig 2 Profile of the wedge block channel

7. The winning tenderer submitted a price of £63K for the RC step spillway, and £43.5K for the wedge block alternative, representing a 30% saving. The prices submitted by the other tenderers showed very similar savings. The price for each block was £17.

CONSTRUCTION DETAILS

8. The overflow channel across the breach is trapezoidal in cross-section, 1.6m wide across the base and 1.5m deep with 1 in 1 sideslopes. The gradient is approximately 1 in 1000. The channel is constructed from pitching reclaimed from the upstream face of the dam laid in no-fines concrete on a layer of waterproof sheeting to prevent seepage into the embankment. A weir was constructed at the upstream end of the channel using the weir blocks from the old overflow.

9. The trapezoidal spillway, 2m wide with 1 in 2 sideslopes, was constructed at a gradient of 1 in 3 on the downstream face of the dam. At the toe, a large anchor block was constructed in reinforced concrete cast against undisturbed ground. The hydraulic jump which will form when the flow in the spillway enters the stilling basin, would have a disruptive effect on wedge blocks if they were laid in this area. Therefore, a series of ten reinforced concrete steps were cast on the anchor block to the same shape as those to be formed by the wedge blocks higher up, Fig 5.

10. The formation for the spillway channel was then excavated and precast edge beams were laid to line and level on a no-fines concrete bed, starting from the bottom of the slope. These provided a guide for subsequent operations. Fibretex F-3200S, a needle punched thermic bonded polypropylene geotextile, was laid on the trimmed formation followed by 200mm of 20mm single sized stone and a layer of Terram PET 42 polyester mesh, Fig 2.

11. At the top of the spillway a reinforced concrete nosing was cast onto the top row of blocks with a polythene membrane to prevent adhesion, Fig 3. A minimum overlap of 525mm was provided to ensure that if the blocks move

down the slope during operation, the leading edge of the top row will be shielded from flow impact.

Fig 3 Crest detail

12. The wedge blocks were lifted into place using the backacter of an excavator. To aid handling, lifting anchors were cast into each block so that it was suspended at a convenient angle. The block was guided into its final position by hand and tamped down with a flagging mall. A spirit level was used to check that the top face of each block was horizontal across the slope and a wooden template to obtain the required angle with the edging blocks.

13. The blocks proved easy to lay. It was not found necessary to slide the blocks on the plastic mesh and no rucking of the mesh occurred.

14. Pitching to the 1 in 2 side slopes was nominally 250mm thick laid on a 150mm bed of no-fines concrete. The pitching operation followed the laying of the wedge blocks up the slope.

ADVANTAGES

15. The advantages of using wedge blocks to construct the spillway at Brushes Clough were considered to be:

(a) Cost. The tenders for the wedge block spillway were 30% cheaper than for the original design, giving a saving of approximately £20,000. On larger spillways the savings are potentially greater.

(b) Factory production. Wedge blocks can be precast off site to a high quality in advance of construction, and delivered as required. This proved to be very convenient on a site with restricted access and storage space.

(c) Fast construction. There was no need to transport and place tonnes of wet concrete on a steep slope with poor access. At Brushes Clough the concrete for the anchor block, the stilling basin and the downstream channel had to be pumped 70m from the crest of the dam 28m above. The blocks were simply transported across the face of the embankment, placed on the

drainage layer and tapped down into position.

(d) Inherent stability. The wedge block system is hydrodynamically stable, the faster the flow, the greater the force holding the blocks onto the slope.

(e) High energy dissipation. The system is hydraulically rough giving safe low velocities. The safety of the general public, especially children, who may stray onto the embankment was a design consideration for both spillway options. A long smooth channel with a vertical drop of 17m would produce very high flow velocities, and if frozen in winter would become a very dangerous slide.

PERCEIVED DISADVANTAGES

16. Possible disadvantages of the wedge block system were also identified:

(a) Susceptibility to vandalism. From the client's point of view, the major fear was the possible susceptibility of the blocks to damage or removal by vandals. The standard block in the Design Guide has been modified by NWW Engineering so that it is very heavy and difficult to remove and the drainage holes are more difficult to obstruct.

(b) Slope stability problems. At low flows the slope beneath the spillway is likely to become fully saturated. The slope stability was checked in this condition and was found to have an adequate factor of safety.

WEDGE BLOCK DESIGN

17. The design of the Wedge blocks was carried out using the rules contained in the draft CIRIA Design Guide although the final shape differed from the standard block in certain respects, Fig 4.

18. The blocks were made almost twice as heavy as the minimum allowed by the guide because of concerns that NWW Operations staff had expressed about possible attack by vandals. Also, the faces of the block were detailed at right angles to the top of the block rather than to the slope, so that the blocks would wedge against each other if an attempt was made to remove them from the completed spillway. It was initially thought that this geometry would cause difficulty in laying the blocks, because they would have to be slid into position, but this did not prove to be a problem on site.

19. The minimum average thickness, t_a, recommended in the draft Design Guide for the 1 in 3 slope and design discharge intensity of 1.525 m^2/s is 100mm; in the NWW design t_a = 212mm. The step height to step length ratio for the block is 4.72, the draft Design Guide recommended that this ratio should be between 4.0 and 6.0 for hydrodynamic stability. The Guide also recommended that the aspect ratio, maximum length to average thickness, should not be greater than 4 for an unreinforced block. The NWW block has an aspect ratio of 3.88. Each full block weighs 120kg.

Fig 4 Details of the NWW wedge block

20. The design of the drainage holes in the block also differs from the recommendations of the Design Guide. It was considered that holes through the block or grooves in the face could easily become clogged either by debris or by the action of vandals. Therefore 127mm × 29mm slots were specified in a relatively long overlap which protected the smaller slots below, Fig 4. Castellated slots were specified in the vertical face with a maximum gap of 18mm. This is smaller than the nominal size of the stone drainage layer on which the blocks would be laid in case the confining layer of plastic mesh was damaged during laying or deteriorates with time. If these slots were to be formed in the downstream face alone casting would be difficult. Therefore, the vertical slots were formed in the upstream face so that when laid they lined up with the slots in the overlap on the block above. The blocks were laid in stretcher bond, requiring half blocks at the ends of every other row.

21. The Design Guide recommended that the area of the drainage slots should be between 2.5% and 5.0% of the exposed surface area. In the NWW block this area is 3.1%.

22. The blocks were made using sulphate resisting cement to increase resistance to chemical attack from the slightly acid water and to achieve the early strength necessary for rapid reuse of the steel moulds. The coarse aggregate was a 20-5 graded limestone and the fine aggregate a 50/50 mixture of limestone dust and washed pit sand. The water/cement ratio was 0.50, and

the cement content 360kg/m³. The specified concrete strength was 40N/mm². Compaction was carried out on a vibrating table.

UNDERDRAIN DESIGN

23. The soil on which the spillway and stilling basin is founded is a brown or grey silty sandy clay fill material, with many sandstone, siltstone and mudstone fragments. The soil has a high uniformity coefficient (ie $D_{60}/D_{10} >> 5$), and therefore has good self filtering properties. It was considered that the soil could be effectively filtered using a single layer of geotextile with an O_{90} of 80μm.

24. As well as preventing fine particles from the underlying soil being washed out at high flows when water is drawn up into the spillway, the underdrain must also provide a free flowing conduit for seepage flow through the drainage slots and joints between the wedge blocks at low flows.. Therefore, a 200mm layer of single sized stone was laid above the geotextile filter. This material is very free draining and has the added advantage of being self compacting thus reducing long term settlement.

25. In addition, a layer of polyester 5mm mesh was laid on top of the stone. This layer was included to prevent the stone becoming trapped in the drainage slots either during the placing of the blocks or by the suction action of the water.

Fig 5 Plan of the stilling basin and underdrain egress pipework with a detail section at the end of the in-situ steps

26. A 100mm perforated clayware pipe wrapped in the geotextile was laid at the downstream end of the underdrain against the in-situ steps, Fig 5. This was done to prevent saturation of the surrounding soil, which might cause local instability. This pipe is connected to two drains which discharge on either side of the stilling basin above the level of the overflow weir. This has the advantage that underdrain flows can be monitored. Underdrain flows above the capacity of this system emerge from the drainage slots in the wedge blocks.

SITE MONITORING

27. Owing to the unique nature of the Brushes Clough spillway, CIRIA awarded a contract to Salford University Civil Engineering Limited (SCEL) to install equipment to monitor flow conditions and movement at the site and to maintain site records up to April 1995.

28. As part of the construction programme NWW financed the installation of a trapezoidal throated venturi flume and stilling chamber located in the channel crossing the breach. An important consideration in the design of the flume was that it should not cause out of channel flow during the 1 in 50 year storm and for this reason a very narrow throat capable of accurately measuring low flow was not used. SCEL have installed a maximum level recorder in the stilling chamber, consisting of a vertically mounted square tube, formed in perspex, with horizontal bulkheads creating 64 cells at 25mm spacing. Each cell has a slot at high level linking it to the main chamber. As the water level ponds behind the flume it fills the stilling chamber and those cells that are submerged. When the flow level recedes, water is trapped in the cells allowing the maximum flow to be ascertained even if it has not occurred at the time of an inspection visit.

29. Two rows of six piezometers were also installed as part of the construction programme as shown on Fig 6. The first row is located about half way down the spillway and the second row is at the toe, 22m and 5.3m respectively from the insitu concrete stilling basin.

Fig 6 Piezometer locations

30. Upon completion of the construction work, SCEL installed a permanent survey station and accurately surveyed the as built spillway. In addition, aluminium plates were fixed to the edge beams with a score mark in line with every tenth step in order to visually detect sliding of the blocks. The site will be re-surveyed after a large storm.

DYNAMIC MEASUREMENTS

31. When the flow reaches significant depth it will be possible to take readings of depth and velocity. Initially it had been hoped to utilise a natural storm for this purpose but experience to date suggests that because the Brushes Clough catchment is very small, run-off occurs too quickly to mobilize and set up equipment. Investigation will thus be carried out on artificially created flow surges.

32. SCEL have manufactured a special instrument bridge to span the channel, adapted from an aluminium ladder. For safety reasons all equipment will be operated remotely from the bank. In order to measure depths in the highly turbulent ill-defined water surface, SCEL have developed an electronic probe that detects the percentage of time that the tip is in contact with water. An electric motor drives the tip up or down until a pre-set contact period (say, 50%) is obtained.

OBSERVATIONS

33. The spillway first came into use in early April 1993. Since then it has primarily been subjected to low flows up to 20 l/s. The peak flow recorded at the time of writing (March 1994) was 700 l/s in early January 1994. This was hardly sufficient to generate skimming flow and so the hydrodynamic element of the blocks has not yet been tested. No movement has occurred.

34. Under low flow conditions the water splashes over the edge of each step and gradually makes its way into the underdrain until, near the bottom of the spillway, the channel runs dry. The water then re-appears from the two pipes draining the underdrain and at larger flow rates some issues from the slots in the bottom rows of blocks.

VANDALISM

35. The spillway is close to a public footpath and recreational area and there is open access to the site. Large numbers of children have been observed playing on the site and this has resulted in some damage.

36. At an early stage pipework for three of the piezometers in the bottom row were damaged and this required extensive repair work by SCEL to bring them back into use. The cast iron covers for the piezometer monitoring boxes have also been broken and these have now been replaced by steel plates.

37. Of more significance is damage to the blockwork itself, about 10% of the blocks have significant pieces broken off the face of their step. It is believed that this has been caused deliberately by boulders, present at the site as a

result of the excavation of the dam material, being dropped onto the spillway or rolled down the steps. There is also evidence that motorcycles have been driven up and down the steps.

CONCLUSIONS

38. At Brushes Clough the wedge block system has provided an easy to construct spillway at less cost than a conventional design. The inherent hydrodynamic stability of the block means that very small units can be used and factors like susceptibility to vandalism are more important considerations than flow related failure criteria.

ACKNOWLEDGEMENTS

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Contingency planning for dam failure

J.R. CLAYDON, BSc(Eng), DMS, CEng, FICE, Resources Manager, Yorkshire Water Services, R.A. WALKER, BSc, PhD, CEng, MICE, Principal Engineer, Babtie Shaw & Morton, and A. J. BULMER, MSc, MErgS, Human Factors Analyst, W S Atkins Consultants

SYNOPSIS: Large dams represent a major potential hazard with historical failures having resulted in significant loss of life and damage to property. In order to minimise this risk of failure it is not only essential to have high standards of construction, inspection and maintenance but also to have practical emergency plans in place for dealing with a disaster should it occur. The preparation of emergency plans needs to be approached from both a technical and a human angle. It must cover matters such as the time of arrival, at any location, of the flood wave after a failure, the likely area of inundation and destruction, the duration of inundation, the lines of communication and responsibility between and within emergency services, sources of expert advice and evacuation procedures. This will normally involve more than one agency and the timing of warnings is esential if effective action is to be taken to reduce the impact of the disaster.

The paper describes the technical and human aspects which need to be addressed in emergency planning and suggest a framework in which such plans can be prepared. The authors' experience in undertaking failure studies for Yorkshire Water's reservoirs is used as the basis for the paper.

Under technical aspects the assumed hydrological conditions prior to failure, secondary failures of downstream structures, the impact of dam failure mode on inundation and the influence of assumed valley roughness on the extent of inundation are described.

Under human aspects the paper addresses the requirement for call out procedures for key personnel, access to control centres, clarification of roles and responsibilities, communication requirements and liaison with external agencies.

BACKGROUND

1. Reservoirs, particularly in urban situations, represent potential hazards to the public. A majority of the reservoirs owned by Yorkshire Water (YW) have been placed in category A defined as "Reservoir where a breach will endanger lives in a community". (Institution of Civil Engineers, 1978)

Reservoir safety and environment. Thomas Telford, London, 1994

2. There have been no lives lost due to dam failure in the UK since 1925. However, the notable failures of Bilberry in 1852 and Dale Dyke in 1864 both with large loss of life are still well known in their localities. Both dams were rebuilt and are in use today. As supervising engineer for one of them, the first author is very conscious of the duty to ensure no further failures. High standards of monitoring, supervision, inspection and maintenance are essential to keep the risk of failure at present low levels.

3. There is no relationship between hazard and risk.

Hazard is defined as "the consequences of failure".

Risk is defined as "the probability of failure".

CONTINGENCY PLANS

4. There is no general legal requirement to prepare contingency plans for dam failure. There are comparable hazards to the community which are required to have contingency plans such as nuclear installation. More relevant to the water industry, owners of large chlorine stores are required by the CIMAH Regulations 1984 to have contingency plans for gas escape, to publish them and to undertake emergency exercises.

5. Yorkshire Water's view on the need to prepare contingency plans takes into account criticisms that have been raised in the past following disasters where contingency plans were inadequate. Even though the risk is small it is imperative that the emergency authorities do have these contingency plans available. Reservoir safety has been addressed in the company's emergency procedures since they were first written, with increasing levels of detail and information since implementation of the Reservoirs Act in 1986.

6. In 1988 and 1989 the Water Research Centre commissioned Binnie and Partners to prepare a methodology for preparation of inundation maps. This culminated in a report in 1990 (WRc, 1990), which is still confidential to the signatories to the WRc common interest research agreement.
This report gave detailed requirements for the content and possible format of inundation maps and on the approach and methodology to be adopted in modelling the dam failure and progression of the resulting flood wave downstream.

HAZARD RANKING

7. The WRc report also suggested an approach to deciding the order in which the many dams of one owner might be tackled, based on ranking dams in order of hazard. Various hazard ranking methods were studied and a simple easily applied one was recommended.

8. Hazard ranking has not been adopted by Yorkshire Water. It was decided to prepare contingency plans for all reservoirs covered by the Reservoirs Act. Preparation of a hazard ranking list could cause public concern and is unnecessary once a decision is taken to prepare plans for all reservoirs.

PROGRAMME

9. Three firms of consulting engineers were appointed to prepare contingency plans, each for a different pilot group of reservoirs. There was a free exchange of ideas and information at joint progress meetings from which agreement was reached on technical issues related to dambreak analysis and preparation of inundation maps. Details are given in later sections of this paper.

10. The other part of the pilot work, preparation of contingency plans including organisational and human factors, proved more difficult to agree. Eventually this was resolved by a Quality Action Team consisting of analysts from W.S. Atkins with Yorkshire Water staff from various parts of the company, including reservoir safety, operations, emergency planning and the control room. Details are given in later sections of this paper.

11. After completion of the pilot studies separate commissions were entered into for:

- preparation of inundation maps for 105 reservoirs
- preparation of a general contingency plan for dam failure and a specific one for each dam

12. The programme is due to be completed in 1996. The cost per annum is less than 3% of the annual capital investment in reservoir safety. The amount spent is regarded as a prudent investment in contingency planning. It does not detract in any way from the continuous task of ensuring that the risk of dam failure is minimised.

INUNDATION MAPS

13. In the preparation of inundation plans, information was drawn from publications by WRc (WRc, 1990) and the Department of the Environment (DoE, 1991), from maps prepared by the Ordnance Survey, from rating curves, flood warning levels and washland areas supplied by the National Rivers Authority, Yorkshire Region, from aerial photographs supplied by the Committee for Aerial Photography at Cambridge University, from reservoir inspection reports and flood studies supplied by YW and from photographic records of site visits.

14. The basic procedure used to prepare the inundation maps was that defined in the draft publication by WRc entitled "Reservoir Contingency Planning - Methodology for Preparation of Flood Inundation Maps" (WRc, 1990). This procedure relies on the use of the computer simulation program DAMBRK UK which models a dam breach and the movement of the resulting flood wave down the river valley. Input to this program includes parameters which can be measured with a reasonable degree of accuracy and parameters which cannot and thus have to be estimated. For those parameters which have to be estimated, average values recommended in the program user manual were used in an initial analysis. These parameters were then varied within prescribed ranges during a sensitivity analysis. Based on a comparison of these results, a recommended procedure was proposed and applied to prepare final inundation maps.

15. Information on running the computer program DAMBRK UK was obtained from the Program User Manual (DoE, 1991). The basic data required are as follows:

(a) hydrological details
(b) reservoir details
(c) dam details
(d) breach details
(e) downstream valley cross-sections
(f) valley roughness characteristics

16 Hydrological details. The hydrological details were the inflow hydrographs to the reservoirs. With a view to producing conservative estimates of flood inundation, PMF inflow hydrographs were assumed for the reservoirs.

17 Reservoir and dam details. The reservoir and dam details were obtained from records supplied by YW and entered into the simulation without change.

18. Breach details. The dam breach details, breach side slope and the time of failure can only be estimated from records of past dam failures. Recommended ranges are given in the DAMBRK UK User Manual for a number of dam types. All the dams in the present study were considered as well engineered earth structures and the breach details were established as those which fall midway between the recommended limits, ie:

(a) breach top width = 2 x dam height
(b) breach side slope = 1:1
(c) time of failure = 1.25 hours

19. Downstream valley cross sections. The downstream valley cross-section details were measured from 1:10,000 scale Ordnance Survey Maps. These cross sections were selected at locations of interest and at significant changes in channel topography. The locations of washlands were obtained from the NRA and their storage characteristics were measured from 1:10,000 scale Ordnance Survey Maps.

20. Valley roughness. The valley roughness characteristics, in the form of Manning's roughness coefficients, were estimated on the basis of a recommendation contained in the publication by WRc (WRc, 1990). Model calibration against suitably documented dam failures, including the failure in 1864 of Dale Dike Dam, have suggested a global value of Manning's 'n' in the range between 0.03 and 0.05. The mid value of 0.04 was used in the initial analysis.

21. Sensitivity Analysis. Using the basic data described above, a dam break simulation for a single reservoir was carried out. The results of this dam break simulation formed the basis of a sensitivity analysis which assessed the impact, with respect to the extent of inundation and structural damage, of the following changes;

(a) changes to the valley roughness
(b) changes to the dam failure duration
(c) changes to the spillway overflow capacities
(d) introduction of upstream reservoirs to form a cascade
(e) introduction of secondary damming in the downstream valley

22. For reasons of confidentiality the dams cannot be identified and are referred to in the text by letter only.

23. Variation of Valley Roughness. The results of changing the value of Manning's roughness coefficient, 'n', are summarised in Table 1 for cross-sections 8km, 18km and 36km downstream of Dam A. These three cross-sections represent conditions in the upper, middle and lower reaches of the river valley respectively. In all but the final case the roughness coefficients were constant throughout the river valley. In the final case, termed "variable", the downstream valley was divided into three sections and a coefficient of roughness assigned to each section, thus:

(a) n = 0.04 for the steep valley section from the dam to the outskirts of a major city

(b) n = 0.09 for the densely industrialised valley section through the major city, and

(c) n = 0.05 for the less densely urbanised valley section downstream of the major city

The results are generally in line with expectations, higher values of roughness produce floods which are slower moving and of greater depth and longer duration. Only with the highest value of Manning's coefficient is there evidence that this increased roughness causes sufficient attenuation in the upstream valley to reduce flood levels in the lower river reaches. Not reflected in Table 1 is the fact that higher values of roughness coefficient lead to reduced areas of structural damage both laterally across the valley and longitudinally down the valley.

Table 1. Sensitivity of maximum flood depths (metres) to change in valley roughness

	Manning's roughness co-efficient (n)					
	0.03	0.04	0.05	0.07	0.09	Variable
Maximum depth 8km from dam	4.2	4.6	5.5	6.1	6.5	6.4
Maximum depth 18km from dam	3.8	4.2	4.3	4.6	4.7	4.3
Maximum depth 36km from dam	2.5	2.7	2.8	2.9	2.6	2.7

24. Variation of dam failure duration. The results of doubling the dam failure duration are shown in Table 2. This change has the effect of slightly reducing inundation levels by some 0.7m in the upper reaches of the valley, and by some 0.4m in the middle reaches of the valley. The duration of flooding increases in the upper and lower reaches of the valley but remains essentially unchanged in the middle reaches, due probably to local valley constrictions. Not reflected in the Table is the fact that the extent of structural damage is also reduced.

Table 2. Sensitivity of maximum flood depths (metres) to changes in dam failure duration

	Dam failure duration (hrs)	
	1.25	2.5
Maximum depth 8km from dam	4.6	3.9
Maximum depth 18km from dam	4.2	3.8
Maximum depth 36km from dam	2.7	2.8

25. Effect of reservoir cascades. A comparison of the results of simulating the reservoir cascades is summarised in Table 3. In the simulation each dam in the cascade is induced to fail starting from the uppermost dam. It is evident from the comparison that maximum depths and thereby inundation levels do not vary greatly, certainly not between the multiple reservoir cases. The maximum difference in inundation levels between multiple reservoir cases is of the order of 0.4m in the upper valley reaches and 0.5m in the lower valley reaches. Not reflected in the Table is the fact that the areas of structural damage are very similar for all the multiple reservoir cases. With regard to flood duration and flood arrival times there appears to be few significant differences between the multiple reservoir cases but more significant differences between those cases and the single reservoir case especially in the middle and lower valley reaches.

Table 3. Sensitivity of maximum flood depths (metres) to the failure of upper dams

	Cascade combination		
	A	A + B	A + B + C
Maximum depth 8km from dam A	4.6	5.2	5.0
Maximum depth 18km from dam A	4.2	4.7	4.8
Maximum depth 36km from dam A	2.7	3.2	3.7

26. Effect of secondary damming. At locations along the valley the river under investigation is crossed at intervals by bridges. Minor bridges may be washed out by the floodwave but it is possible that the larger structures, especially those some distance from the dam could become blocked thus creating secondary dams. In the sensitivity analysis a bridge carrying a motorway across the river was modelled as a secondary dam some 7m high. The situation created a backwater upstream of the bridge and on overtopping and subsequent failure, it also created a secondary wave downstream of the bridge. A comparison of the results is summarised in Table 4, it is evident that upstream of the bridge, backwater effects give an increase in maximum flood levels of less than 0.5m. This increase becomes insignificant beyond 6km upstream of the bridge. Downstream of the bridge the secondary wave creates increases in maximum inundation levels of the order of 1.0m.

Table 4. Effect of secondary damming

Upstream of Secondary Dam at chainage 36km from Dam A

CASE		DISTANCE FROM DAM A (km)			
		30	32	34	36
No secondary Damming	Water Level (m AOD)	15.7	14	14	14
	Maximum Flow (cumecs)	500	350	345	334
With Secondary Damming	Water Level (m AOD)	15.8	14.5	14.2	14.1
	Maximum Flow (cumecs)	470	360	440	972

Downstream of Secondary Dam at chainage 36km from Dam A

CASE		DISTANCE FROM DAM A (km)			
		37	38	39	40
No secondary Damming	Water Level (m AOD)	11.4	11.0	10.4	9.8
	Maximum Flow (cumecs)	326	332	320	320
With Secondary Damming	Water Level (m AOD)	12.3	11.9	11.4	10.9
	Maximum Flow (cumecs)	775	622	480	443

Recommended Procedure

27. In establishing the recommended procedure the following criteria were adopted :

(a) the predicted extent of flood inundation and damage should be conservative,

(b) the degree of conservatism should be tempered by the fact that large scale, swift evacuation at short notice would in itself pose a significant threat to life and property.

(c) the procedure should be sufficiently practicable in order that it can be applied within reasonable time and budgetary constraints.

With regard to the aspects of valley roughness, dam failure duration, cascades and secondary damming the following sections describe the adopted procedures.

28. Valley Roughness. The extent of flood inundation and damage is mapped as the envelope of two cases, being :

(a) the case with a constant global value of Manning's coefficient equal to 0.04, and

(b) the case with different and higher estimates of Manning's coefficient more appropriate to the physical characteristics of the river valley.

29. Dam Failure Duration. Dam failure duration for a well engineered embankment dam is set at 1.25 hours. This value is in the centre of the range of durations observed or deduced from real dam failures.

30. Cascades. If the failure of an upper dam causes overtopping of a lower dam then that dam is also assumed to fail. A comparison is made between the extent of flooding by single and multiple failures. If the differences are not significant then the flood inundation sheets should be produced only for the worst case and included as copies in the inundation plans for the other reservoirs.

31. Secondary Damming. Secondary damming and subsequent failure caused by blocking of river crossing structures is, in general, not modelled in detail since its impact is covered by the conservatism inherent in the other assumptions such as reservoir inflow conditions and valley roughness characteristics. However for major river crossing structures in valley reaches where the recommended procedure produces a lesser degree of conservatism (eg in the lower reaches of a valley where low valley roughness coefficients have been estimated) then the effects of secondary damming and subsequent failure are modelled to establish the downstream extent of flood inundation and damage.

32. Recommended Procedure. The procedure has been applied to produce inundation maps defining the areas of flooding and structural damage. These maps also contain the predicted flood hydrographs at key locations together with information for authorities dealing with an emergency. A sample of the information contained in an inundation map together with an explanation of that information is contained in Figure 1.

Figure 1:Key to Flood Map Sheets

CONTINGENCY PLANS

33. Introduction. It is of fundamental importance that emergency planning for dam breach scenarios takes into account not only 'hardware' related issues such as provision of sufficient evacuation pathways, alarm systems, warning signs and communication facilities, but also addresses the 'software' or human behavioural aspects. These include clarification of roles and responsibilities, provision of information, lines of communication and the need for clear leadership and guidance.

34. It is important that both hardware and software aspects are given equal weight if successful emergency management is to be achieved in practice. Unfortunately, hardware considerations have tended to dominate emergency management considerations. This is due to the fact that emergency planning has in the past been considered as a purely engineering problem with engineering solutions.

35. The purpose of this section is provide a brief description of the standard format of the dam breach contingency plans prepared on behalf of Yorkshire Water, and to outline the human behavioural aspects incorporated.

36. Each contingency plan consists of the following:

User Guide Describes the purpose of the contingency plan and summarises each section.

Section 1 Summarises the information required during the handling of an incident at the reservoir.

Section 2 To aid in the diagnosis of the severity of the incident in the first line response, a set of 10 questions, each with four levels relating to severity of outcome, are presented. The resultant score will provide an immediate indication of how serious the situation is and who needs to be contacted. This should ensure that personnel responding to the incident have a clear idea of what action to take.

Section 3 Intended to be a decision support tool during the incident and as such, outlines the major decisions needed to be taken during the response.

Section 4 Provides further information on the key steps outlined in Section 3. This section should be used for reference purposes rather than followed in detail during an actual incident.

Section 5 Provides a list of external telephone numbers which will need to be referred to in an incident.

Section 6 Illustrates the layout of warning messages to be sent to the Police.

Section 7 Outlines the emergency roles and responsibilities of all Yorkshire Water personnel involved in the management of an incident.

Section 8 Series of Inundation Maps showing the extent of flooding following a dam breach for the reservoir.
(Figure 1 shows the type of information shown in the Inundation Maps)

37. The human behavioural aspects incorporated include:

38. Decision/Action Tree. Provides an outline of the major decisions needed to be taken during the response. This acts as a decision support tool for use during an incident, thereby ensuring vital actions are not omitted and all relevant personnel are contacted promptly.

39. Contingency Procedure. This section is broken down into a number of steps, the key elements of which are:

- *Initial Contact* - On detection of the likelihood of a dam breach clear instructions are laid out as to the initial point of contact, whether Yorkshire Water personnel, a member of the public or an organisation such as the Police. This should ensure a swift and effective initial response.
- *Access Routes* - Information on the safest routes to the reservoir is extracted from the Inundation Maps and clearly presented. Where access by land is not possible, information on the provision of a helicopter is included.
- *Control of the Incident* - To avoid confusion during the handling of the incident, the procedure clearly defines when to seek more specialist or senior advice. Instructions for alerting the Police are also included. For instance, if the Person In Charge of the Incident believes a dam breach is imminent, a standard 'Yellow Dam Breach Warning Message' will be faxed to the Police. However, if the dam has already breached, a standard 'Red Dam Breach Warning Message' will be faxed. This will ensure the Police have a clear idea of the status of the incident and the potential need for evacuation.
- *Obtaining Technical Advice* - Details for gaining information on Reservoir Safety are presented, together with alternative contacts if reservoir-specific personnel are unable to be contacted. This will ensure that specialist technical advice is always accessible.
- *Contacting External Organisations* - The Police are responsible for contacting other emergency services, eg fire and ambulance. However, Yorkshire Water may wish to liaise with these and other organisations. Therefore an up-to-date list of external contact numbers specific to the reservoir are contained within individual contingency plans.
- *The Emergency Control Centre* - Criteria are presented on the requirement for an Emergency Control Centre (ECC) to be set up. It is recommended that the ECC should be located at the relevant Control Room. Details on the provision of facilities necessary to ensure efficient handling of the incident, eg additional telephones, computer terminals and stationery, are also included.
- *Alerting Yorkshire Water Facilities Down-stream of the Flood-Water* - In the event of a potential/actual dam breach, information is contained in the contingency plan to alert managers responsible for Yorkshire Water facilities down-stream of the flood water. For each facility identified, the following information will be conveyed: the estimated time of arrival of the flood wave following the dam breach;

the estimated maximum depth of water to be expected; and, the resulting type of damage, structural or inundation.

- *Call-Out Procedures* - Key internal personnel involved in the response to the incident are identified, eg Emergency Liaison Officer, Group Insurance Manager, Customer Services, Administration. Both daytime and out of hours contact numbers are included, as well as nominated deputies in the event of personnel unavailability.
- *The Emergency Committee* - Information on the establishment and membership of the Emergency Committee are contained in the contingency plan. The role of the committee is to coordinate the action required to resolve the incident. This will ensure an efficient and effective response to the incident.
- *Termination of the Incident* - In order to subsequently assess the effectiveness of the incident response, and for the purpose of any legal proceedings, instructions are included on logging the events of the incident and filling in the appropriate incident response forms.

40. Roles and Responsibilities. In order for an incident to be managed effectively, all personnel must be fully conversant with their role and required duties during an incident. The issue of role clarity has been shown to be a major determining factor in the efficiency of incident response following research into recent major accidents. Therefore all key personnel involved with a dam breach incident have a written description of their interface with other personnel/external agencies. This section is intended as an aid to familiarise personnel with their role before an incident and therefore should only be used for reference and training purposes.

41. Summary. Integration of 'hardware' and 'software' elements in the development of contingency plans for dam breach scenarios should ensure an efficient and effective response to an incident. However, it is important that the contingency plans are supplemented with training, simulated exercises and refresher courses if personnel are to respond so that the incident has the minimum impact possible.

GENERAL CONCLUSIONS

42. If contingency plans are needed, effort spent on hazard ranking may not be worthwhile .

43. A method exists to prepare inundation maps for dam failure with adequate acuracy and reasonable cost.

44. Integration of maps into emergency procedures needs to be done very carefully, taking into account many human and organisational factors.

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The seismic behaviour and design of reservoir intake towers

W.E. DANIELL, BSc, PhD, Research Fellow, and C.A. TAYLOR, BSc, PhD, MICE, Reader, Earthquake Engineering Research Centre, University of Bristol

SYNOPSIS. The Earthquake Engineering Research Centre at Bristol University is engaged on a research programme to develop and validate analytical and design techniques for reservoir intake towers subjected to earthquake loading. This paper describes the current state of knowledge of the linear and nonlinear seismic behaviour of intake towers and how they interact with the surrounding water, the foundation soil and any adjoining structures. Current methods for the analysis and design of reservoir intake towers are discussed. Examples from the current research programme are employed to illustrate aspects of tower behaviour and the techniques adopted for their design.

INTRODUCTION

1. Intake towers form the entrance to reservoir outlet works and often house equipment for regulating the release of the impounded water for vital public services. The failure of an intake tower during an earthquake could cause the disruption of an important public utility and at worst the catastrophic failure of a dam. Thus, it is desirable that intake towers, in seismically active regions, are designed for earthquakes by using rational analytical methods based on a sound understanding their behaviour. This paper discusses the seismic behaviour of reinforced concrete reservoir intake towers and procedures for their design.

SEISMIC BEHAVIOUR OF TOWERS

2. The seismic response of an intake tower is complicated by a number of factors which are introduced, briefly, in Paragraphs 3 to 18. The response of a tower will be either linear-elastic or nonlinear and this response may be influenced by the dynamic interaction of the tower with the surrounding reservoir, the foundation soil and any adjoining structures.

Linear-Elastic Behaviour of Towers

3. A reinforced concrete tower will exhibit linear-elastic behaviour provided that its constituent materials are not strained beyond their linear-elastic limits. Under earthquake loading, this means that the concrete would

not have cracked and hence the reinforcing steel would not be carrying any significant load. In effect, the behaviour of the tower would be equivalent to one constructed of mass concrete.

4. The seismic and linear-elastic response of a tower depends on its natural (or resonant) frequencies of vibration, their associated mode shapes and the internal material damping. The mode shapes are a series of deformed shapes in which the tower can vibrate. A mode shape is caused to vibrate with relatively large amplitude when the exciting earthquake contains significant energy at frequencies close to the corresponding natural frequency of that mode. The amplitudes of the modal responses are controlled by the material damping of the concrete.

5. During an earthquake, the lowest modes of vibration of a tower will be excited most significantly, as the energy in the ground motion is concentrated in a low frequency band (typically up to about 20 Hz). However, the total seismic response of the tower will be dominated by the fundamental bending mode of vibration; its cantilever bending mode. The natural frequencies of a structure depend on its mass and stiffness properties. Stiffer structures have higher natural frequencies of vibration and structures carrying more mass will have lower natural frequencies.

Nonlinear Behaviour

6. The factors which contribute to the nonlinear seismic behaviour of a tower are introduced, in stages, in Paragraphs 7 to 13 by considering first, the nonlinear response of a tower to a static load, then to a cyclic load and finally to a dynamic ground motion.

7. Nonlinear Behaviour under Static Loads This is illustrated by the results of a recent nonlinear finite element study into the seismic behaviour of a 50 metre high and 8 metre diameter intake tower (Daniell, 1992). A horizontal force, representative of the inertia forces generated during an earthquake, was applied gradually to the tower in one direction. This forced the tower to bend and cracks formed in the concrete on the tension side of the tower. The response of the tower is illustrated in Figure 1 which shows how the displacement at the top of the tower varies with increasing load.

8. The graph is made up from four distinct portions. The linear portion, AB, corresponds to the constant stiffness of the tower before significant cracking is initiated at its base. The nonlinear portion, BC, represents a gradual decrease in stiffness as cracks propagate over the tower. At point C, the cracks through the base section of the tower have extended to their maximum length. Beyond C, cracks continue to propagate over the tower but portion CD of the graph is almost linear indicating a constant stiffness for the tower even though crack propagation is continuous. This suggests that

Figure 1: Nonlinear displacement of tower under static loading

the stiffness of the tower is dominated by the bending stiffness at its base, and is not influenced by reductions in the bending stiffness of sections above this level. From D to E the stiffness decreases much faster as the reinforcing steel begins to yield and the displacement of the tower becomes excessive. In this example, the steel yields before the concrete crushes. However, in a more heavily reinforced tower the concrete may crush first or at the same time as the steel yields.

9. The nonlinear behaviour of this tower can be split into two types; nonlinear-elastic behaviour, when the concrete in tension has cracked but the concrete in compression and reinforcing steel are within their elastic material limits and nonlinear-plastic behaviour once either the concrete or steel become plastic.

10. Nonlinear Behaviour under Cyclic Loads The nonlinear-elastic behaviour of the tower, discussed in Paragraphs 7 to 9, was investigated for cyclic horizontal loading also (Daniell, 1992). Under reversed loading, critical sections of the tower became cracked completely. The response of the tower was dominated by its bending stiffness which did not deteriorate with cyclic loading and the relationship between the stiffness of the tower and its applied load was found to be the same as that for unidirectional loading.

11. There are no reported studies on the nonlinear-plastic behaviour of intake towers under cyclic loading but, experimental studies have been conducted for reinforced concrete chimneys (Mokrin and Rumman, 1985). These tests showed that once the reinforcing steel has yielded, the cyclic response is dominated by the plastic behaviour of the steel because the cracks

which extend across the whole of the critical cross-section remain open. Under cyclic loading, the stiffness of the steel is reduced significantly once it has yielded.

12. Dynamic Nonlinear Behaviour During an earthquake, the nonlinear response of a tower will depend on its stiffness, which varies with the continually changing inertia forces on the tower. The concept of resonance, at the natural frequencies of vibration of the tower, applies no longer and the nonlinear dynamic behaviour of a tower cannot be described in simple terms.

13. Reported studies into the nonlinear dynamic behaviour of intake towers are scarce. However, the nonlinear-elastic behaviour of the tower, discussed in Paragraphs 7 to 9, was studied in response to an impulsive ground motion (Daniell, 1992). This was compared against the response of the same tower treated as a linear-elastic structure to investigate the effect of cracking and the resulting changes in stiffness. Both towers responded in the fundamental mode of vibration only and their resulting displacement time histories are shown in Figure 2. After two cycles of response, the period of vibration of the linear-elastic (uncracked) tower settles to that of its fundamental mode. For the cracked tower, the effect of the reduced stiffness is seen as an increase in the period of vibration and an increase in the maximum displacement.

Figure 2: Dynamic response of cracked and uncracked towers

Interaction with Adjoining Structures

14. Often intake towers are connected to other towers or a dam either via a footbridge or through a shared base, or both. The tower may interact dynamically with an adjoining structure when their natural frequencies are

close, as the resonance of one structure will excite a relatively large response in the other.

15. This effect was found to be significant in the dynamic response of the intake tower in Wimbleball reservoir (Daniell, 1992). The top of this tower connects to the crest of the dam by a footbridge. Using the finite element method, the tower was analysed for its response in the first and second bending modes when excited by harmonic horizontal forces. In an initial numerical model, the tower and bridge were represented while the dam was assumed to be rigid. The computed responses for this model compared unfavourably with actual measurements from dynamic tests on the tower (Daniell, 1992). With further analysis, it became apparent that the fundamental mode of the dam interacts with the tower and that the dam should be included in the numerical model. Hence, the model was modified, the analysis repeated and a far better fit was achieved with the test data.

16. For the second model, the response of the second mode of the tower increased by about 23% while the response of the first mode remained unchanged. The computed natural frequencies for the tower were 1.68 Hz and 7.80 Hz for the first and second bending modes and 6.81 Hz for the fundamental mode of the dam. This example demonstrates that the dam behaves as a rigid body when its natural frequencies are much higher than the modes of vibration of the tower but will interact dynamically with the modes of vibration of the tower when their natural frequencies are close.

Fluid-Structure Interaction

17. During an earthquake, the motion of the ground causes an intake tower to vibrate and to set up fluctuating pressure waves in the surrounding water which, in turn, exerts forces back on the tower. The effect of this fluid-structure interaction on the dynamic response of a tower is equivalent to an increase in its mass. This added mass reduces the natural frequencies of the tower which, in turn, alter the response of a tower to a specific earthquake. The reduction in natural frequencies is significant only when the depth of the reservoir is at least equal to half the height of the tower. Then depending on the reservoir depth and the slenderness of the tower, the fundamental frequency of a tower can be reduced by up to 45% (Liaw and Chopra, 1974).

Soil-Structure Interaction

18. When the foundation soil is flexible relative to the stiffness of the tower, their dynamic interaction leads to a reduction in the effective stiffness of the tower and an increase in the overall damping. The reduction in stiffness results in lower natural frequencies for the tower which in turn, influence its seismic response. The increased damping results from internal material damping in the soil and from radiation damping as energy

propagates away from the tower through the soil. The extent of radiation damping depends on the local geology. Where the soil is uniform to a large distance from the tower, the radiation damping may be significant in reducing the response of the tower. However, where a stratum of soil is relatively shallow and overlies a stiffer soil, radiation damping will be reduced as a result of wave reflection at their interface.

SEISMIC ANALYSIS OF TOWERS

19. A number of methods have been proposed for the seismic response analysis of intake towers. Differing in levels of refinement, they include simple methods for preliminary design purposes and refined finite element analyses for the final design.

Linear-Elastic Analysis

20. Simplified Methods Any standard simplified method of analysis normally employed for towers can be adopted providing that an allowance is made for the added hydrodynamic mass and that consideration is given to the limitations of the particular method used. An approximation to the added hydrodynamic mass on a towers can be evaluated using the Goyal and Chopra method (Goyal and Chopra, 1989) for a range of cross-sectional shapes.

21. The available methods consider the effect of the horizontal component of ground motion only, and adopt procedures for evaluating a static force equivalent to the maximum inertia force on the tower generated from the ground motion. This force is then used in a static analysis to determine the maximum seismic stresses and displacements in the tower. The equivalent static forces are evaluated using lateral force procedures or a response spectrum approach.

22. In the lateral force procedures (Applied Technology Council, 1978, ACI307-79, 1979, Montes and Rosenbleuth, 1968), the fundamental frequency of the tower and the peak ground acceleration are required to evaluate the distribution of shear and moment. These procedures are suited only to uniform or gradually tapered towers which have a regular variation of mass and stiffness, as they can produce inaccurate results for towers with concentrated masses or sudden changes in cross-section. These procedures have additional limitations when applied typical intake towers. The ACI procedure for reinforced concrete chimneys (AC1307-79, 1979) applies to very slender towers and tends to underestimate overturning moments (Chopra and Fok, 1984). The Montes-Rosenbleuth method (Montes and Rosenbleuth 1968) has been found to produce fairly accurate results for towers with uniform cross-section but it tends to overestimate the response of tapered towers (Chopra and Fok , 1984).

23. A modal, or response spectrum analysis, overcomes the limitations of the lateral force procedures because any irregularities in mass and stiffness are accounted for implicitly in the evaluation of the mode shapes and natural frequencies. The response spectrum analysis forms the basis of another simplified method developed specifically for intake towers (Goyal and Chopra, 1989). This method allows for the contribution from the second bending mode of vibration which has been shown to be relatively more important in the seismic response of squat towers (Chopra and Fok, 1984).

24. The effects of soil-structure interaction can be included in some of these simplified methods by modifying the natural frequency and the effective damping of the fundamental mode of vibration using standard expressions (Velesto and Meek 1974) for ground conditions which can be idealised either as a viscoelastic half-space or as a stratum of softer soil overlaying a stiffer soil.

25. These simple, but approximate, methods are quick and easy to use for the preliminary design of a tower. However, with developments in personal computers and finite element programs, more reliable results can be obtained quickly from the finite element analysis of a simple model. Intake towers can be represented by beam elements provided that account is taken of the effects of shear distortion (Goyal and Chopra, 1989), and the dynamic fluid effects can be represented with lumped mass elements. The effects of soil-structure interaction can be modelled using springs and dampers at the base of the tower or by discretising a finite extent of the soil with finite elements (Dowrick , 1987).

26. Refined Finite Element Methods Finite element programs have been developed specifically for the linear-elastic earthquake analysis of intake towers where the tower and the reservoir are discretised using finite elements (Liaw and Chopra, 1975) and the effect of soil-structure interaction can be modelled (Goyal and Chopra, 1989). The advantage of these programs is that the fluid elements model accurately the effects of dynamic fluid-structure interaction. Their disadvantage is that they do not have the flexibility to model different foundation conditions or complex structural arrangements e.g. continuity with the bridge and dam. These limitations can be overcome by using general finite element packages which include the facility of a fluid element.

Nonlinear Analysis

27. Currently, there are no proven methods for the nonlinear seismic analysis of reinforced concrete intake towers. The finite element method can be used but, the numerical material models are not sufficiently proven to make their use justifiable in most seismic problems and the analyses are time consuming and hence, costly. At present, there is a requirement for the

development of a simple and reliable method for the nonlinear analysis of intake towers. Although a simple procedure has been proposed and even adopted (ICOLD, 1991) for towers with little or no ductility, it still needs to be fully validated.

28. This procedure is recommended for use when the results of a linear-elastic response spectrum analysis predict cracking in the concrete. The over-stressed sections of the tower are assumed to crack and the natural frequencies and the response of the tower are re-evaluated by substituting the reduced moments of inertia at the cracked sections. The analysis then follows an iterative procedure until further cracking is inhibited. The validity of this procedure was examined recently (Daniell, 1992), by comparing it against the results of the nonlinear seismic analysis of the intake tower described in Paragraph 13. The simplified procedure under estimated significantly, the maximum stresses in the steel and concrete and over estimated the maximum displacement of the tower.

29. In any simplified analytical procedure for "non-ductile" or nonlinear-elastic towers, it is necessary to calculate the position of the neutral axis in cracked sections for a given bending moment and axial load so that the stresses in the steel and concrete and the moment of inertia can be evaluated. The moment of inertia is used in assessing the overall stiffness of the tower and hence the displacements. Charts and formulae, which provide this information, are available for reinforced concrete chimneys (ACI307-79, 1979) and it has been recommended that these can be applied to circular intake towers (ICOLD, 1991 and Chopra and Liaw, 1975). Initially, this recommendation may seem valid. However, when the underlying assumptions are examined, it becomes apparent that these formulae may not apply to intake towers. Chimneys typically have ratios of wall-thickness, t, to radius, r, of about 0.03 to 0.08 and therefore, are considered as thin shelled structures which have no significant variation in longitudinal strain across the thickness of their walls. Intake towers, on the other hand, have much thicker walls with the ratio t/r, varying typically between 0.15 and 0.30. Hence, the variation in longitudinal strain across the thickness of an intake tower wall may be significant and to ignore this fact in an analysis may lead to errors in evaluating the stresses and displacements in a tower.

30. To determine whether this is the case, design formulae were developed for sections of circular intake towers accounting for the variation in strain across the thickness of the wall (Daniell, 1992). Using both the chimney design formulae and the new tower formulae, steel and concrete stresses were evaluated for a range of sections typical of intake towers. A comparison of these values revealed that the chimney formulae tend to over estimate the maximum stress in the steel and under estimate the maximum

stress in the concrete. Therefore, the application of the chimney design formulae to intake towers will lead to inaccurate results.

31. The new design formulae give relationships between the bending stiffness of a section and the applied load. These formulae may be used in the development of a relatively simple finite element program for the nonlinear-elastic analysis of towers for their earthquake response where the tower may be modelled using nonlinear beam elements. The nonlinear seismic behaviour of the intake tower, which was the subject of the recent study, was shown to be dominated by its bending stiffness. In addition, the relationship between the bending stiffness and the applied load under unidirectional loading was shown to hold under cyclic loading. Assuming that these two findings apply to intake towers generally, then the new design formulae can be implemented directly in to a finite element program.

CONSIDERATIONS FOR SEISMIC DESIGN

32. It has been proposed that design criteria recommended for dams should be adapted to the seismic design of intake towers (Daniell, 1992). These criteria would require a tower to resist frequent earthquakes of moderate intensity without its serviceability or durability being compromised, while for less frequent severe earthquakes, the structure or function of the tower should not be impaired to the extent where a catastrophic failure of the dam could occur.

33. The acceptable deterioration of a tower under either of these criteria will depend on factors such as the type of tower, its function and its location. In towers with dry wells, cracking of concrete would impair both serviceability and durability requirements. Leakage of water into the tower and the exposure of reinforcing steel to corrosion would present problems. These problems would be less severe in towers which contain water and hence, limited cracking may be acceptable during a moderate intensity earthquake.

34. Any equipment, housed in intake towers, which is critical to the safety of the dam must remain operational after a severe earthquake. This requires attention to the detailing of the equipment and its supports while the tower may be permitted to suffer extensive cracking and even become ductile. The location of a tower is also an important factor in determining the permissible level of damage. If it does not house important equipment and is remote from any other safety critical structures, then severe damage may be acceptable during a maximum design earthquake.

35. Once the criteria for the seismic design of a tower have been established, then the type of analysis can be selected for the moderate and severe earthquakes. Linear-elastic analysis applies to towers where no

cracking occurs, nonlinear-elastic analysis to towers where cracking is permitted but neither the reinforcing steel nor the concrete become plastic and in the extreme case a nonlinear-ductile analysis could be employed.

36. There is dispute over whether free-standing towers should be allowed to become ductile during an earthquake, because their stability depends entirely on the structural integrity of the single member which forms its structure. Good engineering practice has been to provide as much redundancy as possible into the seismic resisting system of a structure. As this can not be achieved in a free-standing tower, it is sometimes recommended that they are designed to remain essentially elastic and hence a nonlinear-elastic analysis would be required.

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Implementing safety recommendations on UK dams — a supervising engineer's view

J. HAY, BSc, CEng, MICE, FIWEM, Associate, Rofe Kennard and Lapworth

SYNOPSIS. Putting safety into practice under the Reservoirs Act 1975 has not been straightforward in certain areas. Undertakers, enforcement authorities, inspecting engineers, and supervising engineers have interpreted their duties and responsibilities in different ways. Some have been more active than others in seeking to ensure that the recommendations of an inspecting engineer, in the interests of safety, are implemented.

INTRODUCTION

1. Under Section 10 of the Reservoirs Act 1975 [the Act], the recommendation of an inspecting engineer as to measures to be taken in the interests of safety is mandatory. The undertaker is required to carry such measures into effect as soon as practicable under the supervision of a qualified civil engineer. In some instances, implementation has either been unduly delayed or not proceeded with at all, because of economic constraints, or legal difficulties, or simply because the undertaker is not convinced of the need for the measures in the first place.

2. In such circumstances, the enforcement authority *may* require the undertaker, by written notice, to carry the recommendation into effect within a time specified in the notice, such time to be determined after consultation with a qualified civil engineer. Thus the enforcement authority has been given the power to enforce the recommendation but may choose not to do so.

3. The failure of an undertaker to carry out a safety recommendation may be followed by a period of consultation when the enforcement authority seeks further advice from a qualified civil engineer or often, in the first instance, from the supervising engineer. The appropriate response of the supervising engineer in this situation is open to question. Should he adopt an active role in seeking to ensure that safety recommendations are carried out? Should he give advice to the enforcement authority at all, bearing in mind that the Act only defines his relationship with the undertaker? In what

context is such advice given? Should he consult the inspecting engineer?

4. Engineers carry responsibilities under common law and the supervising engineer can find himself in a "Catch 22" situation when asked for advice by an enforcement authority. He may wish to respond but can only do so outwith the Act. The terms of reference of the supervising engineer should therefore clearly define when he is fulfilling his duties under the Act and when he is acting in an advisory capacity, whether to the undertaker or to others.

5. The contents of the paper are mainly based on the author's experience as a practising supervising engineer and also draw on the knowledge, experience and concerns of a number of other engineers with responsibilities for reservoir safety. Case histories have necessarily been presented in general terms. The views expressed are personal. The paper has been prepared with the aim of promoting discussion on what must surely be one of the most important challenges facing the dam engineering community - putting safety into practice.

SAFETY

What does it mean?

6. The meaning of "safety" in the context of the Act has been the subject of lengthy debate by practising dam engineers. Is the Act aimed solely at safeguarding persons or property against escapes of water from a reservoir? Is the dam itself to be safeguarded? The Reservoirs Committee of the Institution of Civil Engineers (1987) advised panel engineers that recommendations as to measures that should be taken in the interests of safety should be clearly distinguished from those in other categories, drawing attention at the same time to the absence of any definition of "safety" in the Act, and pointing out that what constitutes safety could only be decided by a ruling of the Court.

7. The Commitee noted that Sections 16(1) and 16(2) of the Act, relating to the use of emergency powers by an enforcement authority specifically to protect persons or property against an escape of water from a reservoir, appeared to indicate that there could be instances where an escape of water might not place persons or property at risk. In other words, there could be occasions when the failure of a dam might have no safety implications. This interpretation seems consistent with the approach used in "Floods and Reservoir Safety : An Engineering Guide" (ICE, 1989), which requires an impounding reservoir to be placed in a particular category depending on the size of community at risk downstream, and is clearly directed towards "public safety" rather than the physical integrity of the dam itself.

8. Deciding how safe a dam is, and hence the probability of it failing, can be a complex process, and very subjective. It may not be surprising that the legislators were unable to define safety, but preferred to rely for the final decision as to whether or not a dam is safe on the judgement of an individual qualified civil engineer. In the absence of a clear definition of safety, and in the knowledge that what constitutes safety will be decided by the Court, some inspecting engineers have reached quite different conclusions from others on aspects such as risk categories and design floods. Because of the uncertainty surrounding the question of safety, some inspecting engineers have felt it necessary to issue a disclaimer to the effect that their inspection report is not to be regarded as an absolute assurance or guarantee of total reservoir safety in the future.

How important?

9. The Reservoirs (Safety Provisions) Act 1930 [the 1930 Act] laid down, in the interests of safety, precautions to be observed in the construction, alteration, and use of reservoirs, and amended the law with respect to liability for damage and injury caused by the escape of water from reservoirs. The Reservoirs Act 1975 strengthened the provisions of the 1930 Act, particularly in regard to the supervision of reservoirs between inspections, and in increasing the duties and powers of local authorities to enforce the legislation. Concern about safety has clearly been the major motivating factor in the development of legislation, and has led to the creation of guidelines for professionals engaged in the design, construction, operation and maintenance of reservoirs. The Department of the Environment's reservoir safety research programme further emphasises the continuing commitment to reservoir safety. Any failure, or near failure of a dam, is always carefully analysed by dam engineers to identify ways of enhancing safety.

Who is responsible?

10. Responsibilities, powers and duties under the Act are held by the following persons or groups of persons:

- the Secretaries of State for the Environment, for Scotland and for Wales
- the enforcement authorities
- the undertakers (generally the owner or lessee of a reservoir)
- the qualified panels of engineers for design, construction, and inspection, and including supervision
- the qualified panel of engineers for supervision.

11. Although the Act does not say so, most people would agree that the undertaker is ultimately responsible for ensuring the safety of his reservoir. The Act was designed specifically to "make further provision against escapes of water from large reservoirs or from lakes or lochs artificially created or enlarged" and the principle that an undertaker has a strict liability for damage to others caused by escapes of water from his reservoir was established by the case of Rylands –v– Fletcher (1868). J W Phillips (1987), however, has pointed out that *"professional aspects"* of reservoir safety are the responsibility of panel engineers and that *"all technical matters relating to safety rely on panel engineers' expertise and judgement".* Precisely what is meant by "professional aspects" and "technical matters relating to safety" may be open to conjecture but a distinction has been drawn, on technical grounds, between the strict liability of the undertaker and the responsibility of panel engineers. Responsibility for safety, therefore, does not lie solely with the undertaker. Panel engineers are involved.

12. The Act, like the 1930 Act before it, embodies the principle that a single qualified civil engineer should be responsible for the inspection of existing reservoirs in the United Kingdom. It is argued that this "concentrates the mind", and ensures that the inspecting engineer will take great care in exercising his judgement and meeting his responsibilities. However, the Act extended the provisions of the 1930 Act by introducing, inter alia, the enforcement authority and the supervising engineer, with longer term roles for overseeing and supervising reservoirs respectively. As a consequence, the concept of an individual qualified engineer, with undivided responsibility for the safety of a reservoir, may now be a thing of the past.

13. Some have argued that the supervising engineer, because of his continuing involvement with a reservoir, carries a much greater responsibility for its safety than the inspecting engineer, who may inspect it, and report on its condition, only once every ten years. The current panel structure of engineers, which requires inspecting engineers to be more highly qualified and experienced than supervising engineers, does not seem to support this argument. What does seem certain, however, is that if there were to be an incident which placed a reservoir at risk, the actions of the undertaker, supervising engineer, and enforcement authority would subsequently be scrutinised just as rigorously as those of the inspecting engineer.

14. It seems that all parties – Secretary of State, enforcement authority, undertaker, inspecting (or construction) engineer, supervising engineer – share responsibility to a greater or lesser degree for the safety of a reservoir. It is therefore important that the legal framework under which they function is effective. Practising dam engineers, who have a wider knowledge and greater understanding of

the factors influencing reservoir safety than the undertaker, local authority or legislators, may have a particular responsibility to ensure that this is so.

IMPLEMENTING SAFETY RECOMMENDATIONS

15. Implementing an inspecting engineer's safety recommendations is an integral part of the process of ensuring the safety of a reservoir, although the inspecting engineer's responsibilities under the Act are fulfilled with the submission of his report. Unless he is retained to give advice or to supervise the carrying out of safety recommendations, he may have no further involvement in a particular reservoir and have no effective part to play in ensuring compliance with the Act, including the implementation of the recommendations. Where undertakers have failed to implement safety measures, individual authorities and supervising engineers are known to have reacted in different ways, some taking no action, others apparently exceeding their authority. The fundamental dilemma facing them both is deciding how much they should become involved. Further uncertainty may arise where the enforcement authority is also the owner of a reservoir, or when the supervising engineer is employed by the undertaker e.g. public water company. The supervising engineer might understandably feel inhibited from becoming too active, particularly if extensive remedial measures were known to be required.

The Role of the Enforcement Authority

16. The enforcement authority has a statutory duty to ensure that the undertaker complies with the Act in all respects. It must send reports at two-yearly intervals to the Secretary of State on the steps it has taken in this regard, and must also maintain a register containing prescribed information on large raised reservoirs wholly or partly within its area. The Secretary of State is empowered to instigate an inquiry as to whether an enforcement authority has failed to perform any of its functions under the Act.

17. Hitherto, the duties of the enforcement authority have been regarded by many as mainly administrative and not technical. However, an enforcement authority may also have to take decisions which have a direct bearing on reservoir safety, e.g. when an undertaker fails to implement the safety recommendations of an inspecting engineer. Under the Act, the authority may choose to serve written notice on the undertaker to carry out the recommendation within a time specified which must be determined in consultation with a qualified civil engineer. An authority may also be called on to decide whether or not a dam is safe, and whether immediate action is needed to protect persons and property.

18. Since reservoir safety is acknowledged to be of paramount importance, the implementation of safety recommendations must be given the highest priority and the enforcement of such recommendations cannot be regarded as purely a matter of administration. The enforcement authority must be relied upon to play an active role in ensuring that recommendations are carried out, where necessary contacting undertakers, consulting qualified engineers, and visiting reservoirs.

The Role of the Supervising Engineer

19. The Act states:

"At all times when a large raised reservoir is not under the supervision of a construction engineer, a qualified civil engineer ('the supervising engineer') shall be employed to supervise the reservoir....." (Author's underlining)

and defines the statutory duties of the supervising engineer as follows:

"... 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."

"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 for the reservoir 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."

"to recommend to the undertakers that the reservoir be inspected under section 10 .... if at any time he thinks that such an inspection is called for."

20. The limitations on the duties and responsibilities of the supervising engineer contained in the legislation are not always fully

understood. Many undertakers, for example, expect advice from their supervising engineers on wider issues, often being unaware of the distinction between the responsibilities of inspecting engineer and supervising engineer. Some inspecting engineers have regarded supervising engineers as assistants, requiring them to carry out line and level monitoring, take instrumentation readings, maintain records in the prescribed form – all tasks which are clearly the responsibility of the undertaker and not the supervising engineer. Some enforcement authorities have expected supervising engineers to pursue dilatory undertakers on their behalf.

21. When an undertaker fails to implement an inspecting engineer's safety recommendations, the supervising engineer has no brief to issue additional or alternative advice to that implicit in an inspecting engineer's safety recommendations. His only formal recourse, if a matter is not resolved to his satisfaction, is to call for a statutory inspection. To some, this approach is too legalistic, inflexible and uncompromising. They argue that the supervising engineer should play a more active role and should urge the undertaker to implement safety recommendations, pointing out the likely consequences if he (the undertaker) fails to do so. This seems a dangerous precedent, even if the supervising engineer knew what the likely consequences might be. Any advice given could be misconstrued, particularly if it in any way contradicted the recommendations of the inspecting engineer.

22. The role of the supervising engineer has changed from the original concept of an ICE Committee (1966) that a chartered civil engineer be appointed to report any significant change in the behaviour of the reservoir, and is now perceived by many to be more onerous than defined in the Act. Recent guidelines issued by the Reservoirs Committee (1993) recommend that applicants to the supervising engineers' panel should have had about five years experience and have been in positions of responsibility in the design or construction or operational supervision for significant periods on reservoirs to which the Act applies. Supervising engineers are also expected to undergo further training to make good any shortfall in experience before being appointed to the panel. Specialist topics such as soil mechanics, hydraulics, and flood hydrology are included in training programmes although it is not always clear how the supervising engineer is expected to apply in practice the knowledge which he has gained. For example, it is not within his brief to revise flood assessments made by an inspecting engineer.

23. It is also argued that the publication of guidelines such as the BRE report "An engineering guide to the safety of embankment dams in the United Kingdom" (1990) and the latest CIRIA guide "An engineering guide to the safety of concrete and masonry dam

structures in the UK" (1994), which are intended to assist "the various personnel who are responsible for the safety of reservoirs", place added responsibility on the supervising engineer. Notwithstanding that these guides are not to be regarded as mandatory nor a code of practice, the supervising engineer will doubtless be expected to have familiarised himself with the contents and to have taken account of them, whilst exercising his own knowledge, experience and judgement.

24. In recent years, many large undertakers have come to rely more heavily on the supervising engineer for safeguarding reservoir safety. A reduction in the number of resident reservoir keepers, who would previously have been responsible for routine surveillance, has deprived supervising engineers of a valuable source of up-to-date information and background knowledge. Of equal concern is the failure of some undertakers to retain adequate construction and monitoring records relating to their reservoirs, thus making it more difficult for the supervising engineer to appreciate the significance of any change that may have been observed.

Relationship between Enforcement Authority and Supervising Engineer

25. The Act defines the duties and responsibilities of a supervising engineer to the undertaker but makes no reference to the relationship between the enforcement authority and the supervising engineer. However, if a supervising engineer gives advice to an undertaker which either recommends an inspection or that any other action be taken, or draws attention to a breach of certain provisions in the Act, he is required to forward a copy of that advice to the enforcement authority. A channel of communication then exists between the authority and the supervising engineer, and the authority may naturally turn for further advice or information from the supervising engineer.

26. At that stage, the supervising engineer must consider carefully how to reply, and bear in mind his specific duties and responsibilities defined in the Act. He must be careful not to compromise the undertaker by giving advice which is outside his area of responsibility, for example, by commenting on the risk category, or the magnitude of the design flood, or the necessity or otherwise of implementing particular safety recommendations. Where an undertaker has failed to implement a safety recommendation, particularly a major one, there is a temptation for the supervising engineer to suggest some less ambitious, or interim stop-gap, measure to improve the situation. This can lead to confusion, and it is preferable for such matters to be referred to a qualified civil engineer.

SAFETY RECOMMENDATIONS

27. The manner in which an individual inspecting engineer formulates his safety recommendations may be influenced by factors such as the age, personality, knowledge and experience of the individual, and his relationship with the undertaker and other parties. It is important that the recommendations are precise and clear. If they are not, uncertainty can arise and the relevance and importance of the recommendations called into question. Particular difficulties have arisen for the undertaker, enforcement authority and supervising engineer where the recommendations have been found to be in one or more of the following groups:

Nothing to do with safety

28. There have been many examples of recommendations in the interests of safety which should have been classified under operation, monitoring or maintenance. Had this been done, they would not then have come within the enforcement authority's statutory powers. Because of confusion surrounding this issue at the time, the ICE Reservoirs Committee (1987) advised panel engineers to distinguish clearly recommendations as to measures to be taken in the interests of safety, which are mandatory, from those in other categories such as operation and maintenance. However, it may not be easy to make such a distinction, particularly where a lack of maintenance, or a failure to carry out adequate monitoring, can endanger a reservoir. In such cases, the inspecting engineer will insist that the maintenance or monitoring is carried out. In one recent inspection report, the inspecting engineer's only recommendation in the interest of safety was to emphasise the need for on-going maintenance.

Unable to be certified

29. Recommendations can be worded in such a way that they cannot be certified as having been completed. Common examples occur when on-going items of maintenance such as the routine operation of valves, or clearance of vegetation at specified intervals, are contained within safety recommendations. The system of monitoring progress on the implementation of safety recommendations requires that a qualified civil engineer certify that the measures have been carried into effect. He can only do so in the case of a completed item which does not require further work at some future date.

Too specific

30. On occasions, inspecting engineers have given specific details of the measures which, in their opinion, are necessary to meet the requirements of a particular safety recommendation. They have

usually done so with the aim of helping the undertaker to understand the scale of the problem and how to deal with it. In a recent example, to meet a shortfall in spillway capacity, an inspecting engineer recommended that a reinforced concrete spillway channel of a certain size should be built on the right abutment of a dam. Other means of accommodating floods, such as increasing the freeboard by lowering the top water level or raising the crest, may have been equally effective but the undertaker was not given the option. Some inspecting engineers now feel that safety recommendations should be couched in more general terms. In this particular case, the lack of adequate spillway capacity could have been referred to without stating precisely how it was to be put right, and any additional advice could have been included in a separate appendix.

Impractical

31. There are examples of an inspecting engineer making a safety recommendation, apparently without considering fully how it is to be put into effect. When this happens, the undertaker, enforcement authority, and supervising engineer, may find themselves in the invidious position of either accepting the recommendation as it stands or seeking to have it modified to something more practical by referring the matter back to the inspecting engineer. A typical example relates to the setting up of an emergency procedure which necessitates the provision and installation of a large number of temporary pumps to lower the water level in a reservoir level quickly. Even assuming it were possible to obtain and install pumps quickly enough, the procedure would probably only be effective if those responsible were highly trained and able to respond in the same way as the emergency services.

Too onerous

32. Increasing the capacity of a spillway to meet modern flood standards can be very expensive. It is understood that some large undertakers have taken the view that implementation of safety recommendations can be deferred if the measures have not been budgeted for and finance is not available. At a time when public spending is strictly controlled, this trend appears likely to continue, and the description "as soon as practicable" may increasingly be taken to mean that the measures need only be implemented when the undertaker can afford them.

Conditional

33. There is a tendency nowadays for specialist advice and studies to be quoted separately from an inspection, possibly as a result of the pressures of competitive tendering. For example, the costs of a

detailed flood study or a stability analysis may no longer be included in the inspection fee. The end result is that any safety recommendations become conditional until such time as the results of the further analyses are available. In such an instance, the inspecting engineer would presumably be retained to give further advice after the submission of his inspection report.

Providing for future development

34. It is reported that some inspecting engineers have placed a reservoir in a higher risk category in the knowledge that a future housing development is planned downstream of the dam and have made safety recommendations on that basis. Others feel that only the current situation at the time of the inspection should be considered and the matter should be referred to the supervising engineer to check from time to time whether any development is taking place, so that the dam can be reclassified if necessary.

Time related

35. A safety recommendation may state that an urgent item must be completed within a certain period of time. If the specified timetable is not met the recommendation strictly becomes impossible to carry out. The Act requires that safety recommendations be carried into effect "as soon as practicable" and, to avoid confusion, it may be advisable to omit any reference to time in the wording of safety recommendations.

CASE HISTORIES

36. In the following examples, the inspecting engineer's safety recommendations, despite being mandatory, were not carried out. The reasons why, and the actions of those involved, are examined. Whilst specific cases are described, it is believed that the features which gave rise to the safety recommendations in the first place, such as inadequate spillway capacity or leakage, are not uncommon.

Example 1 – The inspecting engineer for an old privately–owned, earth embankment recommended, in the interests of safety, that a new overflow structure and spillway channel should be built. The works were not carried out and the inspecting engineer retired. Another panel AR engineer was appointed to advise the undertaker. He considered that the dam could be made safe by raising the dam crest and widening the existing spillway. These works were carried out and he issued a certificate, stating that in his opinion the works were adequate to meet the principles behind the original recommendation of the first inspecting engineer. The supervising engineer, in the knowledge that a panel AR engineer was advising the undertaker, did

not call for a statutory inspection but simply recorded in his annual statements that the original inspecting engineer's recommendations had not been carried out.

Example 2 – A final certificate for a newly completed reservoir contained several conditions specifying additional works. The first annual statement of the supervising engineer noted that these works had not been carried out. The statement also included advice to take certain measures against erosion and was copied to the enforcement authority. Subsequently, the enforcement authority requested the supervising engineer to inform them of the action he was taking to ensure the conditions were complied with. The supervising engineer replied that he would be reporting what had been done in his next statement.

Example 3 – A report from an inspecting engineer recommended that the top of an old embankment be levelled to its lowest elevation to provide an auxiliary overflow. A road ran along the top of the dam and local people objected to the lowering of the road, citing an increased risk of spray on the road, leading to ice build-up in freezing conditions. The supervising engineer was requested to attend meetings with the undertakers' representatives, local people and statutory bodies and comment upon options proposed. His comments on hydrology, desirability of overtopping, alternative options, etc were also requested. The inspecting engineer finally amended his report to provide the auxiliary overflow at a higher elevation. Throughout there was no contact between the inspecting engineer and the supervising engineer.

CONCLUSIONS

37. From the point of view of a supervising engineer, the practical application of parts of the Act has not been straightforward in practice. A particular example is deciding what to do when an undertaker fails to implement safety recommendations. Judging from the varied reactions of enforcement authorities and engineers to the same situation, others are also uncertain, and there appears to be a clear need for a more uniform and consistent approach. It is suggested that the following measures may be worth considering:

- the issuing of guidelines to bring about a greater understanding and awareness of the duties and responsibilities of the various parties under the Act, and the relationships between them.
- the issuing of guidelines giving further clarification on the nature and wording of safety recommendations.
- a review of the effectiveness to date of the enforcement role of local authorities, including the implementation of safety

recommendations. The establishment of a nationwide database would be beneficial.

- the introduction of a legal framework in which the precise terms of reference of the various parties, particularly the enforcement authority and the supervising engineer, are defined. This is regarded as essential for professional indemnity purposes. The legal framework should:
  - clarify whether the enforcement authority has a technical role,
  - take account of the engineer's duty of care under common law,
  - distinguish between the supervising engineer's duties defined under the Act and his other responsibilities outside the Act,
  - clarify what is expected of an individual supervising engineer responsible for supervision of a reservoir *"at all times"*.
  - extend the responsibility of the inspecting engineer beyond the submission of his report to enable him to play an effective role in the implementation of his safety recommendations.
- the formation of a central regulatory authority to bring about some uniformity and consistency of approach to reservoir safety in the UK.

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A practical appraisal of the overtopping of embankment dams

A.K. HUGHES, BSc, PhD, DMS, CEng, FICE, FIWEM, MIM, Partner, and C.G. HOSKINS, BSc(Eng), CEng, MICE, Associate, Rofe Kennard and Lapworth

SYNOPSIS. The presence of a good development of vegetation, and particularly an extensive root structure, will help to minimise erosion damage and maintain the integrity of a dam. The paper discusses the behaviour of vegetated crests and downstream slopes under overtopping flows and suggests that present concerns over overtopping may be misplaced in many instances. Each situation must be judged on its merits and the site specific influences considered. The paper gives practical guidance and outlines simple works to improve the response to overtopping. Various examples are presented.

INTRODUCTION

1. Many recent small dams have been designed to overtop on rare occasions with flow passing over an engineered auxiliary overflow. Older dams often show signs of frequent uncontrolled overtopping yet have functioned satisfactorily for many decades. The behaviour under overtopping conditions depends on many factors and influences and their complex interaction, but it is evident that the vast majority have been able to withstand such flows without significant damage. Furthermore it appears that the instances of overtopping may, in some instances, be more frequent than envisaged with some dams overtopping as often as annually.

BACKGROUND

2. Embankment overtopping has often been cited as the primary cause of all documented failures of large dams, and the majority of failures involving small dams. Overtopping can occur at any stage during the life of a dam, the risk of serious damage or failure being a function of design and other factors.

3. Defects which may lead to overtopping include inadequate spillway capacity, lack of maintenance for control equipment installed on or in th dam, settlement caused by seepage or by insufficient compaction, embankment stability, incorrect operation of the dam, and even vandalism, as shown in Fig. 1 and Table 1.

 Reservoir safety and environment. Thomas Telford, London, 1994

FIGURE No.1 FACTORS WHICH MAY CONTRIBUTE TO OVERTOPPING

Table 1. Factors cited as reasons for overtopping failures/accidents

Lack of Engineering Supervision during Construction
Problems of Spillway Provision and River Diversions during Construction
Interruption of Work during Construction
Problems of Access
Inadequate Spillway Capacity
Sabotage
Vandalism
Inappropriate raisings of embankment
Flashboards
Roller Gate problems
Loss of Power Supply
Icing of Gates
Insufficient maintenance provided to Valves and Gates
Empty/Abandoned Reservoirs
Landslides
Settlements
Cascade Failures

4. However, there may be a multitude of other factors which may have initiated the failure process. For example, within each of these causes of overtopping, a piping failure, or internal erosion, may result in crest settlement and lead to overtopping and then perhaps failure. Hence it is often difficult to specify a single reason for failure. Nevertheless, several analyses of types of failure have been carried out over the years, the majority of which assign about 30% of all failures to overtopping.

5. An extensive survey of nearly 700 incidents/failures due to overtopping, including over 250 incidents/failures in the UK has yielded a large amount of information on the mechanism of overtopping failure and the means by which resistance to overtopping can be increased (Hughes, 1981).

RESPONSE TO OVERTOPPING FLOWS

6. Many of the dams built in Britain may be described generally as being of small or medium size, elderly and built of cohesive material at a time when technology of design was rudimentary, if actually non-existent. Many structures associated with mills and other industrial processes were built when hydrological information was scarce and hence their spillways are often found to be hopelessly inadequate by modern standards. As a result, many small dams are overtopped frequently by floods having low return periods – yet they still survive.

7. Their survival appears often to have resulted from two main reasons; firstly, the period of overtopping is short and secondly, the dam has good vegetative cover offering protection against the overtopping waters. Other features such as the topsoil and fill type, general arrangement of the dam etc, can also be significant.

8. Surface flows from overtopping can cause sheet erosion or gulleying if flows are concentrated. The extent of erosion depends primarily on the complex interaction of the water flow, the topsoil and uppermost layers of the fill, the slope and its extent, and the vegetation/ground cover. Other features to be described later, can have a significant effect in adverse circumstances.

9. The influence of the water flow is determined by depth of flow, velocity and duration and whether there is a tailwater; these are largely dependent on the topography, storm experienced, and spillway arrangements at a particular site. A clear distinction must be made between flow conditions before and after the occurrence of surface discontinuities which occur at the onset of failure. Erosion will occur when the shear stress on the surface exceeds the resistance of the soil or of the fill material; the more coarse granular materials on a slope will also be influenced by gravity.

10. The maximum velocity reached on the downstream face of a dam depends on the overtopping head, the roughness, and the head loss across the embankment. The maximum erosive potential exists when the flows allow critical depth conditions to exist at the crest, the flow remains in contact with the downstream slope and produces a hydraulic jump at the toe. On small dams there may be insufficient length of slope to enable terminal velocity to be reached in which case the erosive potential is reduced. Intermediate flattening of the slope or berms may decrease the velocity of flow but in the case of the latter may cause increased erosion due to the turbulence at the change of slope. Steepening of the slope may encourage erosion. Erosion may be increased at the toe or on the mitres of a dam as a consequence of the change of slope, concentration of flow, increased turbulence, granular drainage material and perhaps the existence of softer ground where inadequate drainage is present.

11. The soil type forming the embankment will clearly influence the rate of surface erosion. The behaviour of the material will be affected by the percentage and type of clay, the plasticity, consistency, the percentage of organic matter present, and the cation exchange capacity. The presence of sub-surface flow occurring in fissures, voids, and holes where roots have decayed, as well as activities which may have loosened or roughened the surface (e.g. rutting, localised erosion, excavations etc) will affect the behaviour of the material.

12. The embankment slope may also be affected by alternate freezing and thawing cycles. These will tend to loosen the soil and cause fissuring resulting in a lowered strength and rainfall infiltration. Desiccation may

have a similar effect and encourage soil creep. Apart from the risks to shallow instability, both processes will tend to disturb the embankment profile and lead to non-uniform flow conditions.

VEGETATION

13. Vegetation on the slope will act in a triple role in providing surface resistance, reducing the speed of flow, and strengthening the topsoil and upper levels of the fill by value of its root system. Hoskins and Rice (1992) discuss the influence of vegetation on embankment dams and conclude that vegetation may add an extra margin of confidence and reduce the likelihood of maintenance and remedial work.

14. The prime function of vegetation is to strengthen the slope by the development of a dense root system and to provide a good protective cover to the erodible materials beneath. The roots will grow to provide support and extract water and thus will develop and die back according to the needs of the vegetation and external influences. Most roots are found typically within the uppermost 50mm for a good grass cover, a few hundred millimetres for herbaceous vegetation and deeper for larger shrubs and trees. At shallow depths, therefore, an extensive development of roots will normally be present to bind the soil physically. The roots will also tend to extract water and increase the strength of the fill.

15. A well developed, dense ground cover of vegetation comprising mainly grasses gives the best means of providing surface resistance and reducing the speed of flow. Hewlett, Boorman and Bramley (1987) discuss the behaviour of a grassed surface under increasing hydraulic loading and show that the best results are obtained when a uniform length of grass sward is laid down by the flow. This will occur regardless of the length of the grass sward and is preferable to tussocky grasses, small shrubs etc which will be more rigid and result in turbulent, more erosive flow. Isolated clumps of these types of vegetation are particularly likely to cause problems including localised turbulence and channelling of the flow.

16. Bare patches, with no plant growth are undesirable, but where present beneath a layer of vegetation are likely to be reinforced by their root growth. Damage due to human or animal access which has prevented establishment and the regeneration of cover, especially if it has caused loosening or disturbance of the ground is to be avoided. Often, an unofficial access path results in a bare track down the slope with an associated gulley after prolonged use, providing a readily available channel for concentrated flow.

17. Many of the aspects relating to vegetation are discussed by Hoskins and Rice (1992) and Hoskins (1993). In particular, the suitability of the type and position of vegetation, particularly trees, should be considered. Vegetation must also be maintained to promote a healthy cover and any deficiencies or losses rectified. In particular mature trees and larger

shrubs will require selective lopping and pruning if they are too large, out of balance, or unhealthy or, perhaps, removal.

18. Clearly, features on a dam, especially at the crest and down the slope which tend to concentrate flow will rapidly increase the risk of overtopping damage. Such features may include a non–structural wave wall which might fail under hydraulic loading, a wave wall with a gap within it, localised settlement, inappropriately placed trees, berms, drainage layers etc etc and in general an attempt should be made to keep the flow as uniform as is possible across the crest or portion subject to overtopping.

FORMATION OF A BREACH

19. An experimental study (Hughes 1981) and evidence from a number of case histories suggest that if the overtopping takes place and is sustained then erosion will concentrate in one or more areas and eventually one area. Here, although some mass movement of the turf and topsoil may also occur, gullies will be cut into the upper layer. Overtopping waters will tend to form a rounding of the crest and what will be described as a 'step–feature' near the crest. This feature will be repeated at fairly regular intervals down the slope with the most prominent steps being those nearest the crest and nearest the toe.

20. The step forms in a manner similar to that in which a waterfall erodes; establishing a vertical face, with some undercutting if the water clings to the face, and a plunge pool at the base of the face. In addition as the erosion process continues the step recedes towards the upstream face, just as a waterfall recedes and forms a gorge of recession, until at some point the water pressure on the upstream face causes a breach through the bank. Once this occurs a channel has been eroded through the upper part of the bank with, in the case of cohesive soils, vertical sides.

21. Reservoir storage, at this stage causes an increase in discharge causing a widening and deepening of the channel through the dam. The final shape of the breach is dependent, to a large extent on the material from which the dam is formed. The stages in the development of a breach is shown in Fig. 2. An early account of the step formation is given below.

22. An account of the failure by J.G. Parker (ASCE, 1891), a civil engineer working in the area at the time, tells of attempts to save the embankment of the South Fork Dam as overtopping threatened in 1889. *"Half of the men were cutting a ditch through the shale rock at one end of the breast. This ditch was cut through the original ground and about 25 feet from the constructed portion of the breast. The shale was so tough that they could not cut it more than about 14 inches deep and about 2 feet wide, but when it was cut through to the lake the water rushed in and soon made it a swift stream, 25 feet wide and about 20*

inches deep, but the rock was so hard that it could not cut it any larger than this."

23. The remaining men "*had a plow at work throwing up a furrow, and thus raising a temporary barrier or breast to retard the water flowing over the dam, should it reach that height which it was gradually doing. The water in the lake rose until it was just passing over the breast, notwithstanding that the lake had then two outlets (the waste weir and the one cut by the labourers). The breast was slightly lowered in the centre and the water washed away our temporary embankment thrown up by the plow and shovels, and the water was passing over in many places in a distance of 300 feet about the centre of the breast.*"

24. Parker reported: "*The water was cutting the outer face of the dam, but not as badly as I feared it would, its greatest effect was on some portions of the roadway which crossed the breast where the roadway had been widened on the lower side by the addition of a shale earth or disintegrated shale, upon which the action of the water was instantaneous, but the heavy rip-rapping on the outer face of the dam protected this wash and the water cut little gullies between each of the large stones for rip-rap.*"

25. Parker considered cutting a spillway through the embankment at one end, but decided against this action as he realised that it would have meant the destruction of the dam, and he also hoped that the level of the reservoir would cease to rise. He later found that "*the water on the breast had washed away several large stones on the outer face and had cut a hole about 10 feet wide on the outer face and about 4 feet deep, the water running into this hole cut away the breast in the form of a step both horizontally and vertically, and this section went on widening and deepening this hole until it was worn so near to the body of the water in the lake that the pressure of the water broke through, and then the water rushed through this trough, and cut its way rapidly into the dam at each side and the bottom; and this continued until the lake was drained.*" Reports state that, on total breaching, the water level dropped some 18m in a 'matter of minutes'.

26. Parker stresses that "*the dam did not break, but was washed by the water passing over it from 11.30 o'clock am until nearly 3 p.m. until the dam was made so thin at one point that it could not withstand the pressure of the water behind it, and the water once rushing through this trough nothing could withstand it!*"

27. The storage, some 22 x 10^6 cubic metres of water, was released into a steep deep valley and the resulting flood causing great devastation and, partly through fires caused in Johnstown, the loss of some 2280 lives.

EROSION RISKS

28. Methods to investigate and assign the erosion risk are given by Coppin and Richards (1990) and Hemphill and Bramley (1989), and

(1) WATER APPROACHES THE CREST

(2) INITIAL OVERTOPPING – STEPS START TO FORM

(3) STEPS INCREASE IN SIZE

(4) RECESSION OF TOP STEP

FIGURE No.2 STAGES IN DEVELOPMENT OF A BREACH

(5) RECESSION OF TOP STEP CONTINUES UNTILL WATER PRESSURE CAUSES BREACH

(6) CONTINUED HORIZONTAL EROSION

(7) EROSION CONTINUES

(8) BREACH COMPLETE

FIGURE No. 3 STAGES IN DEVELOPMENT OF A BREACH

although these are essentially for waterway slopes under surface runoff conditions can be used to give an indication of the various soils, vegetation etc. Work by Fread (1991), Hughes (1981) and MacDonald and Langridge–Monopolis (1984) has attempted to quantify erosion rates and breaching characteristics for dams.

PRACTICAL GUIDANCE TO MINIMISE EROSION RISKS

29. Experience to date by reference to case studies suggests that many dams are more tolerant of occasional overtopping than considered in the past. Many of the old small dams in this country have probably been overtopped, some on a frequent basis, with no major ill–effects nor threat to their integrity.

30. Many features can be identified which can have a direct influence on the erosion resistance of a dam; their appreciation and application is an essential feature of assessing the adequacy or otherwise of existing arrangements and features. Each dam is unique and each situation is different and most must be judged on its merits with all site specific influences considered.

31. Assessing the response to overtopping essentially falls into four main areas:–

Hydrological and associated features

(a) shape of hydrograph – water levels attained, period of time that water is likely to overtop
(b) upstream slope protection and crest protection

Hydraulic considerations

(a) is there a wave wall? is it structural?
(b) condition of slope – uniformity
(c) horizontal crest (over all or restricted length?)
(d) changes in slopes – berm/drainage channels etc
(e) mitres
(f) instrumentation buildings and other obstructions

Geotechnical conditions

(a) knowledge of soil type
(b) embankment stability
(c) influence of fissures etc
(d) presence and influence of drainage features
(e) presence and influence of any geotextile, slope protection or other features

Biological conditions

(a) basic knowledge of vegetation
(b) likely root structure
(c) suitability of type and position of vegetation
(d) health of vegetation
(e) likely behaviour under overtopping flow
(f) possible changes with time – seasonal effects/erosion/vandalism

CONCLUSIONS

32. In conclusion, it is considered that many UK dams are more tolerant to overtopping than perhaps is imagined. In particular, if minor works are carried out, such as levelling of the crest or the installation of a kerb and downstream protection to assist in trying to ensure uniform overtopping and removal of features which would lead to a concentration of flow or turbulence then many can withstand overtopping flows for significant periods.

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Analysis methodologies for rigorous and approximate seismic assessment of dams

C.I. ROBERTSON, MA, CEng, MICE, Principal Engineer, Sir William Halcrow & Partners, F. GRANSHOAB, BSc, BEng, MSc, Post-graduate Student, Heriot-Watt University, and J.L. HINKS, MA, MSc, CEng, FICE, Principal Civil Engineer, Sir William Halcrow & Partners

SYNOPSIS. The seismic hazard to dams is a matter of ongoing concern and the assessment methodology is subject to debate. In this context, the technical background for rigorous and approximate seismic analyses of dams is discussed and related to a practical example of a 3-D analysis of the Foz Coa dam in Portugal. The effect of different modelling approaches is determined from follow-up studies in two dimensions and the results of these discussed. Conclusions are drawn to assist practising engineers in determining appropriate methodologies for the seismic assessment of dams.

INTRODUCTION

1. Throughout the world earthquakes can pose a significant risk to the integrity of dams and reservoirs. In areas of low seismicity such as the UK, the risk is more remote but the potential hazard is still sufficiently large that it should be considered. To address this, a guidance document to assist engineers in the seismic assessment of dams in the UK has been published by the Department of the Environment, Charles et al (1991). However, there is little awareness of the merits of alternative analytical routes, software packages and degree of rigour required. This paper discusses these in relation to a practical example.

2. Sir William Halcrow and Malcolm Dunstan and Associates were asked to assist Balfour Beatty and Andrade Gutierrez in preparing an alternative design in Roller Compacted Concrete (RCC) for the 127m high Foz Coa dam in Portugal. The arch-gravity design would have had a crest length of approximately 555m.

3. A requirement to assess the seismic capability of the dam was introduced at a relatively late stage in the process and it was therefore necessary to undertake a three-dimensional seismic analysis of the dam within a very compressed timescale. A relatively simple mesh was therefore used, so that it was quick to generate, check, analyse and process.

4. The results of the seismic assessment were presented within the allotted timescale. However, it was felt that a follow-up study to determine the quality of the results from

the greatly simplified analysis would be worthwhile. This paper presents the results of this follow-up and gives recommendations based on the findings of this case study.

TECHNICAL BACKGROUND

General

5. There are three basic components of the system which need to be accounted for in the analysis, namely, the dam structure itself, the foundation strata and the water contained by the dam. Whilst in a rigorous analysis it is possible to explicitly represent all three components, in a simplified analysis this is not possible for the foundation and reservoir water, instead some major simplifications are introduced. The main considerations in possible representation of the system are discussed below.

Representation of the Structure

6. The finite element method is now widely used within civil engineering, especially for the analysis of large or complex structures, and there are many packages available on the market. Those suitable for carrying out dynamic or seismic analysis are somewhat fewer in number but increasing steadily. If a non-linear representation is required then the choice of suitable programs becomes relatively restricted.

7. In a 3-dimensional analysis of a concrete dam the choice of elements will normally be restricted to thick shell finite elements or to solid or "brick" elements. The former would be suitable for an arch dam with a relatively thin cross-section, the latter for a gravity dam or thick arch dam.

8. After the choice of element type, the principal consideration is the degree of mesh refinement. The factors to be considered will reflect the purposes of the model and the degree of accuracy required, for example whether the analysis is for a scoping study or for a detailed design. Similarly, if only mode shapes and accelerations are required as opposed to stresses, a less detailed model will provide satisfactory answers.

9. In formulating a model it is therefore important to know:

- The capabilities and accuracy that can be achieved by the different element types within the software code being used;
- The tolerance of the elements to aspect ratio and element shape considerations;
- The purpose of the model and the type of results required;
- The approximate answers to be expected.

10. It can be seen that previous experience of the finite element package and of the problem to be solved is a pre-requisite.

Representation of the Foundation and Ground Motions

11. The proper representation of the ground in dynamic ground structure interaction problems is so complex that no current software package fully addresses the problem.

12. The principal effects in terms of the seismic motions at foundation level are:

- Modification of the seismic ground motions by the configuration and characteristics of the site in the absence of the structure.
- The effect of excavating and inserting a foundation into the site; this has a tendency to average the motions at the interface between the structure and ground and is termed "kinematic interaction".
- The seismic loads imposed on the foundation by the structure will cause deformations in the ground. This is termed "inertial interaction".

13. In comparison to a "fixed base" analysis, the response characteristics of the structure are modified by the ground. The principal effects being a lowering of the natural frequencies of the system because of the added flexibility of the ground and an increase in damping by propagation of foundation motions into the ground. This latter effect is termed "radiation damping".

14. As it is virtually impossible to specify rationally the true free field motions in three dimensions, it is common practice in this form of analysis to assume a uniform ground motion input into the model with horizontal and vertical motions being represented by vertically propagating shear and compression waves respectively. Time history or response spectrum inputs may be used to represent the input.

15. Varying degrees of sophistication are possible in representing the ground compliance. The simplest is to represent the foundation by a system of linear springs, however if dashpots are included to represent damping than a non-linear software package is required. An alternative is to use a linear finite element mesh in which the ground is assumed massless, thus neglecting inertial and damping effects in the ground. Some packages, such as EACD-3D, Fok et al (1986), will allow substructuring which permits separate idealisations of the ground and structure.

16. More sophisticated analyses will make allowances for the non-linear and frequency dependent nature of the ground behaviour. Some software packages will also allow special boundary conditions which represent the infinite extension of the mesh into the free field to allow the proper representation of damping. However, the realisation of this sophistication in three dimensions is very complex and to date has only been implemented for practical purposes in 2-dimensional applications such as in the program FLUSH, Lysmer et al (1975).

Representation of Impounded Water

17. The acceleration of a surface in contact with water results in hydrodynamic pressures being exerted on the surface. This effect is often termed the "added mass effect" as the resultant is akin to an additional mass moving in sympathy with the surface acceleration.

18. A number of studies have investigated the dam-reservoir interaction. Factors which need to be considered are the inclination of the dam face, the depth of water, the length and shape of the reservoir and whether the effect of compressibility in the water is significant. For small dams the assumption that water is incompressible and inviscid will only introduce a small error. A rule of thumb which has been applied is that if the natural frequency of the dam is less than half the frequency of the reservoir then compressibility can be ignored, where the frequency of the reservoir is given by the velocity of sound in water divided by four times the reservoir depth. Similarly, if the length of the reservoir is greater than twice its depth, the effect of length can be neglected. In any decision on modelling of dam-water interaction, reference should be made to the extensive literature on the subject, eg. Hall et al (1983), Nath (1982) and Zienkiewicz et al (1986).

19. The simplest representation is to model the water as an added mass, using for example the work of Zangar (1952) to obtain a distribution for this mass. Some consideration is required to avoid introducing unrepresentative effects such as inertial forces that are not perpendicular to the dam face under motions that may be in any direction. More sophisticated models will represent the water with finite elements adjacent to the dam and a boundary condition on the upstream side of these elements to represent an infinite extent of the reservoir as is possible when using EACD-3D.

20. In programs such as EACD-3D it is possible also to represent the absorptiveness of the bottom and sides of the reservoir which can have a modest effect on the hydrodynamic pressures.

FOZ COA DAM DESCRIPTION

21. The design was for a roller compacted concrete arch-gravity dam, with a crest length of 555m and is 127m high at its centre. It had a central portion with a 227m constant radius arch with a 92° included angle. To either side are straight sections with a length of approximately 100m. The valley sides are such that the dam is approximately symmetrical about its centre line. A plan view and elevation are illustrated in Figures 1 and 2.

22. The dam is founded on rock with an assumed dynamic Young's modulus of 12GPa, the comparable figure for the concrete being 24GPa.

MODELLING CONSIDERATIONS FOR TENDER ANALYSIS

23. The seismic capacity of the structure was to be determined in relation to the peak ground acceleration which caused the vertical direct stress to exceed the tensile strength of the structure. The timescale limitation was such that the simplest possible model, commensurate with obtaining an indication of seismic capacity for tender purposes, was required. IMAGES-3D was the software package used for the analysis. This is a menu driven linear elastic package for P.C.s with excellent graphics facilities and is ideal for producing quick answers from reasonably simple models.

24. The sectional thickness of the dam is such that brick elements are required to model the dam. At the base of the model, two elements are the practical minimum in the through thickness direction, this was reduced to one element at the top in order to limit the total number of elements and to maintain aspect ratios within sensible limits. The elements are elongated in the circumferential direction, again to minimise the number of elements. This is acceptable as the stress gradients in this direction are much less than in the other directions. Figures 3 and 4 illustrate the model used.

25. The foundation was modelled by springs. The spring stiffness being determined from the frequency independent values for elastic half spaces, ASCE (1986). Each foundation node was supported by springs in three orthogonal directions with the vertical stiffness distributed in the through thickness direction to give the desired rocking stiffness about a circumferential axis.

26. Added mass effects from impounded water were represented by lumped mass, with components applied in the global orthogonal directions such that the inertial forces would act normal to the surface. The magnitude of the lumped sum masses was determined in line with Zangar (1952).

27. A response spectrum analysis was performed to determine the seismic stresses. Modal and spatial combination used the Square Root of the Sum of the Squares method. These stresses were combined with those from the gravity and hydrostatic load cases. By considering two levels of earthquake, it is possible to predict the ground acceleration level at which the allowable tensile stress in the RCC would be exceeded.

28. The vertical tensile strength of the RCC at lift joints had been taken as 1.0MPa under static loading. Under transient loading it is appropriate to increase this figure by a factor of between 1.2 and 1.5. Taking the higher value as suggested by J.M. Raphael (1984) we obtain a dynamic tensile strength of 1.5 M Pa for the RCC.

29. At the upstream face a tensile stress of 1.5M Pa is only allowable at the crest of the dam. Below this level a suitable reduction must be made for water pressure as recommended in USBR Monograph 19 (1977).

Using the USBR formula we obtain an allowable tensile stress of about 1.0 MPa at the heel of the dam.

30. It was found that the allowable tensile stress would be exceeded for a peak ground acceleration greater than about 0.2g.

31. It is worth noting that a more refined mesh would be expected to give higher stresses at points of stress concentration although this would be offset to some extent by reduced conservatisms in a more rigorous analysis.

2-D COMPARATIVE STUDIES

Outline of Models Investigated

32. For ease of comparison, two dimensional models were used to test the sensitivity of results to various modelling assumptions. Aspects investigated were mesh refinement, foundation modelling and the inclusion of added mass effects. The Table below gives a schedule of the models along with the first three natural frequencies.

TABLE 1 : SCHEDULE OF 2-D MODELS

MODEL	MESH	ADDED MASS	FOUNDATION	Natural Frequencies (Hz)		
1	Coarse	No	Rigid	2.4	5.9	8.5
2	Coarse	Yes	Rigid	1.8	4.7	8.2
3	Coarse	Yes	Mesh	1.5	3.8	5.3
4	Fine	Yes	Mesh	1.3	3.3	4.1
5	Fine	Yes	Rigid	1.8	4.6	8.2
6	Coarse	Yes	Springs	1.3	3.1	3.6
7	Coarse	No	Springs	1.7	3.2	4.5
8	Fine	Yes	Springs	1.2	3.5	3.9

33. The coarse mesh of the structure had 13 membrane elements as against 200 in the fine mesh. For those models where the foundation was idealised by a mesh the coarse representation has eight elements whereas the fine representation had 300 elements.

34. For the coarse mesh there are only 3 springs on the model base and the distribution of stiffness is uniquely defined by the need to match both rocking and vertical stiffnesses. However, for the fine mesh it is necessary to assume a distribution of spring stiffness, for this study a triangular type distribution was assumed which gives a simple approximation to the parabolic distribution of pressure under a rigid footing. Spring stiffnesses were therefore largest on the two outside faces of the dam and a minimum at the centre.

Discussion of Results

35. The significance of including added mass can be easily seen by comparing models 1 and 2 where the added mass reduces the frequency of the fundamental mode by 25%, this mode being the most significant. A similar reduction in frequency occurs between models 7 and 6.

36. Similarly the effect of foundation flexibility is demonstrated by comparison of models 5 and 4 where the introduction of a mesh to represent the rock foundation reduces the fundamental frequency by 28%. From models 4, 6 and 8 it can be seen that the use of springs to model foundation flexibility can give a good approximation to the results obtained from a more sophisticated approach and that in terms of frequencies the coarse mesh for the structure is capable of giving good results. The introduction of model 3 into the comparison shows that the very crude foundation mesh in this model is overstiff.

37. The comparison of stresses is a more demanding test of the models. The comparison here is restricted to models 4, 6 and 8 as being the three which are both compatible with each other and representative of the physical nature of the problem in two dimensions. Figures 5, 6 and 7 show stress plots for the vertical direct stress when the seismic loads are added to the gravity and hydrostatic load cases for a peak ground acceleration of 0.3g.

38. Some care is necessary in the interpretation of these stress plots as the output from a seismic response spectrum analysis is unsigned which reflects the mathematical techniques employed in the summation of responses of the various modes and also that seismic loading can act in either direction. When the seismic load case is added to a signed static loading the output is an envelope of maximum (tensile) stresses. When subtracted from a static load case the plot shows an envelope of minimum (compressive) stresses.

39. Figures 5, 6 and 7 show a remarkably good correspondence indicating that the assumptions on spring stiffness are capable of giving excellent results. Figure 7 shows, not surprisingly, some differences in the stress plot for the coarse mesh, although the trends and magnitudes are broadly similar.

40. The envelope of maximum tensile stresses for model 6 shows a general reduction for the coarse mesh. The Peak stress is some 15% below that of model 4 and occurs lower in the dam, this being a consequence of the very crude modelling at the higher levels in the dam.

41. In the 3-D situation, this discrepancy will be reduced, probably to less than 10%, due to a proportion of the lateral loads being carried by arch action. In this context it is worth noting that the effect of arch action is to significantly increase the dam stiffness, the first mode being at a frequency of 2.0Hz as opposed to 1.3Hz in the two dimensional studies.

It should be noted that the three-dimensional model was not able to reproduce the possible opening and closing of the contraction joints under seismic loading. This effect should desirably be modelled in a more detailed exercise.

42. Coincidentally the direct vertical stress under the seismic load case is very similar in the 2-D and 3-D models. This arises because the stiffer 3-D model attracts higher seismic accelerations and hence inertia loads, but this is balanced by the out of plane load path.

43. It is worth noting that in the above analyses that the contribution to overall damping from the ground is underestimated and that the analyses will therefore be conservative.

44. It should be further remembered that the uncertainties is seismic engineering are very large, ranging from the ground motions to how structures actually respond. Any analysis, no matter how sophisticated, can therefore only give an indication of a general level of robustness to a seismic event.

CONCLUSIONS

1. The effects of impounded water and foundation flexibility should be incorporated in the modelling approach.
2. The use of springs to model foundation flexibility can give acceptable results in approximate analyses.
3. The fineness of the mesh should be chosen to reflect the purposes of this model; for preliminary assessment coarse meshes can give satisfactory results and this will be related to the performance of the elements in a given package.

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Figure 1. - Plan showing outline of dam and foundation contours.

Figure 2. - Developed elevation of dam showing existing ground and foundation levels.

Figure 3. - Three-dimensional mesh of dam used for tender purposes.

Figure 4. - Cross-section of dam showing finite element representation.

Fig. 5 - Stress plot showing distribution of vertical direct stress for model 4. (fine mesh of dam and foundation)

Fig. 6 - Stress plot showing distribution of vertical direct stress for model 8. (fine mesh on springs)

Fig. 7 - Stress plot showing distribution of vertical direct stress for model 6. (coarse mesh on springs)

Inundation mapping for dam failure — lessons from UK experience

F.R. TARRANT, BSc, MSc, DIC, GMIWEM, MBHS, Assistant Modeller, L.A. HOPKINS, BSc, GMIWEM, Environmental Scientist, and J.M. BARTLETT, MA, MICE, MIWEM, MBHS, Hydraulic Modeller, Binnie & Partners

SYNOPSIS. UK water companies being aware of their liabilities as dam owners have carried out dam break analyses to provide potential inundation and damage maps forming the basis on which the emergency services produce contingency plans. The potential dam failure flood waves from over 35 UK studies and advice to 5 of the major water PLCs have been statistically analysed to establish relationships between reservoir parameters, types of failure, flood wave characteristics and extent of potential inundation and flood damage. The resulting relationships can be used as broad guidelines for other dam owners considering the potential hazards posed by their dams.

INTRODUCTION

1. In 1984, the EC Directive on the Control of Industrial Major Accident Hazards (CIMAH) was introduced. This requires major industrial complexes in the UK to carry out risk assessments of their sites and operations, which enables contingency plans to be implemented in the event of an incident occurring which could pose a major threat to the environment.

2. Despite the fact that reservoirs are one of the major man made hazards, risk assessment legislation does not yet apply to them in the UK. However, there is legislation which imposes a system of safety checks on reservoir construction and operation in Great Britain, namely the 1930 Reservoirs (Safety Provisions) Act, superseded by the 1975 Reservoirs Act. The current legislation applies to all reservoirs that are designed to hold, or are capable of holding, more than 25 000 m^3 of water above the natural level of the surrounding ground.

3. There are 2 500 reservoirs which fall within the ambit of the 1975 Reservoirs Act, 20% of which are concrete or masonry dams and 80% of which are embankment dams. Approximately 70% of the 2 000 embankment dams in the UK were built prior to the year 1900 and only 7% since 1960. Less than 10% of existing embankment dams have been built since theories of modern soil mechanics have influenced the design and construction of modern dams.

4. Largely as a result of the requirements of the 1930 and 1975 reservoirs safety Acts, dams in the UK have one of the best safety records in the world. More recently though, there has been increasing consideration given to the hazard posed by

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reservoirs, particularly since privatisation of the water companies has lead to greater awareness by the new PLCs of their liabilities.

5. In an attempt to quantify their liabilities, many of the water companies have undertaken risk assessments and contingency planning for dam failures, in particular for many of the larger dams located upstream of major settlements.

6. There have been very few real dam failures and those that have occurred are invariably poorly documented. Therefore, the results presented in this paper are based on dam failure simulations using the computer program DAMBRK.UK rather than real events. The lack of past experience of actual failures means that there is limited information on which to validate the results of the program, although model simulations of a few real events for which there is some information have shown a good correlation. An example is the failure of Dale Dyke dam in 1864, details of which are given in a paper by L.A. Hopkins (1993).

RESERVOIR HAZARD ANALYSIS METHODOLOGY

7. Reservoir hazard analysis methodology to produce inundation mapping for a potential dam failure involves carrying out a dam break analysis. Such a study consists of the following tasks :-

- data review
- site visit and local survey
- mode / mechanism of failure
- breach characteristics / dimensions
- model of reservoir and dam failure using DAMBRK.UK
- breach outflow hydrograph
- valley data capture using MOSS 3-dimensional digital terrain modelling package
- valley routing using DAMBRK.UK
- sensitivity analyses
- flood damage analysis
- inundation and flood damage mapping using MOSS 3-dimensional digital terrain modelling package
- hazard assessment

Data review

8. The data review involves collating and defining details of the dam, such as crest and base elevations, spillway dimensions, outflow structures, tunnels and culverts, slope of upstream and downstream faces of the embankment, construction details, embankment materials, and inspection reports. Reservoir details are also collected, such as normal (NWL) and top (TWL) water levels, elevation - surface area / volume information. Hydrological data for the reservoir is also collected, such as the Probable Maximum Flood (PMF) or an alternative suitable inflow hydrograph and any direct inflows. All this information is collated on a spreadsheet which is designed to convert it into the appropriate input data for DAMBRK.UK.

Site visit and local survey

9. The site visit includes an investigation of the downstream valley in terms of the characteristics of the river channel, topography, bridges and embankments across the valley,

vegetation, buildings, constrictions and obstructions in the valley. Any structures considered significant enough to affect the flood wave in its passage downstream are surveyed.

Mode / mechanism of failure

10. To establish a possible failure mechanism for the dam, geotechnical and structural aspects of the embankment are studied, such as dam stability, embankment materials, construction history, local geological conditions, monitoring data and maintenance problems. From this study a "least unlikely" mode of failure is suggested. It must be emphasised that in the UK under normal high standards of embankment monitoring and maintenance, the possibility of dam failure is extremely remote.

Breach characteristics / dimensions

11. Breach characteristics are predicted using four methods for earth embankment dams; a physically based mathematical model (BREACH) written by D.L. Fread (1991), a method suggested by Hughes (1981) based on dam dimensions, and relationships derived by MacDonald and Langridge-Monopolis (1984) adapted for UK data, and a method produced by Binnie & Partners (B&P) under contract to the UK Department of the Environment (DoE) (1989). These methods provide a range of results. Breach dimensions chosen for modelling are usually those provided by the B&P/DoE method because they are based on statistical analysis of actual UK dam failures.

Model of reservoir and dam failure using dambrk.Uk, breach outflow hydrograph

12. The computer simulation package DAMBRK.UK is used to model the reservoir and dam failure mechanism, by overtopping or piping, to produce a breach outflow hydrograph.

Valley data capture using moss 3-dimensional digital terrain modelling package

13. The topography of the valley downstream of the dam is captured from 1:10 000 Ordnance Survey maps of the area using the MOSS 3-dimensional digital terrain modelling package. Any local survey data available is also included in the topographical model. A long profile and cross sections describing the valley are then produced by the MOSS package.

Valley routing using DAMBRK.UK

14. The dam failure flood breach outflow hydrograph is then routed down the valley using DAMBRK.UK, taking into account any obstructions or constrictions encountered by the flood wave. The output from the program gives tables and figures indicating maximum flood discharges, elevations, velocities and times of peaks with distance downstream from the dam, as well as discharge and elevation hydrographs at specified locations.

Sensitivity analyses

15. Depending on the particular characteristics of the dam and the conditions of the downstream valley, sensitivity analyses are

carried out on the flood routing. The most important variables subject to sensitivity analysis are the mode of failure and the breach dimensions. Valley roughness is also tested to consider its effect on peak flood levels.

Flood damage analysis

16. Flood damage analysis is carried out to estimate the level of damage that could occur at a series of locations downstream of the dam. The flood damage parameter at a specified location is related to the product of the maximum depth and velocity at that location (Binnie & Partners / Flood Hazard Research Centre, 1991). Threshold values for this parameter are 7, above which total destruction could generally occur, and 3, above which partial structural damage could occur. Below 3 there would generally be inundation damage only. The maximum elevation up to which each of these zones of damage could occur is calculated.

Inundation and flood damage mapping using moss 3-dimensional digital terrain modelling package

17. The results of the dam failure flood routing are then interpreted and mapped using the MOSS terrain modelling package. An inundation map is produced showing isolines of flood depth down the valley. A flood damage map is also produced showing zones of the three different types of flood damage - total destruction, partial structural damage and inundation damage only. This flood damage map is used as the basis for contingency planning by the emergency services.

Hazard assessment

18. A hazard assessment is then carried out to estimate the number of buildings and structures that could be effected by the different types of flood damage. The population at risk in the inundated areas downstream of the dam where the suggested critical minimum warning time of 1.5 hours for effective evacuation would not be available is also assessed. The potential loss of life is calculated using standard US Bureau of Reclamation procedures (US Bureau of Reclamation, 1986).

LESSONS FROM UK EXPERIENCE

19. The results of 35 dam failure studies using DAMBRK.UK have been statistically analysed to establish relationships between reservoir parameters, types of failure, flood wave characteristics and extent of potential inundation and flood damage. The relationships are shown in the following graphs.

20. The graphs relate a known reservoir parameter such as dam height or reservoir capacity to a parameter derived from DAMBRK.UK analysis results so that the graphs can be used as a predictive tool. In general, for each graph there is a maximum and minimum line enveloping the points, as well as a line of best fit. Therefore, when using the graphs as predictive tools to estimate the likely consequences of a particular dam failing, each graph provides a range of values within which the dam could fall, as well as a best estimate.

Fig. 1. Dam height and maximum breach discharge by mode of failure

Fig. 2. Reservoir capacity and maximum breach discharge by mode of failure

21. Plotting dam height against maximum breach discharge (Figure 1) provides a band of values showing a trend of increasing discharge with dam height. The trend is the same regardless of mode of failure by overtopping or piping. The best fit line shows remarkable similarity to the curved relationship between dam height and peak flow from Figure 2 in the Institution of Civil Engineer's "Floods and reservoir safety : an engineering guide". This curve is based on estimated flood peaks from actual dam failures provided by Kirkpatrick (1977). The similarity

indicates that the DAMBRK.UK program provides a realistic estimate of the potential peak discharge arising from a dam failure given the height of the dam, whilst the spread of results indicates the potential variability due to other factors.

22. The relationship between reservoir capacity and maximum breach discharge (Figure 2) shows a similar good correlation between the two parameters. Again there is no distinct difference in the trend between the two types of failure mode.

Fig. 3. Dam height and time to maximum breach size by mode of failure

Fig. 4. Dam height and time to maximum breach size by embankment type

23. For many of the dams studied there was a direct relationship between dam height and time to maximum breach size (Figure 3), assuming that the breach erodes to the base of the dam, because for earth embankments breach height is one of the parameters used in estimating the failure time. However, in several cases, mainly where the mode of failure was by overtopping, there were reasons why this relationship was not straight forward. In cases where there were small earth embankment dams and large inflows, the time to maximum breach size would not just be a function of dam height because once the breach had reached the base of the dam it would continue to erode width ways while a high rate of inflow was sustained.

24. When the relationship is plotted by embankment type it is clear from Figure 4 that masonry dams do not fit the general relationship. From past evidence of the failure of concrete dams, they tend to have a partial breach as one or more monolith sections formed during the construction of the dam are forced apart and over turned by the escaping water. It is difficult to predict the number of monolith which may be displaced or fail. The time for breach formation could be in the order of a few minutes.

Fig. 5. Dam height and breach base width by mode of failure

25. As for time to maximum breach size, there is a direct relationship between dam height and breach base width (Figure 5) because for earth embankments dam height is one of the parameters used in estimating the base width of the breach. However, as for the breach time, there are exceptions to this rule. Again, small earth embankment dams subject to large inflows would be eroded to the base of the dam but then the erosion would migrate sideways while a high rate of inflow continued.

Fig. 6. Dam height and length of total destruction

Fig. 7. Dam height and length of partial structural damage

26. The relationships between dam height and the length in kilometres downstream of the dam of the two types of flood damage, total destruction (Figure 6) and partial structural damage (Figure 7), show a broad correlation between increasing dam height and increasing length of damage. A curve can be drawn on each graph indicating the maximum damage that could occur.

For example, from Figure 6, if a given dam is 25 m high, there could be total destruction downstream for a distance of up to 45 km. The actual length of damage, and the elevation up to which the different types of damage could occur, will depend on the topography of the downstream valley, in particular the valley slope and shape.

27. The grouping of points on Figure 6 indicates that for about 90% of dams total destruction would probably not affect more than 10 km downstream of the dam, irrespective of dam height. Figure 7 shows that for 90% of dams, regardless of height, partial structural damage is unlikely to affect more than 20 km of the downstream valley.

28. Reservoir capacity plotted against total destruction or partial structural damage shows similar relationships to dam height related to flood damage.

CONCLUSIONS

29. The general relationships described in this paper provide broad guidelines only which can be used as tools to estimate the potential consequences of a dam failure based on known dimensions of the dam.

30. Given dam height and/or reservoir capacity, an estimate of the maximum breach discharge resulting from the failure of the dam can be obtained. Knowing the dam height, the DAMBRK predicted time to maximum breach size, the breach base width, and the possible extent of total destruction and partial structural damage in the valley downstream of the dam can be estimated.

31. These estimates can be obtained from the best fit lines on each graph. However, it must be appreciated that the relationships are based on only 35 dam failure simulations using DAMBRK.UK and not on real events, although the results fit well with results from known dam failures. Therefore, minimum and maximum envelopes are also provided on each graph within which the results for the dam could fall.

32. Not every case fits these relationships. There is variability due to factors other than the known dam dimensions used in the relationships. In addition, the limitations of the DAMBRK.UK program are generally mathematical where certain assumptions are made which may or may not be applicable to the model of a particular dam.

33. This means that although these guidelines provide broad outline estimates of some measures of the potential consequences of a dam failure, they do not avoid the necessity for carrying out a dam failure simulation as part of a reservoir hazard assessment for each dam.

34. Each dam should be considered and modelled individually, especially in terms of its potential modes of failure and breach dimensions since these are critical in determining the characteristics of resulting dam failure flood wave.

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The seismic behaviour of gravity dams in areas of low seismicity

C.A. TAYLOR, BSc, PhD, MICE, Reader, W.E. DANIELL,BSc, PhD, SERC Research Fellow, R.A. MIR, Research Assistant, M.S. SIMIC, Research Assistant, Earthquake Engineering Research Centre, University of Bristol, and J.L. HINKS, Principal Engineer, Sir William Halcrow& Partners

SYNOPSIS. Methods for assessing the potential seismic behaviour of typical British gravity dams are reviewed. The results of research in progress at the Earthquake Engineering Research Centre at Bristol University, including physical scale modelling and numerical analysis, are used to illustrate the basic mechanics of the dynamic behaviour of gravity dams.

INTRODUCTION

1. Over the past decade, engineers have become increasingly aware of the potentially damaging impact of a moderate magnitude earthquake occurring in a populous area of generally low seismicity. An example was the magnitude 5:3 Newcastle, Australia earthquake of December 1989 which killed eleven people and caused damage estimated at over £500m (EEFIT, 1990). Although a relatively small earthquake by global standards, the Newcastle event occurred near a large population centre where structures had not been designed for seismic loads. Such conditions are typical of Northern Europe, including the United Kingdom. A similar magnitude event occurred at Bishops Castle, mid-Wales, in 1990, but because of its remote location caused little damage (Pappin, 1991). The risk of such events occurring anywhere in the UK has prompted a review of the seismic performance of all types of dams and the production of a guidance document (Charles et al, 1991).

2. This paper examines methods for assessing the potential seismic behaviour of typical UK gravity dams and draws on research in progress at the Earthquake Engineering Research Centre at Bristol University. It reviews the basic mechanics of the dynamic behaviour of gravity dams, citing evidence from prototype and laboratory model tests and from numerical analyses. Reference is made to a study of the 56m high Claerwen gravity dam in Wales, but the overall conclusions are applicable to most gravity dams.

UK SEISMICITY

3. The starting point of any seismic review is the assessment of the likely seismicity. A detailed review of the seismicity of the UK is beyond the scope of this paper, but a brief overview is appropriate. The Newcastle, Australia

earthquake has already been cited as a typical example of an intra-plate earthquake of the type expected in the UK. Unfortunately, no ground motions were recorded for this event. However, witnesses reported that strong motions lasted for about five seconds and probably included two or three strong cycles. There was some evidence to suggest a strong vertical acceleration component. Intra-plate earthquakes are usually the result of local fault movements rather then direct tectonic plate movements. As a consequence, their focus is often shallow and close to the site in question, leading to locally strong ground motions even though the total energy released is small.

4. Recently, the nuclear industry's Seismic Hazard Working Party (SHWP) has conducted extensive studies of UK seismicity (Mallard et al, 1992, Muir Wood and Mallard, 1992). The SHWP studies have suggested a horizontal Peak Ground Acceleration (PGA), for a 1 in 10,000 year event of around 0.33g (Mallard and Woo, 1991), based on a magnitude 6.0 earthquake. This is broadly consistent with the PGA of up to 0.375g for a 1 in 30,000 year event suggested by Charles et al (1991). Such a PGA is significant and would be expected to cause more than superficial damage to many types of structure. No strong motions of this level have been recorded in the UK, but Fig.1 shows a synthetic ground acceleration history considered to be representative of UK conditions. It lasts for approximately 10 seconds and has a period of about 4 seconds of significant shaking. It was derived from the response spectrum shown in Fig.2. This response spectrum was developed in an early study of UK seismicity (Principia Mechanica, 1981) and has been the basis for most nuclear facility design since 1980.

THE SEISMIC BEHAVIOUR OF GRAVITY DAMS

5. The seismic behaviour of any structure is a vibration problem, the exciting force being the random motion of the ground on which the structure is founded. The dynamic loads act in addition to the normal static loads and lead to fluctuating displacements, strains and stresses in the structure. The objective of a seismic analysis is to determine the magnitude of these parameters and to assess whether the structure will suffer any distress due to the combined action of the static and dynamic loads. Providing the stress and strain levels in the concrete and foundation remain in the elastic range, a gravity dam will resonate at specific natural frequencies. If the stress and strain levels exceed the elastic range, the dam will respond inelastically and the concept of resonance will not be strictly applicable. The dynamic response of a gravity dam is influenced significantly by its dynamic interaction with the reservoir and the foundation rock. These factors will be considered briefly.

Dam-reservoir interaction

6. When the reservoir interacts dynamically with the dam, its effect can be seen as a combination of an apparent added-mass to the structure and also added damping. The added mass results from hydrodynamic pressure waves in

the reservoir remaining close to the structure and it can increase the effective mass of the dam by up to 30%. This will tend to reduce the natural frequency of the dam and alter the inertia forces associated with the body of the dam. The added damping results from hydrodynamic pressure waves which propagate away from the dam. This radiation damping is thought only to be important when the ratio of the fundamental resonant frequency of the reservoir to the fundamental frequency of the dam is less than 2.0 (Fenves and Chopra, 1984). The fundamental frequency, f_r, of a reservoir of constant depth, H metres, is given by $f_r = c/4H$, where c = velocity of sound in water in m/s (approximately 1440m/s). f_r represents the first vertical, or 'bouncing', mode of the reservoir. If the dam vibrates at or close to this frequency; energy will be transferred from the dam into the reservoir and will radiate away through the water, thus damping the response of the dam. The reservoir will also have higher order modes, dominated by surface displacements. These will interact with the dam, changing its natural frequencies and damping.

7. Fig. 7 shows analytical modes of a 100m high gravity dam computed from a finite element model which included displacement-based fluid elements modelling the reservoir (Greeves and Taylor, 1992). The many, closely spaced modes of the reservoir are clear, but it is interesting to note that the dam behaves in the first cantilever mode shape in a number of them. Such 'repeated structural mode' behaviour is a consequence of properly considering the dam and reservoir as an integrated system with distributed mass and stiffness (Greeves, 1991). Prototype measurements by the Bristol EERC on a number of gravity dams have suggested the presence of repeated structural modes. Further detailed prototype tests on Claerwen dam are in progress to verify their existence and to assess their significance with respect to earthquake response.

Dam-foundation interaction

8. Similar effects to those of dam-reservoir interaction occur when the foundation rock is flexible relative to the stiffness of the dam. The dynamic interaction leads to a reduction in the effective stiffness of the dam-foundation system and hence a reduction of the overall natural frequency. Radiation damping will occur through the foundation in the same way as through the water. Material damping in the rock will also occur. Both these phenomena will remove energy from the dam and reduce its overall response.

9. The extent of radiation damping through the foundation depends on the local geology. Where the rock is uniform to a large distance from the dam, radiation damping may be significant in reducing the response of the dam. However, where the foundation rock is relatively shallow and overlies a rock of different consistency, radiation damping will be reduced as a result of wave reflection at the interface between the rock layers. Neglecting radiation damping in an analysis can lead to an unacceptable overestimation of the response of the dam, through energy being trapped in the model.

The significance of dam-reservoir-foundation interactions

10. Fig.3 shows some of the lowest natural frequencies of a 2-D section of the 56m high Claerwen dam. With the reservoir empty and a foundation rock modulus of 10GPa the dam section has upstream-downstream modes at 5.1Hz, 11.6Hz and 20.4Hz, and a vertical mode at 8.4Hz. Acceleration and displacement distributions throughout the dam are proportional to the mode shapes. It can be seen that the response tends to be amplified towards the top of the dam. When hydrodynamic effects are included, the three upstream-downstream modes reduce to 4.0Hz, 9.7Hz and 17.3Hz respectively. Increasing the foundation modulus to 22GPa, and still including hydrodynamic effects, raises these frequencies to 4.8Hz, 11.6Hz and 19.4Hz. Low level ambient vibration tests on Claerwen dam have indicated a fundamental natural frequency of about 5.0Hz.

11. The significance of these changes can be appreciated by examining the UK ground response spectrum in Fig.2. A point on the spectral curves is a measure of the amplitude of response of a structure having the given natural frequency and damping characteristic associated with the point. Noting that the curves are normalised to a PGA of 1.0g, a structure with natural frequencies between about 5Hz and 13Hz and a damping of 5% critical, will experience a *response* acceleration of nearly 3.0g. Outside this frequency band the response acceleration, and hence inertia force experienced by the structure, reduces. The first upstream-downstream natural frequency usually contributes around 90% of the overall seismic response of a gravity dam. If, as in the case of Claerwen, the first natural frequency of the dam falls close to a break point in the ground response spectrum, then consideration should be given to the accuracy of the dam-reservoir-foundation modelling and its influence on the conservatism of the analysis.

METHODS OF ANALYSIS

12. Meaningful seismic analysis of a gravity dam requires some form of finite element analysis. Simple pseudo-static approaches are usually inadequate as they neglect the dynamic response of the dam-foundation-reservoir system. There are three common approaches to finite element analyses of gravity dams; a standard finite element analysis, a frequency domain analysis and a full, non-linear analysis.

Standard finite element analysis

13. The first method uses a standard finite element program to model the dam and foundation as elastic regions. The reservoir may be modelled explicitly using fluid finite elements, or by approximate 'added-mass' techniques (Greeves, 1991). The earthquake time history response of the dam may be computed by direct integration of the equations of motions or by modal superposition techniques. Such analyses usually apply the earthquake motion

at a rigid boundary which prevents radiation damping through the foundation rock and leads to overly conservative predicted responses. Recent evaluations of various radiating boundary formulations by the Bristol EERC has led to the conclusion that the most practical numerical boundary is to attach simple viscous dampers at the boundary (Simic and Taylor, 1992a). The foundation rock should be modelled as a massless medium with elastic stiffness. The mass of the dam is included in the analysis and the earthquake motion is input at the base of the dam (Simic and Taylor, 1992b). Although this overall approach has a number of simplifying assumptions, it yields results which are comparable with more sophisticated methods, yet can readily be handled by most standard F.E. packages.

Frequency domain analysis

14. If the foundation rock and reservoirs are modelled as infinite or semi-infinite half-spaces, closed form impedance functions can be derived for each. These impedance functions are expressed in complex number form. In concept, they can be seen as a set of frequency dependent springs and dashpots applied to the boundary of the dam itself. They also implicitly model infinite, radiating boundaries for both the fluid and foundation. Being a complex number representation, the use of such impedance functions lends itself to a frequency domain rather than time domain solution. This allows frequency dependant viscous and radiation damping effects to be correctly modelled. This technique is the basis of the widely used computer program EAGD84 developed by Chopra and his co-workers (Fenves and Chopra, 1984b). The dam is modelled by elastic, 2-D continuum finite elements while the foundation and reservoir are modelled by impedance functions.

15. There are, however, some important limitations to the EAGD84 approach. It can predict overall damping of the order of 15% critical damping which reduces significantly the overall response of the dam. Such high damping is largely a consequence of the perfect, visco-elastic half-space model of the foundation rock. Obviously, this is a major simplification of the actual non-linear, jointed nature of the rock. The authors of this paper are cautious of the damping levels predicted by EAGD84, the concern being that they can be too high and therefore unconservative. This should be kept in mind when interpreting the results of an EAGD84 analysis, but the program is still a very useful tool. A further limitation of EAGD84 is that it yields no information about stresses and displacements in the foundation.

Non-linear finite element analysis

16. This is really an extension of the standard finite element analysis technique. The elastic material properties of the dam and foundation can be replaced by non-linear models which can handle cracking, crushing, joint opening and other non-linear features. A dynamic non-linear model is an expensive solution. Great care must be taken to ensure that the model

correctly represents the progression of non-linear behaviour through the dam. This is particularly important for brittle materials. It is possible for a poor model to predict entirely the wrong crack pattern, for example. Careful modelling can, however, predict the observed behaviour of real dams, such as the non-linear analysis of the Koyna gravity dam (Greeves and Taylor, 1990). Non-linear seismic analysis of gravity dams requires considerable understanding, skill and experience. It is still in its infancy and its application to most UK gravity dam problems is probably not warranted.

Strategy for analysis

17. Of the three analytical methods discussed, the EAGD84 approach is the most effective, providing its limitations are considered when interpreting the results. For the analysis of Claerwen dam and other dams, the authors have adopted EAGD84 as the primary analytical tool, but for comparison have also carried out a number of standard finite element analyses including the dashpot radiating boundaries. A set of at least six, spectrum compatible ground acceleration time histories have been used to explore the effects of time history characteristics on the response of the dam.

DAM RESPONSE

18. As already noted, the seismic response of a gravity dam tends to be dominated by its first cantilever (i.e. upstream-downstream) mode of vibration. This leads to an amplification of accelerations over the height of the dam as it oscillates to and fro. For a 50m high dam subjected to UK-type ground motions, peak crest displacements relative to the foundations are typically of the order of 20-50mm, excluding any static loads. Of greater interest, however, are the stresses induced in the dam and foundation, since from these any tendency for cracking, crushing or shearing can be evaluated.

19. Parametric studies of a typical UK gravity dam are in progress in the Bristol EERC (Daniell et al, 1991) as well as the Claerwen studies. The cross-section being considered is shown in Fig.4. Note that it has two distinct changes in the slope of the downstream face near the crest. Such a neck at the crest is a common feature in gravity dams, since it minimises the volume of material needed to resist the static loads. However, from a dynamic point of view, it introduces a potential weakness. Stresses are concentrated at the changes in slope, from where tensile cracking can be initiated. This is illustrated in Fig.6(a) which shows the worst-case tensile principal stress distributions in the typical dam for combined static and seismic loading. Two clear tensile zones are induced at the upstream heel and at the lower change in slope near the crest on the downstream face. The results shown are for peak horizontal and vertical ground accelerations of 0.3g and 0.15g respectively. No foundation radiation damping was included and so the peak tensile stresses of 1.48 N/mm^2 and 3.18 N/mm^2 are conservative, perhaps by a factor of two. Maximum compressive principal stresses were of the order of 6.0 N/mm^2.

20. Fig.6(b) shows similar stress distributions for different horizontal and vertical ground acceleration time histories, but which have the same response spectrum as those used to obtain Fig.6(a). Here, the maximum tensile stresses on the upstream face occur near the crest at the same elevation as the lower change in slope on the downstream face. In this case, the maximum tensile stresses were 3.39 N/mm^2 and 3.10 N/mm^2. Fig.6 shows clearly the influence of the ground motion time history characteristics on the dam's response. It is important, therefore, to use several time histories in an analysis.

21. The results of the parametric study to date have shown four critical zones where high tensile or compressive stresses can be induced (Fig.4). As might be expected, two of these are at the base of the dam at the upstream heel and downstream toe. The other two zones tend to occur on the upstream and downstream faces near the top of the dam, especially at the bottom of a neck and for high dams in excess of about 80m. The latter case can be explained partly by the fact that the natural frequencies of high dams are typically in the range 2-5Hz and, therefore, higher order cantilever and vertical modes, which increase flexure near the crest, are more likely to be excited in an earthquake. Also, hydrodynamic pressures are greatest a few metres below the reservoir surface. The Bristol parametric studies indicate that for well-proportioned dams (i.e. without a neck) up to about 60m high, critical seismic stress zones tend to be at the base only.

22. The discussions so far have assumed that a 2-D model is an adequate representation of a dam's behaviour. This can be justified if the dam is constructed as a series of independent monoliths and if the joints between the monoliths can be expected to allow free movement during an earthquake. But it should be remembered that earthquake ground motions have three orthogonal degrees of freedom (and most probably three rotational degrees of freedom, although these are not recorded by strong motion instruments) and a monolith will experience motions in all these directions. If, for example, the across valley *response* of a monolith is considered significant, then there might be a case for a 3-D analytical model of at least an individual monolith and perhaps the whole dam. If the whole dam acts as a single monolithic structure, its seismic strength may be enhanced, since there is the possibility of load spreading to the abutments. This, in effect, leads to wedging of the dam in the valley. Obviously, the occurrence of such actions is dependent on the overall geometry of the dam, particularly the ratio of the crest length to the dam's height, and its applicability should be assessed for each structure.

POTENTIAL FAILURE MECHANISMS

23. The essence of a seismic safety review of a dam, or indeed any structure, is the assessment of the likelihood of a failure mechanism to develop.

It is therefore appropriate now to consider potential seismic failure mechanisms in a gravity dam system.

24. A gravity dam is a simple structural system. The forces acting on the structure are easily identified (but not necessarily easily quantified) and the load paths are also clear. The rigid body diagram in Fig.5 shows the principal static and seismic loads acting on a gravity dam. The loads can be superimposed to yield a resultant force at any instant, remembering that the magnitude and direction of the seismic forces change with time. By considering the line of action of this resultant in conjunction with a knowledge of the material properties of concrete and rock, potential failure mechanisms can be identified.

25. Under static loading, the resultant is inclined downstream and ideally remains within the middle third of the base to avoid the generation of tensile stresses at the upstream heel. A thrust zone is developed roughly parallel to the downstream face, inducing the largest compressive and horizontal shear stresses at the dam base near the downstream toe. If the dam is caused to accelerate downstream, the direction of the resultant will move downstream. This has the effects of increasing compressive stresses at the toe and reducing them at the heel, often leading to tensile stresses here. Should such tensile stresses exceed the tensile capacity of the dam materials, then a crack will form. The path this crack takes will depend on the relative stiffnesses and strengths of the dam and foundation. If the foundation rock is much stiffer than the concrete, the crack will tend to propagate horizontally. If, however, the stiffnesses and strengths are similar, or if the rock is weaker than the concrete, the crack path will propagate downstream, but will also tend to be forced downwards by the thrust zone acting parallel to the downstream face. Scale model studies in a centrifuge at Boulder, Colorado (Waggoner at al, 1993) and on the SERC shaking table at Bristol (Mir and Taylor, 1994), support this view.

26. Once a crack forms, the stiffness of the dam foundation system changes. Under the action of the static load component, the crack will tend to remain open, only closing momentarily when the dam accelerates upstream. This behaviour has been confirmed by the shaking table tests. It is reasonable to assume that in a prototype structure, water would penetrate the crack at a significant pressure, probably full reservoir head, thereby increasing the uplift forces. The resultant overturning moment on the dam will increase, tending to open the crack further until a new equilibrium situation is reached. Simple, iterative hand calculations indicate that, depending on the material strength and magnitude of the dynamic response of the dam, cracking need not necessarily propagate through to the downstream face. For the dam to remain in equilibrium, the compressive stresses in the thrust zone at the toe will increase, in turn increasing the compressive and shear strengths of the materials. This would tend to inhibit horizontal crack propagation, instead forcing the crack to

follow a line of least resistance which dips downwards into the foundation. This explanation is consistent with the failure mechanisms observed in the Boulder centrifuge tests which included a flexible foundation and the Bristol shaking table tests which included a rigid foundation. In some of the shaking table tests it proved impossible to force the crack to propagate to the downstream face, despite using scaled table accelerations equivalent to prototype ground motions exceeding 1.0g. While seismically induced cracking at the base of gravity dams might not lead to catastrophic failure during the seismic event, the post-earthquake static stability could clearly be compromised owing to changes in longer term seepage paths and uplift pressures.

27. Progressive tensile cracking at the base of the dam has been suggested as the most likely failure mechanism, but this needs to be related to the magnitude of ground accelerations in order that sensible safety assessments can be made. This is the subject of continuing research in the Bristol EERC, involving non-linear finite element analysis as well as careful physical scale modelling. Initial conclusions of this work are that cracking is initiated when critical dam response accelerations are exceeded. This is to be expected of brittle materials such as concrete and rock. Shaking table tests using a single horizontal component of ground motion, which matches a UK hard rock response spectrum, have indicated that significant cracking occurs for ground motions exceeding 0.5g. The models were subjected to a series of shakes of increasing amplitude. It was observed that cracks initiated and propagated during acceleration pulses which exceeded 0.5g. Had vertical ground motions been included in these tests it is thought likely that the models would have cracked at slightly lower ground accelerations. In this case the forces acting on the dam could reasonably be expected to be increased at certain instants due to the coincidence of peak horizontal and vertical input motions, although this would depend on whether or not these peak components were in phase. Given the likely nature of a UK earthquake (i.e. a shallow focus, near-field event) it is not inconceivable that peak horizontal and vertical motions would coincide.

CONCLUSIONS

28. The Bristol research work is not complete and care must be taken when drawing conclusions. However, at this stage, it would seem that it is possible for the maximum credible earthquake in the UK (Richter Magnitude 6.0) to cause some cracking near the base of a gravity dam, but that the ground motions are unlikely to contain sufficient high acceleration pulses to cause catastrophic collapse. It should be remembered that magnitude 6.0 earthquakes are quite common world-wide. Many gravity dams have been shaken by them, but, as yet, there have been no catastrophic failures.

29. There is sufficient evidence to indicate that seismic loading on British dams deserves serious consideration. A balanced, incremental approach to seismic safety assessment is warranted whereby the methodology and detail of

the assessment process is matched to the hazard and risks associated with each structure. Complexity of analysis should only be introduced when clearly justified.

30. Of equal concern should be the seismic performance of secondary structures and systems. The natural amplification of accelerations up the height of a dam can lead to significant shaking of equipment, such as spillway gates, and secondary structures, such as parapets and bridges. Failure of these could compromise the overall safety of the dam without endangering its structural stability. Falling debris obviously constitutes a danger to personnel and ancillary structures. It is important to remember that earthquakes search out the weakest parts of a structure. Even the failure of a minor component, such as the collapse of a bookshelf onto an electrical control panel, can have serious consequences. Successful earthquake engineering, therefore, requires common sense as much as sophisticated analysis.

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Fig.1 Synthesised UK ground acceleration time history scaled to 1.0g

Fig. 2 UK hardrock response spectrum (5% damping)

Fig. 3 Mode shapes of Claerwen Dam

Fig 4 Parametric cross-section & critical stress zones

Fig. 5 Rigid body forces and crack path

Fig 6 Parametric studies - worst case stresses

Fig 7 Repeated structural modes

Rapid hazard ranking for large dams

G. THOMPSON, MSc, MICE, Partner, and P.B. CLARK, MA, MICE, Senior Hydraulics Engineer, Binnie & Partners

SUMMARY. The purpose of this paper is to present the results of a research study carried out into the prioritisation of reservoirs according to their hazard potential. The initial objective was to define a quick and simple means to rank reservoirs to define the order in which to carry out detailed hazard mapping, but the procedure has a wider application in the general prioritisation of dams according to the potential hazard they pose.

HAZARD AND RISK

1. Dams in the UK are amongst the safest in the world. There has been no incident resulting in loss of life for the last sixty years. However UK reservoirs rank among the major man-made hazards in the country (Parr, 1988). For this reason there is strict UK reservoir safety legislation. Internationally there is an increasing awareness of the hazards posed by reservoirs and the potential need for emergency planning. Of 34 countries included in a recent survey (Committee on risks to third parties, 1988), more than half require hazard mapping and emergency plans, at least for major dams.

2. There are a considerable number of reservoirs in the UK. The majority are owned by a relatively small number of Water Companies, Regional Councils, and other utilities and industrial companies. If potential hazard mapping were to be required for all UK reservoirs, these few owners would be faced with a significant task.

3. In the study several methods were considered for ranking a population of dams/reservoirs according to the relative hazard that they pose. This is different from the risk of their failure. In this paper the definitions used are that 'Hazard' relates to the consequences of a failure occurring i.e. the amount of damage and potential casualties, whilst 'Risk' is the probability of that failure.

RANKING METHODS

4. There are a number of methods that have been developed in various countries for prioritising expenditure on dams or for specifying safety standards. Most of the methods involve some assessment of the risk of dam failure. These are generally too complex to be applied rapidly to a large number of dams.

 Reservoir safety and environment. Thomas Telford, London, 1994

Table 1. *Comparison of ranking methods against the standard data set*

Reservoir Ref No	Hazard Ranking from dambreak	Height (H)		Res Vol X Height (V.H)		Subjective Assess (Then by V.H)		Res Vol (V)		Cullen (Eqn 3)		Equation 11	
		Ranking	Error	Ranking	Error	Ranking	Error	Ranking	Error	Ranking	Error	Ranking	Error
17	1	13	12	6	5	15	14	4	3	7	6	1	0
5	2	11	9	4	2	3	1	3	1	1	1	13	11
6	3	15	12	17	14	16	13	15	12	3	0	9	6
12	4	18	14	13	9	8	4	11	7	12	8	3	1
10	5	22	17	15	10	9	4	9	4	9	4	4	1
9	6	20	14	18	12	23	17	14	8	11	5	8	2
7	7	5	2	8	1	5	2	10	3	6	1	2	5
11	8	9	1	16	8	10	2	18	10	13	5	11	3
2	9	17	8	9	0	19	10	8	1	10	1	12	3
23	10	1	9	2	8	2	8	2	8	5	5	18	8
20	11	8	3	20	9	11	0	22	11	14	3	6	5
8	12	16	4	19	7	20	8	19	7	18	6	7	5
13	13	6	7	5	8	4	9	5	8	4	9	5	8
4	14	10	4	7	7	18	4	7	7	8	6	14	0
22	15	12	3	12	3	14	1	13	2	19	4	23	8
16	16	2	14	1	15	1	15	1	15	2	14	10	6
25	17	21	4	24	7	25	8	24	7	25	8	21	4
18	18	19	1	21	3	21	3	23	5	23	5	18	0
1	19	14	5	14	5	22	3	12	7	21	2	18	1
14	20	23	3	23	3	13	7	21	1	17	3	16	4
19	21	24	3	22	1	12	9	20	1	22	1	15	6
15	22	7	15	11	11	7	15	16	6	15	7	22	0
24	23	3	20	10	13	6	17	17	6	16	7	20	3
21	24	25	1	25	1	24	0	25	1	24	0	25	1
3	25	4	21	3	22	17	8	6	19	20	5	24	1
		Sum of errors	= 206	Sum of errors	= 184	Sum of errors	= 182	Sum of errors	= 160	Sum of errors	= 116	Sum of errors	= 92

5. Simple hazard ranking methods are used by many organisations, (in some countries enforced by legislation) to categorize dams for hydrological and earthquake design requirements. These include for example the Institution of Civil Engineers' guideline, "Floods and Reservoir Safety" (ICE, 1989), and the American Society of Civil Engineers' classification system (ASCE, 1988).

6. The ICE method, and others developed for the same purpose - ie. design standards - do not provide sufficient differentiation in their ranking system to be of practical use for hazard ranking. Thus a large proportion of the reservoirs in the UK, which come under the requirements of the Reservoirs Act are classed in Category A under the ICE guidelines. What is required is a system which will allow a more detailed comparison of the reservoirs within the broad categorisations of that and similar systems.

BASELINE FOR COMPARISON OF THE RANKING METHODS

7. To compare the accuracy of the ranking methods a "standard data set" of 25 typical UK reservoirs has been analysed in detail by carrying out dam break analyses and assessments of the potential downstream damage.

8. In order to accurately rank the 25 reservoirs according to their hazard potential the computer program SMPDBK was used, this is a simplified version of the US Weather Bureau program DAMBRK.

9. For all the reservoirs considered a standard set of failure parameters were adopted as follows:-

- failure (of earth dams) by overtopping;
- the breach erodes vertically at 0.6 m/min;
- final breach width = twice the height of the dam;
- the water level was set at the dam crest level;
- there was no inflow to the reservoir.

10. The cross-sections of the valley downstream were taken from the OS maps at a scale of 1:10,000, and were chosen to be reasonably representative of the valley. The maximum extent of any model was approximately 100 km. Inundation boundaries were interpolated and properties in the inundation zones were counted. Buildings such as schools, hospitals, police stations, public utilities and factories were identified where possible and noted separately.

11. The consequences of a dam failure can be classified in two general categories: damage to property, and damage to people. Property damage, whether direct or indirect (i.e. damage caused directly by the flood, e.g. a property destroyed; or damage caused for example, as a result of loss of production facilities) can be expressed in monetary terms. It is a much more contentious subject as to whether potential damage to people - i.e. injury and loss of life - can or indeed should be so expressed. Recent research in the UK (Binnie & Partners 1989) concluded that for most purposes connected with dam safety, potential consequences of a failure should be expressed in terms of both the monetary value of the potential property damage and the potential number of casualties, but no attempt should be made to assess the cost of a life.

12. The population at risk downstream of a dam is a function

of the properties affected although it is complicated by day/night and weekday/weekend movements of the population between private dwellings and places of work and education etc. In these studies the population at risk was derived from a count of the relevant building types and typical occupancy rates. For consistency we assumed occupancy rates applicable to day time during a week day.

13. The number of casualties in a dam failure event is very greatly affected by the warning time available. With sufficient warning the whole population could be evacuated. The US Bureau of Reclamation (USBR) have analysed the relationship between the Loss of Life (LOL) and the Population at Risk (PAR) in 27 actual failure events in the United States. They found that:-

- For warning time less than 1.5 hours: $LOL = (PAR)^{0.6}$
- For greater warning time : $LOL = 0.0002\ (PAR)$

14. Assuming warning was first given less than 1.5 hours before the dam failed it could be expected that there would be some reaches of the valley downstream of the dam with very high percentages of casualties among the population. In the lower reaches where there would be more than 1.5 hours warning before the flood wave arrived the casualty figures could be expected to be very much lower.

15. The cost of property damage is determined by both inundation damage and structural damage to property. The criteria for damage costs assessment were developed during a joint project by Binnie & Partners and the Flood Hazard Research Centre and Middlesex University, (Binnie & Partners, 1989). These relate a damage potential factor, depending on both velocity and depth, to three levels of damage, i.e. total destruction, partial structural damage and inundation damage only. The pattern of damage typically shows a high cost per building in the upper reaches, and a very much lower one in reaches distant from the dam. The dam break analyses show that the magnitude and power of the flood waves are maintained for a considerable distance and the changeover from 'high' cost per building to 'low' cost occurs a surprisingly long distance from the dam - 20 km or more being not unusual.

16. The cost of property damage was found to follow a very similar pattern to the that of casualties down the valley. Therefore the estimation of either one or the other will give a relative measure of the total hazard posed by the reservoir considered. As it is slightly easier to define, the calculated potential "Loss of Life" was therefore used as the relative hazard index for each of the 25 reservoirs.

HAZARD RANKING METHODS

17. The basic approaches are discussed in the following sections.

Reservoir size

18. Hazard ranking based solely on parameters associated with the reservoir is the simplest and easiest of all methods to apply. It requires no subjective assessment and the reservoir dimensions should be readily available.

19. The criticism of such a method is that it takes no account

of the reservoir location relative to any community or urban areas or the size of such communities.

20. The most obvious parameters to use are:-
- the reservoir volume, V;
- the height of the dam H;
- the volume x dam height, which is often used as an index for breach outflow.

Subjective assessments

21. The methods classed under the general category of 'subjective' methods are those in which an assessment of the relative hazards of a population of dams is made without calculation. Such a judgement can best be made by someone who knows the valley and the reservoir - e.g. the supervising engineer. He will intuitively take into account the height of the dam and the volume of the reservoir, even if he makes no direct reference to the actual values.

22. A subjective assessment can be made extremely quickly. If that person also has experience of dam break modelling then the judgement may be reasonably accurate for they would be able to make some assessment of the passage of the flood wave and the likely order of magnitude of depths down the valley. Such an approach may be quite adequate to rank an owners' dams, particularly if the number of dams is not high. However it is likely that at best such a method could realistically rank a population of dams in only a few broad categories, probably no more than 5, ranging from low to high hazard.

Index/points systems

23. A more objective ranking system is to utilise actual parameters of the dam and valley. The method based on reservoir size is the simplest form of such an approach and this can be extended to cover the downstream valley and the location and size of communities.

24. This concept was first proposed by Sarkaria. He developed the idea of a Dam Hazard Potential Index and suggested that it would be a function of the following parameters:-

Reservoir volume	V
Dam height	H
Distance to principal population centre	L
Drop in elevation over distance L	Δy
A value index of the population centre	P
Index of relative stability of the dam	T_d

From which he proposed a Dam Hazard Potential Index as:-

$$DHPI = f(V, H, 1/L, \Delta y^2, P, T_d) \qquad .....(1)$$

25. Strictly, the inclusion of the index, T_d, into the function brings an element of risk into the assessment as it involves a judgement on the likelihood of failure of the dam. The problem in applying Sarkaria's ideas lies in how to assign relative values to the various parameters, what the actual function is and how it can be calibrated.

26. An extensive literature review has uncovered no applications of Sarkaria's method, or indeed any development of his ideas for hazard ranking. Elements of his proposals have been included in many mainly American risk-based systems, e.g. McCann at Stamford, Vanmarke, Van Thun. These all require estimation of the risk of failure of the individual dams in varying degrees of detail and are thus not 'hazard assessment'. Similarly Hagen (Hagan, 1982) has developed for the USBR a points-based rating system which is frequently described as a 'Hazard Rating' System. However Hagen's method is primarily a risk-based analysis, rather than a hazard analysis as defined above.

27. Cullen, in a private communication, has suggested a hazard ranking system based on the potential energy of the reservoir relative to the centre of the community (or communities) at risk:-

$$\text{the Hazard Index (H.I.)} = f(V.\Delta y) \qquad .....(2)$$

where V is the reservoir volume and Δy the drop in elevation between the reservoir and the centre of the community.

28. Though elegantly simple to apply, there are two drawbacks. Firstly, it takes no account of the size of the community downstream. Secondly, the Index indicates that the further the community is downstream of the dam, and hence the greater the potential drop of the flood wave, the greater the hazard. In reality the hazard is greater for communities near to the dam.

29. Including a population index in Cullen's basic index, equation (2), creates a complication in that the Δy term also applies differently to each community, and it is perhaps better to redefine the index as:-

$$H.I. = V.\left(\frac{P_1.\Delta y_1}{L_1} + \frac{P_2.\Delta y_2}{L_2} + \ldots + \frac{P_n.\Delta y_n}{L_n}\right) \qquad .....(3)$$

30. However, the equation still does not include all of the relevant parameters. In order to overcome these deficiencies we returned to the generality of Sarkaria but without the index for dam stability. We concluded that the parameters he chose are probably the most important ones, but there are others, such as length and surface area of the reservoir, shape of the valley cross-section and resistance characteristics of the valley. Generally these are likely to have only secondary influences, though the valley shape could be more significant. To test the form of the function we used the generalised power function form given by:

$$H.I. = V^t\ H^u\ L^v\ S^w\ Sh^x\ p^y\ d^z \qquad .....(4)$$

where the variables are as defined above but in addition:

S = the valley slope;
Sh = a valley shape factor;
p = the linear extent of community along the valley;
d = a density factor for the community.

31. In order to calibrate the model we grouped the parameters

in three groups each representing a major feature of the dam break wave and the potential damage. These were:-

- the breach formation and the release of a flood at the dam;
- the routing of that flood downstream;
- the location and extent of the communities at risk.

The Hazard Index can then be defined as:-

$$H.I. = f(F, A, P) \qquad .....(5)$$

where:

F is a measure of the flood released from the reservoir
A is an attenuation factor along the valley and
P is a population factor defining the extent of any communities.

32. The magnitude of the flood released from a reservoir is largely a function of the size and speed of development of the breach, which can be related to V.H. i.e.:-

$$F = K.V.H \qquad .....(6)$$

where K is a coefficient of proportionality.

33. The routing of a flood down the valley is a complex phenomenon, hence the need for sophisticated mathematical modelling techniques. Any attempt to produce a relationship of the simplicity required for a hazard index can only be a very broad brush approach giving a general indication of the influence of the major parameters. Experimentation with DAMBRK program led to the finding that there is a general relationship of the form:-

$$A = \frac{Q_L}{VH} = \frac{S}{(a + bs + cL)} \qquad .....(7)$$

For the 25 dams considered the values the coefficients were best fitted :-

$a = 10.0$
$b = 1.5 \times 10^4$
$c = 5.0$

where:

V and H are in cubic metres and metres respectively;
Q_L is in m^3/s and is the peak discharge at a distance L km from the dam; and,
S is the average valley slope from the dam to the community.

34. It is not suggested that this equation enables dam break routings to be carried out simply by hand, but it does provide an approximate measure of the flow at a distance L downstream of the dam.

35. The Population Factor, P, can be considered as the multiple of p, the linear measurement of the urban length along the valley, and the width of inundation at peak discharge. This

width was subsequently assessed as a function of the local peak discharge and the valley shape parameter, Sh, which was arbitrarily selected as the measurement of valley width at a standard height of 5 m above river bank level.

36. Loss of Life was taken to be the primary Hazard Index. However, the probability of loss of life occurring is significantly higher when the warning time is less than 1½ hours. Therefore the population parameter was restricted to those areas within the zone which would be inundated within 1½ hours of the breach occurring, ie. it is assumed that no prior warning of the failure is given.

37. In order to asses this limit, we analysed the dam break simulation results and found the relationship between reservoir capacity and the distance travelled by the flood wave in 1½ hours was best fitted by a curve with equation:-

$$L_{1.5} = 13.9 + 0.00076\ V - \frac{302}{V} \qquad .....(8)$$

where $L_{1.5}$ is the distance (in km) travelled by the flood wave in 1½ hours and V is the capacity of the reservoir (in 1000 m^3). Again this is only a general approximation.

38. With these assumptions it was deduced that the Hazard Index, could be written in the general form:-

$$H.I = \frac{Q_L^x . Sh^y . p}{S^z} \qquad .....(9)$$

39. In order to determine the three powers, x, y, and z, the equation was correlated against the data set for the 25 dams and the best correlation with the ranking in produced by the dam break loss of life calculation was achieved was with:

$$x = 0.2$$
$$y = -1.5$$
$$z = 0$$

i.e. $$H.I = \frac{Q_L^{0.2} . p}{Sh^{1.5}} \qquad .....(10)$$

That z = 0 is perhaps not surprising especially as the influence of slope is to some extent catered for within Q_L. However the inverse relationship between Sh and the Hazard Index is more difficult to account for. The shape factor may influence the travel time more significantly than appreciated, and hence the distance downstream for which the parameters have been assessed.

40. Substituting for Q_L and allowing for a multiplicity of downstream communities gives:

$$H.I = (V.H)^{0.2} \sum \frac{S_n^{0.2} . p_n}{(10 + 1.5 \times 10^4 S_n + 5L_n)^{0.2} . Sh_n^{1.5}} \qquad .....(11)$$

COMPARISON OF METHODS

41. The methods for hazard ranking discussed above are compared in Table 1. In each case a measure of the accuracy of the ranking is given based on the total of the errors in the ranking of each reservoir related to the ranking produced by the dam break simulation.

42. Methods based on reservoir size give similar results as might be expected with volume tending to be dominant parameter. The general tendency is correct, but there are several major discrepancies. Thus the largest two reservoirs are in fact only medium ranking in terms of their hazard, because of their locations. Similarly the reservoir with lowest hazard is ranked by V.H as being in the upper 50%. Ranking by height alone gives some very misleading hazard estimates.

43. Subjective assessments of the relative hazard of each reservoir were made by supervising engineers responsible for those dams. They were ranked in terms of 'high hazard'; 'high/medium hazard'; 'medium hazard' etc., and these assessments have been converted to a points system ranked from 1 to 10 where 1 represents a high hazard and 10, very low hazard. One of the problems of these assessments lies in the finer ranking of reservoirs at the high hazard end of the scale. Thus over 50% of the reservoirs were ranked as 'high hazard', category 1. Generally these reservoirs do lie in the top half of the standard ranking table but there are some notable exceptions. Thus the most hazardous reservoir, as ranked by the Loss of Life calculation was assessed subjectively as only category 3; while the least hazardous was assessed in category 1.

44. Variations on the subjective assessment method were considered to try to provide a more detailed ranking The reservoirs that had been subjectively assessed were further ranked by the volume x height . It is this double ranking - i.e. firstly by general category, secondly by V.H - that is shown in Table 1. The ranking is not particularly good and ranking by volume alone appears slightly better.

45. Cullen's method, despite its simplicity, gives a reasonably good correlation. There are one or two notable discrepancies with the reservoir with the highest hazard being ranked several places lower by Cullen's method, whilst that placed second by Cullen's method had an actual hazard below average. In fact both these reservoirs have somewhat unusual features which may have influenced their hazard ranking in the reference data set. The former is near to the centre of an urban community with several large buildings requiring individual assessment of the numbers. The total Population at Risk and hence the hazard assessment could have over estimated. The latter reservoir is large and with several communities downstream but these contain very few factories or commercial premises. The resulting population at risk is thus relatively low.

46. The final derived equation (11) provides the best correlation, as would be expected in view of the fact that it has been calibrated against the ranked data set. The most notable discrepancy is the second most hazardous reservoir which equation (11) ranks much lower. It is however the only method which correctly identifies the most hazardous reservoir.

CONCLUSIONS ON RANKING COMPARISONS

47. The various hazard ranking methods do give very different answers. It could not be expected that any of the methods would give exact correlation with the detailed dam break analysis of the standard data set as in many cases there are small differences between the relative hazard assessments in that standard set. Minor changes in the assumptions would undoubtedly change the ranking of some of the reservoirs.

48. Overall however, as might be expect in view of the way the method has been developed, the proposed hazard index system in Equation 11 provides the best comparison with the standard ranked set.

49. Hazard ranking based on reservoir dimensions only has a low accuracy and can give misleading indications.

50. The subjective assessments were generally moderately good but with some surprisingly erroneous hazard assessments which significantly reduces the overall accuracy. This is probably because of the difficulty in such cases of visualising the flood release and the damage that could occur.

51. The methodology was derived to determine the order in which to prepare flood hazard maps. However, the methodology has a wider potential application in that it allows large numbers of reservoirs to be rapidly compared in terms of their hazard potential.

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