Reservoir management, risk and safety considerations

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SYNOPSIS. Risk assessment techniques are being increasingly applied to portfolios of reservoirs in the UK and overseas. While hydrological and mechanical/electrical risk can be reliably evaluated using modern techniques, geological and geotechnical risks are more difficult to quantify. The calculation of seismic risk might appear fairly straightforward, but it poses a number of challenges because a severe earthquake may discover weaknesses in the dam or reservoir rim that were not identifed before the event. At larger dams with gated spillways, the probability of mechanical/electrical malfunction can be significant. A simple methodology for the quantification of each major class of risk is described with the aim of calculating a probability of failure for each dam. This can then be multiplied by a figure representing the financial consequences of failure in order to yield an annualised figure of the magnitude of the risk, which can then be used in ranking the portfolio.

INTRODUCTION

Risk analyses have been increasingly used for engineering applications over recent years. In 1982 a House of Lords Select Committee recommended that the techniques should be applied to reservoir safety and this led to the publication, in 2000, of CIRIA Report No C542 entitled “Risk Management for UK Reservoirs”.

The paper describes techniques of risk analysis for reservoir safety that have been developed for use in the Balkans, the Caribbean and elsewhere. The methodology has many similarities to that in the CIRIA Report but adopts a definition of risk which is in use in Canada (Hartford, 1997) and Switzerland:

Risk (£/year) = consequences of failure (£) x probability of failure (per year)

Long-term benefits and performance of dams. Thomas Telford, London, 2004

The methodology differs from that in the CIRIA Report in that it seeks to quantify likelihood as an annual probability and consequences in terms of £ or $. The advantages of this approach are:

(a) the risk can be expressed in £/year and represents the premium that would be payable in a perfect market to insure the dam

(b) a portfolio of dams can be ranked according to the calculated risk that they pose

(c) account can be taken of all the undesirable consequences of dam failure including interim costs (e.g. provision of temporary water supplies) and the cost of rebuilding the dam.

The disadvantages of the approach include the following:

(i) the difficulties of putting reliable probabilities to certain types of failure (e.g. internal erosion)

(ii) the need to allocate a monetary value to the loss of a human life

(iii) the inability to handle uncertainty other than through sensitivity analyses.

PROBABILITY OF FAILURE

The historical annual probability of failure of large embankment dams up to 1986 is given by Foster et al (2000a) as 4.5×10^{-4} per dam-year and this reduces to 4.1×10^{-4} per dam-year if construction failures are excluded. This figure should be compared with the statement by Hoeg (1996) that the probability of failure of embankment dams had reduced over a period of 30 or 40 years from 10^{-4} towards 10^{-5} per year. Charles et al (1998) have shown that in the period 1831-1930 in Great Britain the occurrence of a failure causing loss of life was 3×10^{-4} per dam-year. However, since the introduction of reservoir safety legislation in 1930 and up to the time of writing, no failures have occurred in Great Britain which have caused loss of life.

Probability of failure may be taken as the sum of the probabilities of failure due to the following causes:

- hydrological failure
- geological/geotechnical failure
- mechanical and electrical failure
- seismic failure

Foster et al (2000b) give the following breakdown for the causes of failure of large embankment dams prior to 1986:

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	% of total failures
Overtopping	46
Piping through embankment	31
Piping through foundation	15
Piping from embankment to foundation	2
Slope instability	4
Earthquake	2

Internal erosion thus accounts for 48 % of the failures of embankment dams. Although the term "piping" is used by Foster et al, 2000a and 2000b, piping is just one particular form of internal erosion and the three categories of piping listed above doubtless include other forms of internal erosion failures that strictly speaking were not piping failures. Where failure has occurred it will often be impossible to determine the precise mechanism of internal erosion.

Although mechanical/electrical failure does not feature in the above list from Foster et al (2000b), a more detailed list in Foster et al (2000a) indicates that 13% of failures are associated with a spillway gate. Where large dams with gated spillways are under study this mode of failure cannot be ignored.

Failures due to earthquakes represent only 2 % of the total, but it should be remembered that there are difficulties in defining failure. Dams are frequently badly damaged in earthquakes without an uncontrolled release of water taking place. This may be partly because irrigation dams are sometimes full for only a couple of weeks per year. For the Nihon-kai-Chubu earthquake in 1983 damage equivalent to failure was defined as follows (Gosschalk et al, 1994)

- sliding of slope
- longitudinal crack more than 50 mm wide
- transverse crack
- crest settlement more than 300 mm
- leakage of water

Hydrological failure

Overtopping is believed to have been responsible for about half of worldwide embankment dam failures and most of the deaths (ICOLD, 1997). This statement is supported by the statistic, quoted by Foster et al (2000b), that 46 % of embankment dam failures are attributable to overtopping.

A relationship will often be needed between return period and percentage of probable maximum flood (PMF). The growth curve in Figure 1 is derived from the figures quoted in "Floods and Reservoir Safety". It is only approximate and should probably not be used overseas without careful checking.

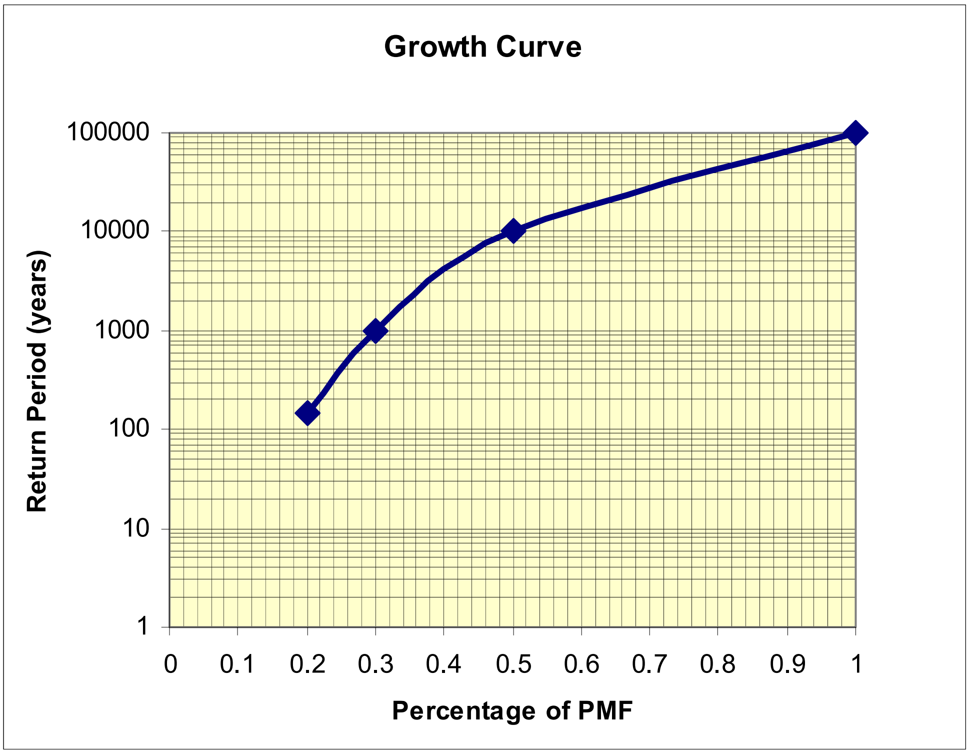

Figure 1. PMF Growth curve for UK (from "Floods and Reservoir Safety")

ICOLD Bulletin 109 argues that where the spillway is designed for, say, the 1,000 year flood the true probability of failure for hydrological reasons will often be an order of magnitude less. This is thought to be for the following reasons:

- the reservoir may not be full at the start of the storm
- wave freeboard may not be taken up by waves
- the dam may be able to withstand some overtopping.

Bearing the above in mind it should be possible to put a probability to overtopping leading to dam failure in a period of risk of, say, 100 years.

Figure 2. Orifice spillway at 51m high dam. The dam is used partly for flood control

Geological/geotechnical risk

Foster et al (2000b) attribute 48 % of embankment dam failures to internal erosion and, when taken across the whole portfolio of dams, the average probability of geological/geotechnical failure will be about the same as the average probability of hydrological failure. About half of all internal erosion failures through the embankment are associated with the presence of conduits. This has been confirmed in a study of internal erosion in European embankment dams where the ICOLD European Working Group on internal erosion in embankment dams found that in almost half the cases where failure occurred, or where failure almost certainly would have occurred very quickly if the reservoir had not been rapidly drawn down, the problem was associated with a structure passing through the embankment (Charles, 2002).

Work by Foster et al (2000b) give the average frequency of failure (during the life of the dam) due to piping through the embankment by dam zoning categories for large dams up to 1986. Some of these figures are reproduced below:

	Average frequency of failure (x 10^{-3})
Homogeneous earthfill	16.0
Puddle core earthfill	9.3
Earthfill with rock toe	8.9
Concrete face earthfill	5.3
Earthfill with filter	1.5
Zoned earthfill	1.2

It should be noted that 49 % of internal erosion failures occurred during first filling of the reservoir, 16 % during the first 5 years of operation and 35 % after 5 years operation.

In areas of steep topography particular account needs to be taken of the risk of landslides into the reservoir such as that which caused the loss of over 2,000 lives at Vaiont in Italy in 1963 (Hinks et al, 2003). This event was particularly disastrous because of the high loss of life (LOL) in the village of Longarone downstream where 94 % of the 1,348 residents perished.

Mechanical and electrical failure

The principal mechanical/electrical risk is the failure of spillway gates to open. However the following also need to be considered under this heading if not elsewhere:

- Non-operation of spillway gates because of human error
- Blocking of spillways with debris
- Non-operation of bottom outlets

During the 1987 floods in south-eastern Norway the percentages of dam owners experiencing problems were reported as follows:

Power failure	50 %
Communication Problems	23 %
Spillways not opened	19 %
Damaged Access Road	17 %
Clogging of spillways	10 %

The above illustrates the high risk of power failures during extreme events; in some environments it may be appropriate to assume that the primary power source will definitely fail. Because of this spillway gates are always provided with a standby power source the reliability of which may itself be questionable. In a recent survey the probability of failure on demand was assessed as between 0.2 % and 1.0 % depending on the details of the particular installations.

For the dam to fail the failure on demand clearly needs to be accompanied by a flood and it may be that the greatest risk to the dam is from the non-operation of all the gates in a flood of relatively modest return period.

Figure 3. 24 metre long by 5 metre high spillway gate. Synchronization between the two ends is not reliable and the gates are at risk of twisting.

Human error in the operation of spillway gates is an important factor since operators will often be reluctant to cause certain flooding downstream. This will particularly be the case if they are subject to high level political pressure not to open the gates. This needs to be factored into the risk calculations.

Blocking of spillways with debris is not strictly a mechanical/electrical problem but there have been a number of serious incidents causing major damage and/or loss of life (Hinks et al, 2003).

The non-operation of a bottom outlet is unlikely to be the main cause of the failure of a dam but it may be an important contributory factor. The problem is often the accumulation of silt or debris in front of the outlet.

Seismic failure

Most of the dams that have failed completely as a result of earthquakes have been small homogeneous dams in Japan, China and India. Another important category of failures are tailings dam, particularly in Chile where there were devastating failures in the earthquakes of 1928 and 1965. For conventional large dams those of greatest concern are those constructed on liquefiable foundations or using liquefiable fill.

CONSEQUENCES OF FAILURE

The methodology provides a mechanism for reducing the consequence of failure to a single number. For the ranking of 33 dams in Albania, Hinks and Dedja (2002) used the number of houses at risk. This worked quite well for relatively small irrigation dams up to 30 m high but is not adequate for large dams where the cost of replacing the dam itself could run into hundreds of millions of pounds. The answer is to calculate the total cost of failure including:

- loss of life.
- loss of housing and commercial property
- agricultural and infrastructure losses
- loss of dam and power station

With the aid of dambreak analyses it should be possible to quantify the above losses, although there may be complications due to uncertainty over the water level in the reservoir at the time of failure.

Loss of life

A particular difficulty arises in determining an appropriate notional cost to allocate to the loss of a human life. It has been suggested that it is inappropriate to put a value on human life and this viewpoint can be readily understood, particularly where the value chosen is much too low. However, it is emphasised that in the context of reservoir risk management, the allocation of a notional cost to the loss of a human life is being done solely to assist in ranking a portfolio of dams by risk and is not meant to reflect on the intrinsic worth of human life.

For overseas work the authors have assigned a notional cost to the loss of a human life by taking the Gross Domestic Product (GDP) per capita of the country concerned and capitalising it at an appropriate rate of interest. In the UK this methodology would give a sum of about £335,000 at 2004 prices assuming capitalisation at 5% rate of interest. This compares with a cost of £1 million to prevent a fatality quoted in the HSE booklet “Reducing Risks, Protecting People” (HSE, 2001). Probably the appropriate notional cost to put on the loss of a life in the UK is somewhere between these two values. However, doubling the assumed cost of human life will often make little difference to the order of ranking by risk.

It is worth noting that priorities for remedial works at a portfolio of dams can be ranked without the need to put a predetermined cost on the loss of a human life. If the cost of remedial works is known at each dam, it is possible to work out what the cost of human life would have to be to justify the expense of those remedial works at each dam. The dams can then be ranked giving the highest priority accorded to the dam where the cost to prevent a fatality is lowest.

In addition to determining the value of each life it is necessary to determine loss of life (LOL) as a proportion of the population at risk (PAR). A number of authors have addressed this issue and various formulae have been proposed which take account of warning time (WT):

- For WT < 15 mins LOL = 0.5 (PAR)
- For 15 mins < WT < 1.5 hrs LOL = $PAR^{0.56}$
- For WT > 1.5 hrs LOL = 0.0002 (PAR)

The data from which the above formulae were obtained were all for developed countries and mostly for the United States. LOL may well be greater in developing countries where there is less personal mobility. DeKay and McClelland (1993) have pointed out some of the limitations of these formulae.

Loss of housing and commercial property

The costs of a dambreak associated with damage to housing can be roughly estimated by taking a standard value for each dwelling. If greater accuracy is required higher values can be put on larger houses and lower values on smaller ones.

For some years various levels of damage have been defined as follows in terms of velocity (m/sec) x depth (m) – see Binnie & Partners, 1991:

V x d < 3 m^2/sec	inundation damage
3 m^2/sec < V x d < 7 m^2/sec	partial structural damage
V x d > 7 m^2/sec	total structural damage

The above relationships may understate the damage caused and it is worth noting that in the 2000 floods when 10,000 properties were flooded, the total damage was estimated at £1.3 billion, ie £130,000 per house (Watts, 2003). This compares with a figure of £ 63,000 per house for flooding in Melton Mowbray in 1998 (Kavanagh, 2003)

Agriculture and infrastructure losses
A dambreak is likely to do permanent damage to fields and agricultural infrastructure near to the dam whereas only temporary damage is likely further downstream. Depending on the season there may, however, be extensive damage to crops. Roads and bridges may also be washed away and financial allowance may need to be made for their replacement as well as for the short-term disruption to commerce whilst the bridges are reconstructed.

Loss of dam and power station
For the valuing of dams and power stations, parametric equations have been developed using dam height, dam length, reservoir capacity, installed capacity of power stations etc. This is, clearly, a very simplified approach but it has proved to be more successful than trying to update figures for the original cost of the facilities. The parametric equation used for 24 large dams in the Caribbean was:

$$\text{Cost (\$m)} = 0.65 \times MW + 0.13 \times Mm^3 + 0.52 \times h + 0.065 \times L$$

Where MW is the installed capacity at the power station in MW
Mm^3 is the capacity of the reservoir in Mm^3
h is the height of the dam in metres
L is the length of the dam crest in metres

Whilst the above equation uses readily available parameters and has proved reasonably successful it cannot be recommended for wider use without careful calibration for the stock of dams to be considered.

Where power stations are underground or a long way downstream of the dam it may be tempting to exclude the cost of their replacement from the estimates on the grounds that they are unlikely to be destroyed. However, if the dam fails, the power station is unlikely to be of much use for several years and expensive alternative generating capacity may have to be installed.

For dams in cascade it will often be necessary to assume that failure of the upstream dam will take those downstream with it.

Other costs
Where dams provide water supply to cities the cost of disruption may be high both in terms of the health of the citizens and in respect of the

development of an alternative source. These, and similar costs, need to be taken into account.

CONCLUSIONS

The methodology described in this paper is suitable for the ranking by risk of a portfolio of dams. The accuracy of the probabilities of failure in absolute terms will depend on the care taken in calculating those probabilities and on the budget available for the exercise. This will, in turn, be dictated by the purpose for which the results are required.

In the words of Cummins et al (2001):

Whilst the precise probabilities and consequences will never be known because each dam is unique and there is a lack of applicable data, these risks can be compared with others faced by the community.

This is just one advantage of seeking to calculate absolute probabilities which form a common language with engineers working in disciplines other than dams.

ACKNOWLEDGEMENTS

The authors would like to acknowledge valuable advice and assistance given by Professor J.Lewin

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