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Lessons from historical dam incidents

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or by telephoning 08708 506506.

Author(s):

J Andrew Charles, BRE
Paul Tedd, BRE
Alan Warren, Halcrow Group Ltd

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Research Contractor:

Halcrow Group Ltd.
Burdorop Park
Swindon
Wiltshire SN10 5BP
Tel. 01793 812479

Environment Agency's Project Manager:

Emma Milner
Phoenix House
Global Avenue
Leeds
LS11 8PG

Collaborator:

Building Research Establishment

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This report was produced by the Research, Monitoring and Innovation team within Evidence. The team focuses on four main areas of activity:

- **Setting the agenda**, by providing the evidence for decisions;
- **Maintaining scientific credibility**, by ensuring that our programmes and projects are fit for purpose and executed according to international standards;
- **Carrying out research**, either by contracting it out to research organisations and consultancies or by doing it ourselves;
- **Delivering information, advice, tools and techniques**, by making appropriate products available.

Miranda Kavanagh

Director of Evidence

Executive summary

This report aims to help those responsible for the safety of reservoirs. These include engineers appointed under current legislation, personnel who visit reservoirs in the course of their duties, staff who operate and monitor reservoirs, and enforcement authority engineers.

The scope of the report is limited to water-retaining structures: some types of waste impoundments, such as tailings dams, may suffer similar types of malfunction, but these are not included. Although the focus of the report is on incidents at dams in Great Britain, reference is made to a few international incidents. International experience is particularly helpful for those types of dam that are not commonly found in Great Britain.

The report begins with an introduction in Section 1. The background to the subject is briefly outlined, the value of the national incident database is demonstrated and the need for post-incident reporting and investigation is emphasised.

In the next two sections, general, technical and regulatory lessons from dam incidents are outlined. Section 2 gives a historical overview of the subject which shows how serious incidents have improved our understanding of dam behaviour and the hazards posed by these structures. This section should not only be of interest to dam engineers but should also help those reservoir owners with limited technical knowledge to develop a basic grasp of the more significant aspects of the subject. Section 3 shows the close links between historical incidents and failures and the development of reservoir safety legislation and guidance.

Section 4 looks at how incidents have been managed, including the role of owners and panel engineers. The significance of drawdown rates and other provisions for dam incidents such as evacuation planning are presented. Some examples of incident management are described.

Section 5 begins with an overview of serious incidents and a classification and brief analysis of the modes of failure. This is followed by descriptions of over thirty major incidents and summaries of seventy other incidents. There is some overlap with information presented in Section 2, but Section 5 in essence constitutes a convenient reference section for readers interested in incidents of a particular type or at a particular dam.

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In the preparation of this report by Halcrow/Building Research Establishment (BRE) substantial use has been made of work previously published on the subject:

- Binnie (1976) *The evolution of British dams*.
- Skempton (1989) *Historical development of British embankment dams to 1960*.
- Kennard (1995) *Four decades of development of British embankment dams*.

Work previously published by BRE has also been of assistance including Charles (1986) *The significance of problems and remedial works at British earth dams*, Charles (1990) *Deterioration of clay barriers*, and Tedd *et al.* (1994) *Remedial works to clay cores of UK embankment dams*.

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1 Introduction

1.1 Background

The number of casualties arising from a breached dam can be greater than from most other kinds of technological disaster. Maintaining reservoir safety has considerable importance for the public in a country such as Great Britain where a number of dams pose a high hazard, being located upstream of heavily populated and industrialised areas. Thus, although the probability of failure of a dam is generally low, the consequences of failure could be great. As most reservoirs constitute a low probability/high consequence scenario, careful management of these risks is essential.

Fortunately, few catastrophic failures have occurred in Great Britain and since 1925 there has been no loss of life due to dam disasters. Table 1.1 lists dam failures that caused loss of life in Great Britain. All the dams are embankments except Eigiau which was concrete and failed due to an inadequate foundation. Since 1925, there have been failures involving breaching of embankments and also many 'near misses' and other serious incidents (Wright, 1994).

Table 1-1 British dam failures that caused loss of life (after Charles, 1993)

Dam	H (m)	Reservoir volume (x 10 ³ m ³)	Date built	Failure		Deaths
				Date	Type	
Tunnel End	9		1798	1799	OF	1
Diggle Moss (Black Moss)			1810	1810	OF	5
Whinhill	12	262	1828	1835	IE	31
Brent (Welsh Harp)	7		1837	1841	OF	2
Glanderston				1842	OF	8
Bold Venture (Darwen)	10	20	1844	1848	OF	12
Bilberry	29	310	1845	1852	IE	81
Dale Dyke	29	3,240	1863	1864	IE	244
Rishton				1870		3
Cwm Carne	12	90	1792	1875	OF	12
Castle Malgwyn				1875	OF	2
Clydach Vale				1910	OF	5
Skelmorlie	5	24	1861	1925	OF	5
Eigiau and Coedty (Dolgarrog)	10 11	4,500 320	1911 1924	1925 1925	FF OF	16

Type of failure: IE = internal erosion, FF = foundation failure, OF = overtopping during flood

Although there has been no loss of life since 1925 due to dam disasters in Great Britain, during the last fifty years disastrous failures overseas have resulted in much loss of life as shown by the examples in Table 1.2.

Much can be learned from these failures, particularly those such as Baldwin Hills, Malpasset, Teton and Vaiont which have been the subject of detailed investigation and substantial literature. A useful start to such a study is given by Jansen in his book *Dams and public safety* (Jansen, 1980) which includes illuminating accounts of the failures of Machhu II, Teton, Frias, Baldwin Hills, Vaiont, Babii Yar, Malpasset, and Vega de Tera as well as of many other failures. Failure of Vaiont and Malpasset

are described in further detail in Section 5 of this report. Both involved large loss of life.

Table 1-2 Some international dam disasters causing loss of life

Dam	Dam type	Country	Height (m)	Res. volume (10 ⁶ m ³)	Date built	Failure		Deaths
						Date	Type	
Vega de Tera	CMB	Spain	34	7.8	1957	1959	SF	144
Malpasset	CA	France	66	22	1954	1959	FF	421
Babii Yar	Emb	Ukraine				1961	OF	145
Vaiont	CA	Italy	265	150	1960	1963	L	2,600
Baldwin Hills	Emb	USA	71	1.1	1951	1963	IE	5
Frias	Emb	Argentina	15	0.2	1940	1970	OF	42+
Banqiao	Emb	China	118	492	1953	1975	OF	#
Teton	Emb	USA	93	308	1975	1976	IE	11
Machhu II	Emb	India	26	100	1972	1979	OF	2,000
Bagauda	Emb	Nigeria	20	0.7	1970	1988	OF	50
Belci	Emb	Romania	18	13	1962	1991	OF	25
Gouhou	Emb	China	71	3	1989	1993	IE	400
Zeizoun	Emb	Syria	42	71	1996	2002	OF	20
Camara	RCC	Brazil	50	27	2002	2004		5
Shakidor	Emb	Pakistan			2003	2005	OF	135+
Situ Gintung	Emb	Indonesia	16	2		2009	IE	100

Dam type: CA = concrete arch, CMB = concrete and masonry buttress, Emb = embankment, RCC = roller compacted concrete.

Type of failure: IE = internal erosion, FF = foundation failure, OF = overtopping during flood, SF = structural failure on first filling, L = 270 x 10⁶ m³ landslide into the reservoir caused overtopping of the dam by a wave 125 m high, but remarkably the dam survived.

= It has been reported that tens of thousands died in this disaster which involved the failure of a number of dams, of which Banqiao was the largest.

News items in *New Civil Engineer* with headings such as *Dam emergency rings checking alarm bells* (23 January 2003) and *Unstable dam assessed after 10 years' neglect* (30 January 2003) show that near misses continue to occur. It was reported in *New Civil Engineer* (21 November 2002) that "reservoir engineers told NCE last week that as many as four dams and reservoirs could be at risk from bursting every year".

Given the broad scope of this subject, the report has been limited in several ways:

- Although reservoir safety is essentially an international subject, and worldwide experience is of considerable value, the report is for the most part limited to dams and reservoirs in Great Britain. International incidents are included only to illustrate types of incident that are not covered by the British database of dam incidents.
- Although waste impoundment structures such as tailings dams can present similar hazards to water-retaining dams, these are not included as separate safety legislation is currently in force.

The Reservoirs Act 1975 applies to large raised reservoirs holding more than 25,000 cubic metres of water, but this report includes information on smaller reservoirs

since, depending on their location and elevation, smaller dams can present a substantial threat to public safety and lessons can be learned from incidents at such reservoirs. Furthermore, such reservoirs are likely to be brought within the ambit of new legislation.

There are several approaches to mitigating the risk and consequences of a dam failure. The risk of failure may be reduced by structural improvements to the dam and its ancillary works and by better surveillance, monitoring and maintenance. Tighter emergency management procedures can reduce the likelihood of failure and risk of casualties should a failure occur.

The majority of British reservoirs are impounded by embankment dams, many of them built in the nineteenth century (BRE, 1994). Considering the emphasis given to slope stability in geotechnical engineering, it may seem surprising that relatively few catastrophic failures have been due to slope instability associated with inadequate shear strength or high pore pressures. Most of the failures which have caused loss of life can be attributed to the embankment breaching due to one of two causes:

- Overtopping of the embankment during an extreme flood. This hazard is largely within the province of hydrology and the selection and estimation of the design flood, and provision of appropriately sized spillway and freeboard.
- Internal erosion associated with processes such as piping or hydraulic fracture. In new dams this should be prevented by appropriately designed filters and careful design of the watertight element. Where overflow arrangements have been improved to meet modern flood standards, internal erosion is likely to be the major remaining threat to an old embankment dam which does not have filters designed to modern standards or which has a draw-off structure (culvert or unprotected pipe) passing through it or which has a deep clay filled cut-off trench.

Knowledge of the dam and its ancillary works, and of processes likely to be at work which could pose a threat to safety, make it possible to assess the hazard of internal erosion Charles (1998, 2002a). An understanding of the performance of similar dams may become more critical as the stock of dams in the United Kingdom ages. The effects of climate change, changes in operating conditions and the ageing process itself might change the patterns of geotechnical behaviour understood from historical performance and incidents. Learning from the recorded performance of dams is fundamental to improving reservoir safety and, consequently, incident reporting and the compiling of case histories are important tasks. Until recently, this was done on an *ad hoc* basis by publishing case histories, but many incidents remain unreported. Nevertheless, major failures have generally been reported and discussed in learned journals or conferences.

Since 1930, reservoir safety in Great Britain has been regulated by Act of Parliament. In the interests of public safety, the Reservoirs (Safety Provisions) Act 1930 required the owners of reservoirs with a capacity of more than five million gallons (22,700 m³) above the natural level of any part of the surrounding ground, to provide for their inspection by a qualified civil engineer who was a member of a panel of civil engineers constituted for the purposes of the Act. The Reservoirs Act 1975 went beyond the provisions of the earlier Act in a number of ways. Local authorities were designated as enforcement authorities, being required to keep registers of all raised reservoirs (defined as those with a capacity greater than 25,000 m³ above the natural level of any part of the land adjoining the reservoir) and to ensure that undertakers, usually the owners, complied with the requirements of the Act. The duties of

undertakers, enforcement authorities and engineers appointed to the various panels were laid down in the Act or set out in regulations.

A major change in reservoir safety occurred in September 2004 when responsibility for the enforcement of safety legislation in England and Wales was transferred from a large number of local authorities to the Environment Agency under the provisions of the Water Act 2003, thereby ensuring a uniform application of safety legislation across the country. The Flood Risk Management (Scotland) Act 2009 transfers the Enforcement Authority role to SEPA. Further legislative changes are planned in the Flood & Water Management Act 2010.

1.2 National incident database

The study of case histories has a major role in subjects as diverse as medicine, law and engineering design. While it is vital that practitioners have a sound grasp of the underlying principles of their subject, their personal practical experience needs to be supplemented by the study of well-documented case studies. Although the teaching of engineering science is primarily concerned with analysis, case histories should play a role in engineering education, particularly for civil and geotechnical engineers who assess the condition of existing works.

Knowledge of the history of a dam is one of the most useful and important elements in making an accurate diagnosis of a reservoir safety problem and in some cases can be more valuable than physical examinations and diagnostic tests. It should include the records of monitoring and surveillance, previous incidents and remedial works. Case histories have useful functions in a number of areas:

- (a) In dealing with an emergency, readily available documentation of the history of the dam can be a crucial factor.
- (b) In the condition assessment of a dam, knowledge of past behaviour of this type of structure is important and should help to identify abnormal behaviour. Just as no competent physician would treat a patient without first ascertaining as much of the patient's relevant medical history as possible, so no engineer should diagnose the nature of a problem or design remedial works at a particular dam, without first researching the history of the structure.
- (c) Where a particular type of problem has been diagnosed at a dam, a study of the case histories of dams with similar problems and remedial works can be a useful guide when considering the best course of action.
- (d) A collection of case histories can give an indication of the prevalence or otherwise of different types of malfunction in a particular type of dam and can provide a useful indication of the need for preventative works and of possible solutions. The Building Research Establishment (BRE) began developing a national dams database in 1988 as part of the Government's Reservoir Safety Research Programme and this included compiling data on dam failures and incidents, and remedial works (Tedd *et al.*, 1992).

Needless to say, there are limitations and problems with the study of case histories. Only a fraction of the information potentially available is likely to be readily available. The reliability of a case study involving a malfunction of a dam may be suspect because it is incomplete, or a full account might have been embarrassing for some of

the parties involved and professional ethics may forbid the exposure of details given confidentially. Published case studies are comparatively rare and may be untypical, and on occasion may have been carefully selected to prove a particular theory. The BRE Bibliography of British Dams provides a comprehensive publicly available list of published references (the bibliography can be accessed on the website of the British Dam Society: www.britishdams.org). The long-term preservation of data presents problems as it is difficult to store data so that it is accessible and yet secure.

The BRE database was superseded by the new national incident database in 2006 as part of work on the post-incident reporting system and this is now administered by the Environment Agency under the guidance of an independent All Reservoirs Panel Engineer. The database holds information on dam characteristics and remedial works as well as information on incidents. The database contains a substantial amount of incident data which has been useful in assessing the probability of a safety incident at a British embankment dam (Brown and Tedd, 2003). The Environment Agency also holds basic information on all statutory reservoirs in England and Wales and these provisions should provide a firm foundation for incident reporting and identifying future research needs. The national incident database is freely available to those with a legitimate need to access the information.

1.3 Post-incident reporting

An incident reporting system can be helpful in enhancing public safety in hazardous situations and such systems have been developed in many high hazard industries (McQuaid, 2002). In the context of reservoir safety, an incident can be defined as an event which differs from normal conditions and which has resulted in, or could have had the potential to result in, an uncontrolled release of reservoir water, with consequent harm to people, property or the natural environment. The most significant uncontrolled release of reservoir water is likely to be associated with the breaching of a dam, but failure of ancillary works can also cause hazardous situations. Serious incidents include:

- (a) failures in which there has been an uncontrolled release of reservoir water with consequent casualties or property damage;
- (b) 'near misses' which have not caused casualties or property damage, but which might have done had there been no human intervention; typically a near miss incident requires emergency action such as rapid reservoir drawdown, the implication being that without such emergency action a breach would be likely.

The Environment Agency reporting system covers gathering, analysing and sharing information about reservoir incidents. The system also provides for the investigation of the more serious, unusual or complex incidents, such as the Ulley incident in 2007. The system of incident reporting should help to identify and quantify trends in the behaviour of dams subject to reservoir safety legislation and provide comprehensive information on incidents that will help determine future research priorities (Charles, 2005). The system aims to:

- gather information on reservoir safety incidents;
- investigate incidents where appropriate;
- learn lessons from incidents;
- inform the reservoir industry of trends and lessons learned;

- provide information that can contribute to reservoir safety research and incident frequency data for quantitative risk assessment.

Where there are particular points of learning that should be shared, bulletins are prepared to provide an insight into an incident or group of incidents. Each year, the Environment Agency publishes an annual report on post-incident reporting to review the incidents reported over the last year and to provide an update on related research and development.

Fortunately, failures are rare and a system of reporting that also includes near misses has much to commend it. Often, more can be learned from a near miss than from a failure:

- (a) When a failure occurs, matters of blame, legal responsibility and the possibility of criminal prosecution can form a difficult environment in which to carry out a satisfactory investigation. The investigation of a near miss does not have these problems to the same extent.
- (b) An uncontrolled release of reservoir water is generally associated with a breach of the dam and evidence of the cause of failure is likely to be destroyed in the failure. With a near miss, the evidence still exists and can be fully investigated.
- (c) The much higher rate of near misses than failures facilitates meaningful quantitative analysis and provides insight into the probability of failure from different causes. Reports of near misses provide a reminder of hazards and encourage timely preventative actions. Potential failures are identified before an accident occurs.
- (d) Reports of near misses help to identify the reasons why failures do not occur. The report of an incident should reveal the barriers that prevented a near miss becoming a failure. This may shed light on whether near misses are a good guide to failures.

Where internal erosion takes place, the reason that the near miss did not become a failure may be associated with some or all of the following factors:

- early identification of the problem;
- rapid drawdown of the reservoir;
- slow development of internal erosion.

A review of the Upper Rivington incident commissioned by the Department for Environment, Food and Rural Affairs (Defra) focused on safety legislation (for which the department has responsibility) and on the effectiveness of Defra's Reservoir Safety Research Programme. The technical causes of the incident were not investigated specifically, but were not entirely excluded from the study. The *Review of operation of Reservoirs Act 1975 in relation to serious incident at Upper Rivington* (May 2002) was carried out with the cooperation of the reservoir undertakers, and made the recommendations shown in Table 1.3 (Charles, 2005).

Table 1-3 Recommendations from report of serious incident in 2002

Recommendations	Actions
The proposal to amend the Reservoirs Act to give the Secretary of State powers to direct undertakers	Following the Water Act 2003, undertakers of specific reservoirs will

<p>to prepare a plan setting out the action they would take to control or mitigate the effects of flooding likely to result from any escape of water from the reservoir should be implemented as soon as possible.</p>	<p>be required to prepare reservoir flood plans. These plans must set out how undertaker will respond in an emergency to reduce the effects of flooding due to an escape of water from the reservoir.</p>
<p>Helpful guidance on appropriate emergency procedures for rapid lowering of the reservoir in an emergency has been given in <i>An engineering guide to the safety of embankment dams in the United Kingdom</i>. However, in view of the vital role such procedures have in maintaining reservoir safety, consideration should be given to whether further guidance is required to emphasise their importance.</p>	<p>A suite of papers has been published in <i>Dams & Reservoirs</i> on the use of low-level outlets in emergency situations. The papers deal with, respectively, target capacity (Hinks (2009), risk assessment (Brown (2009a) and British Waterways' approach (Brown, 2009b).</p>
<p>A formal system of reporting serious incidents should be developed. Investigations of those incidents which might be termed a "near miss" would also be helpful. The requirements for incident investigation in the nuclear industry, for chemical hazards and for railways have been reviewed by the Health and Safety Executive (2000).</p>	<p>A system for reporting serious incidents has been developed (Gosden and Brown, 2004). The Environment Agency has taken the lead role and has developed processes and procedures for a voluntary system which was implemented in 2007 (Warren and Hope, 2006).</p>
<p>Further research on internal erosion should be undertaken. A preliminary study should define the scope of the work to ensure its relevance to the threat internal erosion poses to the British population of old embankment dams.</p>	<p>This long-term objective is being facilitated through international collaboration through the ICOLD European Working Group on internal erosion (Charles, 2002a). An outline strategy has been devised (Brown and Gosden, 2004).</p>

This report provides many other examples of how serious incidents and 'near misses' have shaped reservoir safety legislation and best practice in dam design.

2 Technical lessons from dam incidents

2.1 Dam and reservoir failure

Dams vary greatly in their design and construction. Their effectiveness and safety also depend critically on their foundation and all sites differ in their geology. Human factors affect how a reservoir is operated and how a dam is maintained, monitored and kept under surveillance. With so many variables, recording and learning from how dam incidents have arisen is challenging. This section provides an overview of the types of major incidents that have arisen over the last 200 years.

If asked to cite failures of British dams, most engineers in the reservoir industry would be able to quote Dale Dyke, Bilberry and Dolgarrog, together with recent serious incidents such as Ulley, but many would struggle to name more of the several hundred incidents that have occurred. The lack of knowledge of dam incidents can give rise to misplaced optimism with respect to the long-term deterioration of dams. This report aims to counter this by providing a broad perspective on the range of incidents that have arisen in the past and can arise again.

With most structural failures damage is limited to an area in the immediate vicinity of the structure, but the breaching of a dam and the consequent uncontrolled release of the impounded reservoir water can cause destruction over a large area downstream of the dam. The structural stability and security of such dams, therefore, is of major importance for public safety, particularly in Great Britain where many reservoirs are located in river valleys upstream of densely populated and industrial areas.

There is a long history of dam and reservoir construction in Britain. In the second half of the eighteenth century, many ornamental lakes were established in the landscaped grounds of country estates and, by the end of the century, reservoirs were needed to supply the canals rapidly being built across the country. During the first half of the nineteenth century, the demand for unpolluted water supplies to the rapidly expanding industrial towns led to a major increase in reservoir construction. This continued throughout the nineteenth and twentieth centuries, but has been followed by a decline in dam construction during the last thirty years.

Figure 2.1 shows the growth in reservoir capacity and number of reservoirs since 1800 (Tedd *et al.*, 2000). It shows that older dams were generally low structures with small reservoir capacities and the effect of constructing much larger reservoirs in the 1950s, in particular the construction of hydro-electric schemes in Scotland.

The timeline in Figure 2.2 shows important incidents and developments in dam construction and legislation.

Before 1900, nearly all British dams were of the embankment type with a notable exception in Vyrnwy, a gravity dam built in 1890 to supply water to the city of Liverpool. Although in the twentieth century a large proportion of dams were built of masonry or concrete, the majority of dams in Great Britain are earth embankments. Reservoir safety is thus intimately concerned with the behaviour and long-term

performance of old embankment dams. Until the 1950s, most embankment dams in Britain were built to a traditional design with a central core of puddle clay. More modern embankment dams typically use a wider core of rolled clay. However, the use puddle clay cores dam continued until 1972, with the completion of Jumbles, north east of Manchester.

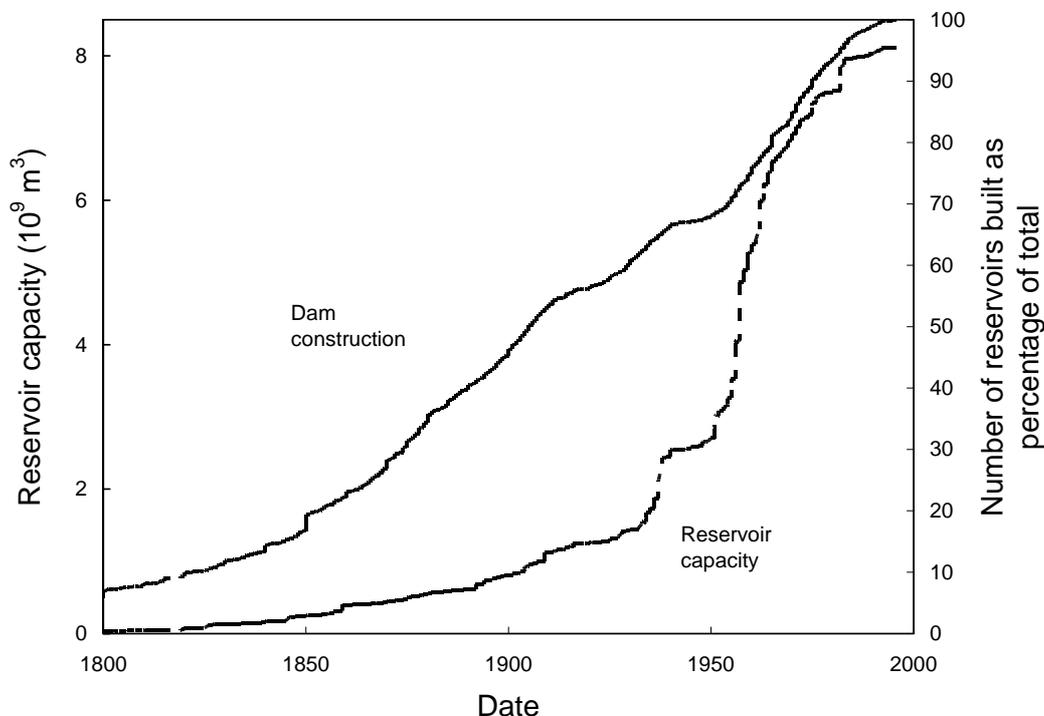


Figure 2-1 Growth in British reservoir capacity and number of reservoirs with time

The vast majority of serious incidents have concerned embankment dams, but this is not surprising since about a tenth of British dams are built of concrete/masonry. It certainly should not be concluded that concrete dams are immune to problems.

While a dam failure can be broadly defined as an incident, occurrence or process whereby a dam does not perform the function for which it has been constructed, namely to safely impound a reservoir of water, dam failure generally means the breaching of a dam with the uncontrolled release of reservoir water. However, the term is also used for the 'failure' of an embankment dam during construction; that is, an event where inadequate shear strength in the fill and possibly the foundation causes instability in one or both embankment slopes and may also involve the foundations. If such instability occurs before the embankment has begun to act as a water-retaining structure, there is less concern for reservoir safety.

When a dam has been constructed and the reservoir basin filled with water, the dam is said to be 'in service'. Failures in service are distinct to those arising during dam construction. Unlike failures during construction, failures in service are usually intimately connected with the particular function of the dam to impound a reservoir of water. Shear failure could, of course, occur with the reservoir fully impounded as well as during construction, but there are other more common modes of failure. If an embankment dam crest is overtopped during a flood, it could be breached by surface erosion of floodwater flowing over the crest and downstream slope. Failure by overtopping during construction is rare and the only incident of this type is included in

Section 5 (Woodhead No1 overtopped on 12 October 1849 releasing $500 \times 10^3 \text{ m}^3$ of water). An embankment dam can also fail by internal erosion associated with excessive seepage and leakage passing through the body of the embankment or the foundation.

Some hazards, such as foundation instability as at Eigliau, threaten concrete dams as well as embankment dams, whereas other hazards relate to concrete deterioration. Examples include long-term seepage, frost, the use of high-alumina cement and alkali-aggregate reaction.

The Environment Agency report *Modes of dam failure and monitoring and measuring techniques* provides further information on the threats to dam safety and the various ways in which incidents might arise.

Significant British failures and incidents	Date	Date	Key developments in British dam construction practice, legislation and guidance
		1730	First use of clay core, Serpentine (Hyde Park)
		1755	Use of clay blanket on upstream face, Petworth
		1766	First foundation cut-off
		1795	Butterley - one of first puddle clay core dams
Failure of Blackbrook due to poor construction	1799		
Whinhill, 31 dead	1835		
		1838	Telford set standard for puddle clay core dam design with hydraulic gradient of 3 across the core
Failure of Bilberry, 81 dead	1852		
		1854-1862	Slope failures during construction at Arnfield, Calf Hey and Piethorne repaired incorporating a toe berm.
		1854-1864	Specifications for zoned fill construction become common practice.
Ainsworth Mill Lodge floods mine workings	1860	1868	Imposition of strict liability "Rylands v Fletcher"
			Specification by Simpson (one of the investigating engineers) incorporates points of learning: railway wagons to be excluded from embankment area during construction; fill to be worked in horizontal layers not exceeding 9 inches.
Failure of Dale Dyke, 244 dead	1864	1864	
		1866	Introduction of Waterworks Bill with many of the features of the Reservoirs (Safety Provisions) Act 1930
		1872	Rankine's paper on gravity dam design published (principal stress and 'middle third' concepts).
Pentwyn serious internal erosion in puddle clay cut-off	1870s	1876	Woodhead No. 2 dam, first use of concrete to completely fill deep cut-off trench.
		1877	Binnie promotes the use of 'long inclines' rather than abrupt changes in the depth of puddle trenches to reduce the risk of differential settlement.
Tunstall and Cowm, severe leakage through foundation	1879	1879	First remedial use of grouting to seal foundations
		1882	Vyrnwy gravity dam designed with a drainage tunnel network to reduce uplift pressures.
Failure of Skelmorlie, 5 dead	1925		
Dolgarrog failure, 16 dead	1925	1925	Edward Sandeman letter published in the Times which led to Reservoirs (Safety Provisions) Act 1930.
		1930	Reservoirs (Safety Provisions) Act 1930
		1933	Publication of "Interim report of the Committee on Floods in relation to Reservoir Practice"
Construction failure of Chingford	1937	1937	Use of modern earth moving and construction equipment led to high pore pressures. Soil mechanics used in re-design for first time of British dam.
Lynmouth flood disaster	1952	1960	Re-publication of "Interim report of the Committee on Floods in relation to Reservoir 1960 Practice"
		1960s	End of puddle clay core construction and beginning of rolled clay core construction in the UK
Balderhead, severe internal erosion on first filling	1967	1968	First use of diaphragm walls to repair watertight element of dam. Development of filter rules based on permeability of filter.
Lluest Wen; internal erosion emergency drawdown	1969		
Warmwithens; failure with uncontrolled release of water.	1970	1975	Reservoirs Act 1975; publication of the 'Flood Studies Report'
		1978	Publication of "Floods and Reservoir Safety: an engineering guide" 1st edition
Construction failure of Carsington	1984	1986	Instigation of Review Panels and the National Database at BRE following recommendations in the Coxon Report into the failure of Carsington dam
		1986	Reservoirs Act 1975 comes into full effect
		1990	Publication of "An engineering guide to the safety of embankment dams in the United Kingdom" 1st edition
Kielder; disruption to upstream blockwork	1984	1995	Publication of "Performance of blockwork and slabbing protection for dam faces"
		1999	Publication of the "Flood Estimation Handbook"
Bolby; failure of stepped masonry spillway	2005	2007	Post-incident reporting system established
		2007	EA Biennial Report sets out proposals for legislative change
Ulley; failure of spillway, emergency drawdown and evacuation	2007	2008	The Pitt Report. Paper on "Security of stepped masonry spillways" published
			Draft "Floods and Water Management Bill" sets out proposed changes to the Reservoirs Act for England and Wales; Flood Risk Management (Scotland) Bill sets out changes for Scotland.
		2009	

Figure 2-2 Timeline of failures/incidents and key developments in British dam construction practice, legislation and guidance

2.2 Nineteenth century

By the middle of the nineteenth century a fairly standard design of embankment dam had been adopted, with an upstream slope of one vertical in three horizontal and a steeper downstream slope of one vertical in 2.5 or two horizontal. Although control of material and workmanship should ensure the integrity and watertightness of the core within the body of the dam, leakage could occur through the natural strata of the valley underneath or around the sides of the dam. Leakage could also occur in the basin of the reservoir. In the early puddle clay core dams, it was usual to extend the puddle clay into a cut-off trench below ground level thus connecting the core to a stratum of low permeability. The trench often continued into the valley sides. Sometimes very deep trenches were dug but the trench was usually narrow, with vertical sides.

The catastrophic failure of two dams in the nineteenth century, Bilberry in 1852 and Dale Dyke in 1864, led to major changes in the design and construction of puddle clay core embankment dams.

Excavation for the cut-off trench of the 29-metre high Bilberry dam began in 1839 and a spring was encountered in the bottom of the trench. The outlet works comprised a masonry culvert which had to cross the puddle clay filled cut-off trench. Serious problems soon became apparent: muddy water came through the culvert in 1841 and in 1843 the leakage became worse and water burst through the culvert. Remedial works were unsuccessful and large settlements occurred. It was claimed that between 1846 and 1851 the bank settled three metres. This settlement eliminated the freeboard and soon after midnight on 5 February 1852 the embankment was overtopped and breached during a storm. The resulting flood claimed 81 lives in the Holme Valley below the dam. It would appear that erosion of and through the puddle clay was the cause of the settlement that led to the catastrophe. The Home Secretary, Sir George Grey, arranged for Captain R C Moody of the Royal Engineers to inspect the remains of the dam and give expert evidence at the inquest. It is noteworthy that the Bilberry dam had three adverse features: a puddle clay filled cut-off trench in which springs were encountered, highly permeable fill on either side of the puddle clay core, and a culvert through the embankment.

Construction work on the 29-m high Dale Dyke dam started in January 1859. The deep puddle clay filled cut-off was completed in 1861 and the embankment was finished by April 1863. Impounding commenced in June 1863 and by 10 March 1864 the water level was 0.7 m below the crest of the weir. In the late afternoon of the following day, a crack was observed along the downstream slope near the crest of the dam. At 23:30 a collapse occurred and the dam was breached. In the resulting flood, 244 lives were lost and extensive property damage was caused, including some in Sheffield. Robert Rawlinson and Nathaniel Beardmore were appointed by the Home Secretary, Sir George Grey, to investigate the failure. In their report Rawlinson and Beardmore were critical of both the design and construction of the dam. They believed that failure was most likely to have been caused by leakage from a fractured outlet pipe which passed through the embankment, but the design and construction of the embankment itself was also criticised (Rawlinson and Beardmore, 1864).

“...the puddle-wall is much too thin, and the material placed on either side of it is of too porous a character....No puddle-wall should ever be placed betwixt masses of porous earth, as puddle, under such conditions, will crack, and is also liable to be fractured by pressure of water.”

However, the cause or causes of the disaster have continued to be disputed (Binnie, 1978). It has, however, been established that the core was susceptible to hydraulic fracture.



Figure 2-3 Postcard photograph of the Dale Dyke dam breach (courtesy of C Hoskins)

As a result of the failures of Bilberry and Dale Dyke and some serious problems encountered with other embankment dams, important lessons were learned which led to developments in design and construction during the latter half of the nineteenth century. Dale Dyke had a puddle clay filled cut-off trench, permeable fill immediately on either side of the puddle clay core, and outlet pipes laid in a trench beneath the embankment. Most attention had always been given to the central vertical core of puddle clay which formed the all important watertight element within the embankment dam. Although the leading engineers of the period realised the need for a substantial width, in some dams the core was excessively narrow. At Dale Dyke the top width of the core was a mere 1.2 m, and, with batters of 1:16, the maximum width at ground level was only 4.9 m. Following the failure, the replacement dam constructed nearby had a much wider puddle clay core.

According to the earliest concept of puddle clay core dams, the embankment shoulders merely served to support the core and only needed to be stable and reasonably solid. Captain Moody strongly criticised this aspect of the design. Moody drew attention to the failure to properly control fill placement and ensure that the more cohesive fill was placed next to the puddle clay core with the more granular fill in the outer slopes. Deficiencies in the supporting fill were also apparent at Dale Dyke and it was recognised that it was not prudent to place poorly compacted, highly permeable fill next to the puddle core. It became recommended practice to place a

selected fill, which was more cohesive and better compacted than the general embankment fill, on either side of the puddle clay core.

Captain Moody remarked that a sinkhole in the crest at Bilberry was located above the culvert *“and is no doubt due to the washing away of the bad puddling over and above the culvert where it passes through the puddle wall below”* (Moody, 1852). Robert Rawlinson, who had no connection with the design and construction of the Bilberry dam or the investigation of the failure, a few years later commented on the disaster to the effect that *“The Holmfirth embankment was said to have been rendered “rotten” by a spring, or springs, in the centre”* (Rawlinson, 1859). J F Bateman, one of the leading dam engineers of the time, claimed that the dam was badly constructed on sandstone rock: *“The water escaped through the fissures of the rock, and gradually washed the embankment down in such a way that the top of the embankment was lower than the top of the swallow which was constructed as a waste weir for the purpose of letting the water off”* (Bateman, 1879). As the dangers of erosion of the puddle clay in the cut-off trench into a fissured rock foundation became better appreciated, the superiority of backfilling the cut-off trench with concrete rather than puddle clay was recognised and there was a general trend from puddle clay to concrete filled cut-off trenches. Grouting of foundations came into use in the late nineteenth century when Thomas Hawksley applied it to a wing trench at Tunstall dam and remedial work at Cowm dam in 1879.

There was a realisation that when puddle clay is in contact with jointed rock, particularly at the bottom of cut-off trenches, water might fracture and erode the clay and escape through joints in the rock. Pentwyn dam (Binnie, 1987a) completed in 1863 used a puddle clay cut-off. A fault across the valley and the presence of limestone resulted in serious leakage from the reservoir, accompanied by settlement. This and other examples of erosion led to the practice of lining the bottom and sometimes the sides of the cut-off trench with brickwork or concrete as was deemed necessary.

For the second Woodhead dam completed in 1876, Bateman used concrete in the cut-off trench, and this appears to be the first occasion when reliance was placed on concrete alone. However, use of puddle-filled trenches carried on into the twentieth century and incidents of internal erosion continued as at Walshaw Dean Lower and Middle dams, completed in 1907.

Initially, the practice with puddle clay core dams was to lay the outlet pipe through the embankment and puddle core. In some cases the pipes were surrounded with fill material, but it became more common to surround pipes with concrete. Rawlinson came to definite conclusions about the unsuitability of these practices (Rawlinson, 1879).

An engineer, whether designing waterworks or other works, should not put any portion of the material liable to decay out of reach: he should not bury such material as cast iron under an embankment having a 500-feet base, so that nothing but the destruction of the bank could ever render it accessible for repairs’.

The practice of placing outlet pipes, or culverts containing outlet pipes, through the embankment was largely superseded by the more costly but much safer expedient of driving a tunnel through the natural ground. When the hazards of a pipe containing water under reservoir pressure were better understood, the early practice of

controlling the flow of water by a single valve at the downstream toe was widely changed and upstream controls became more common.

The problems experienced with puddle clay core embankment dams turned the thoughts of Major Hector Tulloch, who later succeeded Rawlinson as the chief engineering inspector of the Local Government Board, toward masonry dams and he wrote to Professor Rankine at Glasgow University *"I consider the fact of the puddle wall in the middle of the dam being virtually all the resistance that the dam can bring to bear against the water, renders all our dams far too weak."* In his reply Rankine stressed the importance of the foundation which *"should be sound rock, if practicable, and should a rock foundation be unobtainable, firm impervious earth"*. He added that *"It may be doubted whether any earthen foundation is thoroughly to be relied on where the depth of water exceeds 100 or 120 feet."* Rankine also warned Tulloch that *"It is not advisable to build a masonry dam on an earthen foundation"* (Tulloch, 1872).

Tulloch's misgivings received further confirmation from troubles experienced at Lower Lliw, a dam designed by Robert Rawlinson, the engineer appointed by the government to investigate the Dale Dyke catastrophe. Construction of the 27-m high dam north of Swansea commenced in 1862 and was completed in 1867. In 1873, water started to flow from downstream drains at a much increased rate and the water was turbid. A spring had burst through the puddle clay core. Erosion of the puddle clay led to settlement of the embankment. Remedial work involved an open cutting 50 m wide at the top and 15 m wide at the bottom to a depth of 11 m below the top of the embankment and a trench nine metres long and six metres wide sunk from the bottom of the cutting to the rock, a total depth of 32 m below the top of the embankment. At a depth of seven metres in the trench, a fissure 0.6 m wide was found in the puddle clay filled with the coarse material of the selected fill. The fissure extended down to the face of the rock. A drain was installed to take away the springwater which acted on the clay at the bottom of the trench and the trench was backfilled with puddle clay. In 1883 after two years of service, leakage again increased. Turbid water came from the drains and settlement occurred at the location of the remedial works.

Another leading dam engineer, Thomas Hawksley, was called in to advise, but his answers to the questions of the dam owner, Swansea Borough Council, were not encouraging. It was not possible to determine the cause of the leak, the materials in the embankment were fit for purpose and, as to remedial work, he could only comment that *"The method in the former instance was, in my judgement, unexceptionable, and nothing better than a repetition of the same method can now be suggested"* (Binnie, 1981). A disillusioned town council did not opt for a second attempt at the remedial works and the reservoir was operated from 1883 to 1975 with the top water level reduced by 5.5 m. It would be nearly another hundred years before an understanding of hydraulic fracture was gained. The embankment was fully rebuilt in 1978.

At the inquest following the Dale Dyke disaster, the jury stated that the legislature ought to take such action as would result in government inspection of all works of this character and that such inspections should be frequent and sufficient and regular. However, no legislation was introduced at that time. For the next 60 years there was no repeat of the major loss of life that had occurred at the two Yorkshire dams. The collapse of Cwm Carne in 1875 was the most serious failure (Smith, 1992). This 12-m high embankment settled over many years due to internal erosion and was overtopped at 17:30 on 14 July, with failure at 23:00. The resulting flood caused the

loss of 12 lives. The failure is reminiscent of Bilberry in that deterioration occurred over a long period with little attempt to improve the obviously defective dam.

The first large masonry dam to be built in Britain was Vyrnwy dam, completed in 1891 and designed by Thomas Hawksley and G F Deacon. Following the failure of Habra dam in French Algeria in 1881, Hawksley became concerned about uplift as an overturning force on the underside of a masonry dam and consequently increased the thickness of Vyrnwy. As a further precaution against uplift, a drainage tunnel slightly above the tailwater level was incorporated in the dam, making Vyrnwy not only the first dam in the world to be designed with uplift acting on its base taken into consideration, but also the first to have underdrainage.

2.3 Early twentieth century (1901-1930)

In 1925 two failures caused loss of life: a small dam at Skelmorlie in South West Scotland failed and a disaster at Dolgarrog in North Wales, the latter failure being the more serious.

Following heavy rainfall, the Skelmorlie lower reservoir failed at 14:00 on 18 April 1925. The flood water, which was probably sufficient to overtop the embankment, was augmented by water from a quarry. This quarry had partially filled due to a blocked culvert and suddenly emptied when the blockage cleared. The embankment breached over a nine-metre length and the reservoir emptied in 15 minutes. Five people were killed in the village of Skelmorlie. The failure was attributed to a grossly deficient overflow and inadequate freeboard.

The verdict of the jury at an enquiry held at Kilmarnock Sherriff Court was: "*The disaster was caused by absence of any regular skilled supervision and inspection.*"

On 2 November 1925, the remote concrete Eigiau dam collapsed, where it was only five metres high, due to a blow-out of the lower part of the dam wall at a point where there had been a seepage path for several years. The reservoir water was released and surged into the nearly full Coedty reservoir below. The 11-m high Coedty dam had been constructed in 1924 with earthfill shoulders supporting a central concrete core wall. When the dam was overtopped, the material supporting the core wall on the downstream side was washed away and the core wall collapsed. Sixteen people were killed in Dolgarrog by the resulting flood. Technical evidence at the inquest was given by Ralph Freeman to the effect that the foundation of the dam had not been sufficiently deep. The jury returned a verdict of accidental death "*caused by the bursting of the dam under the wall in consequence of the wall lacking a proper foundation.*" The coroner's jury recommended regular government inspection.



Figure 2-4 The breach in Coedty dam (courtesy of Chris Hoskins)

Only 11 months before the Dolgarrog disaster, Cowlyd dam in the adjacent valley was nearly breached due to overtopping during a storm on New Year's Eve of 1924 (Knight, 1975). A V-shaped area of the downstream fill was eroded down to foundation level, exposing the concrete core-wall. Frenzied backfilling on the following morning saved the dam. Had the central core not been of concrete, it is possible the dam could have failed leading to certain loss of life. The reservoir capacity was probably more than twice that of Eigiau and Coedty combined. However, floodwaters from Cowlyd would not have gone down the same valley as those from Coedty and were likely to have missed the main part of Dolgarrog. Subsequently, the spillway crest of Cowlyd was lowered and the wave wall was raised.

The reservoir failures of 1925 led to the Reservoirs (Safety Provisions) Act 1930. Since this Act was brought into force, and periodic inspection by a qualified engineer became mandatory, there have been no dam failures in Britain which have caused loss of life. Although complete failure and breaching of embankment dams has been relatively rare in Britain, there have been many serious incidents affecting dams in service. In some instances these incidents have warranted emergency drawdown of the reservoir and costly remedial works or permanent lowering of the top water level.

Whilst after 1900, traditional methods of gravity dam construction continued to be used in which large rocks were laid on mortar or a thin layer of concrete, mass concrete started to replace them in which concrete was placed in thick layers and large plums or displacers of rock were embedded. Blackbrook dam, completed in 1906, was the first dam constructed using mass concrete with displacers. The minimal damage during the earthquake incident at this dam illustrates the robustness of the construction.

2.4 Mid-twentieth century (1931-1960)

Failures which occur during the construction of embankment dams are generally associated with slope instability and are similar, therefore, to the failure of any other type of earth embankment. The presence and shape of a clay core, especially a wet puddle clay core, has a considerable influence on the stability of the embankment. The core should consolidate with time and therefore stability should generally improve, with the end of construction or the rapid first filling of a reservoir likely to be critical periods for this form of construction. Studies of instability during the construction of earth dams have had a major influence on the development of soil mechanics in Great Britain.

During 1937, major slips occurred at three embankment dams under construction. The slip at Abberton took place in July and at Hollowell in October. However, the best known of the three failures is the instability which occurred at the end of July in the earth embankment under construction for the William Girling storage reservoir at Chingford in Essex. With eight metres of the planned 10-m height completed, a 90-m long section of the downstream (outer) slope moved. The embankment had a central puddle clay core and was founded directly upon a layer of soft yellow clay. A geotechnical investigation was carried out by the Building Research Station (Cooling and Golder, 1942). The failure surface passed through the puddle core and then followed a path contained within the layer of soft yellow clay. The undrained shear strengths of the yellow and puddle clays were measured by laboratory direct shear tests yielding values of only 14 and 10 kPa respectively. A stability analysis was carried out in terms of total stresses and a factor of safety close to unity was obtained.

The Chingford reservoir embankment was one of the first to be built in Britain using what at that time would have been described as 'modern earth-moving equipment'. Thus, the construction rate would have been faster than had previously been common practice. It seems likely that the development of high pore water pressures in the yellow foundation clay due to rapid loading by the embankment was a major contributing factor to the failure.

In September 1941 movements were observed in the pitching on the upstream slope over the central section of Muirhead dam which was under construction in southern Scotland. The embankment was 21-m high at this stage and a further five metres of fill had still to be placed. The embankment had slopes of one in three, a central puddle clay core and shoulders of boulder clay. The Building Research Station carried out an extensive investigation of the failure. An initial survey established that the upstream slope had moved outwards up to 1.2 m and that a berm on the downstream slope had moved 0.6 m. Movements were horizontal and the toe walls had not moved. When 0.5 m of fill was added, further horizontal movements of about 0.3 m were monitored. It was believed that the embankment had failed through the lower part of the shoulder fill. The strength of this material was found to be very variable but the average measured value of undrained shear strength was close to 40 kPa which corresponded with limiting equilibrium. The final height of the dam was limited to 21 m and the upstream slope was stabilised by a substantial berm (Banks, 1948).

At the time of the Muirhead failure, a similar embankment was under construction nearby at Knockendon and the fill had reached about one-fifth of the full height. As a result of the events at Muirhead, the cross-section of Knockendon was modified by adding a toe weight to the upstream shoulder and by including a zone of stronger

granular fill in the downstream shoulder. Standpipe piezometers were installed to monitor construction pore pressures in the fill and check the rate of consolidation. Measured pore pressures were used together with the results of drained shear box tests to calculate the stability of the embankment (Banks 1952).

The measurement of pore water pressures by the Building Research Station at the site of the 33-m high Usk dam during the early 1950s had an important influence on the development of embankment dam design and construction techniques in Britain (Penman, 1978). Usk has a central core of puddle clay and a cut-off trench filled with concrete. The shoulders of the embankment were of boulder clay. A silt layer was found to be present under the downstream shoulder and as a consequence, a sand drain system was installed. Twin tube hydraulic piezometers were installed in the silt layer to check on the performance of the drains. Also three piezometer tips were installed at the mid-depth of the first season's fill in the downstream shoulder in July 1952. The piezometer tips in the silt layer in the foundation measured no significant pore pressures, indicating that the drainage system was effective. However, pore pressures in the fill were large. Effective stress stability analyses indicated that the factor of safety would be unacceptably small if the dam was brought to full height with the average pore pressure ratio (r_u) greater than 0.5. The pore pressure dissipation that occurred during the winter shutdown period was insufficient to ensure stability.

Advice was sought from Professor Skempton of Imperial College London. Fifteen steel standpipes driven into the fill confirmed the BRS pore pressure measurements and it was decided to place horizontal drainage layers within the embankment shoulders: it is believed that this was the first use of drainage blankets of this type in an earth dam to control construction pore pressure (Sheppard and Little, 1955). The use of instrumentation and associated construction techniques such as those described above have considerably reduced the risk of embankment failure during construction.

Slope instability is by no means confined to the construction period. A major downstream slip was discovered on 18 December 1951 at Harlow Hill, an embankment dam forming an open service reservoir which had been built at Harrogate in 1868. The slip occurred following an extremely wet autumn. A vertical movement of 0.3 m had occurred on the slip plane adjacent to the puddle clay core, with 0.23 m uplift at the toe against the concrete retaining wall. Movement was continuing at 0.01 m per hour. There was a clear danger of a catastrophic dam breach and emergency actions were taken: the reservoir was lowered as fast as possible; sandbags were placed on the toe of the slip to improve stability; and tarpaulins were placed to prevent further ingress of rainfall into the embankment. Movement monitoring commenced and the police were alerted to be ready to evacuate the downstream population. The 1:1.9 slopes of the embankment were too steep for the clayey embankment fill and slope stability must always have been marginal. The embankment was therefore a disaster waiting to be triggered by some phenomenon such as unusually heavy rainfall.

Some important lessons were drawn from the incident (Davies, 1953); the most important was that all embankment dams of clay constructed before the advent of soil mechanics should be regarded as suspect. Furthermore, the normal visual inspection of a dam, unsupported by any real knowledge of the properties of materials of construction, is insufficient to determine the stability of the structure. Soil strength tests are essential to determine actual stability, but they must be sufficiently numerous to provide a proper statistical average, and must be taken from locations on potential slip planes. However, soil strength information is currently only available for a minority of Britain's embankment dams.

Despite the clear warnings given at the time, the lessons of Harlow Hill were not generally learned. However, corroboration of the likelihood of some old embankment dams having only marginal slope stability came when a comprehensive programme of investigations and remedial works was undertaken in the 1970s in Northern Ireland. Most of the seventy or so reservoirs in the province were impounded by embankment dams and at nine of these, it was necessary to install filters with overlying rockfill stabilising berms (Cooper 1987).

Slope stability failures were by no means the only type of incident to afflict embankment dams in this period. One particular type of problem was associated with the precautionary partial drawdown of reservoirs during the Second World War. The 7.6-m high King George V reservoir near Chingford consists of clay fill with a puddle clay core. The embankment was built on unstripped grass and topsoil without any special provision for underdrainage of the downstream (outer) slope. In September 1939 it was decided to reduce the top water level by 1.5 m and this restriction was maintained until February 1945 when raising the water to its previous top level began. Water leakage appeared at the toe of the embankment as the original top water level was approached. The possibility of the leakage indicating an incipient major failure was recognised and the water level was lowered. An investigation initiated in association with the Building Research Station revealed the presence of roots in the puddle clay down to the previous temporary top water level 3.1 m below the crest of the bank. This evidence together with extensive field observations and soil testing indicated that the passage of water was through the upper part of the puddle clay core which had been subject to drying, shrinkage and cracking during the years 1939-1945 (Bishop, 1946).

Refilling of a long-empty reservoir impounded by an embankment dam having a puddle clay core should always be undertaken with the utmost caution due to possibility of desiccation and cracking of the upper part of the core. Additional settlement resulting from the major drawdown could lead to water flow over the top of the core. The possibility of hydraulic fracture during refilling should also be considered. Caution is necessary in the case of a dam having any or all of the following features: (a) a puddle filled cut-off trench, (b) permeable fill on either side of the puddle clay core, and (c) outlet works passing through the body of the embankment.

Earth dams may fail due to inadequate spillways when an exceptionally large flood occurs, however the damage caused by a severe rainfall event may be considerable even without a dam failure. On 15/16 August 1952, 230 mm of rain fell in 24 hours on the upper valleys of the East and West Lyn rivers in Devon. At 20:30 on 15 August 1952 the services of the fire brigade were requested above Barbrook on the West Lyn where a dam had burst and flooded Radsbury Farm (Delderfield, 1981). With such immense rainfall the failure of a small dam was largely irrelevant in the subsequent catastrophe in which the West Lyn river burst its banks and a torrent swept through the town of Lynmouth, resulting in 34 deaths and making a thousand people homeless.

Earthquake damage to British dams is comparatively rare, but on 11 February 1957 Blackbrook concrete and masonry gravity dam was affected by an earthquake with local magnitude 5.3. The dam suffered fairly superficial damage including displacement of 0.75 tonne copings and manhole covers which sheared and were displaced up to 20 mm. However, it has been asserted that "*Had the line of the*

tremor been 90 degrees displaced the result would have been catastrophic as the dam is located only four miles north of the epicentre” (Kennard and Mackey, 1984).

2.5 Late twentieth century (1961-2000)

Some of the most troubling incidents in this period have involved internal erosion and such problems were not confined to old puddle clay core dams. On first impounding in 1966, just before the reservoir was full, the main underdrain flow at the newly built 48-m high Balderhead dam increased. The dam has a rolled boulder clay core, relatively stiff shale fill shoulders and a concrete cut-off. The top 10.8 m of the clay core has vertical sides. Immediately downstream of the core is a crushed limestone filter which connects with the ground drainage blanket. The filter and the drainage blanket were designed according to standard filter rules. Subsequently, localised settlements occurred along the crest and in 1967 two sinkholes formed in the crest. The reservoir was immediately drawn down by 9.2 m and the underdrain flow returned to its previous level. It was established that the main underdrain flow had turned cloudy about a month before the first sinkhole appeared, but after drawdown the water became clear.

Exploratory boreholes revealed erosion within the core at several locations; the boulder clay material had become segregated and the finer particles lost by water erosion. The damage was associated with cracking which had been initiated by hydraulic fracture of the core under almost full reservoir pressure. Low stresses in the core were caused by arching between the clay core and the shoulders and possibly by longitudinal strain due to differential settlement across foundation discontinuities. It was also postulated that once the cracks had formed they were kept open by the water pressure and under the low flow conditions the coarser eroded material had segregated in the cracks. On drawdown the seepage paths closed up due to the decrease in water pressure. Over the central 200 m of the dam, covering the zones of worst damage, the core was repaired by constructing a 0.6 m wide diaphragm wall down to the concrete cut-off. As well as serving as an additional water barrier, the diaphragm wall was intended to prevent migration of eroded material through the core (Vaughan *et al.*, 1970).

The internal erosion problems at Balderhead led to major investigations and research by Professor Peter Vaughan at Imperial College London, which has not only built a much better understanding of the mechanisms involved in the internal erosion process, and particularly the role of hydraulic fracture, but has also led to important developments in filter design (Vaughan and Soares, 1982).

On 23 December 1969 a horse fell into a two-metre deep hole in the crest of the 24-m high Lluest Wen dam in South Wales, which had been built in 1892. Subsidence had occurred previously in 1912 and 50 tonnes of cement grout had been injected in the area of the valve shaft in 1915-16. It was feared that the dam would collapse and an emergency was declared by George Thomas, the Secretary of State for Welsh Affairs. The infirm and elderly were evacuated from their homes on the night of 12/13 January 1970 (Gamblin and Little, 1970). The 0.38-m diameter draw-off pipe was inadequate for rapidly lowering the reservoir water level and a large number of pumps, some positioned by helicopter, were brought in to lower the water level. Also, an emergency cut was made through the spillway lowering the overflow level of the reservoir. The reservoir level was lowered by 9.1 m in twenty days (Twort, 1977).

Meanwhile, over 18 tonnes of clay/cement grout were injected into a single hole in the neighbourhood of the valve shaft where subsidence had occurred.

With the emergency over, full grouting of the puddle clay core was done and fifty tonnes of clay/cement grout were injected. After completion of the grouting works, further investigation was carried out. The puddle clay core was found to consist of sandy silty clay with pockets of silt and sand. The core had a series of cracks, many of them open and iron-stained by seeping water. About 75 per cent of the cracks were within five degrees of the horizontal. Open water-worn cavities were found in two drill-holes. The water content and undrained shear strength of the puddle clay fluctuated widely and erratically. The core was very soft in the vicinity of the valve shaft. In view of these findings, it was decided that grouting alone could not provide a satisfactory solution and a new plastic concrete core was installed using the slurry trench method.

During the excavation of a six-metre diameter shaft at the valve tower, it was discovered that the brickwork of the draw-off tunnel had not been bonded into the masonry at the back of the valve shaft. Puddle clay had eroded through a 50-mm gap and then through a crack in the 0.15-m diameter pipe. At the time of the emergency, there was a 0.06 cubic metre pile of puddle clay at the downstream end of the 0.15-m diameter pipe. It was of concern that so much hinged on a tiny detail which might never have been detected until perhaps too late but for the requirement for major remedial works. The extreme seriousness with which the incident was viewed, and the emergency measures put in place by the Welsh authorities, were undoubtedly influenced by the Aberfan disaster three years earlier, when on 21 October 1966, following heavy rainfall, a colliery spoil tip collapsed and 150,000 m³ of spoil flowed downhill into the mining village, killing 144 people, 116 of them children (Bishop *et al.*, 1969).

Warmwithens dam was a 10-m high clay fill embankment built more than 100 years ago near Oswaldtwistle in Lancashire. The reservoir it impounded lay in series above two other small reservoirs: Cocker Cobbs and Jackhouse. During the period 1965 to 1966, the dam was raised to provide adequate freeboard and the old cast iron draw-off pipe was replaced by a reinforced concrete segmental tunnel driven through the embankment. The tunnel contained a steel pipe for the water outlet. At 7:30 on 24 November 1970 an escape of water was detected and by 13:30 the dam was completely breached to foundation level (Wickham, 1992). The water impounded by the dam was discharged into the two lower reservoirs. The embankment dam of Cocker Cobbs was overtopped, but it did not fail, and the water passed the spillways of the lowest reservoir, Jackhouse, without causing serious damage. Had a cascade failure of the two lower dams taken place, the resulting flood could have caused serious damage in Oswaldtwistle. The breach occurred along the line of the outlet tunnel. It therefore seems possible that seepage through or along the perimeter of the abandoned cast draw-off pipe, or along the perimeter of the new tunnel, could have played a part in causing the failure. This incident showed how rapidly an internal erosion incident can develop and confirmed the hazard where a structure passes through the clay core of an embankment dam.

On the afternoon of 7 March 1983, a member of the public taking two dogs for a walk noticed a depression in the asphalt of the crest roadway of the 35-m high Greenbooth dam about 20 m from the west abutment. The dam, built near Rochdale, was completed in 1962 and was one of the last dams to have a puddle clay core. The depression deepened quickly over a few days. By mid-morning the next day it measured three metres by one metre in plan and had subsided by 0.16 m. The

depression was directly above the toe of a concrete wing wall where there was a sharp change in direction of the concrete/puddle clay core interface. A panel engineer was appointed to supervise the investigation and the reservoir level, which was 1.65 m below top water level, was reduced by 9.3 m over an eight-day period. An investigation identified voids in the puddle clay. These were grouted by tube-à-manchette used a specially designed bentonite, cement, fly ash, clay grout (Flemming and Rossington, 1985).

Serious incidents have not been confined to reservoirs that came within the ambit of the Reservoirs Act 1975. Since the provisions of the Act only applied to reservoirs impounding in excess of 25,000 m³ above natural ground level, the Act did not apply to a planning application submitted for a 14,000 m³ “pond” at Horndoyne farm near Aberdeen in March 1989. Impounding took place in late autumn 1990 and the embankment dam breached during the night of 17-18 November 1990. Water had been seen to trickle along the side of the outlet pipe and this developed into a stream taking earth with it. Eventually a breach was formed and a wall of water, a metre or more deep, swept down the small valley. Four houses were flooded causing considerable damage to the buildings and their contents. A large residential caravan was swept over 100 m from its site, but there were no injuries to people. The failure illustrated the dangers posed by small reservoirs outside current reservoir safety legislation. Measurements made subsequent to the failure suggested a likely storage capacity of 23,000 m³, half as much again as the approved scheme and close to the 25,000 m³ threshold for the provisions of the Reservoirs Act to then apply.

Advances in soil mechanics through the 1960s should have greatly reduced the possibility of instability during embankment construction, but this type of failure was not eliminated as the major upstream slip at Carsington in 1984 demonstrated. At the beginning of June 1984, a 400-m length of the upstream shoulder of the embankment dam slipped some 11 m. At the time of the failure, embankment construction was virtually complete with the dam approaching its maximum height of 35 m. Horizontal drainage blankets were incorporated in both the upstream and the downstream shale fill shoulders. Piezometers had been installed and pore pressures were being monitored in the foundation, in the clay core, and in the shoulder fill. Effective stress stability analyses had been carried out. The failure surface passed through the boot shaped rolled clay core and a relatively thin layer of surface clay in the foundation of the dam. Investigation of the events at Carsington has made important contributions to the fundamental understanding of the behaviour of large earthworks of this type (Vaughan *et al.*, 1989; Dounias *et al.*, 1996).

In his report to the Secretary of State for the Environment on the Carsington failure, Roy Coxon made a number of recommendations including the use of review or advisory panels for major dam construction projects (Coxon, 1986).

“There is merit in involvement of a Board or Panel of Specialists in projects of this kind to review key elements relevant to design and construction. Such a Board can in no way relieve other parties of their normal responsibilities.”

This sensible recommendation, which follows international good practice, has been followed on a number of new dams and major works, including the reconstruction of Carsington, the construction of Queen's Valley dam, Jersey (1986-1993), modification of Woodhead dam (1988-1991), rebuilding of Audenshaw No 3 reservoir (1988-2002) and the raising of Abberton dam (2008-present).

A wet spot on the downstream slope of Lambieytham dam, near St Andrews, was observed during routine surveillance in November 1984. Within 24 hours seepage had increased and muddy water was observed at the base of an eight-metre long crack, which had a maximum width of 0.4 m. There was evidence of uplift six metres downstream of the crack. Following inspection by a panel engineer, it was decided that the reservoir should be emptied as quickly as possible. On the night of 20 November, engineers and the police assessed the likely consequences of failure and householders in the area were alerted to the situation. Pumps were brought onto the site by helicopter and the reservoir level was lowered by five metres in three days. The dam was demolished in October 1985 and BRE carried out extensive investigations as fill was excavated. It was concluded that downstream slope instability was triggered by high pore water pressures associated with large volumes of water from the north-west valley side flowing into and saturating the lower half of the downstream shoulder fill.

Other types of serious incidents at embankment dams have involved damage to the upstream slope protection due to wave action. As a result of these incidents, work on wave prediction has been carried out by Hydraulics Research and cases of failure have been evaluated at dams with three types of upstream protection: pitching, concrete blockwork and concrete slabbing. On the basis of this research, guidance has been produced on best practice in the design of upstream slope protection (Herbert *et al.*, 1995).

In some cases, wave action under storm conditions is so great that downstream slope stability is affected. In February 1962, a major storm at Blithfield reservoir created severe wave action that overtopped the dam, saturated the downstream fill and caused a slip in the downstream shoulder (Leach, 1975). The downstream slip at Combs in January 1976 also occurred during a storm and was probably triggered by wave action saturating the downstream fill through cavities in the wave wall (Ferguson *et al.*, 1979).

Two serious incidents at service reservoirs are worthy of note, although neither incident led to a catastrophic release of the reservoir water. The roof of Sheephouses reservoir, near Bacup, consisting of pre-stressed concrete beams collapsed in 1962. The failure of the beams has been attributed to a reduction in the strength of the high-alumina cement concrete (Neville, 2009). In October 1979, a sudden subsidence occurred in the south-west corner of the No 1 Mill Hill service reservoir, which is built on a limestone foundation. Part of the structure collapsed and the division wall was also affected. Water stored in the damaged compartments drained into the subsided area and then into underlying strata (Millmore and Heslop, 1988).

Although incidents at concrete dams have been relatively rare, major investigations and remedial works have been required at a number of such dams, often associated with uplift pressures not being allowed for in the original design of older dams or with concrete deterioration. A statutory inspection of the Carron dam confirmed doubts about the stability of the concrete gravity section and the structural inadequacy was remedied by installing pre-stressed rock anchors to increase the factor of safety against overturning (McKenna, 1996; Kennard *et al.*, 1996). At Upper Glendevon the dam was strengthened by the addition of downstream rockfill (Johnston, 1995). At Argal dam, there was concern about the condition and performance of post-tension cables which had been installed during raising works. This led to an investigation which included deformation monitoring using electro-levels (Tedd *et al.*, 1995). A concrete buttress was subsequently placed on the downstream side.

Stability checks on gravity dams in the north of Scotland identified three masonry and concrete gravity dams where the stability under the design flood was not satisfactory. A fourth dam, Mullardoch, a 48-m high mass concrete gravity structure, required measures to be taken to improve stability due to a developing situation (Peacock and Sandilands, 1993). It was reported that on 4 July 1986 leakage at Mullardoch had increased to 5.2 litres per second from 0.16 litres/second on 25 June. Existing cracks had opened up and there was some evidence that uplift pressures had increased (Johnson, 1986). Rock anchors were installed to overcome concerns about cracking and leakage (Gosschalk *et al.*, 1991).

It has been suggested that alkali aggregate reaction (AAR) alone is unlikely to destroy a structure's capability. Maentwrog dam in north Wales was built of cyclopean concrete with no contraction joints and substantial vertical shrinkage occurred. Subsequent damage was caused by both frost and alkali aggregate reaction, and the dam had to be replaced (Davie and Tripp, 1991; Tripp *et al.*, 1994; Dodd and Sawyer, 1999).

2.6 Twenty-first century

Serious incidents have continued to occur in recent years. On 9 January 2002, an operative driving across the top of the Yarrow embankment of the Upper Rivington reservoir noticed a stream of discoloured water emerging from the culvert. A jet of water was issuing at a rate of 15 l/s from a half-brick opening provided for drainage in the sidewall of the culvert. The leak was downstream of the puddle clay core. Minor leakage through the roof of the culvert had been monitored for many years with little change observed and the previous day there had been no change in the leakage measurement. The supervising engineer was contacted and the undertaker's operational response centre was alerted to the situation. The reservoir safety manager arrived at the site and contacted a panel engineer, who was on site early the following day. Attempts to block the point of leakage into the culvert did not prevent the ingress of water, but greatly reduced the amount of larger gravel particles in the eroded material in the leakage water. However, leakage occurred elsewhere in the culvert and through the low retaining wall at the toe of the dam. The scour valves were opened but the reservoir was still overflowing the following morning. Pumps were brought in overnight and the reservoir ceased to overflow on the afternoon of 10 January and was seven metres below top water level by 18 January 2002, an average rate of drawdown of about one metre per day. Following an investigation, grouting was carried out (Gardiner *et al.*, 2004).

The two key factors in preventing a disaster at Upper Rivington were early detection of the new leak and rapid lowering of the reservoir. Although the reservoirs were under close surveillance, in the event discovery of the leak was through a providential sighting. Early detection of this type of situation, which can develop rapidly, is fundamental to avoiding a breach and to the evacuation of those threatened. In an emergency, rapid lowering of the reservoir may be crucial. It is vital that valves are operational, that the capacity of outlet valves and pipework is known and, where such capacity is inadequate, pumps are available. At Upper Rivington it was possible to lower the reservoir quite quickly.

The failure of a stepped masonry spillway and associated erosion of the downstream shoulder fill at Ulley dam near Rotherham during a flood on 25 June 2007 led to the evacuation of many people from their homes and closure of the M1 motorway for two days. A number of recommendations emerged from a post-incident investigation by

the Environment Agency which can be grouped in three areas: firstly, the need to better understand the defective behaviour which caused the incident at Ulley; secondly, better preparedness for emergency situations; and, thirdly, enhancement of a reservoir surveillance culture (Mason and Hinks, 2009).

Research is needed into the way in which hydrodynamic forces act on masonry spillways and affect their structural integrity. The current stock of masonry spillways will need to be assessed in the light of this research, but in the meantime such spillways should be inspected and maintained bearing in mind incidents such as that at Ulley and at Boltby in June 2005 where failure of the spillway endangered the embankment dam.

The Pitt Review, which investigated the impact of widespread flooding in the summer of 2007, supported the need for flood plans. Inspecting and supervising engineers need good access to downstream hazard information, including information on access routes in the event of widespread flooding. Full details of drawdown capacities and assumptions should be available in on-site emergency plans for reservoirs, together with additional pumping or siphon capacities needed to enable drawdown in extreme conditions and contact details for local plant and material suppliers. Undertakers should appoint supervising engineers for at least three or four years at a time, for continuity. Since reservoir owners are not always aware of the factors which might affect the safe operation of their asset, operatives like the park rangers at Ulley could usefully benefit from some general training on the subject. A reliable reservoir level gauge board is especially useful at times of emergency and should be sited where it will not be affected by high velocity flows.

Work following the Ulley incident has shown the usefulness of the recently introduced voluntary post-incident reporting system. However, there is evidence that not all major incidents are being reported. The Environment Agency proposes that post-incident reporting be made mandatory and this was supported by the Pitt Review.

2.7 Lessons from serious incidents

From this study, important lessons are highlighted and an indication is given of the probable hazards that British dams face as they continue to age in service. While lessons can be learned from each of the events described, it is useful to group these into three subject areas concerning, respectively, technical understanding of dam behaviour, maintenance, monitoring and surveillance of dams and reservoirs, and development of emergency procedures. The study shows the value of a national incident database and the need for post-incident reporting and investigation. Information on the national incident database and post-incident reporting and investigation are found in Section 1.2 and Section 1.3 respectively.

2.7.1 Dam behaviour

Most serious incidents have been associated with unsatisfactory behaviour of embankment dams, and developments in soil mechanics over the last fifty years have greatly improved our understanding of the strength of fill and foundation materials. Unfortunately, the phenomenon of internal erosion is much less well understood. A major uncertainty concerns how rapidly internal erosion will develop and reports of incidents can give some indication. The study of failures at Warmwithens and Horndoyne provide a necessary reminder of how rapidly a dam may breach due to internal erosion. In both cases, internal erosion appeared to be

associated with a structure or pipe passing through the embankment. A study of European dams found that in almost half the cases where failure occurred, or where failure almost certainly would have occurred if the reservoir had not rapidly been drawn down, the problem was associated with a structure passing through the embankment (Charles, 2002a).

A critical question relates to whether movements, measured in some cases more than 100 years after the construction of an embankment dam, are due to an incipient malfunction which could lead to failure if remedial action is not taken. Embankment movements may be caused by deleterious processes such as internal erosion or slope instability, but in many cases they are due to generally benign or innocuous processes such as secondary compression of the core, creep of the shoulder fill, volume change in puddle clay due to seasonal water content variations, and stress changes associated with fluctuations in reservoir level during normal operation. Investigations by BRE at a number of dams have made it possible to give guidance on this subject (Tedd *et al.*, 1994a, 1997; Vaughan *et al.*, 2000).

2.7.2 Reducing vulnerability

By understanding how incidents arise, reservoir operators can reduce their vulnerability, taking into account the population at risk from dam failure and the cost of improvements. For impounding reservoirs, one of the most important threats is from floods which can lead to overtopping and erosion of a dam embankment where the overflow capacity is inadequate. The development of flood risk guidance in the twentieth century allowed engineers to apply a consistent approach to reservoir flood risk. Improvements in flood estimation techniques have led to spillway improvements at many statutory reservoirs such that overtopping events are now relatively rare. However, the 2007 flooding showed that many smaller non-statutory reservoirs and some statutory reservoirs continue to be at risk from floods.

Other recent technological improvements include wave protection systems, wave walls, erosion protection systems, and relining systems for draw-off conduits.

2.7.3 Maintenance, monitoring and surveillance

In terms of public safety, it is particularly important to identify factors that prevent a 'near miss' becoming a catastrophic failure. Much depends on early identification of a developing internal erosion problem. In several incidents the problem was not detected during routine surveillance, but it would seem unwise to rely on the keen powers of observation of dog-walkers as at Greenbooth or the misfortunes of horse-riders as at Lliest Wen! Frequent surveillance visits are essential and a key issue is how frequent the visits should be. In recent years modern telemetry and remote sensing equipment has reduced surveillance frequency at some dam sites. This trend is not widely welcomed as remote monitoring is not an effective substitute for trained personnel regularly visiting dam sites. The demise of the Victorian approach of having a reservoir keeper for each dam (often housed at the dam) is lamented by many in the industry.

Advances in monitoring equipment enable engineers to monitor the performance of dams under construction and in service with better reliability and accuracy. However, the great majority of dams in Great Britain are old and feature little instrumentation unless specific problems have occurred.

2.7.4 Emergency procedures

Once a developing problem has been identified, much depends on the speed of response and, in particular, the capacity to draw the reservoir level down rapidly. At four serious incidents, Llest Wen, Greenbooth, Lambieytham and Upper Rivington, drawdown rates varied between 0.8 m and 1.6 m per day. Three well-documented serious internal erosion incidents, Fontenelle in the USA in 1965, Martin Gonzalo in Spain in 1987 and Peruca in Croatia in 1993, were saved by rapid reservoir drawdown at a rate of 1.2 m per day at Fontenelle (Bellport, 1967), 1.5 m per day at Martin Gonzalo (Justo, 1988) and 0.9 m per day at Peruca (Rupcic, 1997). These three cases can be regarded as 'near misses' as maximum leakage rates reached 600 l/s at Fontenelle and Peruca, and 1,000 l/s at Martin Gonzalo.

There is considerable debate about required minimum drawdown rates, which have major cost implications for owners. With some large reservoirs it will not be possible to lower the reservoir level at a rate that is likely to inhibit the development of internal erosion and during periods of heavy rainfall, it will not be possible to lower the reservoir at some British dams. It is important to know about diversion facilities such as bywashes and equally important that they are maintained, unlike the one at Maich Water.

The size of a reservoir is only one measure of its potentially destructive force; other factors are its relative elevation and what stands in the potential path of destruction. The need for flood plans and safe evacuation routes is paramount in an emergency situation. Incident management is covered further in Section 4.

3 Dam incidents and the development of reservoir safety legislation

3.1 Nineteenth century

During the first half of the nineteenth century, the demand for unpolluted water supplies for the rapidly expanding industrial towns led to a major increase in reservoir construction. Nearly all the reservoirs were impounded by embankment dams of a traditional form of earth embankment construction with a narrow central core of puddle clay. Prolonged overtopping of these dams, particularly where the embankment had not been well maintained, was likely to result in the dam breaching with a catastrophic release of the reservoir water.

Overtopping failures usually occurred when the overflow works were not adequate to cope with unprecedented rainfall. Such an event occurred when, following unusually heavy rainfall, the Whinhill dam breached at 23:00 on 11 November 1835 and the catastrophic release of the reservoir water resulted in the loss of more than thirty lives at Greenock. The reaction to this type of dam failure at that time is well illustrated by the reporting of the catastrophe in *The Times* one week later. A paragraph headed 'inundations in Scotland' was largely devoted to the flooding and resultant damage caused by the Clyde overflowing its banks during the torrential rain in the area and only briefly mentioned the Whinhill dam failure. The only implied criticism of those responsible for the dam was the assertion: "*The original banks are said to have been raised without a sufficiently strong foundation*".

The same attitude to dam failure was exhibited when thirteen years later, on 23 August 1848, a small dam overtopped during a storm at Darwen in Lancashire with the loss of 12 lives.

The verdict of the jury at the inquest was that "... all the deaths inquired into occurred by an accidental cause, that cause being the excessive rains of Tuesday night and Wednesday morning, by which the reservoir at Bold Venture Lodge overflowed and washed away the embankment ..."

A few years after the failure at Darwen, the view of dam failures as unforeseeable accidents changed when two dam failures in Yorkshire, Bilberry in 1852 and Dale Dyke in 1864, caused major loss of life and were not associated with floods. In both cases, a large element of human culpability was obvious and popular demand for reservoir safety legislation arose.

Bilberry dam collapsed following a long period of leakage and settlement caused by internal erosion within the embankment: the reservoir emptied in 30 minutes causing 81 deaths and much property damage in the Holmfirth area. This was the first dam disaster in Great Britain to draw major attention to reservoir safety and the Home Secretary, Sir George Grey, arranged for Captain R C Moody of the Royal Engineers to give expert evidence at the inquest. The verdict of the jury at the inquest criticised

“*the gross and culpable negligence*” of the dam owners and pointed to the need for legislation.

“We also hope that the legislature will take into its most serious consideration the propriety of making provision for the protection of the lives and properties of Her Majesty’s subjects exposed to danger from reservoirs placed by corporations in situations similar to those under the charge of the Holme Reservoirs Commissioners.”

The position was changed by changes in government in 1852. However, Sir George Grey was again Home Secretary from 1861 to 1866 and during this period legislation was introduced. Under the Waterworks Clauses Act of 1863, any interested person could make a complaint to two justices of the peace that a reservoir was in a dangerous state and, if satisfied that the complaint was well founded, the justices had powers to order the lowering of the reservoir and the execution of works to remove the cause of the complaint.

Dale Dyke failed during first filling of the reservoir on the night of 11 March 1864. The reservoir was far larger than that at Bilberry, and the catastrophe claimed 244 lives in the vicinity of Sheffield. The Home Secretary, Sir George Grey, appointed two civil engineers, Robert Rawlinson and Nathaniel Beardmore, to assist in the enquiry into the cause of the catastrophe. A leader in *The Times* on 17 March 1864 argued that those threatened by reservoirs could not be expected to defend themselves and needed protection.

The verdict of the jury at the inquest on 24 March 1864 included the statement: “that, in our opinion, there has not been that engineering skill and that attention to the construction of the works, which their magnitude and importance demanded; that, in our opinion, the Legislature ought to take such action as will result in a governmental inspection of all works of this character; and, that such inspections should be frequent, sufficient, and regular.”

In their report to the Home Secretary, Rawlinson and Beardmore were critical of both the design and construction of the dam but they did not endorse the recommendation from the jury for government inspection.

“We cannot, however, recommend it for adoption. Any approval of plans or casual inspection of waterworks embankments cannot ensure ultimate safety in such works. The responsibility must remain, as at present, with the engineer and persons immediately connected with the works. Magistrates have jurisdiction under clauses inserted in recent Waterworks Acts.”

On 23 February 1865, Sir George Grey announced that a draft bill was being prepared on reservoir safety and on 23 June 1865 the Select Committee on the Waterworks Bill 1865 reported, recommending that when it was proposed to construct a large reservoir, the undertakers should submit to the Home Office or Board of Trade plans and sections of the site and of the works, together with descriptions of the mode of construction. Furthermore, it should be the duty of the appropriate government department to send a competent person to the site to verify and report upon the plans and sections and descriptions. The undertakers should give notice of the completion of works to the government department and the reservoir should not be filled with water until after the elapse of a specified time. Since some large reservoirs had been allowed to decay and become dangerous, the Committee suggested that supervision over all large reservoirs should be maintained by a government department; and competent persons should be sent to inspect and

report upon such reservoirs. Resistance to safety legislation arose not only from concerns over proposals for a government inspectorate, but also from fears that safety legislation would lessen the dam owner's responsibility and the Committee emphasised that it did not intend to diminish the responsibility of undertakers of waterworks to pay for all damage that resulted from the water stored by them.

A new bill, introduced in 1866, largely followed the radical recommendations of the Select Committee, including a clause imposing strict liability, and was restricted to reservoirs impounding over a million cubic feet of water (about $28 \times 10^3 \text{ m}^3$). However, the Waterworks Bill of 1866 ran out of time and did not become law. There was a change in government in July 1866 and in February 1867, the new President of the Board of Trade announced in the Commons that there was no intention of reintroducing the bill.

Many years would elapse before comprehensive reservoir safety legislation was enacted and a number of reasons account for this delay. Changes in government during the 1850s and 1860s happened at critical moments when legislation appeared to be imminent. The two dam failures which caused the greatest loss of life occurred in 1852 and 1864 and in the following years, no disasters of a comparable magnitude occurred. Furthermore, there were concerns that legislation would not increase safety and that the formation of a government inspectorate would lessen the responsibilities and liabilities of engineers and owners (Charles, 2002b).

3.2 Twentieth century

After the Dale Dyke catastrophe some sixty years elapsed without a major dam disaster, except for Cwm Carne which collapsed in 1875 with the loss of 12 lives. In 1925, two failures caused further loss of life. Following heavy rainfall, the Skelmorlie lower reservoir in South West Scotland failed on 18 April 1925 and five people were killed in the village of Skelmorlie. The verdict of the jury at an enquiry held at Kilmarnock Sherriff Court was as follows: "*The disaster was caused by absence of any regular skilled supervision and inspection.*" An even more serious event occurred in North Wales. Eigiau was a small mass concrete gravity dam built which impounded a large reservoir ($4,500 \times 10^3 \text{ m}^3$). It breached on 2 November 1925 and some 1.5 million cubic metres of water were discharged through the breach in the hour following failure. The Coedty embankment dam, which was situated 2.5 miles downstream of the Eigiau dam, impounded only $300 \times 10^3 \text{ m}^3$ of water and the water flooding in from the Eigiau reservoir filled the almost full Coedty reservoir within minutes. Coedty was overtopped and its concrete core-wall collapsed. A wave of water and mud hit the village of Dolgarrog and ten adults and six children died in the disaster. The jury returned a verdict of accidental death: "*caused by the bursting of the dam under the wall in consequence of the wall lacking a proper foundation.*" The coroner's jury recommended regular government inspection.

Following these two disasters, a critical step towards reservoir safety legislation was made when a letter to *The Times* from a leading dam engineer, Edward Sandeman, was published on 4 December 1925. He drew attention to the report made by the Select Committee on the Waterworks Bill 1865 and their recommendations, expressing regret "*that some such control as was then suggested has not been brought into effect.*" He asserted that there was a vital need for enquiry into the condition of the reservoirs of the country, from the point of view not only of their sufficiency to withstand the pressures to which they were subjected, but also to ascertain whether, in view of exceptionally heavy rainstorms in recent years, they were provided with overflows of sufficient capacity. His wide experience showed that

the inspection of existing reservoirs was necessary and, in the interests of the public safety, the matter should receive attention without delay.

At the Thirtieth Winter General Meeting of the Institution of Water Engineers on 4 December 1925, the Secretary read out the letter published that day in *The Times*. The following resolution was put to the members and was carried.

“That, in view of the number of reservoirs in the country used for the impounding of water, the desirability of holding a full enquiry into the circumstances of the recent disasters at Dolgarrog, in north Wales, and Skelmorlie, Scotland, be represented to the appropriate Departments, having regard to the uneasiness caused generally by the occurrence of such events.”

It was proposed to send copies of the resolution, embodied in a letter, to the Prime Minister, Home Office, Ministry of Health, Board of Trade, Electricity Commissioners, and the Scottish Office (Council of Institution of Water Engineers, 1925).

At a meeting at the Home Office on 11 December 1925, crucial issues related to whether there should be government supervision and what was required for existing reservoirs, as opposed to new works. A draft report was sent to the Ministry of Health on 31 December 1925. A reply from the Ministry dated 4 January 1926 questioned the reasonableness of requiring an annual inspection of all reservoirs containing more than one million gallons ($4.5 \times 10^3 \text{ m}^3$). An interdepartmental conference with representatives from the Home Office, Ministry of Health, Ministry of Transport, and Board of Trade reported in December 1926. It was asserted that the main legal safeguard against defective reservoirs was the Common Law liability of reservoir owners. Another four years were to pass, with a change of government in 1929, before legislation was introduced.

The Act which became operative on 1 January 1931 is described as: *“An Act to impose, in the interests of safety, precautions to be observed in the construction, alteration, and use of reservoirs, and to amend the law with respect to liability for damage and injury caused by the escape of water from reservoirs.”* Some of the main requirements of the legislation, which was applicable to reservoirs holding more than five million gallons ($22.7 \times 10^3 \text{ m}^3$), were as follows:

- design and supervision of construction by a qualified civil engineer;
- reservoir not to be filled until a qualified civil engineer has issued a certificate;
- inspection by a qualified civil engineer at intervals of not more than ten years;
- undertakers to keep a record of water levels, leakages and settlements.

The Act stated that where damage or injury was caused by the escape of water from a reservoir constructed after the passing of the Act under powers granted after that date, the fact that a reservoir was so constructed would not exonerate the undertakers from any proceedings to which they would otherwise have been liable.

A qualified civil engineer is defined as a civil engineer who is a member of a panel constituted for the purposes of the Act and, with the advice of the President of the Institution of Civil Engineers (ICE), the Secretary of State constituted two panels: panel A engineers being qualified to design and supervise construction and to inspect all reservoirs to which the Act applied and panel B engineers qualified only to inspect reservoirs. In 1946, a four-panel system was introduced: engineers in all the

panels could now act as construction engineer as well as inspecting reservoirs, the differences in the panels related to the type of reservoir.

The President of the ICE set up a small informal advisory committee, but it was 35 years after the Act came into force before the then President, Sir Robert Wynne-Edwards, proposed that advice to the Secretary of State on the composition of panels should come from a special committee set up by the Institution, as provided under Section 8(1) of the 1930 Act. This was formed as "The Institution Committee under the Reservoirs (Safety Provisions) Act 1930".

The 1930 Act contains no technical standards and shortly after it became law, the ICE set up a committee, with W J E Binnie as chairman, to determine the maximum intensity of flood for which provision should be made. Their conclusions were published in 1933 as the *Interim report of the Committee on Floods in Relation to Reservoir Practice* (Institution of Civil Engineers, 1933). The maximum intensity of rainfall was related to the catchment area and a formula was given for calculating the run-off. It was recommended that the catastrophic flood be estimated as at least twice the normal maximum flood. *Floods in relation to reservoir practice* was "a guide to water engineers when determining their own particular requirements. The subject does not lend itself to rigid treatment by means of precise rules and regulations." This was the first attempt to instil a common approach to the subject of reservoir flood safety. As a result, there was a significant increase in the amount of remedial work carried out at dams following the 1930 Act. However, the flash flood at Lynmouth in 1952 which killed 34 people raised further concerns about the accuracy of predicting extreme events and the impact on reservoir safety. Engineers had to wait until 1978, following the publication of the *Flood Studies Report* for new guidance in *Floods and reservoir safety*.

Following implementation of the 1930 Act, there have been no dam failures causing loss of life; however, there have been a number of serious incidents and failures. Wright (1994) listed 10 failures in the period 1960-1971 (including some in Northern Ireland). Major catastrophes abroad have also had an impact on the British approach to reservoir safety. Following the failures of Malpasset (1959) and Baldwin Hills (1963) and the devastating overtopping of Vaiont (1963) that killed more than two thousand people (Jansen, 1980) the proposal was made during the Eighth International Congress on Large Dams, held in Edinburgh in 1964, that all member countries of the International Commission on Large Dams should review their reservoir safety regulations. An *ad hoc* committee was set up by the Institution of Civil Engineers to prepare detailed recommendations for a revision of the 1930 Act (ICE, 1966).

In 1970, the government announced its intention to introduce new legislation and five years later the Reservoirs Act 1975 received Royal Assent on 8 May 1975. In effect, the Reservoirs Act 1975 repealed the 1930 Act and re-enacted reservoir safety legislation in a strengthened form, embodying many recommendations from the 1966 Institution of Civil Engineers report. It was not until 25 April 1983, during a debate in the House of Lords on the Report of the Select Committee on Science and Technology on the water industry, that Lord Skelmersdale stated that the government had decided to follow the Select Committee's recommendation and implement the Reservoirs Act 1975.

The Reservoirs Act 1975 was implemented between 1983 and 1986. The Act applies to "large raised reservoirs", that is reservoirs designed to hold or capable of holding more than $25 \times 10^3 \text{ m}^3$ of water as such above the natural level of any part of the land adjoining the reservoir. The 1975 Act, together with the associated statutory instruments, provides the legal framework within which qualified civil engineers make

technical decisions relating to the safety of reservoirs. In effect, the legislation imposes a system of safety checks on reservoir construction and operation. The Act recognises four types of person or organisation with distinct functions and responsibilities:

- undertakers (duties as owner or operator);
- enforcement authority (to ensure compliance with legislation);
- qualified civil engineer (to advise on safety);
- Department of the Environment [now Defra] (legislator).

The 1975 Act kept the same approach to reservoir safety as the 1930 Act, maintaining the panel system of engineers, but creating the new role of supervising engineer. The enforcement role was given to local authorities. However, in his report to the Secretary of State for the Environment on the failure during construction of Carsington embankment dam, Coxon (1986) recommended that consideration should be given to centralisation of key records relating to certification and inspection of dams. Like its predecessor, the 1930 Act, the 1975 Act contains no technical standards and, with some government funding, a series of guidance documents has been produced (Wright *et al.*, 1992):

- *Floods and reservoir safety* (third edition). (ICE, 1996).
- *Investigating embankment dams: a guide to the identification and repair of defects*. Building Research Establishment Report BR 303. (Charles *et al.*, 1996) .
- *Engineering guide to the safety of concrete and masonry dam structures in the UK*. Report 148, CIRIA, (Kennard *et al.*, 1996).
- *An engineering guide to the safety of embankment dams in the UK* (second edition). Building Research Establishment report BR363. (Johnston *et al.*, 1999).
- *A guide to the Reservoirs Act 1975*. (ICE, 2000).

3.3 Twenty-first century

The Water Act 2003 introduced the requirement for reservoir undertakers to prepare reservoir flood plans where directed by the Secretary of State in England or the National Assembly in Wales. The role of enforcement authority in England and Wales was transferred from local authorities to the Environment Agency in October 2004.

The serious incident at Ulley in June 2007, which resulted in one thousand people being evacuated and the closure of the M1 motorway, was highlighted by Sir Michael Pitt in his review of flooding (Pitt, 2008): he recommended that “*The government should implement the legislative changes proposed in the Environment Agency biennial report on dam and reservoir safety (Environment Agency, 2007) through the forthcoming flooding legislation.*” Environment Agency proposals included a risk-based definition for reservoirs within the Act and mandatory post-incident reporting. Major incidents at smaller non-statutory dams such as Maich Water and Cottage Pool underlined a need for legislation to capture smaller reservoirs where the human cost of their failure could be high.

In 2009, a conflict arose between the requirements of work to be carried out in the interests of safety under the Reservoirs Act 1975 and provisions of the Protection of Badgers Act 1992. Those provisions make it an offence to interfere with a badger sett

unless a licence is granted by Natural England, the agency responsible for policing the Protection of Badgers Act.

During a statutory inspection in September 2008, signs of a large badger sett were observed on the 'dry' side of a six-metre high barrier embankment which is part of a Category A flood risk, off-stream flood storage reservoir. The reservoir fills from a tidal river during its operation. A breach of the embankment could result in several farmhouses, residential and commercial development, public and private roads and a large area of land being inundated. Concerns about the stability of the embankment led the inspecting engineer to make a recommendation in the interests of reservoir safety that the badger-damaged part of the embankment should be repaired "*without delay*". The undertaker agreed the timetable of work but was unable to obtain a license from Natural England until six months after the agreed completion date, by which time they were confident that any badger cubs would have left the sett. The undertaker's representative involved in implementing the repair works chose not to breach the Protection of Badgers Act, thus putting the undertaker in breach of the Reservoirs Act 1975. Had the reservoir operated during the at-risk period, there could have been loss of life and much damage. In the event, weather conditions were such that the reservoir did not operate during that period and the remedial work was satisfactorily completed during July and August 2009. It was discovered that the actual amount of material removed from the embankment by the animals was approximately 50 tonnes from a network of tunnels totalling 250 m in length, extending nearly as far as beneath the embankment crest.

4 Incident management

4.1 Provisions for managing incidents

Responses to dam incidents vary greatly from case to case. The main factors influencing the response are:

- nature of the incident (sudden event or worsening long-term trend in behaviour);
- time when the incident was recognised as requiring urgent action;
- knowledge and experience of those responsible for the safety of the reservoir;
- likely consequences of a sudden uncontrolled release of water from the reservoir.

Most owners of statutory reservoirs are aware of the value of monitoring and surveillance and have the benefit of regular visits by their supervising engineer. Major reservoir owners operate their own teams of monitoring and surveillance personnel. Some statutory reservoir owners also have arrangements with inspecting engineers for 'on call' advice and assistance in dealing with any reservoir safety concerns. In contrast, incidents at smaller, non-statutory reservoirs normally come to light indirectly through government departments or the emergency services.

In recent years, Defra and the Environment Agency have sponsored the development of proposals for formalised emergency planning for dams. Emergency planning for flooding from reservoirs has three parts:

- A Reservoir flood (formally **inundation**) **map**. Prepared by the Environment Agency in many cases, this identifies the extent of inundation from a credible worst case scenario from an uncontrolled release of water. The preparation of flood emergency plans associated with dam breach was provided for in the Water Act 2003 and the Environment Agency led work producing flood maps for over 2000 reservoirs in 2009.
- An **on-site reservoir emergency plan**. Prepared by the undertaker, this sets out what the undertakers will do in an emergency to try to contain and limit the effects of the incident. It includes a plan for communicating with external organisations, mainly the emergency services.
- An **off-site reservoir emergency plan**. Prepared by the local resilience forum (in Scotland, the strategic coordinating group), this sets out what the emergency services will do to warn and protect people and property downstream in the event of an incident which could lead to dam failure.

The aim of emergency planning is to plan, as far as is practicable, for a major dam incident. However, every incident is unique. Commonly, incident management will involve reducing the reservoir level, thereby reducing the load on the dam structure and also the volume of escapable water if the incident response is unsuccessful. This can be achieved by:

- maximising the draw-off flow rate from the reservoir, augmented where possible by opening a bottom (scour) outlet;
- emergency pumping of water from the reservoir;

- where practicable, reducing the rate of flow into the reservoir.

Steps should be taken to warn and evacuate people in the likely affected downstream areas.

The post-incident reporting system aims to capture experiences in dealing with incidents as well as factual information on incident development and effectiveness of remedial works. The following section provides some insight into the management of four serious incidents that occurred between 2001 and 2008.

4.2 Examples of dam incident management

The four examples here have been chosen to focus on the management of incidents rather than their technical causes. Each of the four incidents listed in Table 4.1 is described in Section 5.2. Information was obtained from published papers and from engineers associated with the incidents. In three of the cases, possible catastrophic failure involving loss of life was averted by emergency actions. Two of the dams are owned by large water utilities that have many dams and have access to major resources. The other two dams had been owned by large water companies but when they became surplus to requirements they were sold, one to a private owner with limited resources and the other to a borough council.

Table 4.1 Case studies selected for incident management

Dam name	Incident date	Incident description
Ogston	2001	Catastrophic pipe and valve failure resulting in uncontrolled escape of water into the valve shaft
Rivington Upper (Yarrow)	2002	Internal erosion into culvert
Ulley	2007	Flood flow – severe erosion of embankment due to out of spillway flow
Maich Water	2008	Flood flow – over topping of non-statutory dam

4.2.1 Incident No. 44: Ogston

Description of dam

The dam is an earth embankment with a central puddle clay core. The incident involved the failure of a recently fitted valve to the scour pipework which resulted in fracture of the pipework (Hughes *et al.*, 2004).

Identification of the incident

Initially, operation of the newly installed butterfly valve was stiff and modifications were made to the gearbox. During commissioning tests on the butterfly valve, the pipework immediately upstream, including the compensation branch, suffered catastrophic failure, resulting in the sudden uncontrolled release of water from the scour pipe into the base of the draw-off tower where two men were trying to operate the valve. The discharge started to fill the draw-off tunnel until doors on the downstream end were forced open, allowing water to discharge back to the downstream tail bay area.

Emergency action

A guard valve was then closed to isolate the discharge by men going back through the discharging water. Uncontrolled release of water was stopped by the quick response of the men working in the commissioning tests.

Notification of incident

The panel engineer who had recently inspected the reservoir in accordance with the Reservoirs Act 1975 was informed of the incident. Technical advice was provided by the panel engineer and by the owner's review panel of experts. It became apparent that the draw-off tower and tunnel were not safe places to work, since catastrophic failure of the pipework had occurred and had left the scour guard valve unsupported and therefore operation of the guard valve was considered unsafe. Emergency remedial works recommended by the panel engineer included installation of a temporary bulkhead on the scour forebay headwall and a new means of scouring the reservoir until the remedial works had been completed.

Lessons

The incident raised safety issues concerning the management of incidents in valve towers and tunnels. Following the incident, the undertaker now has a procedure in place to ensure that no works are undertaken that have a reservoir safety implication without first referring it to the reservoir supervising engineer for approval. This will diminish the possibility of someone with inappropriate competency carrying out high-risk works on reservoirs.

An on-site emergency plan should always include the advice *"If valves are difficult to operate, seek out the reason why rather than reverting to more force"*.

4.2.2 Incident No. 39: Rivington Upper (Yarrow embankment)

Description of dam

The 12-m high embankment completed in 1857 has a central puddle clay core with a clay filled cut-off trench. Draw-off is by means of a culvert placed in an excavation beneath the embankment.

Identification of incident

At the time of the incident, reservoir keepers habitually visited the dam three times a week. The reservoir keeper was in the culvert on 8 January 2002 to monitor a small leak from the roof which had existed for many years. On 9 January 2002, the keeper was driving across the top of the embankment en route to another dam and saw a broad band of discoloured water emerging from the culvert. The identification of the sudden increase in leak was through a providential sighting. A major leak was occurring through the sidewall of the culvert.

Notification of incident

The reservoir keeper contacted the supervising engineer who reached the site an hour later, about 14:30. The reservoir safety manager reached the reservoir soon after and contacted an all reservoirs panel engineer who arrived on site early the following day. The undertaker's operational response centre was alerted to the situation and dealt with all the contacts with the police, Lancashire County Council and local residents. Some 53 high-risk properties were identified; a warning letter was distributed, but in the event no one was evacuated.

Emergency action

An attempt was made to block the point of leakage with rags, a board and wooden struts. The scour valves were partially opened to avoid disturbing the temporary struts but this had little effect on the reservoir level, as the reservoir was still overflowing the following morning. Pumps were used overnight to lower the reservoir.

Inflow into Upper Rivington from the upper reservoir Anglezarke was halted by raising its spillway level. An all-night watch was mounted on the dam. The leak was plugged and struts the following morning and the scour valves were opened fully. Upper Rivington was drawn down at a rate of one metre per day. The rapid drawdown caused some minor flooding of house foundations under construction.

Lessons

Early detection of the leakage and rapid lowering of the reservoir were key factors in the successful management of the incident.

The main lessons learnt were:

- Internal erosion incidents can develop rapidly.
- Regular and frequent surveillance is essential to detect serious defects at dams while there is still time to take effective action. As a result of the incident, the frequency of surveillance was increased.
- The need for an on-site emergency plan which details the actions to be taken in the event of serious progressive internal erosion, including means of diverting inflows, was highlighted by this incident.
- Contractors should be on call to carry out works at short notice, overnight if necessary.
- The value of a functioning bottom outlet, which allows the reservoir to be lowered rapidly in the event of a problem, is essential.

4.2.3 Incident No. 80: Ulley

Description of dam

The 16-m high puddle clay core dam was completed in 1873 and came under ownership of the borough council in 1986. The reservoir is the centrepiece of the Ulley Country Park.

The spillway channel, which ran down the left mitre of the embankment, disintegrated under flood conditions leading to rapid erosion of the downstream shoulder. Emergency action was taken to divert flow from the damaged spillway, lower the reservoir using temporary pumps and import rockfill to repair the embankment.

Identification of incident

Country park rangers were asked to check on the reservoir periodically in the afternoon and evening of the 25 June 2007 following heavy rain that caused flooding in the area. At about 20:00, a ranger reported damage to the dam from disintegration of the spillway channel.

Notification of incident

The country park manager who was one of the forward liaison officers for the borough council was notified and went to the site. In consultation with the local police, a major incident was declared. An engineer from the council arrived on site at 22:00 and recognised the situation was serious. Once the incident had been identified, council emergency planning procedures were invoked. These were in place following the requirements of the Civil Contingencies Act 2004 and the duty of county councils to establish resilience forums. The supervising engineer for the reservoir was notified and went to the site immediately, arriving at about midnight. At 1:20 on 26 June, the chief executive of the council requested that the M1 be closed and residents from the areas at risk were evacuated.

Had the reservoir been in a remote location, without the level of responsible surveillance and emergency procedures in place, failure of the dam with loss of life might have occurred.

Emergency action

The council's emergency planning procedures had been established under three hierarchical levels described as Bronze (Operational), Silver (Tactical) and Gold (Strategic). A Bronze Command centre was established at Ulley reservoir comprising on-site experts able to identify immediate needs and resource requirements, such as pumps, plant and materials. Silver Command was located at the local district police centre with the purpose of dealing with any requests from Bronze Command such as obtaining plant. Gold Command operates at county level and monitors crises in different areas. The resources needed to mitigate the potential failure of Ulley reservoir were given precedence by Gold Command over flooding elsewhere in the borough.

Once an all reservoirs panel engineer had been appointed, the supervising engineer's role changed to a support role for the panel engineer who made arrangements for the attendance of an experienced contractor to carry out the emergency stabilisation works.

The local authority duty forward liaison officer provided a single point of contact through which requests for information, materials and services from council or contractors could be made.

There were no evacuation plans on site. A decision about which properties to evacuate was based on a brief study of maps at both Bronze and Silver control, as there were no evacuation plans for premises downstream of Ulley.

Lessons

Although good generic arrangements for contacting the emergency services were in place, an on-site emergency plan did not exist. The incident highlighted the need for an emergency plan that would have key drawings, valve procedures, capacity of the scour pipe, emergency contacts, and standby procedures for emergency works (contractors, staff and suppliers).

The importance of the role of the supervising engineer was highlighted by the incident (Crook, 2008). When a supervising engineer is advised of an incident and is asked to go immediately to the dam, he should have a checklist of equipment and know where relevant documents are stored. The supervising engineer's own file on the reservoir should include the last inspecting engineer's report, selected drawings, the last annual statement and Section 11 records, the contact details of the panel engineer who last inspected the dam and a list of all reservoirs panel engineers from the Defra and/or Environment Agency website.

Events need to be recorded that could be used at the inquiry in the event of a catastrophic failure involving loss of life. Digital cameras can play a vital role and should always be available and charged. Both the supervising engineer and undertaker should be aware of mobile telephone network coverage at the site and if none exists, the quickest way of communicating with the emergency services. The possibility of the mobile network being overloaded should be considered. Rotherham Metropolitan Borough Council, as part of their emergency plan, are part of the ACCOLC (ACCess OverLoad Control) for mobile telephones whereby the telephone system is restricted to those who have a recognised essential requirement to use the telephone.

The incident raised the issue of availability of transport, the time taken to get to the site and the possibility of multiple dam incidents in an extreme rainfall event. At Ulley, there was a problem in getting a crane to the site because of the amount of water in the area. In practice there will be many situations where the supervising engineer will not be able to get to the incident site in reasonable time. At Ulley the supervising engineer lived reasonably close. The undertaker should take into account the availability of the supervising engineer.

Knowledge of the reservoir and dam, their behaviour and the potential consequences of failure are important. Supervising engineers need to be aware of emergency plans and assist undertakers in their preparation. In formulating an emergency plan, the following should be included:

- access to the site
- welfare on site
- communications
- working hours
- multiple incidents
- involvement of local authority emergency planning.

The effectiveness of the supervising engineer depends on a long association with the reservoir and frequent changes do not facilitate this. Appointments of at least three to five years are recommended.

The incident at Ulley was well controlled by the prompt and efficient action of the borough council, the emergency services and a framework contractor for Yorkshire Water. However, the incident at Ulley could have led to failure of the dam had it occurred in a remote location or without modern communications and the ability to mobilise plant and materials rapidly.

4.2.4 Incident No. 83: Maich Water

Description of dam

The nine-metre high clay core dam has an impounded depth of 6.3 m and a capacity of $24.3 \times 10^3 \text{ m}^3$. It was built in the second half of the nineteenth century for water supply. The dam was originally an off-stream reservoir with a substantial bypass channel. Collapse of the bypass channel and failure of subsequent repairs to the bypass channel a few years before the incident led to the flood flow going into the reservoir. This, together with settlement of the dam and placing of screens partially blocking the overflow, led to overtopping of the dam following heavy rain and caused erosion of the downstream shoulder.

Description of incident

The incident occurred in the early hours of 1 August 2008. The length of time that water was flowing over the dam is not known but it was still overtopping when the fishing club manager arrived at the start of the day and informed the owner's representative. After the fishing club manager had taken down the screens from the overflow, the seriousness of the incident became apparent and the police were contacted; the police, in turn, contacted the council emergency planning officer. Following inspection of the dam and a report by one of the council's engineers, an

emergency response was activated which involved local evacuation and closure of the main A760 trunk road by the police.

The owner was helpful in giving access and information on the dam, but was absolutely non-committal on engaging or agreeing to pay for any contractor to carry out emergency work. By this time the council had brought pumps, direct labour and their contractor and materials to site and had reinstated the bypass and emptied the reservoir at substantial cost. Emergency works started on the morning of 4 August 2008 and were completed on 9 August 2008.

Under the Civil Contingencies Act, the council's civil contingencies unit provides a contact point for the emergency services for a major incident, and it was through this system that the first call was received of the incident at Maich dam.

Strathclyde police, as the lead agency, agreed to engage a qualified engineer. The all reservoirs panel engineer was only involved to advise on the safety of the dam and its safe removal. To resolve the incidents, there is a normal and routine progress to resolution. At Maich, legal permission had to be sought from the landowner for the removal of the dam.

Lessons

During the multi-agency debriefing the following issues were raised:

- Clarification and guidance needs to be sought from government on future emergency operations where public safety is involved.
- A pool of expert contacts for unusual or unprecedented incidents should be created.

Reservoirs not under the Reservoirs Act 1975 that pose a threat to life or infrastructure need to be identified.

4.2.5 Role of the all reservoirs panel engineer

Concern over the role of the assisting all reservoirs panel engineer was raised by the incidents described above. Unless there is an agreement for the engineer services with the undertaker, as at Ulley, it needs to be established who the engineer is working for. What is the status of advice given to the emergency services and if the engineer is called in by the enforcement authority or local authority, what is the engineer's liability?

5 Description of incidents

5.1 Introduction

One hundred reservoir and dam incidents in Great Britain are summarised in this section. Each incident has been given a unique reference number. Section 5.2 provides summaries of 30 key incidents that have influenced reservoir legislation or dam practice, or provide good examples of typical failure modes. Four overseas incidents are also described. These have all occurred during the last two hundred years. Section 5.3 provides brief summaries of a further 70 incidents. In most cases these are typical of the modes of failure already described within the 34 case histories. In some of these cases, limited information is available. Although the sample of incidents is relatively small, the distribution of incident types is similar to those published by others (Moffat, 1975; Charles and Boden, 1985; Tedd *et al.*, 2000). Many more incidents have occurred but those listed are generally in the public domain and include the major failures that involved loss of life.

The selected incidents and failures have been classified according to type of incident or mode of failure such as internal erosion, slope instability and overtopping. Not all the incidents fall easily into the group classification used in Table 5.1; some could be in more than one of the groups. For example, leakage through the core of an embankment can lead to internal erosion or downstream slope instability depending on the nature of the fill. Wave action can cause damage to the upstream protection but it can also lead to spray being carried onto the downstream slope, causing slope instability.

Two lists summarise the 100 incidents described in Section 5.2 and 5.3 to allow identification and cross-referencing of incidents. The highlighted incidents in both lists are the 30 key incidents described in Section 5.2.

- List 1 arranges the incidents alphabetically by dam name together with the unique reference number, incident year and nature of the incident.
- List 2 arranges the incidents into the types shown in Table 5.1 and chronologically by incident date within each type. The list provides basic features of each dam together with a brief description of the incident type and damage.

Table 5-1 Classification of incidents

Group - Incident type	Brief description
1: Internal erosion or leakage on first filling	'First filling' includes the first five years of operation; cause and location of erosion or leakage are not distinguished.
2: Internal erosion or leakage in service	Incidents that only involve leakage or internal erosion through the core or main body of the dam.
3: Internal erosion or leakage in service associated with ancillary works/cut-offs/abutments	Internal erosion or leakage associated with structures passing through an embankment; erosion into a culvert, erosion along a culvert or shaft, erosion from a pipe under reservoir pressure. Also includes erosion in deep cut-offs and adjacent to abutments.
4: Incidents due to pipe or valve failure	Failures due to fractured pipes that allow water under pressure into the embankment or draw-off works.
5: Slope instability during	Only includes slope instability failures associated with

Group - Incident type	Brief description
construction	construction before the dam starts to impound water.
6: Slope instability in service	Caused by leakage through the dam, rain, spray over the dam, side valley run-off, tree removal and rapid reservoir drawdown.
7: External erosion due to flood flow	Overtopping incidents and those when spillway is damaged.
8: Wave damage to upstream protection	These incidents, in some cases, result in spray causing slips on the downstream slope.
9: Reservoir basin leakage and instability	Generally, these incidents are serviceability problems and not a safety issue.
10: Concrete and masonry dams	Failure modes and deterioration associated with concrete and masonry dams including foundation failure, concerns about stability and material deterioration.
11: Other incidents	Includes aircraft strike.
12: Overseas incidents	Four overseas incidents involving major failures.

Incidents have also been classified according to the severity of the incident. The severity classification system given in Table 5.2 is that used in the Environment Agency's post-incident reports. The system does not take into account the consequence of failure.

Table 5-2 Classification of incident severity

Incident level	Definition
1	Failure (uncontrolled sudden large release of retained water)
2	Serious incident involving any of the following: <ul style="list-style-type: none"> • emergency drawdown • emergency works • serious operational failure in an emergency.
3	Any incident leading to: <ul style="list-style-type: none"> • an unscheduled visit by an inspecting engineer • a precautionary drawdown • unplanned physical works • human error leading to a major change in operating procedures.

Analysis of the incidents

This analysis only includes the British incidents described in this report. Figure 5.1 shows the number of incidents in each category and when they occurred with respect to British reservoir legislation, pre-1930 Act, post-1930 and post-1975 Act (1983 is assumed, being the start of implementation of the 1975 Act). Figure 5.2 shows the number of incidents for each category for the different incident severities.

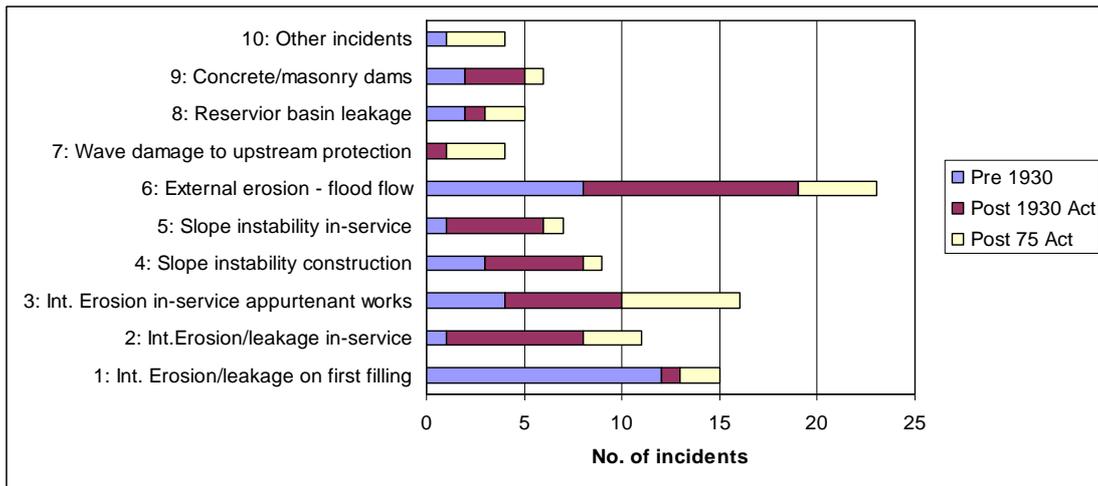


Figure 5-1 Distribution of incident according to type and date

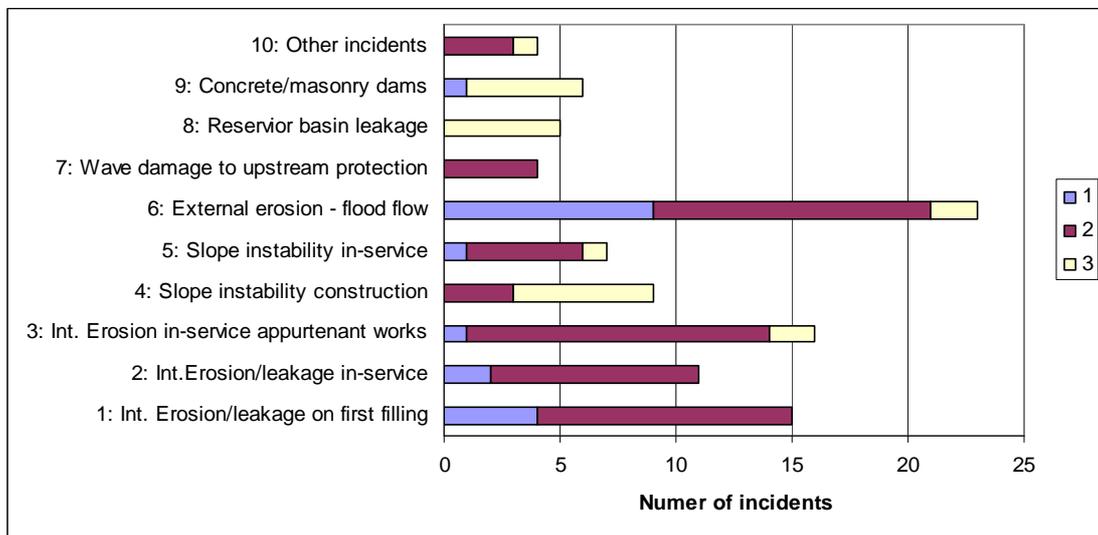


Figure 5-2 Distribution of incidents according to type and severity

Figure 5.3 shows the number of incident included in this report per 25-year period (except for 2001-2008) since 1799.

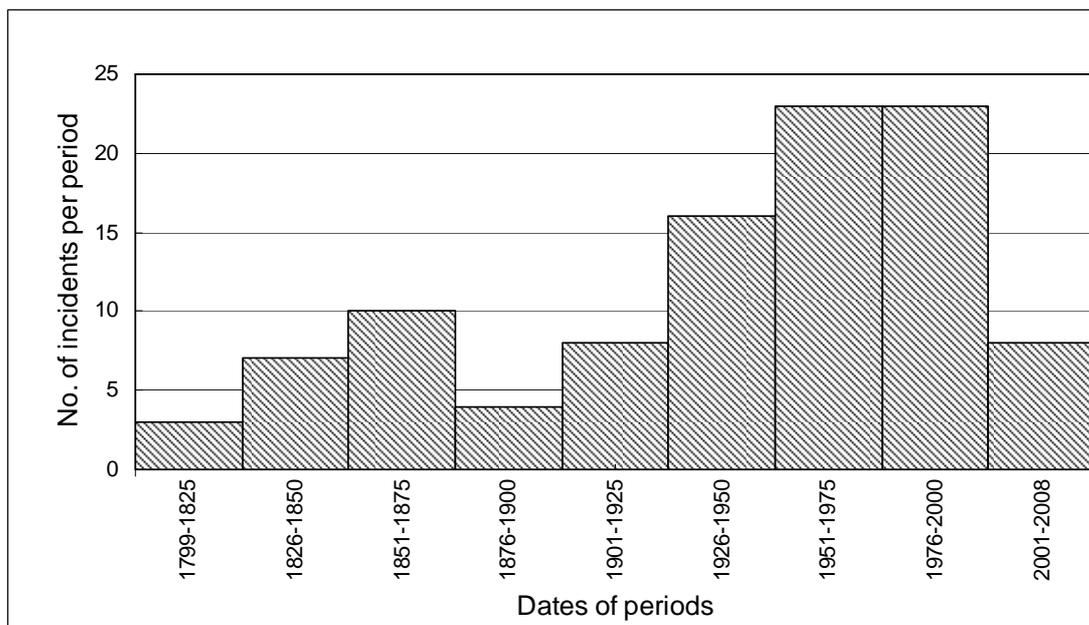


Figure 5-3 Number of incidents per 25-year period

Group 1: Internal erosion or leakage on first filling

Internal erosion and leakage on first filling was not uncommon for pre-1930 embankment dams but is rarer since the introduction of more effective methods of treating foundations such as concrete filled cut-offs, grouting and diaphragm walls and of rolled clay cores.

Most of the incidents listed in this group occurred during the nineteenth and early part of twentieth century and involved embankment dams of traditional construction with a puddle clay core and clay filled cut-off trench. Incidents were related to the poor performance of the puddle core or the clay filled cut-off. The two most catastrophic dam failures involving loss of life, Bilberry and Dale Dyke, are in this group. Puddle clay had long been regarded as the only reliable material for resisting water under pressure. Bateman was one of the first engineers to use concrete in cut-off trenches in the 1870s. By the beginning of the twentieth century, the merits of using “cement concrete” in deep cut-off trenches in fissured ground were appreciated. At least five of the listed incidents stem from the presence of springs encountered in fissured ground in deep cut-offs and there are known to be many more cases.

Other incidents relate to the use of pipes and culverts passing through embankments which fractured due to embankment movement or created stress conditions which triggered leakage along the outside of the pipe.

The introduction in the 1950s of well-compacted rolled clay cores did not entirely eliminate serious leaks and erosion problems as was demonstrated by Balderhead which was completed in 1965, with a relatively narrow core and an ineffective downstream filter.

Although only six dam dams in the UK have an upstream asphaltic lining, each reservoir is large and has generally performed well. The unsatisfactory performance of the lining at Winscar shortly after first filling involved differential settlement of the fill and culvert structure.

Group 2: Internal erosion or leakage in service

This group includes leakage or internal erosion incidents that occurred after the reservoirs had been in service for at least five years. All of the dams have puddle clay cores or are described as homogeneous. The incidents chosen are not related to deep cut-off trenches, pipes and culverts. Most of the incidents occurred many years after construction - up to 70 years - although problems may have been present since construction. Some incidents are associated with changes at the top of the core, such as settlement or drying shrinkage. As settlement continues, leakage can be expected at other dams although it is generally unlikely to be serious. In many cases monitored leakage rates have increased significantly when the reservoir level has approached top water level. Animal burrows were reported as the main cause of internal erosion that resulted in breaching of two embankments.

Group 3: Internal erosion or leakage in service associated with ancillary works/cut-offs/abutments

Most internal erosion incidents that have continued to occur at embankment dams with puddle clay cores are associated with structures (draw-offs and overflow culverts, unprotected pipes) passing through the embankment or foundation cut-offs. This group includes internal erosion incidents associated with structures passing through an embankment; erosion into a culvert or shaft and erosion along a culvert. It includes erosion in deep cut-offs in fissured rock with springs and also in cores adjacent to abutments. As with the previous group, most of the dams have puddle clay cores. Most have deep puddle clay filled trenches and it is these that have been the cause of long-term erosion in a number of cases. With the exception of Warmwithens, where internal erosion occurred along a recently constructed tunnel through the embankment, there have been no reported internal erosion incidents of dams in service in Great Britain involving uncontrolled release of water since the start of the twentieth century. However, there have been many near misses and many cases of ongoing internal erosion.

Group 4: Incidents due to pipe or valve failure

This group includes failure of pipes that have resulted in water under reservoir pressure penetrating into the embankment or draw-off works. The early practice of laying unprotected pipes through embankments and puddle cores caused concern. However, there are few documented cases where failures of the pipes or joints have led to incidents. At both Bilberry and Dale Dyke, failure of the pipes was cited as a contributory cause to the failure of the dams. Corrosion of cast iron pipes has long been recognised as a potential problem and condition assessments and refurbishment are common practice (Reader *et al.*, 1997). High-density polyethylene pipes have been used to line cast iron pipes on many old dams. The incident at Ogston highlights the problem of valves being replaced with ones inappropriate for the required duty and also the brittle nature of grey cast iron when subjected to impact loading.

Group 5: Slope instability during construction

There have been many incidents of slope instability during the construction of earth dams. The majority described here were deep-seated slips which involved a major change in the design of the dam and included addition of berms, flatter slopes, toe weights and in some cases total reconstruction of sections. Although most of the incidents described are post-1930, numerous earlier incidents also occurred. Three of the incidents described, Abberton, Chingford and Muirhead, which occurred between 1937 and 1945, were due to rapid construction following the introduction of modern earth-moving equipment such that excess pore water pressures had less time to dissipate. Dams of similar design had been constructed successfully prior to the introduction of rapid construction methods. Studies of instability during construction of embankment dams led to major developments in soil mechanics.

Group 6: Slope instability in service

The number of in-service slope instability incidents is small compared with the total number of leakage/internal erosion incidents. With the exception of Earlsburn in 1839, over which there is some doubt about the failure mechanism, there have been no slope instability incidents involving uncontrolled escape of water, but some well-documented 'near misses'. A variety of causes of slope instability have been recorded in both upstream and downstream shoulders; particularly the presence of water causing a decrease in effective stress, from rain, leakage through the dam, broken supply pipe within the dam, spray over the dam or flows from valley sides. Other causes include removal of trees, rapid reservoir drawdown and construction.

Group: 7: External erosion due to flood flow

The flood guidance that has evolved since publication of the interim report of the Committee on Floods in Relation to Reservoir Practice in 1933 has resulted in large numbers of enlarged or additional spillways being constructed, particularly since the Floods Studies Report of 1975, but incidents involving overtopping of dams and damage to spillways at statutory and non-statutory dams continue to occur whenever there is an extreme rainfall event. Some of these are due to spillways inadequate for the design flood and others are due to construction type, poor engineering or lack of maintenance.

Often there is little information about these incidents as they generally occur over a relatively short period associated with an extreme flood event of short duration. The rapidity with which overtopping can lead to catastrophic failure is illustrated by Skelmorlie and Trewitt Lake. Overtopping incidents at smaller dams with little evidence of damage is common knowledge, but there are few published case histories. Archer (1992) lists some 11 cases, three of which are included in the list of 70 incidents. None of the overtopping incidents since 1930 has involved breach of a major dam.

The 2007 floods had widespread reservoir safety impacts on a regional scale (Warren and Stewart, 2008). They highlighted inadequacies in some masonry spillways in terms of performance, maintenance and location close the mitres and toes of dams.

Group: 8 Wave damage to upstream protection

The upper part of the upstream face of embankment dams requires protection against wave action to avoid serious damage of the underlying shoulder fill and potential breaching of the embankments. Stone and rough-dressed masonry pitching has been the traditional and well-tried means of protection, with over 80 per cent of earth dams in the UK protected in this manner. Pitching has generally performed well, with long-term deterioration being the main safety issue. Few recorded incidents associated with wave damage have raised concern over the safety of the dam.

Since 1945, other methods including pre-cast concrete blocks, cast *in situ* concrete slabs and rip-rap have been used generally on grounds of cost. The earliest use of concrete slabs on a large scale was at Staines North and South reservoir embankments completed in 1902. Wave damage to several embankments with concrete blocks and slabs highlighted the lack of guidance on the design, maintenance and performance of upstream protection. This led to the report *Performance of blockwork and slabbing for dam faces* by Herbert *et al.* (1995) being commissioned which summarises the results of a survey into the performance of pitching, blockwork and slabbing at British dams and gives details of failures and

repairs. The methods used to assess performance of upstream protection against wave are summarised by Besley *et al.* (1999).

Group 9: Reservoir basin leakage and instability

This group of incidents refers to the reservoir basin as distinct from the dam, its foundation or abutments. Different types of incident relate to the performance of the reservoir basin and the effect of the reservoir on the stability of the ground adjacent to the reservoir. There are many cases, particularly on first filling, where it has been necessary to line the reservoir basin with clay or construct extended wing cut-offs to avoid excessive leakage. Generally, these incidents have not posed a threat to safety or property, but have prevented the reservoir fulfilling its function of holding water and have caused financial loss to the owner. Damage to property or threat to human life has occurred where there have been leaks into mine workings, as at Ainsworth Mill Lodge which led to the Rylands versus Fletcher legal action.

The effect of the reservoir on the stability of surrounding ground due to increased pore water pressure or wave action has never, as far as is known, caused a serious incident in Great Britain. However, the massive slide into the reservoir at Vaiont, Italy, led to a major loss of life. This incident is summarised in Section 5.2.12.

Group 10: Concrete and masonry dams

Six incidents at concrete or masonry dams are included. Eigiau dam, which failed and subsequently caused the failure of Coedty embankment downstream, was a roughly made concrete dam. The dam was intrinsically unsafe, having been built on an inadequate clay foundation at a depth significantly less than that specified in the design. This is the only example of a masonry or concrete dam failure in the UK.

The other incidents in this guide relate to cracking, leakage through or under the dam, uplift pressures and concerns about stability. Three of the incidents are a result of inadequacies in the design or construction which could not have been foreseen at the time of construction. None of the incidents involved “drastic” emergency actions, nor could they be considered as resulting in imminent failure of the dam, but all required remedial works (significant in two cases) to improve the stability and safety of the dam. The approach to remedial works was significantly different in each case.

Failures of masonry or concrete dams have occurred abroad. Bouzey dam in France, completed in 1881, failed in 1895 killing more than 100 people. The dam was slender, with a base width of 11.3 m for a height of 22 m and was underdesigned compared with UK practice at Vyrnwy and Derwent. The cross-section did not satisfy the middle third rule and the mortar used to bond the masonry was suspect. The Vega de Tera Dam in Spain, a masonry-faced concrete buttress dam completed in 1958, failed on first filling with the loss of 144 lives. This appears to have failed due to excessive tension in the upstream face. However, most failures of concrete and masonry dams are attributed to foundation and abutment rock failures.

Group 11: Others incidents

This includes two incidents that do not readily fall into the other groups.

Group 12: Overseas incidents

Four overseas incidents are included to illustrate incident types that have not occurred in the UK. Other major overseas incidents causing loss of life are listed in Section 1. Catastrophic failure of the arch dam at Malpasset illustrates the need for thorough site investigation and the importance of thin weak layers in an otherwise competent rock formation. The disaster at Vaiont emphasises the significance of the potential instability of reservoir slopes.

List 1 - Incidents at British dams arranged alphabetically

Incident No.	Dam name	Date Built	Incident Year	Nature of incident
48	Abberton	1940	1937	Deep seated slope instability due to rapid construction
88	Ainsworth Mill Lodge	1860	1860	Leakage into and flooding of disused mine workings
59	Aldenham	1795	1975	Downstream surface slip triggered by tree removal
46	Alston no 1	1932	1927	Deep seated slope instability during construction
38	Anglezarke (Heapey)	1857	1997	Leakage associated with abutment
57	Auchendores	1922	1968	Downstream surface slip caused by overtopping waves
13	Balderhead	1964	1967	Internal erosion of rolled clay core on first filling
21	Barrow Compensation	1863	1968	Leakage of abandoned dam that filled in flood
47	Bartley	1930	1927	Upstream slope instability during construction
100	Beggars Hall Lake		1999	Plane crash on to small embankment
87	Bewl Bridge	1975	1987	Wave damage to concrete slabbing
3	Bilberry (1852)	1845	1852	Internal erosion caused settlement and eventual overtopping and breaching of dam
71	Bilberry (1944)	1853	1944	External erosion of downstream shoulder - flood from hillside
1	Blackbrook (1799)	1797	1799	Internal erosion caused embankment settlement and eventual overtopping and breach
96	Blackbrook (1957)	1906	1957	Earthquake damage of concrete dam
93	Blackwater	1907	1909	Thermal cracking of concrete dam
84	Blithfield	1953	1962	Wave damage to upstream protection and shallow downstream slip
65	Bold Venture (Darwen)	1844	1848	Breached during flood
81	Bolby	1880	2005	Serious damage to spillway and erosion of downstream toe during flood
29	Bottoms (Macclesfield)	1850	1929	Downstream slip associated with leakage over core and damaged pipes.
64	Brent (Welsh Harp)	1837	1841	Breached due to overtopping during flood
58	Buckieburn	1905	1970	Substantial shallow downstream slip triggered by heavy rain and high winds
40	Carno Lower	1911	2005	Internal erosion associated with draw-off culvert
53	Carsington	1984	1984	Major deep seated upstream slip during construction
77	Chew Magna	1850	1968	Serious damage to spillway stilling basin during flood flow
94b	Coedy (Dolgarrog)	1924	1925	Breached due to flood flow from failure of Eigiau
89	Colt Crag	1884	1888	Basin leakage due to sink holes
60	Combs	1805	1976	Major shallow downstream slip due to wave and spray action
78	Corsham Lake		1968	Flood flow, embankment overtopped causing significant erosion
12	Coulter	1908	1912	Internal erosion caused sudden settlements
68	Cowlyd	1921	1924	Dam overtopped during flood and nearly failed
8	Cowm	1875	1877	Internal erosion of cut-off in fissured rock
18	Craig-y-Pistyll	1877	1939	Internal erosion of homogeneous dam
67	Cwm Carne	1792	1875	Breached due to overtopping during flood following years of settlement and neglect
20	Cwmtillery	1870	1954	Internal erosion of core associated with settlement caused by mining
6	Dale Dyke	1863	1864	Internal erosion and catastrophic failure on first filling
9	Den of Ogil	1878	1881	Leakage on first filling around discharge pipe
63	Diggle Moss (Black Moss)	1810	1810	Breached during flood
5	Doe Park	1861	1863	Internal erosion associated with draw-off culvert across the core and the deep cut-off
73	Doxford Lake	1910	1948	Dam overtopped but no serious damage occurred
70	Dunblane	1933	1943	Dam overtopped during flood causing some erosion
99	Dundreggan	1957	1998	Damage to spillway gate due to vibration
54	Earlsburn		1839	Breach associated with earthquake
94a	Eigiau (Dolgarrog)	1911	1925	Breached due to inadequate foundation
95	Glendevon Upper	1955	1954	Leaks and concerns about stability of concrete dam since first filling
35	Greenbooth	1962	1983	Internal erosion associated with hydraulic fracture close to abutments
7	Grizedale	1866	1867	Internal erosion associated with deep clay filled cut-off trench
55	Harlow Hill	1871	1951	Deep seated downstream slip triggered by rain
30	Holden Wood	1841	1945	Internal erosion associated with culvert through the core
36	Holmestyes	1840	1993	Internal erosion into the valve shaft
15	Horndoyne	1890	1990	Internal erosion and breach associated with outlet pipe on first filling
31	Island Barn	1911	1950	Internal erosion associated with tree roots
22	Kellington East		2008	Breach caused by internal erosion associated with animal burrows
86	Kielder	1982	1984	Wave damage to upstream concrete blockwork
19	King George V	1912	1945	Leakage and stability concerns associated with prolonged drawdown
41	Knypersley	1827	1828	Pipework passing through core pulled apart
79	Kype	1898	1977	Significant erosion adjacent to spillway
61	Lambieatham	1899	1984	Shallow slip associated with heavy rain and runoff from valley side
92	Larksheath	1995	196	Failure of HDPE membrane laid over sinkholes in foundation
26	Lliw Lower	1867	1873	Internal erosion associated with spring in deep clay filled cut-off
32	Lluest Wen	1896	1969	Internal erosion into drawoff tunnel
43	Longwood Compensation	1828	1988	Downstream fill saturated due to fractured outlet pipe
24	Luxhay	1905	1994	Leakage through upper part of core
83	Maich Water		2008	Overtopping of dam and severe erosion of downstream fill during flood
85	Megget	1982	1984	Wave damage to upstream pitching
90	Mill Hill	1939	1979	Collapse of concrete service reservoir associated with mining activity
17	Monkswood	1895	1931	Leakage through upper part of core caused saturation of downstream fill
50	Muirhead	1942	1941	Deep seated slip due to rapid construction
97	Mullardoch	1951	1986	Leakage increase due to cracking of concrete dam raised concerns about stability
23	Oakdale Lower	1890	1986	Sinkhole on crest caused by internal erosion
44	Ogston	1959	2001	Valve and pipe failure in draw-off shaft caused by fitting an inappropriate valve
27	Pentwyn	1863	1882	Internal erosion associated with spring in deep clay filled cut-off
91	Pen-y-Rheol	1863	1985	Collapse of mine workings caused sinkholes in reservoir basin

List 1 - Incidents at British dams arranged alphabetically, continued

Incident No.	Dam name	Date Built	Incident Year	Nature of incident
2	Redmires Lower	1849	1850	Erosion of embankment from water in overflow culvert passing through dam
4	Rhodeswood	1852	1858	Internal erosion of very narrow core caused settlement and turbid leakage
39	Rivington Upper (Yarrow)	1857	2002	Internal erosion into draw-off culvert
28	Roddlesworth Upper (1904)	1865	1904	Internal erosion associated with spring in deep clay filled cut-off
56	Roddlesworth Upper (1954)	1865	1954	Shallow slip associated with heavy rain and runoff from valley side
69	Skelmorlie	1861	1925	Flood flow, overtopped and breached with loss of life
33	Slade Lower	1889	1970	Internal erosion of core close to spillway
45	Spade Mill No.1	1862	1860	Shallow slips during construction
74	Thorters	1900	1949	Dam overtopped during flood
52	Tittesworth	1962	1960	Slope instability of old dam due to partial removal of toe during construction of new dam
76	Toddbrook (1964)	1840	1964	Significant damage to spillway and erosion adjacent to spillway during flood, but dam not overtopped
22	Toddbrook (1977)	1840	1977	Internal erosion of upstream fill
42	Torside	1851	1854	Draw-off pipes fractured and pulled apart
75	Trewitt Lake		1963	Dam breached due to overtopping due to snow melt
72	Tumbleton Lake	1885	1946	Dam overtopped during flood causing some erosion
62	Tunnel End	1798	1799	Breached due to overtopping during flood
82	Ulley	1874	2007	Serious damage to spillway and erosion of downstream embankment during flood
51	Usk	1955	1953	Potential for slip during construction due to high pore water pressures
98	Val de la Mare	1962	1971	Cracking of dam due to alkali silica reaction
10	Walshaw Dean Lower (1907)	1907	1907	Internal erosion on first filling associated with clay filled cut-off
80	Walshaw Dean Lower (1989)	1907	1989	Damage to spillway during flood
11	Walshaw Dean Middle	1907	1907	Internal erosion on first filling caused large settlements
37	Walshaw Dean Upper	1907	1997	Sinkhole thought to be caused by internal erosion
34	Warmwithens	1870	1970	Internal erosion along draw-off culvert
16	Whinhill	1821	1835	Internal erosion caused breach with loss of 31 lives
49	William Girling (Chingford No 2)	1951	1937	Major deep seated downstream slip due to weak foundation and rapid construction
14	Winscar	1975	1976	Leakage through asphaltic membrane close to draw-off culvert
66	Woodhead 1	1849	1849	Breached during construction due to overtopping during flood

Note: Highlighted incidents are described in section 5.2

List 2a. Incidents at British dams categorised by nature of incident and incident year - Incident groups 1, 2, 3 and 4

Incident No.	Dam name	Date Built	Incident Year	Height m	Res Vol 10 ³ x m ³	Watertight element	Cut-off	Nature of failure/incident/cause	Incident severity
Group 1 - Internal erosion or leakage on first filling									
1	Blackbrook (1799)	1797	1799	13		PCC	PC	Internal erosion caused embankment settlement and eventual overtopping and breach	1
2	Redmires Lower	1849	1850	14	634	PCC	PC	Erosion of embankment from water in overflow culvert passing through dam	2
3	Bilberry	1845	1852	29	310	PCC	PC	Internal erosion caused settlement and eventual overtopping and breaching of dam	1
4	Rhodeswood	1852	1858	21	2,273	PCC	PC	Internal erosion of very narrow core caused settlement and turbid leakage	2
5	Doe Park	1861	1863	18	486	PCC	PC	Internal erosion associated with draw-off culvert across the core and the deep cut-off	2
6	Dale Dyke	1863	1864	29	3,240	PCC	PC	Internal erosion and catastrophic failure on first filling	1
7	Grizedale	1866	1867	22	332	PCC	PC+C	Internal erosion associated with deep clay filled cut-off trench	2
8	Cowm	1875	1877	16	1085	PCC	PC	Internal erosion of cut-off in fissured rock	2
9	Den of Oqil	1878	1881	15	604	PCC	?	Leakage on first filling around discharge pipe	2
10	Walshaw Dean Lower	1907	1907	24	727	PCC	PC	Internal erosion on first filling associated with clay filled cut-off	2
11	Walshaw Dean Middle	1907	1907	24	1111	PCC	PC	Internal erosion on first filling caused large settlements	2
12	Coulter	1908	1912	24	2,182	PCC	PC	Internal erosion on first filling	2
13	Balderhead	1964	1967	48	19,684	RCC	C + G	Internal erosion of rolled clay core on first filling	2
14	Winscar	1975	1975	53	8,296	UAC	G	Leakage through asphaltic membrane close to draw-off culvert	2
15	Horndoyne	1990	1990	5	14+	Hom	?	Internal erosion and breach associated with outlet pipe on first filling	1
Group 2 - Internal erosion or leakage in-service									
16	Whinhill	1821	1835	12	262	Hom	?	Internal erosion caused breach with loss of 31 lives	1
17	Monkswood	1895	1931	15	231	PCC	PC	Leakage through upper part of core caused saturation of downstream fill	2
18	Craig-y-Pistyll	1877	1939	13	350	Hom	?	Internal erosion of homogeneous dam	2
19	King George V	1912	1945	8	12,400	PCC	PC	Leakage and stability concerns associated with prolonged drawdown	2
20	Cwmillery	1870	1954	13	148	PCC	?	Internal erosion of core associated with settlement caused by mining	2
21	Barrow Compensation	1863	1968	12	115	PCC	PC	Leakage of abandoned dam that filled in flood	2
22	Toddbrook (1977)	1840	1977	24	1,288	Hom	?	Internal erosion of upstream fill	2
23	Oakdale Lower	1890	1986	11	20	PCC	?	Sinkhole on crest caused by internal erosion	2
24	Luxhay	1905	1994	19	544	PCC	PC	Leakage through upper part of core	2
25	Kellington East		2008	3		Hom	?	Breach caused by internal erosion associated with animal burrows	1
Group 3 - Internal erosion or leakage in-service associated with ancillary works/cut-offs/abutments									
26	Llŵ Lower	1867	1873	27	1,300	PCC	PC	Internal erosion associated with spring in deep clay filled cut-off	2
27	Pentwyn	1863	1882	10		PCC	PC	Internal erosion associated with spring in deep clay filled cut-off	2
28	Roddesworth Upper	1865	1904	21	739	PCC	PC	Internal erosion associated with spring in deep clay filled cut-off	2
29	Bottoms (Macclesfield)	1850	1929	9	180	PCC	PC	Downstream slip associated with leakage over core and damaged pipes.	2
30	Holden Wood	1841	1945	17	367	PCC	?	Internal erosion associated with culvert through the core	2
31	Island Barn	1911	1950	9	4,190	PCC	PC	Internal erosion associated with tree roots	3
32	Lluest Wen	1896	1969	24	1,096	PCC	PC	Internal erosion into drawoff tunnel	2
33	Slade Lower	1889	1970	15	150	PCC	?	Internal erosion of core close to spillway	2
34	Warmwithens	1870	1970	10	102	PCC	?	Internal erosion along outside of recently constructed draw-off tunnel	1
35	Greenbooth	1962	1983	35	3,182	PCC	C	Internal erosion associated with hydraulic fracture close to abutments	2
36	Holmestyes	1840	1993	24	314	PCC+USCB	PC	Internal erosion into the valve shaft	2
37	Walshaw Dean Upper	1907	1997	24	213	PCC	PC	Sinkhole thought to be caused by internal erosion	3
38	Anglezarke (Heapey)	1857	1997	14	4,169	PCC	PC	Leakage associated with abutment	2
39	Rivington Upper (Yarrow)	1857	2002	12	1,090	PCC	PC	Internal erosion into draw-off culvert	2
40	Carno Lower	1911	2005	27	800	PCC	PC	Internal erosion associated with draw-off culvert	2
Group 4 - Incidents due to pipe or valve failure									
41	Knyppersley	1827	1828	16	930	PCC		Pipework passing through core pulled apart	2
42	Torside	1851	1854	26	6,700	PCC	PC	Draw-off pipes fractured and pulled apart	2
43	Longwood Compensation	1828	1988	13	182	Hom	?	Fractured discharge pipe caused saturation of downstream fill	2
44	Ogston	1959	2001	20	6,180	PCC	C	Valve and pipe failure in draw-off shaft caused by fitting an inappropriate valve	1

Notes: Highlighted incidents are described in section 5.2
 All dams are embankments
 Cut-off type: PC = puddle clay, C = concrete, G = grout curtain

List 2b. Incidents at British dams categorised by nature of incident and incident year - Incident groups 5, 6 and 7

Incident No.	Dam name	Date Built	Incident Year	Height m	Res Vol 10 ³ x m ³	Watertight element	Nature of failure/incident	Incident severity
Group 5 - Slope instability during construction								
45	Spade Mill No.1	1862	1860	11	900	PCC	Shallow slips during construction	3
46	Alston no 1	1932	1927	13	1,177	PCC	Deep seated slope instability during construction	3
47	Bartley	1930	1927	20	2,400	PCC	Upstream slope instability during construction	3
48	Abberton	1940	1937	16	25,721	PCC	Deep seated slope instability due to rapid construction	2
49	William Girling (Chingford)	1937	1937	10	13,500	PCC	Major deep seated downstream slip due to weak foundation and rapid construction	2
50	Muirhead	1942	1941	21	3,572	PCC	Deep seated slip due to rapid construction	3
51	Usk	1955	1953	33	12,253	PCC	Potential for slip during construction due to high pore water pressures	3
52	Tittesworth	1962	1960	30	6,400	PCC	Slope instability of old dam due to partial removal of toe during construction of new dam	2
53	Carsington	1984	1984	35	35,000	RCC	Major deep seated upstream slip during construction	2
Group 6 - Slope instability in-service								
54	Earlsburn		1839	6		Hom	Breach associated with earthquake	1
55	Harlow Hill	1871	1951	9	65	PCC	Deep seated downstream slip triggered by rain	2
56	Roddlesworth Upper	1865	1954	21	739	PCC	Shallow downstream slip associated with heavy rain and runoff from valley side	2
57	Auchendores	1922	1968	10	857	?	Downstream surface slip caused by overtopping waves and spray	3
58	Buckieburn	1905	1970	23	875	Hom	Substantial shallow downstream slip triggered by heavy rain and high winds	2
59	Aldenham	1795	1975	8	78	Hom	Downstream surface slip triggered by tree removal	3
60	Combs	1805	1976	16	1,484	Hom	Major shallow downstream slip due to wave and spray action	2
61	Lambieatham	1899	1984	15	54	PCC	Shallow slip associated with heavy rain and runoff from valley side	2
Group 7 - External erosion due to flood flow								
62	Tunnel End	1798	1799	9		Hom	Breached due to overtopping during flood with loss of life	1
63	Diggle Moss (Black Moss)	1810	1810			Hom	Breached during flood with loss of life	1
64	Brent (Welsh Harp)	1837	1841	7		PCC	Breached due to overtopping during flood with loss of life	1
65	Bold Venture (Darwen)	1844	1848	10	20	Hom	Breached during flood with loss of life	1
66	Woodhead 1	1849	1849	16		PCC	Breached during construction due to overtopping during flood	1
67	Cwm Carne	1792	1875	12	90	PCC	Breached due to overtopping during flood following years of settlement and neglect with loss of life	1
68	Cowlyd	1921	1924	14	9,430	CC	Dam overtopped during flood and nearly failed	2
69	Skelmorlie(1861)	1861	1925	5	24	Hom	Flood flow, overtopped and breached with loss of life	1
70	Dunblane	1933	1943	12		?	Dam overtopped during flood causing some erosion	2
71	Bilberry	1853	1944	16	105	USCB	External erosion of downstream shoulder - flood from hillside	2
72	Tumbleton Lake	1885	1945	11	100	Hom	Dam overtopped during flood causing some erosion	2
73	Doxford Lake	1910	1948	2	29	Hom	Dam overtopped but no serious damage occurred	3
74	Thorters	1900	1949	15	286	Hom	Dam overtopped during flood	2
75	Trewitt Lake		1963	5	80	Hom	Dam breached due to overtopping due to snow melt	1
76	Toddbrook	1840	1964	24	1,288	Hom	Significant damage to spillway and erosion adjacent to spillway during flood, but dam not overtopped	2
77	Chew Magna	1850	1968	12	114	PCC	Serious damage to spillway stilling basin during flood flow	2
78	Corsham Lake		1968	3	80	?	Embankment overtopped causing significant erosion	2
79	Kype	1898	1977	17	670	?	Significant erosion adjacent to spillway during flood	2
80	Walshaw Dean Lower	1907	1989	24	727	PCC	Damage to spillway during flood	3
81	Boltby	1880	2005	19	130	PCC	Serious damage to spillway and erosion of downstream toe during flood	2
82	Ulley	1873	2007	16	582	PCC	Serious damage to spillway and erosion of downstream embankment during flood	2
83	Maich Water		2008		20	PCC	Overtopping of dam and severe erosion of downstream fill during flood	2

Notes: Highlighted incidents are described in section 5.2
 All dams are embankments ; All reservoirs were empty for Group 5 incidents
 Watertight element: PCC = puddle clay core, CC = concrete core, USCB = upstream clay blanket, Hom = earthfill no core,

List 2c. Incidents categorised by nature of incident and incident year - Incident groups 8, 9, 10, 11, and 12

Incident No.	Dam name	Date Built	Incident Year	Height m	Res Vol 10 ³ x m ³	Dam type - watertight element	Nature of failure or incident	Incident severity
Group 8 - Wave damage to upstream protection								
84	Blithfield	1953	1962	16	18,172	Emb - PCC	Wave damage to upstream protection and shallow downstream slip	2
85	Megget	1982	1984	56	61,400	Emb - CAC	Wave damage to upstream pitching	3
86	Kielder	1982	1984	55	200,000	Emb - RCC	Wave damage to upstream concrete blockwork	2
87	Bewl Bridge	1975	1987	31	31,400	Emb - RCC	Wave damage to concrete slabbing	2
Group 9 - Reservoir basin leakage								
88	Ainsworth Mill Lodge	1860	1860		20	Emb	Leakage into and flooding of disused mine workings	2
89	Colt Crag	1884	1888	17	4,850	Emb - PCC	Basin leakage due to sink holes	3
90	Mill Hill	1939	1979	6	110	CS	Collapse of concrete service reservoir associated with mining activity	3
91	Pen-y-Rheol	1863	1985	10		Emb - PCC	Collapse of mine workings caused sinkholes in reservoir basin	3
92	Larksheath	1995	1996		325	Emb - HDPE	Failure of HDPE membrane laid over sinkholes in foundation	3
Group 10 - Concrete and masonry dams								
93	Blackwater	1907	1909	25	111,300	CG	Thermal cracking of concrete dam	3
94a	Eigau (Dolgarrog)	1911	1925	10	4,500	CG	Breached due to inadequate foundation	1
94b	Coedy (Dogarrog)	1924	1925	11	320	Emb -CC	Breached due to flood flow from failure of Eigiau	1
95	Glendevon Upper	1955	1954	45	5,040	CG	Leaks and concerns about stability of structure since first filling	3
96	Blackbrook (1957)	1906	1957	22	2,300	CG	Earthquake damage of concrete dam	3
97	Mullardoch	1951	1986	37	223,000	CG	Leakage increase, cracks and concerns about stability	2
98	Val de la Mare	1962	1971	29		CG	Cracking of dam due to alkali silica reaction	3
Group 11 - Other incidents								
99	Dundreggan	1957	1998	16	1,640	CG	Damage to spillway gate due to vibration	3
100	Beggars Hall Lake		1999			Hom	Plane crash on to small embankment	2
Group 12 - Overseas incidents								
101	Malpasset (France)	1954	1959	61	22,000	CA	Catastrophic failure of thin arch dam	1
102	Vaoint (Italy)	1960	1963	265	150,000	CA	Massive landslide into reservoir caused overtopping of dam and extensive loss of life	1
103	Folsom (USA)	1956	1995	103	1,205,129	CG	Failure of flood gate	1
104	Taum Sauk (USA)	1962	2005	16	5,366	Emb	Catastrophic failure due to overtopping by pumping	1

Notes: Highlighted incidents are described in section 5.2

Dam type: Emb = embankment, CG = concrete gravity, CA = Concrete arch, CS = Concrete service

Watertight element: PCC = puddle clay core, CC = concrete core, CAC = central asphaltic core, RCC = rolled clay core, Hom = earthfill no core, HDPE = upstream/basin lining

5.2 Description of major incidents

5.2.1 Group 1: Internal erosion on first filling

3. Bilberry

Incident date: 5 February 1852

Description of dam

Height	29 m	Dam type	Earthfill
Reservoir capacity	310 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1845	Cut-off	Puddle clay

Construction of the dam started in 1839 and was beset with problems. The reservoir owners, the Holme Reservoirs Commissioners, did not wish to incur the expense of frequent site visits by George Leather, the engineer who designed the dam, and supervision of construction was inadequate. The dam was 29 m high but the outlet was only 20 m below the crest. The dam was founded on alternating beds of sandstone and mudstone of the Millstone Grit Series which was described as open and pervious. The puddle clay core was 2.4 m wide at the top and 4.9 m at ground level. A “*very considerable spring*” was encountered at the bottom of the cut-off trench which was not sealed. The puddle at the base was described as “*rather slush than puddle*”.

Outlet works comprised a masonry culvert which crossed the puddle trench. Serious problems were soon apparent. Muddy water came through the culvert in 1841. In 1843 the leak became worse and water burst through the culvert. Remedial works were unsuccessful. Leather, and later J F Bateman, proposed lining the upstream face with puddle and keying it into a trench at the toe, but the owners did not agree to the work. This approach was used at Holmestyes dam, one of the other dams designed by George Leather, following the failure of Bilberry.

Large sinkholes appeared on the crest of the embankment. The embankment settled three metres such that the original freeboard of 2.4 m had reduced to 0.6 m below the overflow weir. An order was made to make an opening in the waste pit in 1846 but this was not done. Because of the leaks and settlement, it was thought necessary “*of never, usually allowing the water to rise to above half its intended height, so that there was always a considerable vacant space in the reservoir to contain any flood water beyond what the outlet-pipe would allow to escape, before it could overtop the bank*”.

Six years before the failure, a flood nearly overtopped the dam and overflow weir.

Incident description

The dam was overtopped after heavy rainfall and breached following the long period of leakage and settlement caused by internal erosion. Collapse of the dam at 01:00 on 5 February 1852 caused 81 deaths and much property damage in the Holmfirth area. The reservoir emptied in 30 minutes. The unsafe state of the dam was recognised by those living closest to it and some escaped the flood which engulfed the inhabitants of Holmfirth lower down the valley.

Lessons

This was the first dam failure in the UK to cause significant loss of life, thereby drawing national attention to reservoir safety. The Home Secretary arranged for Captain R C

Moody of the Royal Engineers to inspect the remains of the dam and give expert evidence at the inquest.

Technical lessons

Moody (1852) drew attention to the failure to properly control fill placement and to ensure that the more cohesive fill was placed next to the puddle clay core, with the more granular fill in the outer slopes. He remarked that a sinkhole in the crest was located above the culvert *“and is no doubt due to the washing away of the bad puddling over and above the culvert where it passes through the puddle wall below”*. Thus, attention was drawn to the dangers posed by culverts or pipes passing through the embankment fill. The circumstances of the failure were discussed by prominent Victorian dam engineers (Leslie, 1852; Rawlinson, 1859; 1879) and by twentieth century experts on embankment dams (Binnie, 1981; Skempton, 1989).

Legislative lessons

A private act for the reconstruction of the reservoir contained some provisions relating to safety, including the appointment of J F Bateman as the engineer. At the inquest, the verdict of the jury on 27 February 1852 contained strong criticism of the Holme Reservoirs Commissioners, and pointed to the need for legislation: *“... and we regret that the reservoir, being under the management of a corporation, prevents us bringing in a verdict of manslaughter, as we are convinced that the gross and culpable negligence of the commissioners would have subjected them to such a verdict had they been in the position of a private individual or firm. We also hope that the legislature will take into its most serious consideration the propriety of making provision for the protection of the lives and properties of Her Majesty's subjects exposed to danger from reservoirs placed by corporations in situations similar to those under the charge of the Holme Reservoir Commissioners.”*

Remedial works

A new dam was built by Bateman on the same site, but with the centre-line nine metres upstream of the old one and with an upstream puddle clay blanket.

6. Dale Dyke

Incident date: 11 March 1864

Description of dam

Height	29 m	Dam type	Earthfill
Reservoir capacity	3,240 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1863	Cut-off	Puddle clay

The dam was engineered by John Towlerton Leather for the Sheffield Waterworks Company. The upstream and downstream slopes were 1:2.5. The maximum depth of the puddle clay filled trench was 47 m below the crest. The top width of the core was 1.2 m and, with batters on both faces of 1:16, it had a maximum width at ground level of 4.9 m. The fill on either side of the core was open and permeable. Two unprotected 18 inch (0.46 m) diameter cast iron pipes ran diagonally from a chamber at the foot of the upstream face in a trench three metres by three metres beneath the embankment. To prevent fracturing, the trench ran downwards 30 m either side of the core to intersect the bottom of the core. Sections of the dam are shown in Figure 5.4.

The cut-off was finished in 1861 and the embankment completed in 1863. By 10 March 1864 first filling was almost complete, with the water level 0.7 m below the crest of the weir.

Incident description

In the late afternoon of 11 March 1864 a crack was observed along the downstream slope near the crest of the dam. In the evening, increasing concern over the state of the dam led to an attempt to lower the reservoir level by blasting a gap in the wall of the bywash channel, but before this was achieved the dam collapsed at 23:30 and the dam breached. The extent of the breach is shown in Figure 5.4. The reservoir was far larger than that at Bilberry, and the catastrophe claimed 244 lives in the vicinity of Sheffield. Several accounts of the failure have been published including Harrison (1864), Amey (1974) and Binnie (1981).

Lessons

This is the most serious dam failure in Great Britain and the country's worst civil engineering disaster. The Home Secretary appointed two civil engineers, Robert Rawlinson and Nathaniel Beardmore, to assist in the enquiry into the cause of the catastrophe. In their report to the Home Secretary, Rawlinson and Beardmore were critical of both the design and construction of the dam. They believed that failure was most likely to have been caused by leakage from a fractured outlet pipe, but the design and construction of the embankment was also criticised: *"the puddle-wall is much too thin, and the material placed on either side of it is of too porous a character...No puddle-wall should ever be placed betwixt masses of porous earth, as puddle, under such conditions, will crack, and is also liable to be fractured by pressure of water."* Modern commentators have stated that the outlet pipes were found to be undamaged at the time of the replacement dam construction when fill was won from the old dam, exposing the twin outlet pipes. Large vertical steps in the longitudinal profile might have led to differential settlement and hydraulic fracture of the core. It is possible that hydraulic fracture of the thin core caused the failure.

Sheffield Waterworks Company retained leading engineers, including Hawksley, Bateman, and Simpson. Contrary to the findings of Rawlinson and Beardmore, these men considered that the dam collapsed as a result of a landslide and was an unavoidable accident. Sheffield Corporation engaged nine engineers, including Sir John Rennie and James Leslie, to report individually on the failure and they agreed with Rawlinson and Beardmore that there had been faulty construction. The failure mechanism has continued to be disputed (Binnie 1978; 1981).

Legislative lessons

The verdict of the jury on 24 March 1864 included the statement: *"that, in our opinion, there has not been that engineering skill and that attention to the construction of the works, which their magnitude and importance demanded; that, in our opinion, the Legislature ought to take such action as will result in a governmental inspection of all works of this character; and, that such inspections should be frequent, sufficient, and regular;"*

However, in their report to the Home Secretary on the failure, Rawlinson and Beardmore (1864) were critical of the design and construction of the dam but did not endorse the recommendation from the jury for government inspection.

Remedial works

The dam was rebuilt in 1875 about 300 metres upstream of the failed embankment.

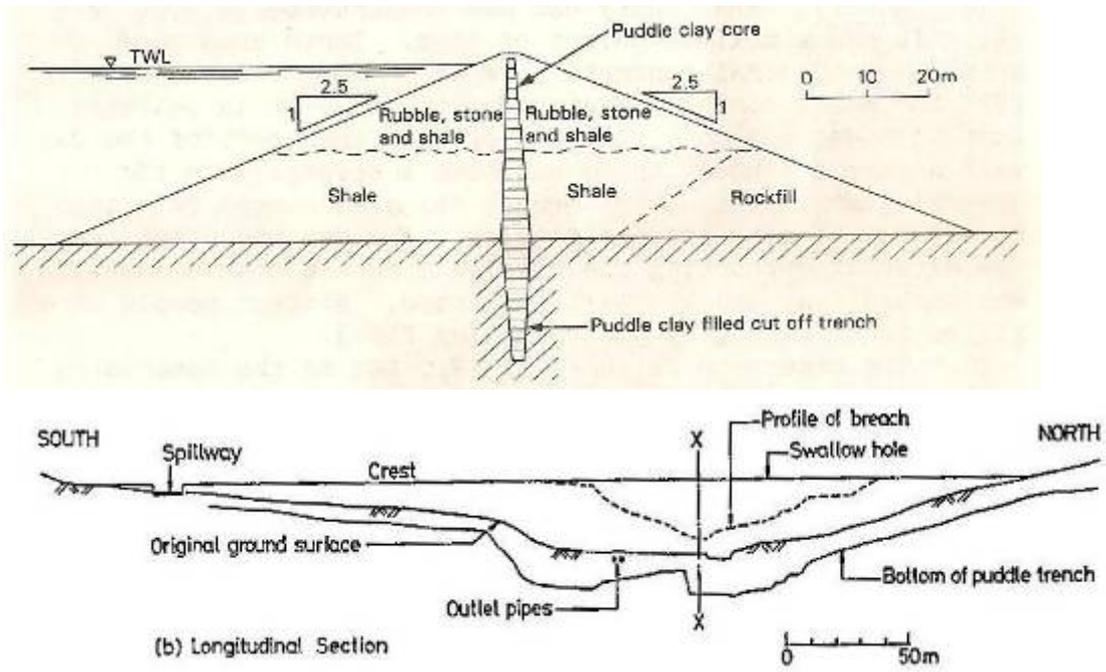


Figure 5-4 Cross-section and longitudinal section of Dale Dyke (after Skempton, 1989)

13. Balderhead

Incident date: March 1967

Description of dam

Height	48 m	Dam type	Earthfill
Reservoir capacity	19,684 x 10 ³ m ³	Watertight element	Rolled clay
Completed	1964	Cut-off	Concrete

The reservoir was constructed on a tributary of the River Tees in County Durham (Kennard, 1964). The dam has a thin, central rolled clay core of boulder clay, relatively stiff shale fill shoulders and a concrete cut-off. The top 10.8 m of the clay core has vertical sides and is 6.1 m wide. Immediately downstream of the core is a crushed limestone filter, 1.5 m wide which connects with the ground drainage blanket. The filter and drainage blanket were designed according to standard filter rules based on particle size distributions. A section of the dam is shown in Figure 5.5.

Incident description

First impounding occurred between October 1964 and February 1966 and, just before the reservoir was full, the main underdrain flow increased. At the end of January 1967 localised settlements occurred along the crest, and in April 1967 a sinkhole about three metres wide and 2.5 m deep developed over the upstream boundary of the crest. The reservoir was immediately drawn down by 9.2 m which reduced the underdrainage and the flow became clear. During the drawdown a second hole appeared. It was established that the main underdrain flow had turned cloudy about a month before the first sinkhole appeared.

Investigations

Exploratory boreholes revealed severe erosion within the core at several locations; the boulder clay material had become segregated and the finer particles lost by water erosion. It was concluded that the leak was associated with hydraulic fracture of the core caused by arching between the relatively stiff shoulders and the narrow clay core. The granular limestone filter would have been particularly incompressible compared to the clay core. It was also postulated that once the cracks had formed they were kept open by the water pressure and under the low flow conditions the coarser eroded material had segregated in the cracks. On drawdown the seepage paths closed up due to the decrease in water pressure.

Remedial works

The dam was repaired by 1968, three years after its initial completion. Over the central 200 m of the dam, covering the zones of worst erosion damage, the core was repaired by constructing a 0.6 m thick diaphragm wall of plastic concrete in six-metre long panels down to the concrete cut-off (Little, 1974; Vaughan *et al.*, 1970). The work was carried out with the reservoir level lowered by 10 m. The plastic concrete mix was designed to have a strength and stiffness similar to that of the rolled boulder clay core and be resistant to erosion. The permeability of concrete was in the range 1×10^{-8} to 5×10^{-10} m/s, the intact core being about 1×10^{-10} m/s. As well as providing an additional water barrier, the diaphragm wall should prevent migration of eroded material through the core.

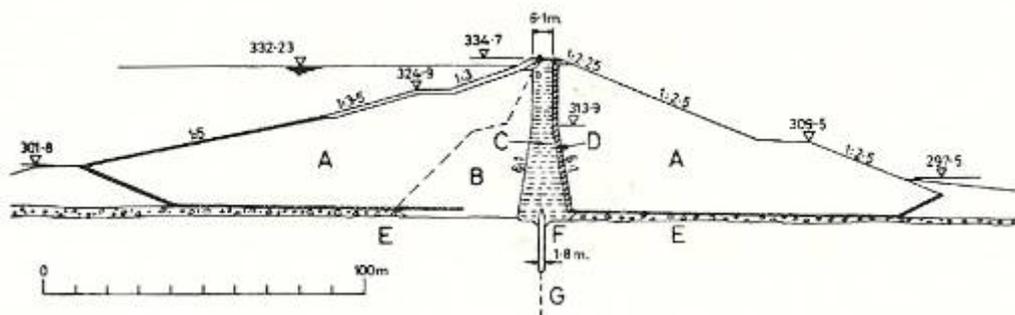
Other lengths of the core where water losses were observed during investigative drilling were grouted using tubes-à-manchette in order to increase total earth pressures within the core and thereby resist hydraulic fracture. Grout tubes were installed at three-metre centres. The core was grouted from the bottom upwards, injections being carried out from alternate sleeves and limited to 0.42 m³ per injection. The grout consisted of half cement and half bentonite with a water/solids ratio of four. The grout was designed, when set, to be slightly less erodible than the boulder clay core material and to have stress-strain characteristics approximating to those of the boulder clay.

When the reservoir had been first filled, leakage had increased to 60 l/s, but on refilling after repair of the core, the leakage was less than 10 l/s.

Lessons

It was concluded that the leakage at Balderhead was initiated by hydraulic fracture. The narrow width of the core may have been a contributory factor leading to hydraulic fracture. The maximum hydraulic gradient across the core was approximately three, which is less than at other rolled clay core dams.

Failure of the filter to retain the finer particles at Balderhead led to intensive research on filter design by Professor Peter Vaughan at Imperial College London. This led to development of a method of filter design for clays in flocculating conditions that could retain all eroded material, to be come known as the “perfect filter method” (Vaughan and Soares, 1982). The ability of a filter to arrest a fine particle was directly related to its permeability. The method was used for the filter design of Cow Green dam (Vaughan *et al.*, 1975).



A – Shale fill; B – Fine shale fill; C – Boulder clay core; D – Crushed limestone filter; E – Shale foundation; F – Concrete cut-off; G – Grout curtain.

Figure 5-5 Cross-section of Balderhead dam (after Vaughan *et al.*, 1970)

14. Winscar

Incident date: 1976-1980

Description of dam

Height	53 m	Dam type	Rockfill
Reservoir capacity	8,296 x 10 ³ m ³	Watertight element	Upstream asphaltic membrane
Completed	1975	Cut-off	Grout curtain

The dam is located in the Pennines, 25 miles north west of Sheffield (Collins and Humphreys, 1974). It was the second largest dam in Britain to have an upstream facing of asphaltic concrete although many had been built in Europe and elsewhere prior to Winscar. Two layers of DAC (dense asphaltic concrete) were used. Only four dams with upstream facing of asphaltic concrete have been built in Britain since Winscar and each has only had one layer of DAC. The dam is founded on jointed sandstone, sandy shales and mudstone-laminated sandstone of the Carboniferous Millstone Grit Series. A grout curtain continues beneath the upstream toe to depths of up to 70 m. A permeable sandstone formation outcrops in the valley floor. The dam is made of compacted sandstone rockfill.

Incident description

Although this incident mainly concerned the performance of the asphaltic lining, leaks through the abutments contributed to flows in the drains. A pattern of rising seepage was recorded during first filling, with flows disproportionately high above a certain reservoir level. Leakage through the left abutment resulted in two stages of grouting in January 1978 and 1979 with the aim of reducing permeability at the contact between two rock strata. A third phase of grouting in 1980 involved emptying the reservoir, which led to the discovery of a series of cracks in the asphaltic concrete in the vicinity of the toe wall and concrete collar surrounding the draw-off culvert. One crack penetrated the full depth of the lining and the hole was “no bigger than a match box”. Fluorescence tests confirmed the connection to the drainage system. Damage had occurred from differential settlement between the rigid concrete collar and culvert, both of which were founded on rock, and the adjacent poorly compacted and inadequately graded rockfill.

Remedial works

Repair involved removal and replacement of around 12 m² of lining and the introduction of a flexible joint between the concrete collar and asphaltic lining using copper sheets (Routh, 1988).

Longer term performance of the lining

The formation of blisters and debonding of the two layers of asphalt occurred over the following years. During late 2000, leakage increased and the appearance of a large spring at the downstream toe in January 2001 with a flow of 15 l/s led to a precautionary reservoir drawdown: leakage flow reduced at half-depth of the reservoir. Defects detected during the 1996 inspection included 20 cracks within the upper layer of DAC varying in length up to about one metre, eight blisters which also had cracks, and debonding between the upper and lower layers of membrane (Wilson and Robertshaw, 1998). Inspection of the membrane in 2001 revealed 60 new defects, with several large cracks at the base of the dam, one of which had opened into a hole through the membrane. In December 2002, a Carpi composite PVC/fabric membrane liner was constructed over the DAC (Carter *et al.*, 2002).

Lessons

It was always recognised that connections between the lining and toe wall and culvert entry to the embankment were areas where large differential settlement between the concrete and rockfill could occur. Concerns at Roadford dam about the differential settlement between the upstream inspection gallery and the relatively compressible low grade rockfill led to the introduction of a wedge of low compressible sandwaste (Evans and Wilson, 1992).

The long-term blistering problems at Winscar have not occurred to the same extent at other British dams with asphaltic linings, although regular maintenance has been required.

5.2.2 Group 2: Internal erosion or leakage in service

19. King George V (Chingford No1)

Incident date: February 1945

Description of dam

Height	8 m	Dam type	Earthfill
Reservoir capacity	12,400 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1912	Cut-off	Puddle clay

The reservoir is one of the larger reservoirs in the Lee Valley. The bunded embankment is 4.5 miles long and was built on unstripped grass and surface soil which overlies a bed of soft clay, which in turn overlies the Lee Valley gravel. A 1.8 m wide clay-filled cut-off trench through the gravel was secured into the London clay some six metres below. The shoulder fill consists largely of surface clay, including topsoil, with gravel and peat. No underdrainage was provided to the outer slope rendering it more susceptible to saturation and instability in the event of leakage through the core. The core comprises very high plasticity alluvial clay. Settlement, as much as 0.9 m in places, led to the core being raised in 1939-40 to restore the freeboard.

Incident description

In September 1939, it was decided to reduce the top water level by 1.5 m (three metres below crest level) as a precautionary measure in case of damage to the embankment by enemy action during the Second World War. On refilling the reservoir in February 1945, after the prolonged drawdown, water appeared at the toe of the embankment to such an extent that its stability was questioned. Leakage virtually ceased when the reservoir was drawn down by 0.8 m. As no water had appeared on the slope, it might have indicated that water was passing under the embankment with the possibility of piping and uplift in the underlying gravel.

Trial pits at the top of the embankment showed that the upper part of the clay core, except for the portion placed in 1939-40, was in a cracked and deteriorated state which was associated with drying out and root formations. Drying had occurred down to a depth of three metres, which had been aggravated by the exposed nature of the embankment and plant roots driven down by the five years of lowered reservoir level. The trial pits showed that water was passing through the puddle clay core just below the reservoir level. The investigation into the properties of the puddle clay is described by Bishop (1946) and summarised by Skempton (1989).

Lessons

Prolonged reservoir drawdown over a number of years, even by a small amount, can lead to cracking of the upper part of a puddle clay core, if it consists of high plasticity clay. Refilling or raising the water level of a reservoir after prolonged drawdown should be done with caution.

Drying and cracking of the upper part of the puddle clay core at other dams in the Lee Valley, including Lockwood and Banbury, occurred as a result of prolonged reservoir drawdown during the Second World War. Ray and Bulmer (1982) described various types of remedial works to reduce the leakage.

24. Luxhay

Incident date: April 1994

Description of dam

Height	19 m	Dam type	Earthfill
Reservoir capacity	544 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1905	Cut-off	Puddle clay

The dam was built for the public water supply of Taunton. Construction drawings show the dam to have a puddle clay core with selected material adjacent to the core. The upstream slope is 1:3 and the downstream slope 1:2.5.

Incident description

Since 1969, the dam has had a history of leakage at the downstream toe associated with high reservoir water levels. Extensive investigation and monitoring had raised concerns about the stability of the lower part of the embankment, with flows from the

toe drain and water level in the downstream fill responding to reservoir level. On a routine visit by the dam owner's staff in April 1994, the sound of running water was heard and the development of new wet areas instigated an instruction to quickly lower the reservoir level by one metre. When the reservoir level was reduced by 0.5 m, the sound of running water ceased and wet areas at the toe dried up.

Investigations prior to the incident

Extensive borehole investigations and laboratory testing had been done prior to the incident. After the 1977 inspection, some 30 shallow observation wells were installed in the downstream shoulder to determine the location of the phreatic surface. Concern was expressed about the lower third of the embankment, resulting in shallow French drains being installed.

A further 15 boreholes with 22 piezometers were installed in 1992. No distinction in material type was identified between the selected fill and outer fill. Permeability measurements indicated a satisfactory core with values in the range 2×10^{-8} to 1×10^{-9} m/s. In the downstream shoulder, measurements indicated a clayey fill with permeability ranging from 1×10^{-7} to 1×10^{-10} m/s.

Investigations after the incident

Following the incident, trial pits were excavated into the core to just below water level but no evidence of seepage was found. It was concluded from the abrupt way in which the leakage flow ceased when lowering the reservoir by a small amount, that overtopping of the core was the main cause of the leak. As the precise location of the leak could not be found without extensive investigation, it was decided to provide a watertight seal at the top of the core to a depth of four metres along the complete length of the dam.

Remedial works

A single phase, self-hardening cement bentonite slurry cut-off wall four metres deep was chosen to make the top of the dam watertight. Despite concerns about the relatively high strength and stiffness of the set slurry, the remedial works using this technique appear to be effective. Details of the construction of the cut-off wall are described by Millmore *et al.* (1998). The use of slurry trench walls to repair clay cores is summarised by Tedd and Jefferis (2000).

Lessons

The importance of frequent supervision by staff is demonstrated by this incident. The occurrence of leakage only when a reservoir is near top water level is a common feature of leakage incidents at many British dams. The limited investigation undertaken following the leakage in 1994 did not identify any definite locations of the leaks. Use of the continuous slurry trench method ensures all potential leakage paths can be sealed.

A shallow wall using typical self-hardening cement bentonite slurry is perhaps not to be recommended in the upper part of a core where an embankment is likely to undergo large differential settlement, because the set slurry is brittle and susceptible to cracking at low strains.

5.2.3 Group 3: Internal erosion or leakage in service associated with ancillary works/cut-offs/abutments

26. Lliw Lower

Incident dates: spring 1873 and January 1883

Description of dam

Height	27 m	Dam type	Earthfill
Reservoir capacity	1,300 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1867	Cut-off	Puddle clay

The reservoir is situated north of Swansea and was one of the highest earth dams in Britain at the time of construction. The dam had a narrow central core of puddle clay with a shallow cut-off trench through glacial and alluvial deposits into the bedrock. The hazards of the springs which emerged from joints in the rock when constructing the cut-off trench were recognised at the time. Hydraulic lime concrete was placed in the bottom of the trench to seal the contact zone and a drainage system was installed on the downstream side of the core to collect water and reduce water pressure under the downstream shoulder. The dam was designed by Robert Rawlinson, the government inspector of the Dale Dyke failure.

Incident description

When the reservoir was filled, leakage was found to vary between 1.4 and 2.9 l/s depending on rainfall, but the leakage rate was not affected by reservoir level. In the spring of 1873, turbid water flowed from the downstream drains at a rate of 26 l/s and the leakage water became turbid. A spring had burst through the fissured rock below the puddle clay core and erosion of the puddle clay caused visible settlement of the embankment by January 1874 (Binnie, 1981; Howe, 1977).

Remedial works

Remedial work started in 1879 and involved an open cutting 50 m wide at the top and 15 m wide at the bottom to a depth of 11 m below the top of the embankment and a trench nine metres long and six metres wide sunk from the bottom of the cutting to the rock, a total depth of 32 m below the top of the embankment. At a depth of seven metres in the trench (18 m below crest), a fissure 0.6 m wide was found in the puddle clay, filled with the coarse material of the chosen fill. The fissure extended down to the face of the rock. A drain was installed to remove water from the spring which acted on the clay at the bottom of the trench. The bottom of the shaft was sealed with Portland cement concrete and the core was repaired with puddle clay.

In 1883, turbid water came from the drains and settlement occurred at the location of the remedial works. The reservoir level was then reduced by five metres for the rest of its working life until the dam was replaced in 1978.

Lessons

In his report to government on the Dale Dyke failure, Rawlinson had been critical of the design and construction of that dam. Now, just a few years later, he was faced with a serious incident at a dam he had designed. Undoubtedly, it shook his confidence in the reliability of traditional puddle clay embankment dams and he agreed that *“any trench dug in rock should, by use of concrete, be put beyond the possibility of water finding vent in the trench”* (Rawlinson, 1883).

Over the following years there was a move away from puddle clay filled cut-off trenches to concrete filled trenches, although cut-off trenches continued to be filled with puddle clay into the early 1900s.

32. Lluest Wen

Incident date: 23 December 1969

Description of dam

Height	24 m	Dam type	Earthfill
Reservoir capacity	1,096 x 10 ³ m ³	Watertight element	Watertight element
Completed	1896	Cut-off	Deep puddle clay

The reservoir is at the head of the Little Rhonda Valley in South Wales. The dam has a puddle clay core and cut-off and boulder clay fill shoulders.

Subsidence had previously occurred in 1912 and 50 tonnes of cement grout had been injected in the area of the valve shaft in 1915-16.

Incident description

On 23 December 1969, a man was riding on horseback across the dam and it is reported that both horse and rider fell into a two-metre deep hole on the upstream side of the puddle clay core close to the valve tower.

On 9 January 1970, an emergency drawdown of the reservoir was deemed necessary (Gamblin and Little, 1970). On 12 January it was decided that people living downstream should be evacuated. An emergency was declared by the Secretary of State for Wales and old and infirm people living in vulnerable areas downstream were evacuated. A colliery in the valley which would have been flooded was temporarily closed. The 0.38 m diameter draw-off pipe was inadequate for rapidly lowering the reservoir and a large number of pumps, some of which were positioned by helicopter, were used. An emergency grouting programme was arranged which involved 18 tonnes of clay/cement grout being injected into a single hole close to the shaft where the sink hole had developed. An emergency cut was made in the spillway, lowering the overflow level by nine metres by 29 January. A certificate for the conditional safety of the reservoir was then issued.

Investigations and remedial works

With the emergency over, grouting of the core was undertaken involving 50 tonnes of clay cement grout. A subsequent borehole investigation found the puddle clay to be soft to very soft with pockets of silt or sand (Little, 1977; Twort, 1977). Many open fissures, iron-stained by the passage of seepage water, were also present. The core was very soft in the vicinity of the valve shaft. In view of these findings, it was decided that grouting alone could not be relied upon and a 0.6 m thick plastic concrete diaphragm wall was constructed. The diaphragm wall was built through the core over the full length of the dam in 4.8-m panels. The wall penetrated the bedrock by between one and four metres, the maximum depth being 34.8 m. Falling head tests on drill holes into the wall gave a permeability value of 1×10^{-8} m/s.

After the remedial works, the main drains to the downstream toe showed very low flows during dry weather with the reservoir full. Piezometers in the downstream fill showed little response to the filling of the reservoir.

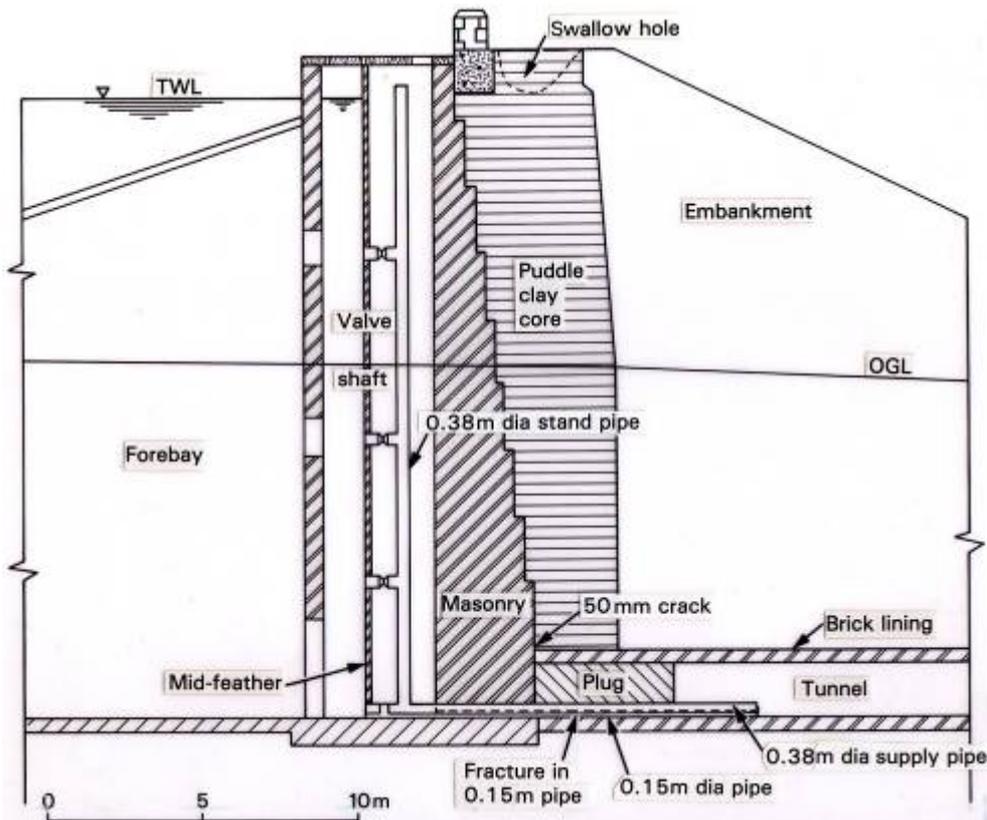


Figure 5-6 Section through the valve shaft at Llest Wen dam (after Little 1977)

It was established that reservoir water pressure had extruded puddle clay from the core into the outlet tunnel, firstly through a 5-10 mm wide gap at the junction between the downstream wall of the valve shaft and the tunnel lining, and then via a 150-mm drainage pipe leading from the base of the valve shaft through the tunnel plug, the pipe having parted and fractured at the junction. At the time there was a 0.06 m³ pile of puddle clay at the end of the 150-mm pipe.

Lessons

The existence of a hole large enough to accommodate a horse was only revealed when the ground gave way under the weight of the horse as it was ridden along the crest of the embankment. Had the surface of the crest been tarmac or concrete, a much larger hole might have formed before it manifested itself.

The extreme seriousness with which the incident was viewed and the emergency measures put in place by the Welsh authorities were undoubtedly influenced by the Aberfan disaster that occurred three years earlier in 1966.

The Secretary of State for Wales, Mr George Thomas, ordered an urgent investigation of all old reservoirs in Wales.

34. Warmwithens

Incident date: 24 November 1970

Description of dam

Height	10 m	Dam type	Earthfill
Reservoir capacity	102 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1870		

Warmwithens reservoir near Oswaldtwistle, Lancashire was built to supply a local textile mill near Accrington. The embankment, which impounded the uppermost reservoir in a chain of three reservoirs, was described as “*consisting of clay filling on the upstream side of the centre and there is some evidence there having been some form of clay core*”. The draw-off consisted of a 250-mm diameter outlet cast iron pipe laid beneath the embankment which was controlled by a valve at the downstream end. Between 1964 and 1966, a 1.5-m diameter tunnel lined with pre-cast concrete segments was driven through the embankment to contain new outlet pipes. The original outlet pipe was plugged at the upstream end and filled with cement/bentonite grout from the lower end (Moffat, 1975).

Incident description

The first indication of an escape of water was detected at 7:30 on 24 November 1970, although the water level recorder indicates it started the evening before (Wickham, 1992). The outflow reached a maximum within two hours. The dam was breached to foundation level by 13:30. The breach, which occurred over the line of the new draw-off works, was 20 m wide at crest level and extended down to the tunnel. Large sections of the concrete tunnel segments were washed out and deposited downstream, as shown in Figure 5.8. Internal erosion appears to have taken place along the line of the tunnel which led to a cavity being formed, leading to subsidence and eventual overtopping of the crest.

Water impounded by the dam was discharged into the two lower reservoirs. The embankment of Cocker Cobbs was overtopped but did not fail and the water passed over the spillways of the lowest reservoir. The incident occurred four years after construction of new draw-off works.

Lessons

As far as is known, the embankment had performed satisfactorily for 100 years before the incident. Construction of a new draw-off tunnel through the embankment very likely provided leakage paths along the tunnel. Tunnelling always reduces the stresses adjacent to the tunnel to zero, which may have led to hydraulic fracture between the lining and ground. Annulus grouting will not necessarily re-establish sufficient earth pressure to prevent hydraulic fracture. This internal erosion incident shows the speed with which this type of failure can develop, and emphasises the need for careful monitoring and interpretation of seepage flows. Remedial works to embankment dams need to be carefully designed and executed with full understanding of how the work could change the behaviour of the dam in terms of stress changes and stability.



Figure 5-7 Breach of the embankment along the line of the recently constructed draw-off tunnel at Warmwithens (from *Engineering Now*, No.4, December 1970)

35. Greenbooth

Incident date: 7 March 1983

Description of dam

Height	35 m	Dam type	Earthfill
Reservoir capacity	$3,200 \times 10^3 \text{ m}^3$	Watertight element	Puddle clay
Completed	1962	Cut-off	Concrete

The dam is located near Rochdale, Lancashire and was engineered by G H Hill and Sons (Omerod, 1962). It is one of the last puddle clay core dams to be constructed in Britain, and like many dams of that era is a hybrid of methods using a puddle clay core in conjunction with modern earth-moving and compaction equipment. The embankment shoulders comprise an inner zone of shale fill and an outer zone of coarse sandstone fill. The boundary between the two zones has a slope of one in one. The core has a base width of 8.2 m, reducing to 2.7 m near the crest. The batters of the core are 1:12. The core sits on a concrete shoe connected to the concrete cut-off wall. At both abutments there are concrete wing walls; the interface between the wing walls and the core is 1:2 (H:V). The dam is founded on the Upper Carboniferous Millstone Grit Series with the valley sides formed of fissured sandstone overlying shale, as shown in Figure 5.9. The valley bottom has a considerable layer of glacial deposits covered with thin alluvium. Grouting was used at the abutments.

When the fill had been raised to within six metres of the final crest level, high foundation pore pressures and some movement in the area of the downstream toe raised concerns that a deep-seated slope instability problem could be developing. Precautionary measures involved the addition of a large buttress of stony material on the downstream toe, as shown in Figure 5.10.

Incident description

In 1975, the crest of the embankment had settled adjacent to the abutment, forming dips in the crest road that are still visible 50 years after completion. In the afternoon of 7 March 1983, more than 20 years after construction, a member of the public taking two dogs for a walk noticed a depression in the asphalt of the crest roadway above the clay core about 20 m from the west abutment, Figure 5.9. The depression deepened quickly over a few days. By mid-morning the next day it measured three metres by one metre in plan and had subsided by 0.16 m. The tarmac road surface was removed over the area of the depression and at one corner, a cavity some 0.3 m in diameter at the surface and 2.2 m deep was revealed between the core and the upstream shoulder. The puddle clay core appeared to be settling with vertical tension cracks at the interface of the shoulder material. No cracks were reported running across the dam. The depression was directly above the toe of a concrete wing wall where there was a sharp change in direction of the interface between the concrete and the puddle clay.

The reservoir level, which was 1.65 m below top water level, was reduced by 9.3 m over an eight-day period.

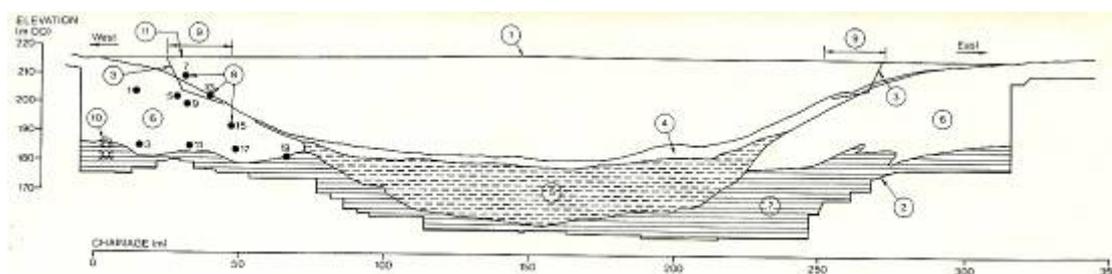
Investigation and remedial works

No separate site investigation was carried out, but a system of recording parameters from the grout hole drilling was used to derive information on the internal condition of the dam and foundations (Flemming and Rossington, 1985). The drilling technique identified voids through the puddle clay core and a cavity upstream of the core. Grouting of the foundation, shoulders and then the core was carried out by tube-à-manchette techniques using a bentonite, cement, fly ash and clay grout. The grout injected was about four per cent of the core volume over the treated area. No significant movement has taken place since the grouting was done.

Lessons

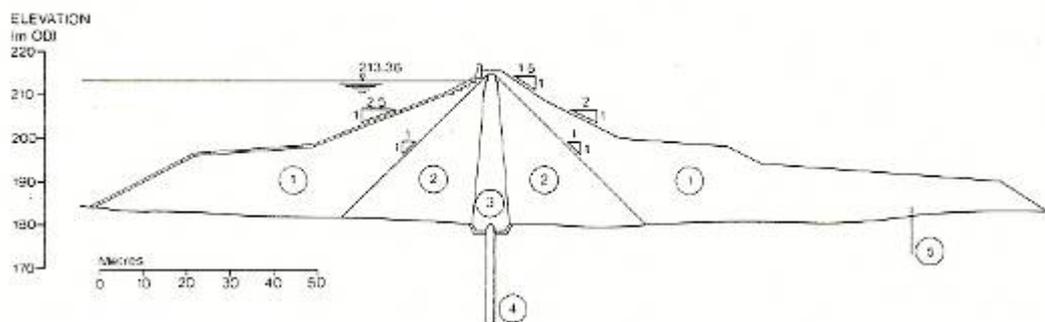
This incident showed that erosion of the puddle clay forming the core had occurred and that the most likely cause of the leak was hydraulic fracture of the puddle clay adjacent to the wing wall where stresses had reduced in the clay due to its settlement relative to the steep wing wall.

The downstream fill and/or the adjacent abutment were not able to behave as a filter to halt erosion. However, it took some 20 years from the end of construction for the internal erosion to manifest itself as a visible defect, although settlements were noticed 14 years after construction.



1 – Dam crest; 2 – Limit of concrete cut-off; 3 – Concrete wing wall at abutment; 4 – Alluvium and colluvium; 5 – Glacial clays; 6 – Sandstone; 7 – Shale; 8 – Piezometers; 9 – Grouted area; 10 – Overflow/discharge tunnels; 11 – Depression.

Figure 5-8 Simplified longitudinal section show geology of Greenbooth dam showing steep abutments and location of depression (after Flemming and Rossington, 1985)



1 – Sandstone fill; 2 – Shale fill; 3 – Puddle clay core; 4 – Concrete; 5 – Sheet piling.

Figure 5-9 Cross-section of Greenbooth dam (after Flemming and Rossington, 1985)

39. Rivington Upper (Yarrow Embankment)

Incident date: 9 January 2002

Description of dam

Height	12 m	Dam type	Earthfill
Reservoir capacity	1,090 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1857	Cut-off	Puddle clay

The eight Rivington reservoirs were completed between 1850 and 1875 to supply Liverpool and other towns to its north. Upper and Lower Rivington reservoirs act essentially as one reservoir separated by Horrobin embankment with a valved connection and a culvert at high level. Upper Rivington reservoir is the third in a cascade of four reservoirs, with inflows from reservoirs in the adjacent valley. The reservoir is also formed by a side embankment, Yarrow, at which the incident occurred. The dam was designed by Thomas Hawksley. Further details of the Rivington Scheme are available in several references (Holye, 1987; King, 1992; Parry, 1903).

Yarrow dam is thought to be of similar design to other dams in the scheme and consists of an embankment of clay and boulders with a central puddle clay core with a puddle clay filled cut-off trench. The dam has a 1.8-m diameter outlet culvert placed in a rock excavation beneath the embankment. The culvert is lined with masonry blue brick and surrounded by puddle clay, with the valve shaft located just upstream of the core as shown in Figure 5.11. The valves are embedded in a concrete plug at the base of the shaft. Twin 600-mm diameter bottom outlet pipes discharge into the upstream end of the 33-m long section of the culvert.

Incident description

When driving across the top of the Yarrow embankment of the Upper Rivington reservoir on 9 January 2002, an operative noticed a stream of discoloured water emerging from the culvert. Minor leakage through the roof of the culvert had been monitored for many years with little change observed and the previous day there had been no change in the leakage measurement. Inspection of the culvert found a jet of water issuing at an estimated rate of 15 l/s at full bore from a half-brick opening used for drainage, and hitting the opposite wall 1.8 m away, Figure 5.12. The leak was downstream of the puddle clay core. Material was being eroded and deposited in the invert.

Emergency actions

The supervising engineer was contacted and the undertaker's Operational Response Centre was alerted to the situation. The reservoir safety manager arrived at the site and contacted a panel engineer, who was on site early the following day. Attempts to block the point of leakage into the culvert did not prevent the ingress of water, but greatly reduced the amount of larger gravel size particles from being eroded, Figure 5.12. However, leakage occurred elsewhere in the culvert and through the low retaining wall at the foot of the dam.

The scour valves were partially opened but the reservoir was still overflowing. There was concern about opening the valves fully, as the flow might wash away the temporary plug in the culvert wall and cause damage to the culvert, especially if there were voids behind the walls. Pumps were brought in overnight and the leak was plugged more securely so that the valves could be opened fully. The reservoir ceased to overflow on the afternoon of 10 January. Eight days later on 18 January, the reservoir level was seven metres below top water level, corresponding to an average rate of drawdown of about one metre per day.

Investigations

Little was known about the geology or layout of the construction. Dynamic probing showed that the culvert was founded on natural ground close to a steep five-metres high rock face three metres away. It was estimated that a one cubic metre void had formed behind the culvert wall.

Remedial works

A grouting programme was undertaken to seal the upstream and downstream shoulders and foundation, followed by grouting of the core during the summer of 2002. The reservoir was slowly refilled to top water level on the 23 February 2003 with daily readings taken on piezometers and seepage flows until November 2003. The reservoir was then inspected every two days. No major leakage has been recorded. Leakage is still under investigation at the dam.

Lessons

The incident shows that internal erosion incidents can develop quickly, and reinforces the need for on-site emergency plans. The two factors in preventing a disaster were early detection of the new leak and rapid lowering of the reservoir (Gardiner *et al.*, 2004; Charles, 2005).

(a) **Early detection of leakage.** The reservoirs were kept under close surveillance yet discovery of the leak was via a providential sighting. Early detection of this type of situation, which can develop rapidly, is fundamental to avoiding a breach and to the evacuation of people.

(b) **Rapid lowering of reservoir.** In an emergency, rapid lowering of the reservoir may be crucial. It is vital that valves are operational, that the capacity of outlet valves and pipes is known and, where such capacity is inadequate, pumps are available. At Upper Rivington, it was possible to lower the reservoir quite quickly. However, the discharge of water directly into an outlet culvert beneath an embankment, which is common practice, needs to take into account the possibility of damage to the lining of the culvert.

Culverts constructed through dam embankments are potential hazards leading to internal erosion, and this incident reinforces the need for vigilant surveillance of such structures. Similar incidents have occurred at other dams including Holmestyes, March

Haigh and Carno Lower. It is likely that this type of incident will occur at other dams in the future.

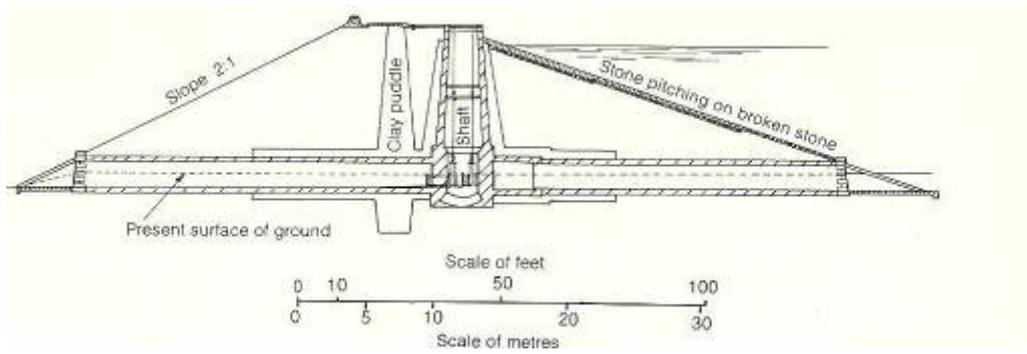


Figure 5-10 Section of Lower Rivington embankment (assumed to be as Yarrow embankment, after Binnie, 1981, p139)





Figure 5-11 Leakage in the draw-off culvert at Yarrow embankment and temporary plug (after Gardiner *et al.*, 2004)

5.2.4 Group 4: Incidents due to pipe or valve failure

44. Ogston

Incident date: 2001

Description of dam

Height	20 m	Dam type	Earthfill
Reservoir capacity	6,180 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1959	Cut-off	Concrete

The earth embankment has a central puddle clay core. The incident involved the failure of a recently fitted valve and therefore the description of the dam is restricted to the outlet and draw-off arrangements. The overflow shaft, valve tower and combined overflow and draw-off tunnel are situated in the centre of the embankment. The overflow tunnel and draw-off are formed as one structure. The complex arrangement of pipework from the scour and the draw-off stack and valves is shown in Hughes *et al.* (2004). A 700-mm butterfly valve on the scour pipework to the overflow shaft was a recent replacement for the Larner Johnson (needle valve) streamline valve which had been found to be in poor condition. Although the use of the butterfly valve was questioned by a panel engineer with regard to velocities and location, the manufacturer had confirmed that the butterfly valve was fit for purpose.

Incident description

Initially, operation of the newly installed butterfly valve was stiff and modifications were made to the gearbox. During commissioning tests on the butterfly valve, the pipework immediately upstream and the compensation branch suffered catastrophic failure, resulting in the sudden uncontrolled release of water from the scour pipe into the base of the draw-off tower where two men were trying to operate the valve. The water quickly started to fill the draw-off tunnel until it forced the doors to open at the downstream end, allowing water to discharge back to the downstream tail-bay area. Fractures occurred at the flanges as shown in Figure 5.13. This is a location of weakness in cast iron pipes, as they are prone to porosity and high cooling stresses.

Emergency actions

The upstream guard valve to the failed scour pipework was shut by men going back through the discharging water. A temporary bulkhead was installed on the scour forebay tunnel headwall to enable safe access into the draw-off tower and provision of a 600-mm diameter washout facility on the raw water draw-off was provided for additional drawdown capacity and to control reservoir levels during remedial works.

Investigations

Even though the butterfly valve had not been installed in the ideal position, immediately downstream of a bend and discharging to zero pressure, investigations were undertaken to find the reasons for the catastrophic failure and to determine the physical condition of the pipework such as remaining wall thickness, degree of corrosion, evidence of welding, flange rating and strength. Investigations established that the gearbox fitted to the butterfly valve was undersized and one of the four screws used for coupling the gearbox to the valve drive shaft was missing. When the connection between the valve and gearbox failed, the instantaneous closure of the valve caused high surge pressures estimated to be in excess of 55 bar.

Lessons

If valves are difficult to operate, seek out the reason why rather than reverting to more force.

Failure was caused by fitting an inappropriate valve and undersize gearbox for the required duty and configuration. Engineers should understand how various valves work and consider their modes of failure. A Larner Johnson streamline valve incorporated into the scour pipework was replaced with a butterfly valve. A key feature of the Larner Johnson valve is that it can handle high flow velocities. The basic operating principles of the valve are described by Reader *et al.* (1997) and Lewthwaite *et al.* (2008). In the case of butterfly valves, failure of the connection between the gate and gearbox will lead to sudden closure and possible generation of high surge pressures. Grey cast iron pipes and fittings are brittle and are therefore susceptible to fracture from sudden surge pressures or impact loading.



Figure 5-12 Fractured compensation pipework (above) and fractured 30-inch (769-mm) diameter scour pipe (below) adjacent to bolted flanges (after Hughes *et al.*, 2004)

5.2.5 Group 5: Slope instability during construction

49. William Girling (Chingford No 2)

Incident date: July 1937

Description of dam

Height	10 m	Dam type	Earthfill
Reservoir capacity	544 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1951	Cut-off	Puddle clay

This fully-banded reservoir embankment, some 3.5 miles long, was built to form a storage reservoir next to the River Lee in Essex. The embankment has a central puddle clay core and cut-off trench through a one-metre thick layer of soft alluvial clay and five-metre thick ballast to the underlying stiff London Clay. The cut-off trench was filled with puddled London Clay. The embankment was built directly upon a layer of soft yellow alluvial clay, as shown in Figure 5.14. The shoulder fill was mixed alluvial clay and gravel.

Incident description

At the end of July 1937, with the embankment eight metres high at a section where the completed height would be 10 m, a 90-m length of the downstream slope failed on a slip surface passing through the core and the layer of soft yellow alluvial clay. Within a few days, the top of the bank had sunk 0.6 m and the toe moved out horizontally four metres, as shown in Figure 5.13. At this section, the embankment had been built to a height of eight metres in 11 weeks using modern earth-moving equipment imported from America. The slip provided a striking contrast to the stable slope at King George V reservoir (Chingford No1) which was completed in 1912 at roughly one-third the rate of Chingford No2, and appeared stable at a height of eight metres on similar foundation strata.

Investigations

An investigation into the cause of the slip was carried out by the Building Research Station (Cooling and Golder, 1942). The undrained shear strengths of the yellow and puddle clays were measured by laboratory direct shear tests yielding values of 14 and 10 kPa respectively. A stability analysis was carried out on total stress and a factor of safety close to unity was obtained. Development of high pore water pressures in the yellow alluvial clay due to rapid loading by the embankment was probably a major contributor to the failure. Samples were also tested eight months after construction and showed a notable gain in strength, and in accordance with consolidation theory, a minimum strength at the centre of the alluvial clay.

Remedial works

With advice from Karl Terzaghi, the embankment was redesigned with wide berms on both slopes, with keys of gravel fill taken down through existing material and alluvial clay to the underlying gravel stratum to obtain a minimum safety factor of 1.5. Work was actively resumed later that year following Terzaghi's recommendations, but the outbreak of war delayed completion until the late 1940s.

Lessons

Features that contributed to the instability were the presence of soft clay in the foundation and the rapid construction rate, such that there was little dissipation of pore water pressure in the foundation. The value of soil mechanics in assessing embankment stability was established (Charles and Boden, 1985; Skempton, 1989).

A similar incident occurred at Abberton dam (Incident No. 48). A major slip in the upstream slope took place during construction on 20 July 1937 with the embankment within two metres of the planned height (French *et al.*, 2000). The slip took place nine days before Chingford. Like Chingford, this was one of the first dams to be built using modern earth-moving equipment such that excess pore water pressures had insufficient time to dissipate from the foundation and fill during embankment construction.

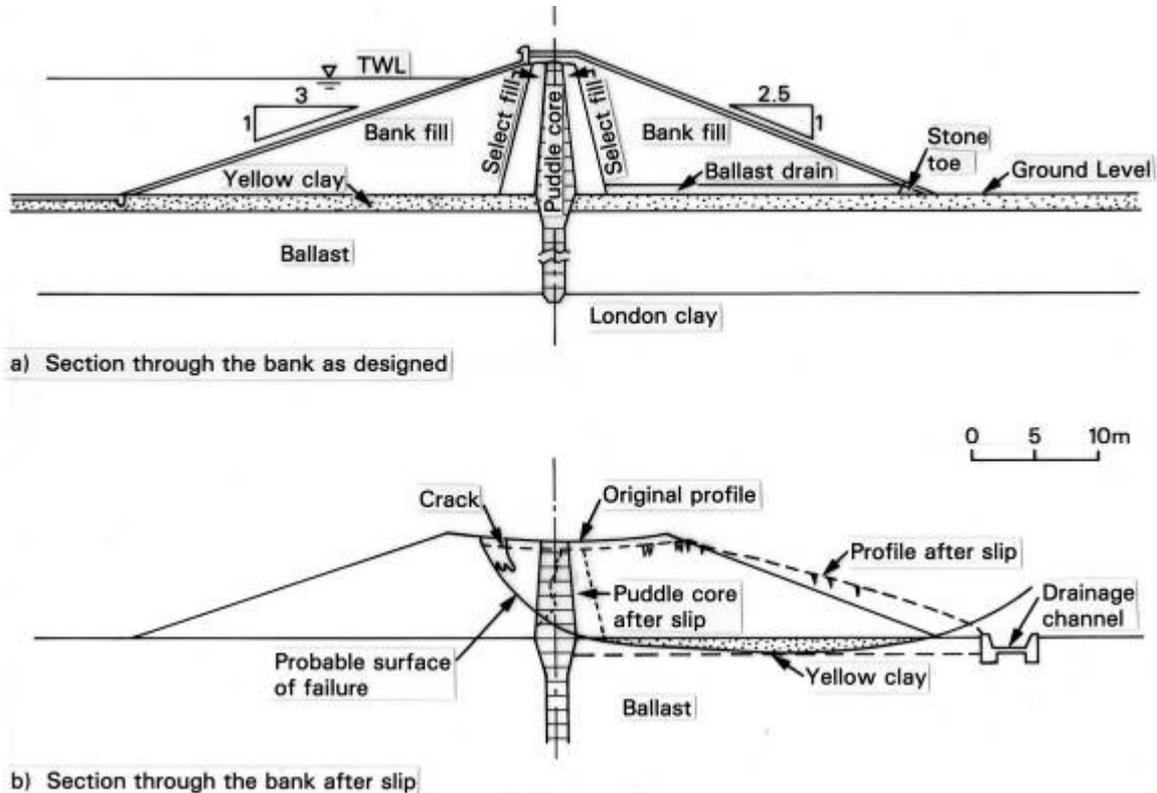


Figure 5-13 Shear failure during the construction of Chingford embankment (after Coosing and Golder, 1942)

50. Muirhead

Incident date: September 1941

Description of dam

Height	21 m	Dam type	Earthfill
Reservoir capacity	$3,572 \times 10^3 \text{ m}^3$	Watertight element	Puddle clay
Completed	1942	Cut-off	Concrete

Construction of Muirhead dam near Paisley, Ayrshire began in 1940 and was designed to be 26 metres high. Wartime conditions demanded early completion of the dam and the introduction of track-laying machinery enabled the upper fill to be placed in a few months. The embankment had slopes of one in three, a central puddle clay core and shoulders of boulder clay. At the end of the first season the fill had reached a height of nine metres and by September 1941, had reached 21 m.

Incident description

In September 1941, noticeable deformations developed in both the upstream and downstream slopes, with tension cracks some six metres from the centre-line on each side. The Building Research Station carried out an extensive investigation into the failure. It was believed that the embankment had failed through the lower part of the shoulder fill. The strength of this material was found to be variable, but the average

measured value of undrained shear strength was about 40 kPa which was close to the limiting equilibrium condition. That the embankment existed in a state of limiting equilibrium was confirmed by adding 0.5 m of fill; immediate outward movements of 0.15 m occurred on both slopes, increasing to 0.3 m before coming to rest after several days (Banks, 1948).

Lessons

The redesign was based on the shear strength determined by back analysis of the existing embankment. The bank was kept at its current height of 21 m and a substantial berm was placed on the upstream slope. The dam was successfully completed in November 1942.

At the time of the Muirhead slip, Knockendon dam in the same locality and built of similar boulder clay had reached a height of six metres (Banks, 1952). Because of its more remote location and inaccessibility it had been decided not to accelerate construction of Knockendon as at Muirhead, and therefore work there proceeded more slowly than had been anticipated. As a result of the events at Muirhead, the section at Knockendon was modified by adding toe weighting to the upstream shoulder and by including a zone of stronger granular fill in the downstream shoulder. Standpipe piezometers were installed to measure construction pore pressures. The measured pore pressures were used together with the results of drained shearbox tests to calculate the stability of the embankment. These were the first observations of construction pore pressures in Britain.

51. Usk

Incident date: March 1953

Description of dam

Height	33 m	Dam type	Earthfill
Reservoir capacity	12,253 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1955	Cut-off	Concrete filled trench and grout curtain

Usk reservoir was built to supply Swansea in South Wales. The dam was the next largest puddle core embankment to be built in the UK after Knockenden. It was founded on boulder clay, although a layer of silt existed under part of the dam which was water-bearing and sandwiched between clay layers of very low permeability. The shoulders of the embankment were made of boulder clay (Sheppard and Aylen, 1957).

Incident description

An incident was avoided by the measurement of pore water pressure during construction of the dam by the Building Research Station and effective stress stability analyses carried out by Imperial College London which indicated that embankment construction could not safely continue (Penman, 1978). The layer of silt which was thicker under the downstream shoulder was drained using a system of vertical and horizontal sand drains (Sheppard and Little, 1955). Twin tube hydraulic piezometers were installed in the silt layer to check the performance of drains. The sand drains proved to be so efficient that little excess pore pressure developed in the silt during construction.

However, pore pressures in the boulder clay fill were large. Effective stress stability analyses indicated that the factor of safety would be unacceptably small if the dam was brought to the design height with the average pore pressure ratio (r_u) greater than 0.5. Dissipation of pore water pressure during the winter shutdown period, with no more fill being added, had only reduced r_u to 0.6.

Horizontal drainage blankets were introduced before the second and third season's fill were placed. Future build-up of pore pressures was restricted, leading to a factor of safety at full height of not more than 1.45. This is believed to be the first use of horizontal drainage blankets within embankment shoulders of this type to control construction pore pressures.

Lessons

The importance of field observations and effective stress stability analyses were demonstrated and soon became standard practice in the construction of embankment dams. The effectiveness of horizontal drainage blankets in clay embankment was also demonstrated.

53. Carsington

Incident date: 4 June 1984

Description of dam

Height	35 m	Dam type	Earthfill
Reservoir capacity	35,000 x 10 ³ m ³	Watertight element	Rolled clay
Completed	1984	Cut-off	Grout curtain

The dam had a rolled clay core with an upstream extension (the boot) and shoulders of compacted mudstone with horizontal drainage layers of crushed limestone about four metres apart (Davey and Eccles, 1983). A typical section is shown in Figure 5.15. The upstream slope was 1:3 and the downstream slope 1:2.5. Effective stress stability analyses were carried out. Fill placing began in May 1982 and took three summers, with winter shutdowns. A small berm was placed at the upstream toe to compensate for a faster rate of construction in August 1983. Earth filling restarted in April 1984 and was one metre below the final crest level on 4 June 1984 when the upstream slope slipped. Observations of pore pressure and settlement were made during construction at four sections and horizontal displacements were observed from August 1983.

Incident description

The failure started in the early hours of 4 June 1984 with a 50-mm crack on the crest over a length of about 120 m. During the night of 5 June a major upstream slip occurred. The slip propagated along the embankment in both directions extending to a length of nearly 500 m, with the embankment crest dropping 11 m, as shown in Figure 5.16, and the upstream toe moving 13 m horizontally by 6 June 1984. The initial slip sheared through the core which already contained shear surfaces due to rutting and along a layer of yellow clay in the foundation which contained solifluction shears. Both materials were brittle with low residual strengths (Skempton and Coats, 1985; Skempton and Vaughan, 1993).

Investigations

Faced with one of the largest geotechnical failures of a structure in Britain, the owners sought independent advice into the cause, and appointed Babbie Shaw & Morton, consulting engineers and Professor Alec Skempton of Imperial College London to report on technical matters relating to the slip. Investigations involved establishing the deformed shape of the dam internally and externally, and extensive sampling and testing of the dam and its foundation (Rocke, 1993).

Remedial works

Reconstruction of the failed dam is described by Banyard *et al.* (1992), Chalmers *et al.* (1993), Macdonald *et al.* (1993) and Vaughan *et al.* (1991). It commenced in February 1989 and was completed in 1991, seven years after the start of investigations. The main differences in cross-section are shown in Figure 5.15. Reconstruction of the new

dam involved excavation of two million cubic metres of the original dam to remove all failed material and lay a sound foundation.

Lessons

The scale of the failure was so great and public concern so high that the Department of the Environment appointed Roy Coxon, who was independent of any interested parties, to report on the actions being taken to investigate the failure. In his report to the Secretary of State, Coxon (1986) recommended that review panels, which are widely used internationally, should be used in the United Kingdom. Failure of the original dam prior to immediate impounding added another seven years to the original programme.

The investigation of the failure by Professor Peter Vaughan led to a better appreciation of the significance of progressive failure, and overestimation of stability by limit equilibrium analysis (Vaughan *et al.*, 1989; Potts *et al.*, 1990). The factor of safety based on peak strengths was about 1.4. Pre-existing shears lowered the safety factor to about 1.2 and progressive failure reduced it to 1.0.

A similar rotational failure occurred at Acu dam, Brazil in 1981 during construction which was due in part to the core shape.

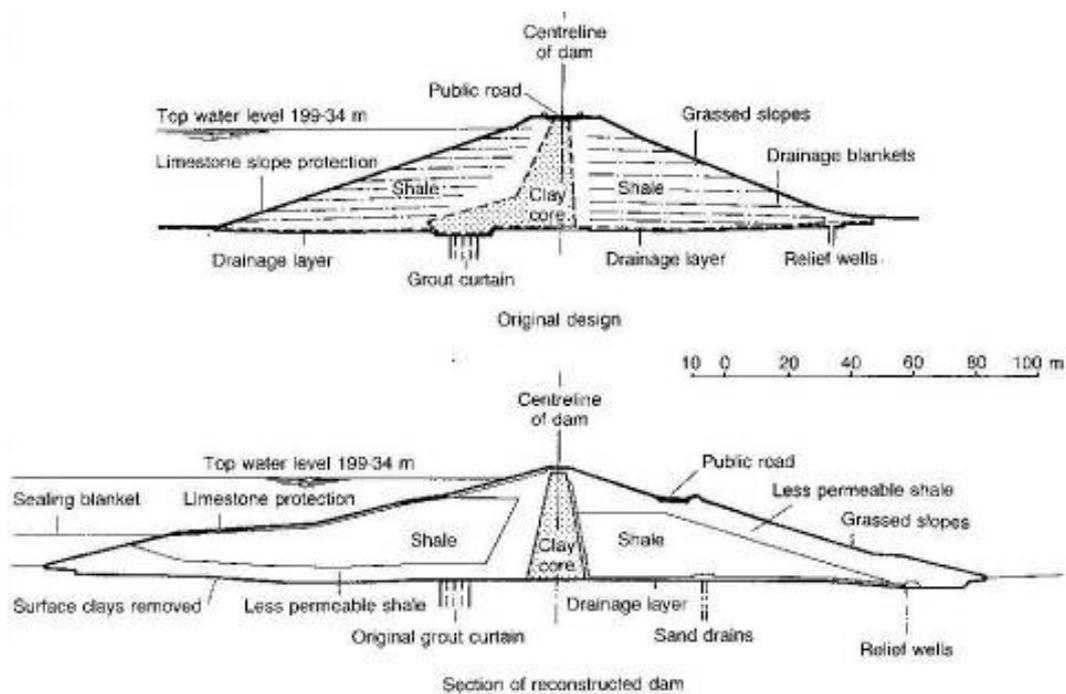


Figure 5-14 Cross-section of the original and reconstructed Carsington dams (after Banyard *et al.*, 1992)



Figure 5-15 Failure of Carsington dam

5.2.6 Group 6: Slope instability in service

55. Harlow Hill

Incident date: 18 December 1951

Description of dam

Height	9 m	Dam type	Earthfill
Reservoir capacity	65 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1868	Cut-off	Shallow puddle clay

The open service reservoir at Harlow Hill lies perched upon a hill, some 70 m above the town of Harrogate which it serves. The reservoir is rectangular and has embankments on three of its sides. The dam is of conventional construction with a puddle clay core supported by “selected fill”. The upstream and downstream slopes are 1:1.9.

Incident description

Following an extremely wet autumn a major downstream slip occurred on 18 December 1951 such that there was a danger of the dam being breached. A vertical movement of 0.3 m had occurred on a slip plane adjacent to the core, with 0.23 m uplift at the toe against the concrete retaining wall (Figure 5.17). Movement was continuing at 0.01 m per hour.

Emergency actions

Such was the concern of a breach that emergency work was carried out by day and night. The reservoir was lowered as fast as possible. Some 33,000 sandbags and 400 tonnes of loose sand contained in “pavement buttresses” were placed on the toe of the slip to improve stability. Tarpaulins were placed to prevent further ingress of rainfall and movement monitoring commenced. Police were alerted to be ready to evacuate the downstream population. Well points at three-metre centres were drilled into the embankments in an attempt to dewater the clay fill. An attempt was even made to drill horizontal holes to drain the embankment.

Investigations

Investigations into the history of the dam showed that upstream slips had occurred immediately after construction and at a later date. During the drought of 1887, shrinkage cracks in the embankment were so deep that rain could percolate into the dam. Stability may have been compromised by the construction of a public road involving the removal of about three metres of the toe and replacement with a “sturdy” concrete retaining wall. Investigations indicated the fill to be largely clay of very low permeability. The 1:1.9 slopes were too steep for the clayey embankment fill and stability must always have been marginal. A major instability was waiting to be triggered by unusually heavy rain or leakage through the dam.

Lessons

Some important lessons were drawn from the incident and are reproduced below from the paper by Davies (1953):

“The size of a reservoir is only one part of the measure of its potential destructive force. The other parts are equally, if not more important, its relative elevation, and what stands in the potential path of destruction.”

“Every dam should be so sited as to provide ample room for emergency measures.”

“All earth dams of clay constructed before the advent of soil mechanics should be suspect.”

“The “normal” visual inspection of a dam, unsupported by any real knowledge of the properties of the materials of construction, is insufficient to determine the stability of the structure.”

“Soil tests are essential to the determination of actual stability, but they must be sufficiently numerous to provide a proper statistical average, and must be taken from locations on potential slip planes.”

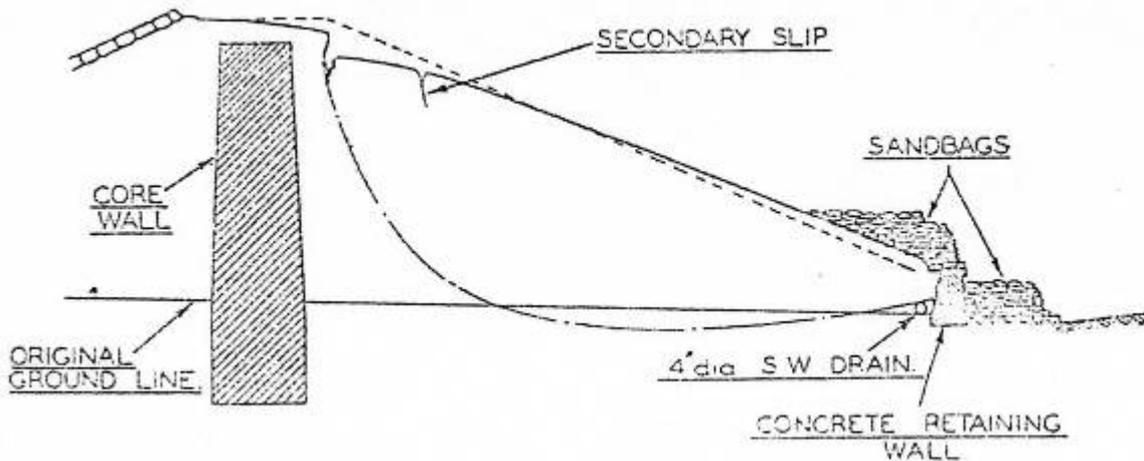


Figure 5-16 Section through the major slip at Harlow Hill (after Davies, 1953)

60. Combs

Incident date: 29 January 1976

Description of dam

Height	16 m	Dam type	Earthfill
Reservoir capacity	$1,484 \times 10^3 \text{ m}^3$	Watertight element	Homogeneous
Completed	1805	Cut-off	Puddle clay

Combs reservoir is upstream of Whaley Bridge in North West Derbyshire and was constructed for the Peak Forest Canal to the design of Benjamin Outram. The homogeneous dam was completed in 1805 but was raised in 1820 by two metres to bring it to its current height. The upstream and downstream slopes are very steep at 1:1.6. A vertical masonry wave wall, up to 1.2 m high, supports the raised top of the embankment. The fill consists of generally firm, very sandy, silty clay with some gravel size mudstone and sandstone fragments.

Various minor incidents occurred and remedial works were undertaken prior to the incident of 1976. Of possible relevance to this incident was the removal of trees before 1948 and the stemming of a top water leak in 1969 caused by decayed roots. There have been repairs to the wave wall and additions to the riprap on a number of occasions.

There are two spillways at the reservoir; the main side overflow weir, 73.5 m long, spills directly into a bywash running along the side of the reservoir and a 1.4-m diameter dropshaft spillway.

Incident description

A shallow slip of the upper part of the downstream slope occurred on 29 January 1976 during a storm in very cold windy weather. The slip was 10 m wide, 12 m slope length and had a slump of two metres. It reduced the crest width from 2.5 m to 1.3 m over a five-metre length. The slipped material tumbled down the lower intact part of the downstream slope onto the lane below, as shown in Figure 5.18.

The slip was discovered at 7:15 when the police and owner were informed. At 9:00, the reservoir draw-off valve was opened to lower the reservoir which was 0.11 m above the main spillway level and 0.91 m below the top of the wave wall. Conditions were freezing and there was ice on the face. Spray was still being blown over the crest on 30 January 1976 when the panel engineer visited the site. On his advice, a deep cut was made in the side overflow weir, pumps were used to lower the water level, and 24-hour surveillance was instigated. It was concluded that cavities at the base of the wave wall were probably created by wave action which led to local saturation of the downstream face and slip. These cavities were plugged as an immediate precaution. Siphon pipes were eventually installed to maintain the water level approximately six metres below the top water level.

Investigations

After an initial investigation of the failed area with trial pits, a ground investigation was carried out with 23 boreholes, 11 trial pits, piezometer installation and measurement of strength parameters. Stability analyses at six sections (Ferguson *et al.*, 1970) based on the piezometer observations indicated marginal slope stability of the downstream slope with some safety factors below one. However, the embankment had stood without recorded failure for 150 years.

Remedial works

Remedial works aimed to restore the failed area, increase the overall stability of the embankment, reduce seepage through the upper part of the dam by means of a cut-off and provide adequate freeboard against waves above the reservoir. The slipped material was removed and glacial till was compacted in 150-mm layers back to the bank profile. The downstream stability was improved by adding an 11-m high buttress of free-draining material consisting of magnesium limestone. Stone drains were found within the original embankment and these were connected into a filter blanket laid beneath the buttress. To reduce the hazard posed by further wave action, a segmental concrete wave wall with a curved deflection shape was installed. The foundation of the wall incorporated a sheet steel pile cut-off to stop any leakage in the upper part of the dam which incorporated the raising of 1820. The gradient of the upstream slope was reduced to one in two using riprap.

Lessons

This incident provides another case of marginal slope stability at a dam which had behaved satisfactorily for 150 years; a major slope instability was waiting to be triggered by some event. In this case, a combination of water being forced by wave action through cavities at the base of the masonry wave wall and spray being blown over the wave wall resulted in the localised saturation of the downstream slope and ice forming on the slope. The cavities had not been identified despite regular inspection and maintenance.



Figure 5-17 The downstream slip at Combs dam (after Dutton, 2001)

61. Lambieytham

Incident date: November 1984

Description of dam

Height	15 m	Dam type	Embankment
Reservoir capacity	54 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1899		

The reservoir was located near St Andrews. The dam had a puddle clay core, an upstream slope of 1:3 and downstream slope of 1:2.5.

Incident description

The incident occurred following abnormally high rainfall in the area. A wet spot on the downstream slope was observed during routine surveillance. Within 24 hours seepage had increased and slurry was observed at the base of an eight-metre long crack, which had a maximum width of 0.4 m. There was evidence of uplift six metres downstream of the crack (Charles, 1986).

Following inspection by a panel engineer, it was decided that the reservoir should be emptied as quickly as possible. On the night of 20 November 1984 engineers from the undertakers and the police assessed the likely consequences of failure and householders in the area were alerted to the situation. Pumps were brought onto the site by helicopter and the reservoir level was lowered by five metres in three days. The dam was demolished in October 1985.

Investigations

An investigation was carried out during demolition to determine the cause of the instability. The downstream fill material was variable but much of it could be described as silty sand. It was concluded that the instability of the lower part of the downstream slope was due to high pore water pressures within the fill.

The puddle clay core appeared to be in good condition and there was no evidence of leakage through the core. Its properties were typical of many puddle clays in good condition, with little of the variability that might be expected in a core that had been subjected to internal erosion. However, remedial works had been carried out in 1934 to prevent leakage from the north west of the dam. From 1941 until 1969, the reservoir was drawn down to 2.2 m below top water level.

The possibility of leakage around the scour pipe or fracture in the scour pipe was discounted during the investigation. Large volumes of water were found to be coming from the north-west valley side and it was concluded that this water had saturated the lower half of the downstream shoulder. This is similar to the incident which occurred at Roddlesworth Upper (Incident No. 56).

Lessons

Like some of the other cases in this section, the embankment performed satisfactorily for many years without serious incident, in this case some 80 years, until triggered by particular conditions. The dominant effect appears to have been heavy rain causing run-off from the valley side, however the contributions of rain on the embankment and overtopping of the top of the core are not known.

5.2.7 Group 6: External erosion due to flood flow

68. Cowlyd

Incident date: 31 December 1924

Description of dam

Height	14 m	Dam type	Earthfill
Reservoir capacity	9,430 x 10 ³ m ³	Watertight element	Concrete
Completed	1921	Cut-off	Concrete

The dam was built for the purposes of water supply for the Conway and Colwyn Bay Joint Water Supply Board and of hydro-electric power for the Aluminium Corporation Limited's power station at Dolgarrog in North Wales. The dam is remote with difficult access. The dam was originally intended to be constructed of concrete and curved in plan, but as the supply of cement was difficult during the 1914-18 war, an earth dam was built. As most of the cut-off trench had been excavated for a curved concrete dam,

it was decided to continue the embankment dam on the curve. The embankment was constructed of moraine fill each side of a central concrete core-wall and founded partly on rock and partly on glacial drift.

The method of placing the fill was described as “rather revolutionary”, consisting of dropping the contents of a four-yard wagon from a considerable height rather than the normal method of spreading the material in thin layers and compacting by rolling. Concerns were expressed over the uniformity of fill placed by dropping method. In the discussion of the paper by Knight (1975), Arthur Penman referred to the method as dynamic consolidation. Information about the dam can also be found in Binnie (1987b), Coutts (1934) Farrington (1921) and Gourley (1922).

Incident description

The reservoir is exposed to prevailing winds, which are funnelled through a neck at the head of the valley. On the night of 31 December 1924, a storm caused overtopping of the dam and a V-shaped area of the downstream fill was eroded down to foundation level, exposing the concrete core-wall.

Emergency actions

Frenzied backfilling on the following morning saved the dam. Had the central core not been of concrete, it is possible that the dam could have failed leading to certain loss of life close to the village of Dolgarrog which was to be devastated 10 months later by the failure of Eigiau dam in the adjacent valley.

Remedial works

Subsequently, the spillway crest was lowered and the wave wall was raised. Geotechnical investigations and remedial works have since been undertaken and are described by Knight (1975). The upstream and downstream slopes are now protected by concrete.

Lessons

The incident showed the limitations of overflow facilities and wave protection in an exposed location. This near failure did not appear to have raised concerns about the state of Eigiau dam.

69. Skelmorlie

Incident date: 18 April 1925

Description of dam

Height	5 m	Dam type	Earthfill
Reservoir capacity	$24 \times 10^3 \text{ m}^3$	Watertight element	Homogeneous
Completed	1861		

The small embankment was located near the Ayrshire village of Skelmorlie on the Clyde coast, south west of Glasgow. The dam was built by local labour to supply water to the village. The overflow consisted of a small diameter pipe through the embankment. Prior to the incident, there had been concern about the lack of maintenance since its construction.

Incident description

Following heavy rainfall, the embankment overtopped and breached on 18 April 1925 killing five people in Skelmorlie (Coutts, 1934; Davidson, 1996). Water from a nearby partially flooded quarry added to the flood water when a blocked culvert cleared. At 14:00 a loud crack was heard and two fissures appeared in the dam. The overflow pipe through the embankment was inadequate and the dam was quickly overtopped. A few

seconds later the embankment gave way and in less than 15 minutes the reservoir was empty. Five people were killed.

Lessons

A public enquiry into the disaster was held on 15 June 1925. The cause of the failure was attributed to the inadequate overflow consisting of a 0.45 m diameter pipe and freeboard which was only 0.45 m. Lack of supervision, maintenance and abnormally high rainfall, plus release of water from the Beithglass quarry were cited as contributory factors. The verdict of the jury at an enquiry was: “*The disaster was caused by absence of any regular skilled supervision and inspection*”. Together with the Dolgarrog failure in North Wales, which occurred in November 1925, this incident led to the Reservoirs (Safety Provisions) Act 1930.

82. Ulley

Incident date: 25 June 2007

Description of dam

Height	16 m	Dam type	Earthfill
Reservoir capacity	582 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1873	Cut-off	Puddle clay

Ulley reservoir is located about three miles south east of Rotherham, Yorkshire and was originally used for water supply. The dam came under ownership of the local borough council in 1986. The earthfill dam has a puddle clay core founded on sandstones of the Middle Coal Measures. The upstream slope is 1:3 and the downstream slope is 1:2. The core was raised by two metres in 1967 using plastic concrete made of local brickworks clay, sand, cement and retarder, placed in short lengths of open trench.

As originally constructed, the dam had narrow spillways down both mitres with masonry retaining walls. In 1943, during the Second World War, a new concrete spillway was constructed at the south end of the dam to discharge, via a chute, to a short stilling basin adjacent to the stream. The new spillway was designed to pass 50 cumecs and one of the original spillways near the left mitre was designed to pass five cumecs, both having the same weir level. In 1967 this was changed so that the reservoir top water level was controlled by the spillway close to the mitre and only when the reservoir rose 1.23 m above top water level did the larger, more modern concrete spillway come into use. Some engineers have questioned the logic of this arrangement.

Incident description

In the evening of 25 June 2007, following a day of heavy rain, damage to the embankment was reported by representatives of the owners. The spillway that ran down the left mitre of the embankment had disintegrated leading to rapid erosion of the downstream shoulder of the dam creating a scour hole 50 m wide and up to six metres deep. Erosion worked up the line of the spillway, eroding material from the downstream face of embankment. The reservoir rose to 1.62 m above top water level. The extent of the damage to the downstream toe and masonry spillway are shown in Figure 5.19.

Calculations have shown that the peak flow in the incident was only 14 per cent of the probable maximum flood (PMF) and 38 per cent of the 10,000-year outflow. Prior to the incident, the dam was classed as Category C (where a breach would pose negligible risk to life and cause limited damage), but this was changed to Category A (where a breach could endanger lives in the community) after the incident because of the infrastructure downstream of the dam.

Emergency actions

At 01:20 on 26 June 2007, the M1 was closed and residents from areas at risk were evacuated. Over 1,000 people were evacuated.

Action was taken to divert flow from the damaged spillway, to lower the reservoir using temporary pumps and to import rockfill to repair the embankment. The damaged spillway was blocked with an eight-tonne skip after unsuccessful attempts with intermediate bulk containers. This reduced flow in the spillway so that water no longer reached the damaged section of the embankment. Emergency pumps were requested from the fire service which became operational at 6:30 to lower the water level. Additional pumps were provided later.

Within a short time of the incident being declared, the erosion hole in the downstream shoulder of the dam was being filled with stone and sheeting placed over the affected area, as shown in Figure 5.20.

Investigation and remedial works

Investigations were carried out to assess the condition of the core, as discrete zones of seepage had been seen in the scour hole at the time of the incident (King *et al.*, 2009). The investigation cast doubt about the watertightness of the two-metre deep plastic concrete extension at top of the dam. This was replaced with a single phase cement bentonite slurry wall.

A new concrete spillway was built over the centre of the dam to take the PMF.

Lessons

The incident shows that design of spillway capacity should not only include spillweir capacity but should consider that significant flows need to be safely discharged away from the embankment. Erosion of the downstream shoulder was initiated by the collapse of the masonry chute running down the left mitre. Mechanisms proposed for the spillway collapse included high turbulence flows in the chute plucking out masonry block or pushing them out by water pressure from behind and by overtopping the chute walls causing back pressure on them (Hinks *et al.*, 2008; Mason and Hinks, 2008, 2009). The incident highlights the hazard posed by spillway channels located down dam mitres and along the toes of embankments. Similar failures are known to have occurred at not less than nine other masonry spillways including Toddbrook in 1985 and Boltby in 2005.

Elevation of the sill of the failed spillway being set at 1.23 m below the sill of the more modern larger spillway made a major contribution to the incident.

Post-incident investigations showed that the plastic concrete used to raise the core was brittle and crumbling and was likely to have been the cause of seepage into the scour hole.

As with other incidents, the key factors that prevented a disaster from occurring included:

- (a) early identification of the problem;
- (b) an effective emergency plan;
- (c) the ability to lower the reservoir and undertake emergency remedial works quickly.

The incident suggests that dambreak studies and emergency planning under the Water Act 2003 should be considered with planning for other sorts of emergencies under the Civil Contingencies Act 2004.

Further discussion of lessons learnt from the near failure of Ulley reservoir is given in Section 4.3.2 of this report.



Figure 5-18 View of collapsed stepped masonry spillway and adjacent erosion of embankment fill at Ulley dam



Figure 5-19 Temporary remedial works to stabilise the embankment following the incident at Ulley dam

83. Maich Water

Incident date: 1 August 2008

Description of dam

Height	9 m	Dam type	Earthfill
Reservoir capacity	24.3 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1850		

The earth embankment is located near Lochwinnoch, Renfrewshire not many miles from Skelmorlie dam that failed in 1925. The reservoir had a similar capacity, lying just outside the ambit of the 1975 Reservoirs Act. It was built in the second half of the nineteenth century for public water supply and was originally an off-stream reservoir, with a substantial bywash channel about four metres wide diverting the river around the reservoir. Some years prior to the incident, a 25-m long section of the bywash channel had collapsed and repairs had also been washed away. An overflow weir about six metres long was located on the right abutment, and screens had been added to retain fish. The outlet consisted of a 150-mm pipe through the dam with inclined sluices operated by rods on the upstream face, but it was inoperable.

Incident description

In the early hours of 1 August 2008, heavy rain fell in west central Scotland causing local flooding and severe damage to Maich dam by overtopping (Mann and Mackay, 2009). Flow had taken place over the entire length of the 35-m long dam and it is estimated that the peak reservoir level was possibly 350 mm above the crest. The overtopping had eroded away a large part of the downstream fill, leaving a vertical face at the downstream edge of the central clay core, Figure 5.22. The edge of the downstream face eroded 150 mm over the next two days, leaving only one metre of crest between the vertical face and the water line. There was concern that complete failure of the dam would take place.

Emergency actions

Several houses at risk were evacuated and two roads crossing Maich Water were closed. Emergency works started on 4 August 2008 with the reinstatement of the bywash channel to divert the river around the reservoir. The reservoir was lowered by about three metres using pumps as the draw-off did not work and the overflow sill was lowered as a precaution in case the reservoir refilled. Further heavy rain washed out the temporary repairs to the bywash channel and the reservoir refilled. Evacuation and road closure were reinstated, and the reservoir level was again lowered by pumping. Two days later, partial demolition of the dam was undertaken leaving a platform that could be overtopped and a retaining pond of water and silt about three metres deep.

Lessons

The cause of overtopping and near complete failure of the embankment was disrepair of the collapsed part of the bywash channel wall and substantial restriction of the overflow by fine screens for fish, combined with long-term settlement of the crest. Screens on overflows are unacceptable for reservoir safety as they restrict flow and allow debris to inhibit flow.

Three factors enabled the threat from this dam to be averted:

- a) The resilience of the relatively cohesive core against erosion by overtopping and loss of support by the downstream fill.
- b) The bywash channel enabling the flow to be diverted. Bywash channels at other reservoirs should not be abandoned without consideration of their value in an emergency.
- c) The low impounded volume of the reservoir.

Lessons on the management of this incident are described in Section 4 of this report.



Figure 5-20 Erosion of the downstream shoulder fill due to overtopping at Maich Water

5.2.8 Group 8: Wave damage to upstream protection

84. Blithfield

Incident date: 16 February 1962

Description of dam

Height	16 m	Dam type	Earthfill
Reservoir capacity	18,172 x 10 ³ m ³	Watertight element	Puddle clay
Completed	1953		

The embankment is built of sand and gravel with selected fill of marl either side of the puddle clay core. The upper two metres of the core are less than two metres wide. The upstream slope is generally 1:3 and the wave protection consists of concrete slabs 4.27 metres square by 305-mm thick with 25-mm wide joints filled with fine gravel. The slabbing is laid directly onto the sand and gravel shoulder fill. The panels on the 1:3 slope are retained within a lattice of concrete beams 305 mm wide by 610 mm deep set into the shoulder fill. Over the upper 2.4 m below top water level, the upstream slope steepens from 1:3 to 1:2.75 in a series of four steps to mitigate wave action. Above the topmost slab, there is a further step at the base of a wave return wall, as shown in Figure 5.22.

Incident description

On 16 February 1962 with the reservoir full, a severe six-hour storm caused the dam to be overtopped by waves and spray which led to saturation of the downstream slope that triggered a slip (Leach, 1975).

On the upstream side, wave damage was confined to the upper row of slabbing in the middle of the dam which was immediately above top water level. Twelve separate panels were displaced with cavities up to 460 mm deep being formed. The damage was located within the central section of the dam and extended over a distance of some 165 m, almost a fifth of the dam. The typical slab movement was rotational, as shown in Figure 5.22. The maximum wind speed down the centre of the reservoir over a 12- to 18-hour period was 37 m/s. The fetch from the north west was 1.7 km.

Had the duration of the storm been longer, it is conceivable that further erosion of the upstream fill could have led to overtopping and failure of the embankment, taking into account the nature of the fill and the narrowness of the core.

Emergency actions

Sandbags were placed in the cavities.

Remedial works

Remedial action was taken the following morning. All displaced and damaged slabs were broken up, the cavities filled with broken concrete and gravel, and the slabs recast. All joints were filled with pea gravel and sealed with mortar to a depth of 100-150 mm. Voids on the top two rows were grouted with a cement/pulverised fuel ash/sand mix. The wave wall was repaired and raised by one metre.

Lessons

Eight other incidents from 1962 to 1990 involving concrete slabbing are described by Herbert *et al.* (1995). The incidents highlight the inadequacies of the original construction.

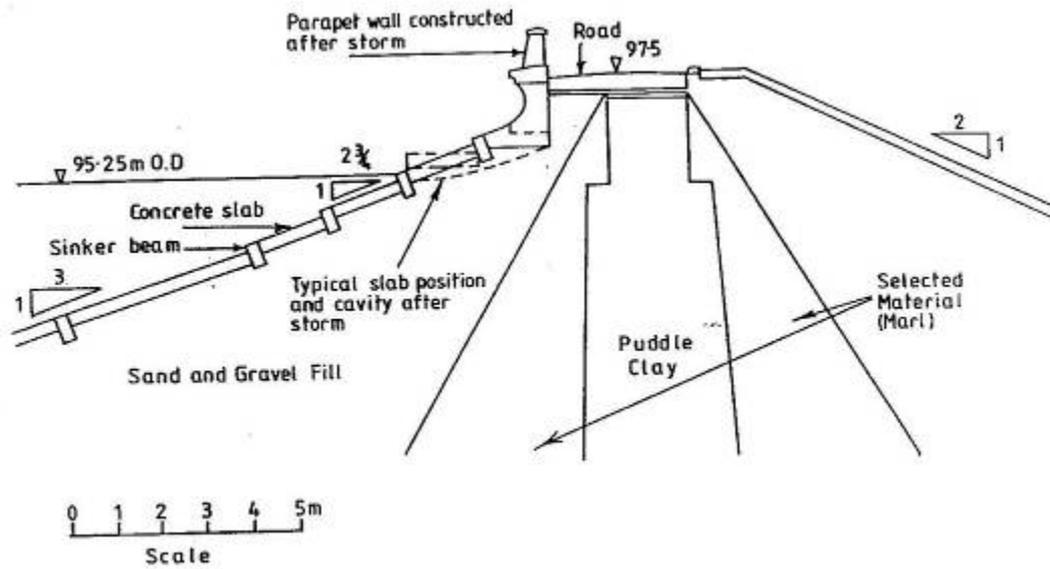


Figure 5-21 Section and damage to the upstream protection at Blithfield dam (after Herbert *et al.*, 1995)

85. Megget

Incident date: February 1984

Description of dam

Height	56 m	Dam type	Embankment
Reservoir capacity	61,400 x 10 ³ m ³	Watertight element	Asphaltic core
Completed	1982		

The reservoir is situated in the Borders Region of Scotland 75 miles south of Edinburgh. It has a maximum fetch of four kilometres and the valley runs in the prevailing wind direction. The long fetch generates substantial waves which have periodically exceeded the design wave height. The dam is a gravel fill embankment, 570 m long, with a central asphaltic core (Gallacher, 1988a). The upstream slope is 1:1.5 and its protection is 1.8 m thick, with the upper 1.2 metres of heavy riprap containing rock up to 900 mm. The specification for the riprap is described by Gallacher *et al.* (1998).

Incident description

Damage occurred to the riprap upstream slope protection during first filling in a severe storm in January 1984 (Gallacher *et al.*, 1988b). The same storm also caused damage to the block work on the upstream face at Kielder reservoir (Incident No. 86). Further damage occurred on a number of occasions during storms. The damage has taken the form of shallow displacement of individual or groups of stones.

Remedial works

No emergency action was required but repairs are ongoing. Five alternatives were considered for the remedial works. Following a value planning study (Gallacher *et al.*, 1998), reinforcement of the existing riprap using bituminous grout was undertaken (Hay-Smith, 1998). Grouting has the effect of increasing the overall mass and binding the stones together, which increases resistance to waves. However, filling the voids also reduces the porosity of riprap and energy dissipation resulting in increased wave run-up and risk of overtopping. It may also lead to the reinforced layer being unable to dissipate uplift pressures. To minimise these effects, pattern grouting was done which involved grouting square blocks of riprap while maintaining ungrouted areas between the blocks for drainage. Pattern grouting has been used on sea defence work where the wave attack is generally more extreme than in inland reservoirs.

Lessons

The simple prediction methods used to determine the design wave heights underestimated the wave heights at Megget. The grid reinforcement method at Megget has proved to be effective, but it is unlikely to be used elsewhere as the conditions of long fetch and steep slopes at Megget are probably unique. However, asphalt has been used at a number of dams for reinforcement of upstream pitching (Hay-Smith, 1998).

5.2.9 Group 9: Reservoir basin leakage

90. Mill Hill

Incident date: 22 October 1979

Description of dam

Height	6 m	Dam type	Service reservoir
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Reservoir capacity Completed	110 x 10 ³ m ³ 1939	Watertight element	Concrete
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Service reservoirs No 1 and 2 at Mill Hill in County Durham were built in 1926 and 1939. The reservoirs were formed by mass concrete walls and reinforced concrete floors and columns. Each reservoir is divided into two compartments by division walls. The reservoirs are founded on drift deposits which overlie Magnesian Limestone and Coal Measures. The reservoirs have a history of cracking and leakage, with the formation of large cavities under the reservoirs associated with coal mining subsidence. Structural repairs, grouting of cavities and waterproofing of reservoirs with a rubber lining were carried out in 1953 and 1964.

Incident description

On 22 October 1979 when the reservoir was being relined, a sudden subsidence occurred in the south-west corner of service reservoir No 1 and part of the structure collapsed (Millmore and Heslop, 1988). The division wall was also affected and 68 x 10³ m³ of water stored in the damaged compartments drained away in six hours into voids beneath the structure, causing an increase in water level in a borehole at the site. There was no threat to life or property.

Remedial works

Partial repairs involving extensive grouting were carried out to maintain some water supply. New storage facilities were built away from the mining area. A thorough borehole investigation was done at the new site to confirm that it was suitable for construction of the reservoirs. Provision was made for continued monitoring of the new foundation and the formation of any cavities. The structures were extensively articulated with movement joints.

Lessons

From detailed site investigation to establish the cause of the damage, it was concluded that the failure was due to the drift deposits, on which the reservoirs were founded, migrating into fissures in the underlying Magnesian Limestone that had been widened as a result of mining.

The incident draws attention to the dangers of mining beneath reservoirs, particularly those that have a rigid structure such as concrete service reservoirs. Knowledge of the geology and foundation conditions beneath reservoirs can help to assess the potential for future incidents.

5.2.10 Group 10: Concrete and masonry dams

93. Blackwater

Incident date: 1909

Description of dam

Height	25 m	Dam type	Concrete gravity
Reservoir capacity	111,300 x 10 ³ m ³	Watertight element	Concrete
Completed	1907	Cut-off	Concrete

The reservoir was built to provide hydro-electricity for smelting aluminium at Kinlochleven, Argyllshire (Roberts, 1911). It was completed in 1907 and has a maximum height of 25 m. It was the first dam to be built with timber shuttering on both sides instead of stone shuttering. The dam is composed of mass concrete with large granite displacers weighing up to 10 tonnes, many of which bridge the lift joints. The hearting concrete in the dam is 1:5 cement to aggregate with up to 50 mm aggregate. A slightly richer mix of 1:4 with 19 mm aggregate is used for the facing concrete. The

foundation rock of mainly fine grained mica schists is tough, with little weathering and no fissures. The maximum depth of excavation to sound rock is only 4.6 m. The section of the dam is narrow in relation to dams of the period. The dam was built at about the same time as Eigiau and like Eigiau was built without vertical construction joints.

Incident description

Vertical cracks were found in the dam shortly after the completion of construction. These opened from the top of the dam and were barely perceptible at the base, Figure 5.23. There were seven main cracks up to 2.4 mm wide through which water seeped. The most severe cracking was concentrated in the northern part of the dam which was explained by the more rapid rate of construction during the summer of 1908 and the greater heat of hydration produced as a result. The appearance of cracks and leakage did not raise immediate safety concerns, but attempts were made to seal the cracks to avoid long-term deterioration.

Remedial works

Unsuccessful attempts were made to seal the cracks using a silicate solution and fine grout, but seepage was reduced by introducing peat into the water which was drawn into the cracks. Despite the appearance of the downstream face of the dam, a number of investigations have indicated that the hearting concrete is in generally sound condition (Wallis *et al.*, 2004; Morgan and Thomas, 1961).

Lessons

It was concluded that the cracks resulted from heat of hydration and resultant cooling and that expansion joints would have alleviated the problem. It is believed that the cracking at Blackwater was one of the cases that instigated the introduction of expansion joints in later dams. Uplift was not taken into account in the design of the dam.



Figure 5-22 Vertical cracks in Blackwater dam

94a Eigiau and 94b Coedty (Dolgarrog)

Incident date: November 1925

The incident involved the failure of two dams, Eigiau and Coedty, built for the North Wales Power Company (which was allied to the Aluminum Corporation) above its Dolgarrog hydro-electric power station. Failure of the concrete Eigiau dam led to the cascade overtopping failure of the embankment dam at Coedty (Anon, 1926; Binnie, 1987; Guthrie Brown, 1964; Walsh and Evans, 1973).

Description of Eigiau dam

Height	10 m	Dam type	Concrete gravity
Reservoir capacity	4,500 x 10 ³ m ³	Watertight element	Concrete
Completed	1911	Cut-off	None

The dam across the Afon Porth Llyd forming the reservoir consists of one kilometre of concrete gravity dam and 250 m of low embankment less than two metres high, Figure 5.24. The concrete contains crushed rock aggregate and large stone displacers. The maximum height above ground level is about 10 m across the river but for much of the long side leg it is less than three metres high. Construction of the dam started in 1907 but work stopped in 1908. A different contractor completed the dam in 1911. In the foundation, a glacial deposit of hard blue clay containing boulders of granite, some of considerable size, is overlain by a layer of peat up to 1.5 m deep near the breach.

Description of Coedty dam

Height	11 m	Dam type	Embankment
Reservoir capacity	320 x 10 ³ m ³	Watertight element	Concrete
Completed	1924	Cut-off	Concrete

Coedty reservoir was built to augment the hydro-capacity of the Cowlyd and Eigiau dams conveying water to the Dolgarrog power station. It is a 250-m long earth embankment with the watertight element described as a thin reinforced concrete wall estimated from drawings to be 0.6 m wide. The upstream slope is 1:2 and the downstream slope is 1:2.5. The material for the dam shoulders was boulder clay excavated from within the reservoir basin.

Incident description

The breach occurred in the concrete dam at 20:45 on 2 November 1925 near the junction of the 1908 and 1910 work where the dam was six metres high. The breach was about 10 m long (Figures 5.24 and 5.25) and scoured a 20-m wide channel three metres below the ground surface. The dam did not fail at its maximum height across the river but at the deepest section of the side leg of the dam. The resulting discharge of 1.5 million cubic metres in the first hour caused Coedty dam, 2.5 miles downstream, to be overtopped and the concrete core wall to collapse forming a breach 60 m wide at the top and 18 m wide at the bottom.

The wall of water, mud, rock and concrete hit the northern end of Dolgarrog village by 21:15. Fortunately, most of the villagers were attending the weekly film show held in the village hall. Witnesses described the wall of water “as being 50 feet wide and quite as high.” Ten adults and six children died. The power station and aluminium works were destroyed. No warning reached the town as the telephone line from the operator at Llyn Eigiau was washed away.

Investigations

Sir Alexander Gibb and Partners were instructed to examine the cause of the disaster and to report on the performance of all dams belonging to the Aluminium Corporation. The investigation in 1926 involved the digging of numerous trial pits at Eigiau up to six

metres deep and a borehole 12.5 m deep at the location of breach to establish how the dam was built.

According to the design, the footings were supposed to have been founded 1.8 m below the surface of the clay but at the point of failure they were only 0.46 m into the clay. A section of the dam at the point of failure is shown in Figure 5.24. It was suggested that the poor concrete and intrinsically soft porous condition of the boulder clay, which was aggravated by fissuring during drying out in the preceding summer months when there was a long period of drought and the lake bed was exposed, contributed to failure.

Detailed examination of the concrete revealed a lack of sand and cement so that the stones forming the aggregate were not cemented together. At least half of the concrete at the breach was badly honeycombed, as shown in Figure 5.26. The stone displacers were larger than desirable for a wall of this thickness and seemed to have been placed carelessly and in several cases there were voids under their bed surfaces.

Eigiau dam was not rebuilt and the breach in the dam remains. The high section across the river was demolished. Coedy was rebuilt and has had various remedial works (Knight *et al.*, 1990).

Lessons

At the inquest, it was stated that the failure was due to criminal neglect upon the part of someone unknown. The jury rendered a verdict of “*accidental death caused by the bursting of Lake Eigiau dam and on the evidence a section of the wall lacked proper foundation.*” The jury recommended that the remaining portion of the dam be examined by government inspectors before reconstruction began.

The incident showed the need for a competent engineer to supervise the construction of dams.

As a direct consequence of the failure, the Reservoirs (Safety Provisions) Act 1930 was passed by Parliament.

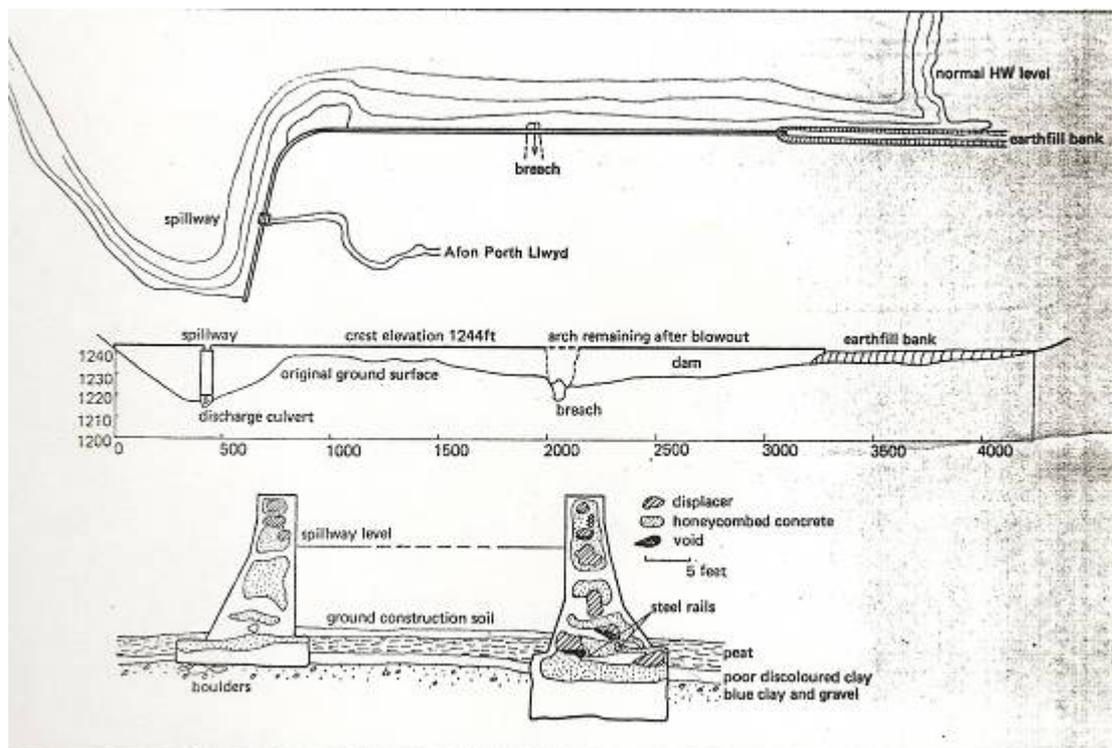


Figure 5-23 Plan and longitudinal sections of Eigiau dam and cross-sections of each side of the breach (after Evans and Walsh, 1973)



Figure 5-24 The breach at Eigiau dam (taken 2009)



Figure 5-25 Typical poor concrete of Eigiau dam showing honeycombing and lack of fines

95. Glendevon Upper

Incident date: July 1954

Description of dam

Height	45 m	Dam type	Concrete gravity
Reservoir capacity	5,040 x 10 ³ m ³	Watertight element	Concrete
Completed	1955	Cut-off	Concrete

The dam was constructed for Fife Regional Council between 1950 and 1955. It has a traditional cross-section for a concrete mass gravity dam with a downstream batter of 0.65:1 (horizontal to vertical) and a small varying upstream batter. It was constructed in 24 monoliths and the water-stops were formed by single copper strips. Generally, the concrete lifts were 0.6 m. There was no evidence that the concrete was vibrated or cured or that the surfaces of the lifts were treated other than brushing away loose material. The concrete was described as harsh and segregated badly and an air-entraining agent was introduced.

The foundation was excavated to shallow depths except for a narrow cut-off trench which varied between about two metres and 12 m deep. No grout curtain was provided. The foundation was described as "large boulder masses with clay joints". The strata consisted of inter-bedded andesite lavas and agglomerate which were brecciated in places and intermixed with siltstones.

Incident description

A series of leaks on first filling raised concerns about the stability of the dam. During impounding, water appeared in such quantity in July 1954 when constructing the stilling basin that sub-drainage was provided. Leakage also occurred through vertical joints between the monoliths. By 1956, leakage measured 27 l/s and fluorescein tests showed numerous and varied flows through the concrete. High uplift pressures caused concern and the reservoir was emptied and remedial measures commenced in 1959. Substantial flows and pressures were obtained from boreholes drilled into the concrete and rock. Drilling rods tended to drop up to 150 mm at the concrete/foundation interface. In 1958, uplift pressures at the downstream toe developed to about 40 per cent of full reservoir head. In October 1969, leakage was identified coming up in the bed of the river 30 m downstream of the dam (Allen, 1975; Allen and Boardman, 1982). Two boreholes drilled from the dam crest in 1990 indicated the concrete to be fissured and honeycombed in places. However, it was generally of acceptable quality in terms of cement content and compressive strength.

Remedial works

Following emptying of the reservoir, the installation of a grout curtain, bitumen seals between monoliths and grouting in and under the dam in 1959-1960 appeared to have been successful. The quantities of grout injected were 520 tonnes for curtain grouting and 200 tonnes for blanket grouting.

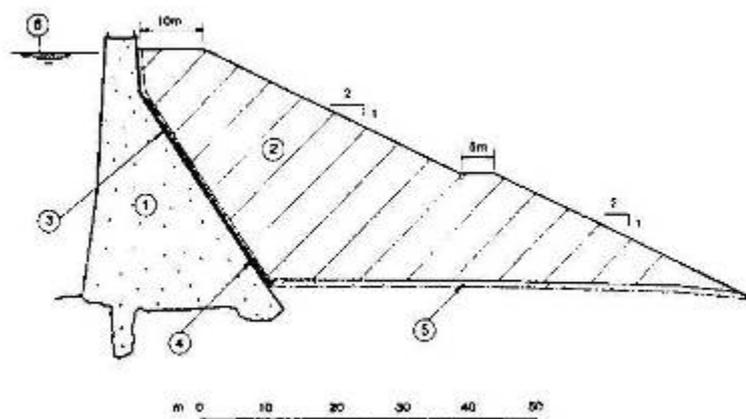
In 1975, ten 76-mm diameter pressure relief holes were drilled under the downstream toe. The total flow was 1.4 l/s with the reservoir full. All piezometers indicated pressure relief. With the reservoir full, pressures were reduced by as much as two-thirds with the average being one-quarter and an uplift load of 4,000 tonnes was removed from the base of the dam. The emergence of flows from the relief holes did not increase overall leakage.

It was recommended that the reservoir level be kept at four metres below top water level. Although this restriction was removed in 1973, it was re-imposed in 1979 following a seismic tremor close to the dam.

The four-metre restriction on top water level represented a reduction of almost 30 per cent in useable storage of the reservoir. In 1992, five options were considered to improve the stability of the dam including rock anchors and adding concrete to the upstream and downstream faces. A rockfill embankment (see Figure 5.27) placed against the downstream slope was selected on the basis of cost and the availability of suitable rockfill material close to the site (MacDonald *et al.*, 1994; Johnston, 1995; Hewitt, 1996).

Lessons

The quality of the concrete was poor, with segregation evident. It was identified as fissured and honeycombed in places, with water seeping through the cracks. The waterstops were ineffective between monoliths and there was a lack of effective cut-off.



1 – Existing mass concrete dam; 2 – New rockfill embankment; 3 – Slip membrane; 4 – Inclined drainage layer; 5 – Drainage blanket on embankment foundation.

Figure 5-26 Section of Glendevon Upper and the remedial rockfill embankment (after MacDonald *et al.*, 1994)

97. Mullardoch

Incident date: July 1986

Description of dam

Height	50 m	Dam type	Concrete gravity
Reservoir capacity	223,000 x 10 ³ m ³	Watertight element	Concrete
Completed	1951	Cut-off	Grout curtain

Mullardoch reservoir, which is part of the Glen Affric hydro-electric scheme, has a 730-m long concrete gravity dam with a vertical upstream slope and 0.7:1 downstream slope. The two flanks of the dam meet at 140° at a central buttress.

Incident description

On 4 July 1986, it was reported that leakage into the longitudinal gallery had increased from 0.16 l/s to 5.2 l/s. Pre-existing nearly horizontal cracks in the gallery walls were found to have opened as much as 1.5 mm. This was mainly evident in the two blocks extending about 15 m either side of the central buttress where the two flanks of the dam meet. At this location, water was issuing from the cracks under pressure. Uplift

pressures had increased significantly. The change in leakage was reported by staff making routine weekly measurements, illustrating the value of routine surveillance by staff familiar with structures. There was particular concern about the suddenness of the increase and the opening of the cracks.

Emergency actions

Detailed visual inspections by experienced dam engineers were undertaken within 24 hours. The reservoir level was lowered to seven metres below top water level. Existing instrumentation was monitored twice daily and the results were assessed.

Investigations

Three-dimensional modelling showed the observed pattern of cracking to be consistent with a build-up of longitudinal compressive stress within the structure, leading to downstream tilting of the central buttress under high temperature loads (Gosschalk *et al.*, 1991). The build-up of compressive stress was thought to be the result of progressive deposition of calcium carbonate in the vertical construction joints resisting thermal expansion and closure of the valley sides.

Remedial works

Improvement of drainage and pressure relief to reduce uplift pressures was achieved by drilling low-level outlets to intersect the longitudinal rubble drain in the foundations. Vertical pressure relief holes intersecting the inspection gallery were cleaned out or, if irretrievably blocked, were duplicated by drilling new holes.

After examining several options, including cutting slots and buttressing the central blocks, it was decided to post-tension the four central blocks of the dam, to improve stability against overturning and to hold the cracks closed under all foreseeable loading conditions, the most severe being that of high temperature and valley closure movements (Hinks *et al.*, 1990). Twenty-six vertical tendons were installed. The tendons were provided with double corrosion protection and were re-stressable and fully detensionable. Transverse tendons were also installed to counteract induced tension in the roof of the inspection gallery. The layout of the post-tensioning is shown in Figure 5.28 and the works undertaken in Figure 5.29. Performance of the dam during the initial post-tensioning period was satisfactory except that the horizontal cracks did not close as much as anticipated when the anchorage cables were stressed.

Lessons

Experiences at Mullardoch dam and elsewhere (Abraham and Sloan, 1978) suggest that sharp changes of alignment should be avoided.

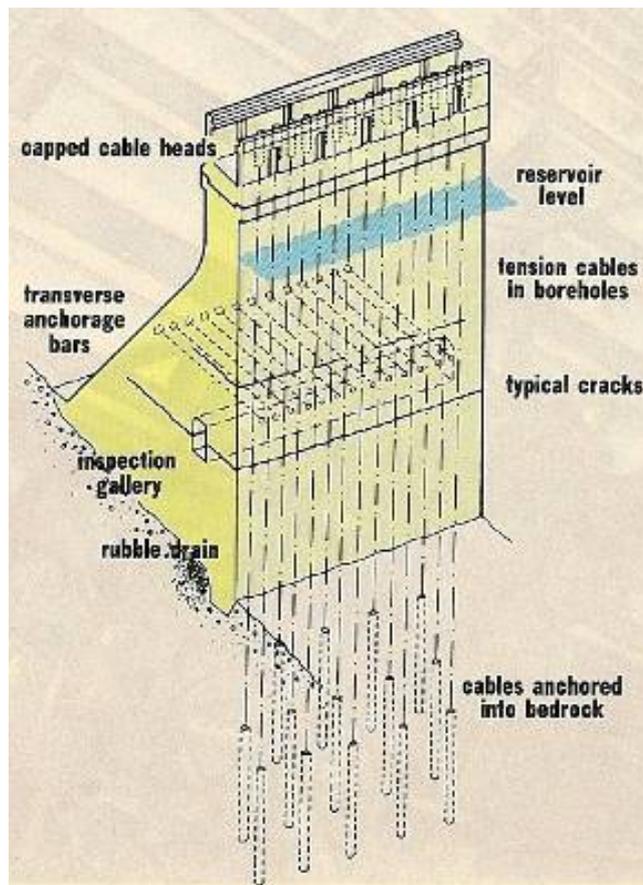


Figure 5-27 Layout of post-tensioning (after Hayward,1990)



Figure 5-28 Post-tensioning works at Mullardoch dam

5.2.11 Group 11: Other incidents

99. Dundreggan

Incident date: 1998

Description of dam

Height	16 m	Dam type	Concrete gravity
Reservoir capacity	1,640 x 10 ³ m ³	Completed	1957

Although there are relatively few spillway gates on British dams, flood flow was controlled by 28 gates at 14 dams owned by Scottish Hydro-Electric in 2000 (Noble and Lewin, 2000). Failure of gates could possibly result in loss of life and severe destruction downstream. Two radial flood gates, 8.25 m high by 8.69 m wide, were installed at Dundreggan dam which is part of the Garry/Moriston Scheme near Loch Ness.

Incident description

Inspection of the gate at Dundreggan was instigated following an incident at Folsom dam (Incident No. 103) in California. The left bank radial gate was taken out of service during the summer of 1998 to install new trunnion bearings of the self-lubricating type. Inspection of the gate during the work revealed cracks in the web of the vertical skin plate stiffener beams.

A change in operation at one of the gates caused severe vibrations at low openings which in turn caused the fatigue cracks.

Lessons

Failure of the spillway gate at the Folsom dam in California had a direct influence on the inspection regime of Scottish Hydro-Electric, which led to the discovery of cracks in the web of the vertical skin plate stiffer beams. Gate vibrations are a cause for concern as they can lead to fatigue failures and should be reported by operators as soon as they are noticed so that the cause can be identified and rectified. The design of the gate lip was the cause of vibration at Dundreggan. Once this had been modified to achieve a clean flow separation, vibration was eliminated. The lip of a gate should have a short cut-off point (Lewin, 1995).

5.2.12 Group 12: International incidents

101. Malpasset (France)

Incident date: 2 December 1959

Description of dam

Height	61 m	Dam type	Concrete arch
Reservoir capacity	22,000 x 10 ³ m ³	Completed	1954

The thin double curvature arch structure, which was located on the French Riviera, had a thickness of 1.5 m at the crest and 6.8 m at its base and was claimed to be the thinnest arch dam of its height. On the right bank it abutted a rock mass whereas on the left bank the topography required the construction of a wing wall. The rock at the site was schist, which featured many minor faults and joints. The foundation was highly deformable in contrast to the rigid arch structure. Construction commenced in 1952 and was completed in April 1954, with reservoir filling starting later in 1954 (James, 1988).

Incident history

No untoward behaviour was identified in the years immediately following dam construction. However, measurements of deflections of the structure made annually in the period 1955 to 1959 indicated a displacement at the base of the dam of 17 mm. Although the chief designer had judged the tolerable deflection to be 10 mm, the design office was not informed of the excessive deflection until after the failure (Jansen, 1980). All measurements were obtained by geodetic surveys on targets fixed on the downstream face of the dam. No instruments were installed within the structure or its foundations to monitor deformations.

In mid-November 1959, following heavy rainfall, the reservoir was 5.2 m below top water level when seepage was discovered at the right abutment 20 m downstream from the dam. By 30 November, following further intensive rainfall, the reservoir had risen to within 3.4 m of top water level and the seepage had increased. On 2 December the reservoir level was almost at top water level and the outlet valve was opened at 18:00. The caretaker left the dam at 20:45.

Incident description

When the caretaker left the dam he returned to his home, which was on the hillside about a mile downstream of the dam. At about 21:10 he heard a loud cracking noise, a violent blast blew open doors and windows and the lights went out. The dam had failed with the reservoir within 0.3 m of being full for the first time (Thomas, 1976). There were no witnesses of the collapse of the dam. After the failure, only the right side of the dam and the base of the central part remained. The surviving elements of the dam were cracked and the joints displaced. In its seven-mile course to the Mediterranean, the resulting flood wave caused 421 deaths and massive damage, including the destruction of the town of Frejus.

Post-failure investigation

A survey of remaining elements of the dam showed a rotation about a pivotal point on the right abutment. A plane of weakness in the left abutment and high uplift pressures were probably two major factors contributing to the collapse. A commission of enquiry appointed by the French government concluded that the construction work was good, particularly with regard to the quality of concrete and bonding of concrete with the rock foundation.

Lessons

The failure of the arch dam was principally due to geological factors and is explicable on the basis of rock mechanics. This indicates the paramount need at the feasibility stage of an arch dam project for appropriate site investigation and assessment under the control of professional engineers and geologists who are familiar not only with the local geology and rock mechanics but also with arch dam design. Since no one individual is likely to be an expert in all these areas, close collaboration between professionals in the different disciplines is essential.

Installation and monitoring of instrumentation, including piezometers and strain meters, are required to monitor deformations of a concrete arch structure and its foundation. First filling of the reservoir is a particularly crucial period for dam safety.

One important issue was highlighted by Karl Terzaghi who asserted that all foundation failures which had occurred despite competent ground investigation and appropriate construction control had one feature in common: "*The seat of the failure was located in thin weak layers*". He affirmed that normal ground investigation techniques did not provide adequate information concerning "*such minor geological details*". However, he considered that a conventional site exploration should have shown that the rock contained numerous joints, some filled with clay, and that from such data an

experienced engineer-geologist should have recognised that it was a potentially dangerous site (Jansen, 1980).

102. Vaiont (Italy)

Incident date: 9 October 1963

Description of dam

Height	265 m	Dam type	Concrete arch
Reservoir capacity	150,000 x 10 ³ m ³	Completed	1960

The double curvature arch dam is near Belluno in Venuto province of Italy. Although it is 265 m high, its crest length is only 190 m. When it was built it was believed to be the world's highest thin arch dam and the second highest dam of any type. The arch structure is 3.4 m thick at the crest and 22.7 m thick at the base of the dam. It is founded on massive limestones (Thomas, 1976).

Incident history

The instability of the valley side had been recognised when the project was planned. On 4 November 1960, when the reservoir retained water to a depth of 130 m, a slide of 700 x 10³ m³ occurred in the left valley side near the dam. Following this incident, the maximum water level was restricted and a network of survey monuments was established. Initially the slope moved at about 10 mm per week, but when in April 1963 the reservoir began to be raised, the creep movement accelerated. By mid-September 1963 the reservoir had been raised 20 m and many of the survey monuments were creeping at a rate of 10 mm per day. Subsequently the reservoir level was raised by another 10 m. On 8 October 1963 it was recognised that a vast mass of rock was moving and action was taken to lower the reservoir. Unfortunately, heavy rainfall had begun at the end of September and the large inflow reduced the effectiveness of this operation such that the total storage volume on the night of 9 October had increased to 120,000 x 10³ m³. By 9 October 1963 the movement rate had increased to 200 mm per day.

Incident description

At 22:15 on the night of 9 October 1963, a resident of Casso, a village on the right valley side 260 m above the Vaiont reservoir, was awakened by the roar of moving rocks. At 22:40 an air blast hit the building, breaking the windows. The roof was lifted and water and rocks came into the room. A vast volume of rock in the left valley side had slid into the reservoir. The 250-m thick slide mass moved 400 m horizontally and moved up the opposite valley side by as much as 140 m (Kiersch, 1988). The volume of the rockslide exceeded 240 x 10⁶ m³ and the material filled the reservoir for a distance of 1.1 miles up to heights of 150 m above reservoir level. The rock mass moved with a velocity as great as 30 m/s and created gigantic waves (Jansen, 1980). Reservoir water displaced by the slide material not only washed up the right valley side into the village of Casso, but also spilled over the dam to a height of 100 m above the crest. The ensuing flood wave reached Longarone at 22:43 and obliterated the town, which was located just one mile downstream of the dam. Some 2,600 lives were lost in this catastrophe.

Lessons

The disaster emphasises the significance of the potential instability of reservoir slopes in ensuring reservoir safety. At an early stage in pre-construction it is essential to understand the geology of the whole area which will be affected by the dam and its reservoir.

When a potentially hazardous instability is being monitored, it is important to measure pore water pressures and movements at depth as well as surface movements. Responsibilities must be clearly defined and there should be guidelines on the

evaluation of data, the point at which action must be taken, and the nature of any actions.

It is remarkable that in this catastrophe, which is one of the world's worst dam disasters, the dam itself suffered no major damage although the rockslide and the overtopping pressures exerted massive forces on the structure.

103. Folsom (USA)

Incident date: 17 July 1995

Description of dam

Height	103 m	Dam type	Concrete gravity
Reservoir capacity	$1,205,129 \times 10^3 \text{ m}^3$	Completed	1956

The dam was built by the US Army Corps of Engineers 25 miles from Sacramento, in North California. It is a 430-m long concrete gravity structure flanked by embankment dams. The reservoir provides flood control, hydro-electricity and water for drinking and irrigation. Five gates, 13 m wide by 15.2 m high, are normally used for flood control. Three additional gates on the dam crest are only used for emergency.

The design loadings for the two arms that support the skin plate of the gate comprise the hydrostatic load and the load from the chain when raising the gate. The two arms each comprise four struts with vertical bracing between the struts and diagonal bracing between the two lower struts, as shown in Figure 5.30 (Todd, 1997). The gates are operated by mechanical hoists with two chains attached to the upstream face of the skin plate.

Incident description

On 17 July 1995 one of the radial spillway gates failed during raising, resulting in an uncontrolled release of $1,133 \text{ m}^3/\text{s}$, some 40 per cent of the reservoir capacity with the reservoir level dropping 11 m before the breach was closed by stoplogs. There were no provisions for stoplogs so they had to be designed, fabricated and installed. No lives were lost, but there was considerable ecological damage with scouring of the river and the resulting increase in fresh water reaching San Francisco Bay causing premature fish migration into the river from the sea.

The right arm of the gate had buckled causing the gate skin to rotate, as shown in Figure 5.31. The two lower struts of the right arm were washed into the spillway. It was found that all of the braces in the vertical plane had failed in tension, with bolt shear being the failure mode.

Investigations

Prior to the failure, there had been no indication of any structural problems, although there had been concern about rusting of the gates over the six years before the failure. All the remaining gates were thoroughly inspected for signs of structural degradation. Some loss of structural integrity had occurred at some of the joints but corrosion of the structural components only had a minor effect on the failure of the gate.

Examination of the failed gate indicated that the diagonal brace joint adjacent to the trunnion (7 in Figure 5.30) was the initial point of failure. Failure of the first brace caused the next diagonal brace to be overloaded and failure of the struts in column bending. Trunnion friction moment was the key factor in the overloading and failure of the gate and had been omitted in the original design calculations. Over the years, corrosion had built up on the trunnion pins, increasing the friction and resulting in higher trunnion moments on the gate arms.

The gates had been operated for flood control since construction in 1956. The normal mode of operation was to raise them prior to the reservoir reaching the top of the gate.

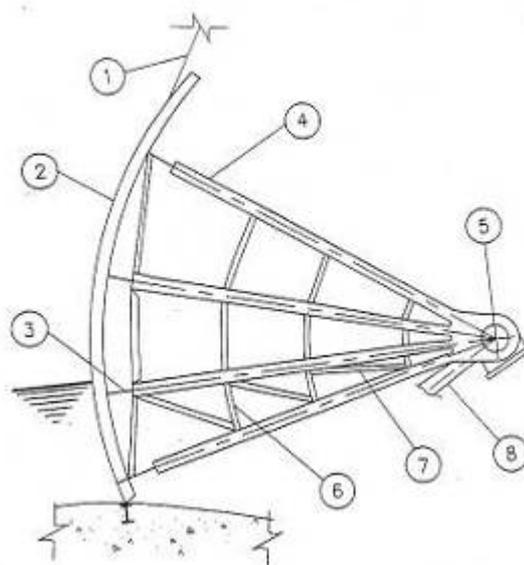
Remedial works

Inherent weaknesses in the original gate design were corrected by adding bracing and reinforcement. The trunnion pins were rotated 180° so that the uncorroded surface would be in contact with the bush, thereby reducing the trunnion moment. A preventative maintenance programme was introduced which includes a full-cycle operation of the gates with grease applied to the trunnions. The cost of repairs was 20 million US dollars.

Lessons

The failure was due to inadequate structural design aggravated by corrosion on the loaded side of the steel trunnion pins and vibration. As a result of the incident the following recommendations were made:

- All radial spillway gates that could involve loss of human life in the event of failure should be inspected thoroughly and design calculations reviewed to ensure the gates meet current design standards.
- Trunnion moments were not considered in many gate designs prior to the mid-1960s and therefore reinforcement may have to be added where trunnion moment was overlooked.
- To prevent possible self-excited vibrations which can cause fatigue damage and damage to the hoisting system, gates should be made adequately stiff.
- Monitoring of a gate's condition over time should be undertaken every five years and could include loading on the hoist, changing geometry of the arms and strain gauge data to check changes in the trunnion moment caused by corrosion.



1 – Chain; 2 – Skinplate; 3 – Girder; 4 – Strut; 5 – Trunnion; 6 - Vertical brace; 7 – Diagonal brace; 8 – Tie anchor.

Figure 5-29 Side view arm forming radial gate at Folsom dam prior to failure (after Todd, 1997)

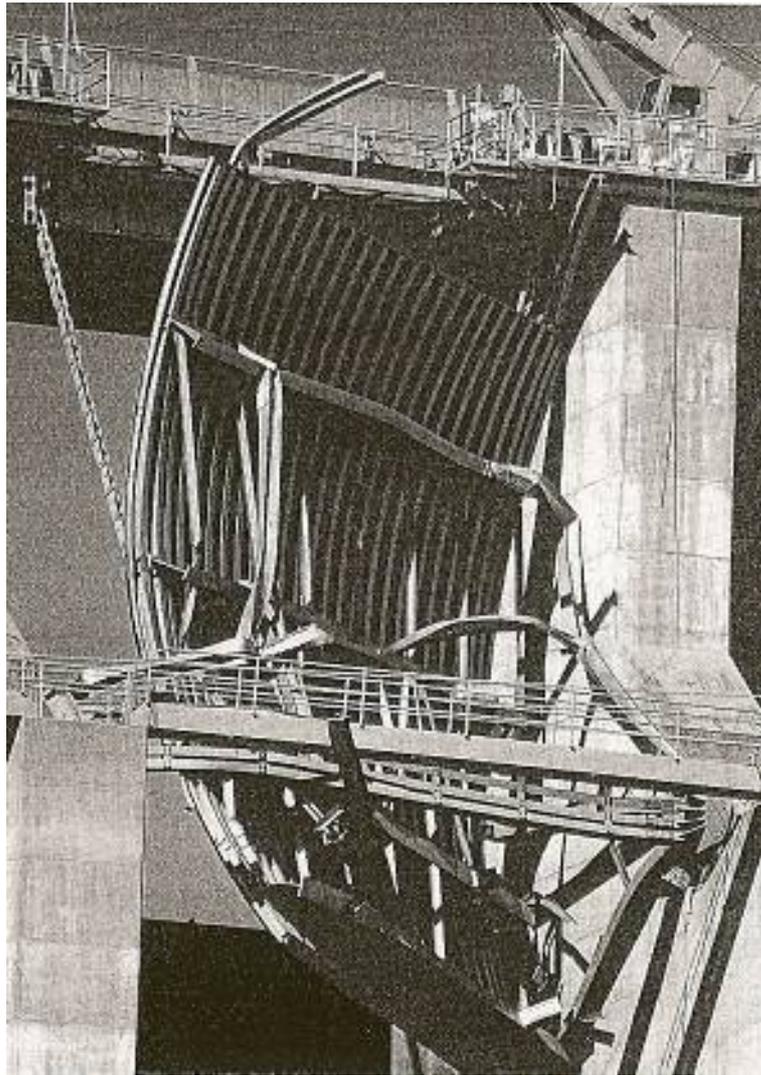


Figure 5-30 Failure gate at Folsom dam

104. Taum Sauk (USA)

Incident date: 14 December 2005

Description of dam

Height	16 m	Dam type	Rockfill
Reservoir capacity	5,366 x 10 ³ m ³	Watertight element	Concrete
Completed	1962	Cut-off	Grout

The Taum Sauk hydro-electric pumped storage scheme involving two reservoirs is located 90 miles south west of St Louis, Missouri. The upper banded reservoir consisted of a 2,000-m long, concrete-faced dumped rockfill dam. Only the upper four metres of rockfill were compacted. The upstream slope was 1.3:1 (horizontal: vertical) and the downstream slope was at the natural angle of repose of the material, approximately 1.3:1 as shown in Figure 5.32. The reinforced upstream concrete face had a design thickness of 300 mm but the actual slab thickness averaged nearly 450 mm due to the unevenness of the rockfill. The upstream concrete face had joints with copper waterstops located at the junctions with the parapet wall, the foundation cut-off slab and with adjacent face panels. Expansion joints between the slabs to accommodate movement, caused by settlement of the rockfill, used 20-mm asphaltic expansion joint material and U-shaped copper water stops. A reinforced concrete plinth

was provided at the toe of the concrete face. The entire reservoir bottom was sealed with two 50-mm layers of hot-mix asphalt concrete placed over levelled and compacted quarry fill. Around the edge of the asphaltic concrete, a single line grout curtain was constructed to limit seepage under the dam. The reservoir had been lined with a membrane in 2004 to minimize leakage.

The reservoir was constructed without a spillway. It had no direct catchment other than rainfall and was filled by pumping. The operation of the pumps and the reservoir level were regulated remotely using data from three water level transducers.

Incident description

The upper reservoir overtopped during the final pumping cycle the morning of 14 December 2005. Overtopping of the three-metre high parapet wall and subsequent failure of the rockfill embankment formed a breach 213 m wide at the top of the rockfill dam and 115 m at the base of the dam, as shown in Figure 5.33. Reservoir data indicated that pumping stopped at 5:15 with the initial breach forming at roughly the same time. Breach widening formed quickly, and complete emptying of the reservoir occurred within 25 minutes. The breach flow passed into the East Fork of the Black River (the river upstream of the Lower Taum Sauk Dam) through a state park and campground area and into the lower reservoir. Upon leaving the Lower Taum Sauk Dam spillway area, the high flows passed along the Black River to the town of Lesterville, located 3.5 miles downstream from the lower dam. The incremental rise in the river level was about 0.6 m which remained within the banks of the river.

The Taum Sauk facility's emergency plan was successfully implemented and the town was temporarily evacuated.

Investigations

Immediately following the incident, the regulator FERC (Federal Energy Regulatory Commission) established a dedicated webpage which provided briefings, updates and reports on the findings of the extensive investigations.

There had been an overtopping of the reservoir on 25 September 2005 which the owners did not report to the Commission until after the December 2005 breach. The owners discovered that one of the three transducers used to measure the water level in the upper reservoir was giving inaccurate readings. Further discrepancies in the reservoir-level monitoring system were found to be due to the failure of the anchoring system securing the level transducers. Planned repairs were not carried out before the failure occurred.

Although it should be assumed in design that all embankment dams will fail if overtopped, some rockfill dams are more sensitive to failure by overtopping depending on the steepness of the downstream slope, compactness of the rockfill, and percentages of sand and fines in the rockfill. Based on the appearance of breach slopes at the Taum Sauk rockfill, it was evident that the embankment in the area of the breach was not constructed as a normal rockfill embankment. Concerns had been expressed about the erosion resistance of the slopes due to rainfall during construction. The "dirty" rockfill found at Taum Sauk, which contained as much as 45 per cent sand plus fines, was not likely to be free-draining for the flows imposed by overtopping. Thus, the flows from overtopping could increase the phreatic levels beneath the parapet wall and within the downstream slope. In the case of a steep downstream slope of 1:1.3, the phreatic levels do not need to be increased much to cause slope instability. The increases in piezometric levels caused by the overtopping flows could have initiated stability failures of various portions of the slope and/or sliding and overturning of the parapet wall, as well as erosion. Once overtopping began, erosion would have started at the downstream toe of the three-metre high parapet wall,

causing instability and resulting in the initial loss of one or two sections of the wall and the sudden release of water over the downstream slope of the dam.

Remedial works

The dam has been rebuilt using roller compacted concrete (Rizzo *et al.*, 2009).

Lessons

The FERC report states: “*The breach was entirely avoidable in that the company knew for over two months that the water level sensors were unreliable, as they had broken free from their anchoring system, but unaccountably failed to make repairs*”.

Although failure of the dam was primarily due to overtopping resulting from the failure of the reservoir-level instrumentation to stop pumping, other contributory factors include the absence of a spillway, poor quality of rockfill that was not free-draining and was readily erodible, weak foundation conditions, steep angle of the slope and sudden failure of the three-metre high parapet wall. No regular visual surveillance of the reservoir was made to prove the reliability of the water-level telemetry readings.

Construction of any dam without an adequate spillway is a potential hazard to the safety of the dam, even when the reservoir is non-impounding.

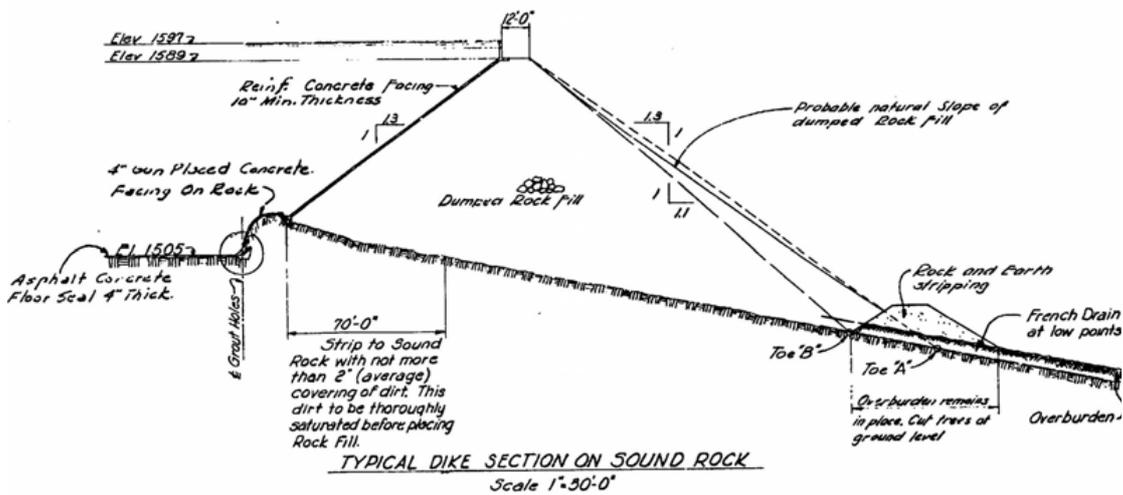


Figure 5-31 Typical section through Taum Sauk embankment



Figure 5-32 Breach through Taum Sauk embankment

5.3 Short summaries of additional incidents

5.3.1 Group 1: Internal erosion or leakage on first filling

1. Blackbrook	Incident date: 1799
Construction details	The 13-m high embankment dam was located in Charnwood Forest and built between 1795 and 1797 for the Charnwood Forest canal, part of the Leicester Navigation. It was engineered by Jessop. It had a vertical sided core only 1.8 m wide, apparently made of poor quality clay, described as riddled soil at a post-failure inspection. The core was only taken to 0.8 m below original ground level. The fill was described as loose rubbly material.
Incident description	The dam leaked and settled 0.45 m prior to failure. The collapse of the embankment dam was attributed to overtopping from snow melt, but it resulted from the crest settlement associated with internal erosion. The breach was repaired in 1801 but the dam failed again within a few months due to overtopping.
Response	The dam site was abandoned until a masonry dam was built in 1906.
Lessons	The incident provides one of the earliest examples of poor design and construction, where no attempt had been made to carry a watertight cut-off trench down to solid rock.
References: Kennard, 1972; Binnie, 1987a; Skempton, 1989.	

2. Redmires Lower	Incident date: 1850
Construction details	The 14-m high embankment dam, completed in 1849, was engineered by John Towleron Leather who also designed Dale Dyke. The shoulders are heterogeneous with predominately clay fill of glacial origin containing varying proportions of silt, sand and gravel cobbles and pockets of organic matter. The clay core was of similar material but without the coarser fraction. The dam was originally built with a 1.8-m internal diameter overflow drop shaft. The culvert was excavated in the foundations below the dam. A section of the dam is shown on page 260 of Binnie (1981).
Incident description	In November 1850, water escaped through the culvert and washed away part of the embankment. It is suggested that the mortar in the stonework joints was of inferior quality and that there was differential settlement causing cracking of the culvert. At the time cements were often of inferior quality to Lias lime, which is unsuitable for hydraulic structures but is still used sometimes.
Response	Following the incident, the vertical shaft was abandoned as a spillway and a spillway weir 7.6 m long was built, discharging to a bypass channel. The top of the shaft was raised and converted to a valve shaft with a 20-inch (0.5-m) standpipe and two 12-inch (0.3-m) inlets at different levels. The work was completed in April 1851.
Lessons	The incident illustrates the problems of spillways passing through or under embankments that are constructed of inferior materials.
References: Binnie, 1981; Swales, 1932; Claydon and Reilly, 1996.	

4. Rhodeswood Incident date: March 1858	
Construction details	Rhodeswood is one of the Longdendale dams engineered by Bateman. Clayey material was placed in 0.6-m layers in the upstream shoulder and rock gravel was placed in 1.2-m layers in the downstream shoulder. Grass sods were placed on both sides of the core, which had vertical sides and was only 3.5 m wide.
Incident description	After the reservoir had been full for a year, a sudden turbid leak developed at the foot of the dam and crest settlement occurred. The investigation found a sand-filled crack across the puddle clay core at a depth of 17 m, probably caused by hydraulic fracture.
Remedial works	In 1860, the defect in the puddle core was excavated and replaced with new puddle. Between 1974 and 1975, the core and foundations were grouted involving 1,500 m drilling and 150 tonnes of cement grout.
Lessons	A very narrow puddle clay core may be particularly vulnerable to internal erosion.
References: Binnie, 1981; Skempton, 1989; Bateman, 1884.	

5. Doe Park Incident date: February 1863	
Construction details	The 18-m high puddle clay core embankment dam was completed in 1861 for the Bradford Waterworks. A section of the dam as designed by John Wignall Leather is given in Binnie (1981) p124. Two adjacent masonry culverts, one metre wide, run to the valve well which is also used as an overflow. A single masonry culvert runs beneath the core and through the downstream fill.
Incident description	Serious leakage through the puddle clay core of the dam at a depth of 12 m occurred after the second filling of the reservoir in 1863. A sinkhole appeared in the crest 0.6-0.9 m in both depth and breadth, possibly due to a fault in the core.
Response	The location of the leakage was investigated by excavating a trench immediately downstream of the core while keeping the reservoir level several feet above the bottom of the shaft. Although this method was successful in identifying leakages, it was condemned as being too hazardous and was discontinued. Imperfections in the puddle wall were repaired on a number of occasions. The cut-off trench was deepened and extended into the hillside.
Lessons	Two main causes of the leakage are cited: differential settlement of the puddle clay adjacent to the masonry culverts where they crossed the puddle trenches; and the cut-off trench not founded on sound impermeable strata, nor the cut-off extended far enough into the hillside.
References: Binnie, 1981; Skempton, 1989; Wood, 1879.	

7. Grizedale Incident date: June 1867	
Construction details	The engineer for the 22-m high dam was Thomas Foster who had worked under Bateman on the rebuilding of Bilberry dam. It was built for the Fylde Water Board. A longitudinal section through the dam is given in Skempton (1989) p26. The cut-off trench was three metres wide with a maximum depth of 36 m. The bottom six metres of the cut-off trench in rock was filled with concrete and the upper part with puddle clay through what appears to be landslide material.
Incident description	In June 1867, when the reservoir had been full for seven months, large turbid leakage occurred: investigations showed that the leak was through the puddle clay cut-off trench at a depth of 24 m.
Response	The subsequent 1876 leakage was traced to a sand-filled cavity in

Lessons	the original puddle trench opposite the loose rock layer some 20 m from the first leak and about the same depth. Attempts between 1867 and 1883 to remedy leakage included driving adits, excavating and remaking the puddle clay core and cut-off. The long-term performance of the dam appears to have been satisfactory. This is probably an example of hydraulic fracture in a puddle clay filled cut-off trench.
References: Arthur, 1911; Skempton, 1989.	

8. Cowm Incident date: March 1877	
Construction details	The 16-m high puddle clay core dam with a puddle clay filled cut-off was engineered by Thomas Hawksley for the Rochdale Corporation. At the start of construction in 1868, the recent collapse of Dale Dyke in 1864 and consequent loss of life was in everybody's mind. The dam had a deep 5.5-m wide puddle clay filled cut-off trench which passed through open-jointed sandstone. The foundations for the dam are complex and included an open-jointed sandstone sandwiched between beds of shale. Rock joints in the cut-off trench were packed with cement and one poor section of sandstone was replaced with a blanket of puddle clay over concrete beneath the upstream slope.
Incident description	Not long after first filling, turbid leakage and crest settlement occurred. This is another case of the failure of a deep clay filled cut-off trench passing through open-jointed ground.
Remedial works	Hawksley dealt with the problem by rock grouting in front of the cut-off trench. He had pioneered the technique at Tunstall dam in County Durham in 1879. It was then used at Cowm later in 1879 and again in 1885-86 following discoloration of water in a well which was accompanied by subsidence in the hillside. The work was entirely successful.
Lessons	The remedial work showed that cement grouting was an effective way of making fissured foundations watertight.
References: Binnie, 1987; Skempton, 1989.	

9. Den of Ogil Incident date: 1881	
Construction details	This traditional puddle clay core dam built for the Forfar Waterworks required a cut-off trench through gravel as much as 24 m below the surface. The lower part of the trench was filled with concrete and the upper part with clay. The draw-off consisted of a 0.6-m cast iron pipe laid in concrete which was embedded in rock except for a short distance at the outer ends.
Incident description	On first filling, reservoir leakage appeared around the discharge pipe at its downstream end. The leakage took place through the very honeycombed concrete surrounding the pipe, leading to the conclusion that the pipe had fractured.
Response	To investigate the cause of the leakage, a heading was driven along the outside of the concrete surrounding the pipe to the concrete foundation forming the trench. The pipe was not fractured. Water was flowing through the concrete and up the downstream side of the concrete cut-off. Complex remedial works involved two additional trenches being excavated and backfilled with concrete, on either side of the original poor concrete.
Lessons	The incident arose because of poorly placed concrete.
Reference: Fox, 1898.	

10. Walshaw Dean Lower		Incident date: March 1907
Construction details	This is the lowest of three typical puddle clay core dams built between 1901 and 1907, some ten miles west of Halifax. The dams were engineered by G H Hill. The lower dam is 24 m high and has a three-metre wide deep puddle clay filled cut-off trench excavated in highly fissured sandstone terminating 40 m below the crest.	
Incident description	Leakage began soon after first impounding in 1907 and increased progressively. In February 1908 leakage increased with major loss of fill material. There was a series of leaks and repairs between completion of construction in 1907 and commissioning in 1915.	
Response	Grouting was carried out immediately upstream of the puddle trench between 1911 and 1915 involving some 173 tonnes of grout. A site investigation in 1980 showed softening of the puddle clay below original ground level and evidence of water paths. Remedial grouting was carried out in the rock downstream of the cut-off in 1982. Nevertheless, settlement continued at an annual rate of 10 mm. Detailed observations that were started in 1990 showed that the settlement was related to reservoir drawdown.	
Lessons	Poor performance of a puddle clay filled cut-off trench required remedial work in the form of grouting following first filling and then much later in the life of the dam.	
References: Barnes, 1927; Charles, 1989; Skempton, 1989; Tedd <i>et al.</i> , 2002.		

11. Walshaw Dean Middle		Incident date: March 1907
Construction details	Construction is similar to Walshaw Dean Lower.	
Incident description	On first filling, substantial settlement of the crest including wave wall and the upstream slope near the crest occurred mainly in the vicinity of the valve shaft. About 20 years later, a two-metre diameter hole appeared on the upstream side of the embankment. Investigations again indicated that the puddle clay in the cut-off was in an unsatisfactory condition.	
Remedial works	Grouting was carried out prior to bringing the reservoir into commission in 1915. It was confined to two areas: firstly, where the outlet tunnel goes through the core and secondly, where there is a steep vertical step of 9.1 m along the base of the cut-off trench. About 1,200 tonnes of cement was used in the grouting by means of 77 vertical boreholes. Extensive gravity grouting was also carried out in the foundation upstream of the core.	
Lessons	The incident highlights leakage problems associated with culverts passing through puddle clay cores and vertical steps in the bottom of cut-off trenches.	
References: Barnes, 1927; Charles, 1989; Skempton, 1989; Wood, 1946; Robertshaw <i>et al.</i> , 1998.		

12. Coulter		Incident date: 1912
Construction details	Coulter reservoir is in the Strathclyde Region of Scotland. The embankment has a maximum height of 24 m and was completed in 1907. It has a very thin puddle clay core with a maximum width of only 3.5 m.	
Incident description	The dam has a history of leakage and settlement. In 1912, sudden settlements occurred and turbid leakage occurred at the toe of the embankment. In 1968, examination of 10 years of movement records indicated that considerable movement had taken place and that there was a risk of sudden failure. Trial pits on the crest showed a settlement and downstream movement of nearly a metre.	
Remedial works	Grouting has been undertaken on a number of occasions and the	

Lessons	<p>top of the clay core was raised in 1936 with 280 tonnes of puddle clay. Following borehole investigations in 1969/70 which indicated the core to have a variable shear strength and the shoulder to be stable, extensive grouting was carried out in 1976/77.</p> <p>Hydraulic fracture of the narrow core was likely and possibly led to the initial internal erosion of the core. The cause of the long-term movements may also have been due to internal erosion; however, the effects of reservoir drawdown may have been a major factor.</p>
References: Charles, 1989; Gallacher, 1988c.	

15. Horndoyne	Incident date: 17 November 1990
Construction details	In March 1989, a planning application was submitted for the impounding of a 14 000 m ³ “pond” by a five-metre high embankment near Aberdeen. As the provisions of the Reservoirs Act 1975 only applied to reservoirs impounding in excess of 25,000 m ³ above natural ground level, the Act did not apply to the proposal. Impounding took place in late autumn 1990.
Incident description	The earth dam was breached during the night of 17 November 1990. Water had been seen to trickle along the side of the outlet pipe and this developed into a stream taking earth with it. Eventually a breach was formed and a wall of water, a metre or more deep, swept down the small valley. Four houses were flooded causing considerable damage to the buildings and their contents. A large residential caravan was swept over 100 m from its site, but there were no injuries to people.
Response Lessons	<p>None</p> <p>The failure illustrated the dangers posed by small reservoirs outside current reservoir safety legislation. Measurements made subsequent to the failure suggested a likely storage capacity of 23,000 m³, half as much again as the approved scheme and close to the 25,000 m³ threshold for the provisions of the Reservoirs Act 1975 to apply.</p>
References: Charles, 1991; 2005.	

5.3.2 Group 2: Internal erosion or leakage in service

16. Whinhill	Incident date: 11 November 1835
Construction details	The 12-m high embankment dam was completed in 1821. Robert Thom, who engineered the waterpower and town water supply for Greenock in 1827, described Whinhill as an ill-constructed embankment, the face next to the water being very steep, with no care taken to make it impervious to water or vermin.
Incident description	The dam failed in 1815 and again in 1835. The 1835 failure resulted in the loss of 31 lives and damage to the extent of six or seven thousand pounds. The dam is said to have failed as a consequence of vermin holes. Moles and water-mice had perforated the embankment like a riddle during the drought in search of water and when the flood came, it rushed through the holes with such force as to sweep away the embankment to its base in a few minutes. In contrast, the flood waters overtopped a dam built by Robert Thom without “injuring” it.
Response Lessons	<p>None</p> <p>The incident illustrates the dangers of a poorly constructed dam and the potential threat posed by animals.</p>
References: Binnie, 1987a.	

17. Monkswood	Incident date: 4 September 1931
Construction details	The 15.5-m high dam located near Bath was completed in 1895. It has a puddle clay core and puddle clay filled cut-off trench taken down to Lias Clay. It is founded on alluvium, peat and solifluction material overlying the Blue Lias Clay. The fill is described as loamy sand and clay and was difficult to place. The puddle clay core is three metres at the surface increasing to five metres at foundation level. Berms were placed on the upstream and downstream slopes to promote slope stability near the end of construction.
Incident description	A localised subsidence developed on 4 September 1931 on the downstream slope. The dam has a long history of leakage initially identified in 1931 and remedial works. Leakage occurred on the downstream berm particularly when the reservoir was within 0.3 m of top water level. The overriding safety concern was slope instability due to saturation of the downstream fill.
Investigations	Following local subsidence in 1931 on the downstream slope, an exploratory excavation revealed water flowing through the fill. It was concluded that the free-flowing water had led to internal erosion. Construction of a gauging chamber in the excavation showed that the flow rate greatly increased when the reservoir was within three metres of top water level. Trial pits in later investigations showed that the top of the core was lower than the overflow sill in places.
Remedial works	In 1931-1932 a grout mixture of cement and sand was injected at depths from three to 24 metres at various positions along the upstream side of the core. Approximately 280 tonnes of cement were used in 47 boreholes. The leakage rate reduced from 1.1 l/s to 0.1 l/s but by 1935 the leakage level had increased to the previous rate prior to remedial works. In 1945 sheet piles were driven six metres into middle of the core over the central part of the dam. This had no effect on the leakage. In 1998 a single phase slurry trench was constructed to a depth of 15 m along the full length of the dam. This appears to have been effective. Full details of the remedial works are described by Penman <i>et al.</i> (2000).
Lessons	Leakage was only significant when the reservoir was nearly full. Although it might have been associated with hydraulic fracture triggered by increased reservoir head, leakage over the top of the core seems more likely.
References: Fox, 1898; Penman <i>et al.</i> , 2000.	

18. Craig-Y-Pistyll	Incident date: 1939
Construction details	The 13-m high dam in South Wales was completed in 1877 and is founded on shale. The embankment is formed of a weathered shaley matrix and appears to have been built without a core or cut-off but a clay blanket was added in 1966.
Incident description	The reservoir was drawn down for repairs to the draw-off tunnel in 1939. The upstream face caved-in during the drawdown. The collapse on the upstream face was filled in. Leakage occurred following refilling of the reservoir.
Remedial works	The leakage was stopped by pressure grouting with 315 tonnes of cement and 14 m ³ of bitumen emulsion.
Lessons	Long-term internal erosion which had probably been going on for many years was not observed until the reservoir was drawn down.
References: Parkman, 1976.	

20. Cwmtillery	Incident date: 1954
Construction details	The 13-m high embankment was built in 1870 in South Wales. Although the dam was standing on a pillar of coal, this protection proved inadequate when deeper mining began in the 1950s. Mining subsidence severely damaged the culvert which had to be supported by steel arches. The puddle clay core was damaged by differential settlement and began to erode into downstream drains. Clay/cement injections in 1954 provided temporary support. More permanent works which were carried out in 1972 when mining was complete included a new lining to the culvert, replacement of the spillway and more extensive grouting.
Incident description	
Response	Deep coal mining has caused significant damage to a number of dams in the UK.
Remedial works	
Lessons	
Reference: Little, 1975.	

21. Barrow Compensation	Incident date: 10 July 1968
Construction details	The reservoir was constructed in 1863 to provide compensation water to mill owners. The 12-m high dam has a puddle clay core and puddle clay filled cut-off.
Incident description	The reservoir was emptied and abandoned in 1882 following several attempts to seal leakage through the foundation. It has normally remained empty since this time, but is prone to fill during floods. During the severe storms of 10 July 1968 the reservoir completely filled and water was seen to leak profusely from the area of the downstream toe during several hours when the reservoir was almost full. The main concern was the effect of leakage through and under the core and the potential for instability of the downstream shoulder during flood events.
Remedial works	In the years leading up to its abandonment in 1882, the core wall and trench were replaced but the leakage continued. In more recent years, a clay blanket was placed on the upstream face of the dam in 1982 and this also proved unsuccessful. The dam was eventually rendered relatively watertight in 2005 following a site investigation which confirmed high foundation permeability and a puddle clay core prone to hydraulic fracture. The core was sealed and widened by grouting works and the foundation was also grouted. The reservoir outlet pipe was closed to allow the reservoir to fill and the effectiveness of the remedial works was tested under controlled conditions with careful monitoring of the embankment stability.
Lessons	The method of abandonment in 1882 was inadequate to prevent the reservoir from filling in the event of a flood and created a hazardous situation with the potential for instability of downstream shoulder.
References: Heaton-Armstrong, 1984; Warren <i>et al.</i> , 2006.	

22. Toddbrook	Incident date: 1977
Construction details	The 24-m high dam consists mainly of boulder clay with sands and gravels. There is doubt about the existence of a puddle clay core even though it is shown on the original construction drawings. The dam is founded on fluvio-glacial sand and gravels, glacial till overlying a faulted sequence of mudstones, sandstones and shales of the Millstone Grit Series and Lower Coal Measures.
Incident history	The dam has a history of leakage. Since 1880, there were complaints about leakage into mine workings. In 1930 leakage was observed at the toe of the downstream slope. As a result of an

Incident description	investigation into the leakage, a depression was found on the upstream slope. This was investigated in 1931 and the area was then reinstated.
Investigations	In November 1975 when the reservoir was low, a depression was noted in the same position on the upstream face as the 1931 depression. In Autumn 1977, 120 mm of subsidence was measured since 1975. The reservoir was emptied to inspect the full extent of the depression and revealed a crater approximately four metres across at the upstream toe partly infilled with silt and into which a tree appeared to have been sucked.
Remedial works	Extensive investigation included boreholes, sampling and piezometers. Exploratory shafts were sunk on the upstream and downstream faces between 1978 and 1980. In 1981, a 1.2-m diameter masonry culvert was found beneath the dam, possibly for stream diversion during construction. Tracer tests showed this to have formed a leakage path through the dam.
Lessons	In 1981, a compacted clay blanket was placed over the suspect area of the upstream toe and the bed of the reservoir. To solve the leakage problem, a single row grout curtain 60 m long within the clay core was formed using the tube-à-manchette system. The reservoir was refilled in December 1983.
References:	Until the reservoir was drawn down, the extent of the crater caused by erosion was unknown. The good practice of periodic inspection of the upstream face of a dam is illustrated by this incident.
References: Anon, 1977; Anon, 1978; Binnie, 1987.	

23. Oakdale Lower	Incident date: November 1986
Construction details	The 11-m high embankment was completed in 1890 in North East Yorkshire near Osmotherly. The shoulders consist of a medium coarse sand downstream and a more clayey material in the upstream shoulder. The puddle core is founded on mudstone over much of its length, but at the north end of the dam adjacent to the culvert it appears to terminate in loose sand.
Incident description	In November 1986, a major sinkhole appeared in the centre of the embankment just downstream of the puddle clay core.
Response Investigation	The reservoir was immediately drawn down. Detailed investigation involving nine boreholes, 18 probe holes and three trial pits revealed extensive voids in the clay core consisting of very soft clay of high organic content. Undrained shear strengths were less than 7 kPa. There were high pore pressures in the downstream fill.
Remedial works	The dam was discontinued.
Lessons	There were no warning signs that internal erosion was taking place until the sinkhole appeared. The reason for the internal erosion appears to be poor quality puddle clay.
Reference: Gallacher, 1988c.	

25. Kellington East	Incident date: January 2008
Construction details	The homogeneous earth embankment constructed of compacted clayey material is part of an eight-mile length of riverside embankment which is part of the reservoir along the right bank of the lower section of the River Aire, between the villages of Beal and just west of Rawcliffe.
Incident description	Following very high river levels which reached the crest of the embankment, a five-metre wide partial breach occurred in the three-metre wide embankment crest, causing it to erode to a maximum depth of about 1.5 m. Had the river remained high, further erosion of

Response	the embankment could have occurred causing significant damage to an adjacent pumping station and its power supply. A sluice gate adjacent to the breach location was opened to allow the reservoir to fill from the river in an attempt to equalise water levels on both sides of the partial breach and a contractor was put on standby.
Investigation	Examination of the breach showed that water had penetrated the embankment via animal holes/burrows, leading to its rapid erosion. At the time of the incident, water spouts had occurred on the inward face of the embankment at two other locations, which were found to have come from mole holes.
Lessons	This incident highlights the damaging and potentially disastrous effects of burrowing animals on embankments, particularly those that are used for flood defence reservoirs adjacent to rivers.
Reference: Environment Agency, 2009.	

5.3.3 Group 3 - Internal erosion or leakage in service associated with ancillary works/cut-offs/abutments

27. Pentwyn	Incident date: 1882
Construction details	The 9.5-m high dam was completed in 1863 under the advice of Thomas Hawksley. Although most of the dam was founded on impervious Old Red Sandstone, part of the dam was founded on Carboniferous Limestone to an unknown depth. The cut-off trench was filled with clay.
Incident description	Serious leakage exceeding 500l/s was accompanied by settlement.
Response	Although the date of the leakage is not recorded, various repairs were undertaken between 1882 and 1923, including drilling and grouting, but improvement was only temporary. The dam was replaced in 1927.
Lessons	Carboniferous Limestone with its open joints is a formation to be avoided at the base of a reservoir. This and other examples of internal erosion led to the practice of lining the bottom and sometimes the sides of the cut-off trench with brickwork and concrete as deemed necessary.
Reference: Binnie, 1987a.	

28. Roddlesworth Upper	Incident date: 1904
Construction details	The 21-m high dam, completed in 1865, is one of the Rivington Reservoirs which were built for the Liverpool Waterworks. The scheme was designed by Thomas Hawksley. The dam, designed by Duncan, is founded on fissured shale and to find solid rock, the cut-off trench was taken down to 40 m in one place. Springs were encountered during the excavation of the cut-off trench. The top width of the core is 1.8 m with batters of 1:12.
Incident description	In 1904 a sinkhole appeared upstream of the core just below top water level. Further subsidence occurred two years later and in 1908 a new sinkhole appeared, 30 m from the first one, again in front of the core extending to a depth of 10 m.
Response	Following the appearance of the second hole, excavation revealed a circular shaft 0.9 m in diameter. On excavating to 10 m, a small spring was found. Examination of the records revealed that difficulties encountered with a spring at the bottom of the cut-off trench had been overcome. Running water was also found at the base of the 1908 sinkhole.
Lessons	Skempton (1989) suggests it was a case of hydraulic fracture

developing 40 years after construction. The incident shows that even where deep trenches filled with puddle clay appear successful for some years after first filling, long-term erosion can continue and manifest itself many years after construction.

References: Binnie, 1981; Skempton, 1989.

29. Bottoms (Macclesfield)		Incident date: May 1929
Construction details	The nine-metre high embankment was constructed in 1850 for water supply for Macclesfield. The embankment had a wide puddle core of good quality but the embankment was fine loamy sand. The outlet consisted of two 300-mm diameter cast iron pipes 0.5 m apart laid directly under the embankment at approximately ground level.	
Incident description	The dam was founded on boulder clay overlying broken gritstone. In May 1929, with the reservoir less than half full, a slip developed in the downstream slope. The slipped portion was saturated with water.	
Response	The reservoir was emptied immediately. Investigation involved making a cut into the downstream shoulder of the embankment on the line of the pipes. Both cast iron pipes were broken close to the flanges near to the upstream side of the puddle core and the broken ends had separated by 50 mm. Other breaks had taken place. The embankment had settled such that the top of the core was 0.5 m below top water level.	
Remedial works	The remedial works involved construction of a culvert of pressed steel and concrete in the cut made for the investigation, a new outlet main, raising the top of the core and embankment, drainage to the outer slope and cement grouting beneath the Millstone Grit below the embankment.	
Lessons	It was concluded that the slip was primarily due to water flowing over the top of the puddle when the reservoir was full.	
Reference: Cover, 1931.		

30. Holden Wood		Incident date: 1945
Construction details	This 17-m high embankment dam was built in 1841.	
Incident description	In 1945, a large hole two metres square by 1.2 m deep appeared in the crest vertically above the outlet culvert.	
Response	Inspection showed that a masonry stopwall across the culvert downstream of the core had many leaks and some masonry blocks were missing. Attempts to plug the leakage by quick setting grout proved unsuccessful. A final repair was done by building a new concrete stopwall and grouting the interspace and surrounding rock.	
Lessons	It was concluded that the masonry wall was protecting the clay core and that leakage had eroded part of the core. This is an example of slow progressive internal erosion into a draw-off structure.	
Reference: Seddon, 1975.		

31. Island Barn		Incident date: 1950
Construction details	The reservoir is formed by earth embankments with a puddle clay core and a cut-off trench keyed into the underlying London Clay. The embankments were completed in 1911 with a maximum height of nine metres. During the passage of the Bill for the construction of the reservoir through parliament, an undertaking was given to plant a portion of the bank with trees that included a mixture of sycamore, wild cherry, willow laburnum, false acacia and lime.	

Incident description	In 1950, 40 years after construction, settlement was observed on either side of the outlet at the location of the planted trees.
Response	Trial pits dug to determine the extent of the roots found that many roots had penetrated the puddle core. Some were 100 mm diameter. Roots were found to a depth of nearly two metres below the top of the core. The greatest distance from the puddle core wall of any tree whose roots had penetrated it was about eight metres.
Remedial works	Roots were cut out of the core which was then repaired. Over 200 trees were removed including all willows and all trees within nine metres of the top of the bank.
Lessons	The planting of trees with extensive root systems such as willows should be avoided on embankment dams, particularly where the fill allows the roots to spread. The adverse effects of trees should be considered when obtaining planning for reservoir construction especially when considering their use as mitigation in an environmental impact assessment. The desiccation and cracking in the upper portion of the clay of core of King George V in the Lea Valley was attributed partly to plant roots which had been driven down by the five years of lowered water level during the second World War.
Reference: Cronin, 1951.	

33. Slade Lower	Incident date: 1970
Construction details	The 15-m high embankment was constructed in 1889 to supply water to Ilfracombe in Devon. The spillway channel is at the north abutment, and in 1970 it was proposed to repair defective brickwork to the spillway channel at the end of the central puddle clay core.
Incident description	Removal of the spillway channel wall during remedial works revealed a small cavity which was connected to a sinkhole beneath the pitching at about top water level. The top of the sinkhole was 0.9 m in diameter and was completely hidden by the pitching.
Response	A borehole investigation revealed "open conditions" (high permeability) at the base of the core and connections to the reservoir when the borehole reached the shillet foundation.
Remedial works	In 1971 grout holes were drilled through the core and three metres into the underlying rock. Leakage into the underflow drain was reduced by a third.
Lessons	Cavities caused by internal erosion must have been present under the core into which considerable quantities of grout were injected, but until the accidental discovery of the sinkhole, it was believed that the dam was satisfactory. Internal erosion had occurred over a long period without disastrous results and without showing any adverse symptoms.
Reference: Kennard, 1972.	

36. Holmestyes	Incident date: 1993
Construction details	The 24-m high puddle clay core dam is one of three dams engineered for the Holme Reservoirs Commissioners by George Leather, one of the others being Bilberry which failed in 1852. Following the failure of Bilberry and concerns about leakages at Holmestyes, Captain Moody (government engineer at Bilberry inquest) inspected Holmestyes and found the valve shaft and culvert to be leaking and running "considerably muddy". The dam was modified by Bateman in 1857 who added a puddle clay blanket to the upstream slope and adjacent sides protected by sandy fill and pitching. The valve shaft is just upstream of the central puddle core such that the full reservoir head would have been acting on the

Incident history	shaft and culvert prior to adding the clay blanket. Prior to adding the upstream blanket the hydraulic gradient across the central clay core would have been 5.7, but this was considerably reduced on adding the blanket in 1857.
Incident description	The dam has a long history of leaks into, and repairs of, the valve tower and culvert: 1938, 1939, 1944, and 1992. In 1944 the total leakage amounted to 0.63 l/s. Geotechnical investigations involving measurement of pore pressures in the upstream fill were carried out in 1982 and again in 1991 to determine the effectiveness of the upstream clay blanket. Measurements showed that pore pressures were much less than reservoir head and therefore the possibility of the upstream clay blanket being damaged due to excess water pressure on reservoir drawdown was unlikely at that time.
Remedial works	A rapid increase in leakage occurred in 1993 at higher levels in the shaft. Also, silt was being deposited in the shaft. The rapid increase in leakage into the valve tower was associated with a long period of heavy rain and build-up of water levels in the upstream fill.
Lessons	Remedial works to stop leakage into the shaft and tunnel included: 1939 - radial grouting from within the shaft, strengthening the culvert with steel arches, grouting and 50-mm reinforced mortar skin. 1944 - caulking of joints in the shaft. 1998 - tube-à-manchette grouting from surface 1.2 m outside valve shaft to seal leaks. Remedial works to scour valve and pipework.
References: Binnie, 1981; Dyke and Williams, 1998; Tedd <i>et al.</i> , 1993.	

37. Walshaw Dean Upper	
Incident date: January 1997	
Construction details	This is the upper of three typical puddle clay core dams built between 1901 and 1907, some 10 miles west of Halifax. The dams were engineered by G H Hill. The upper dam is 12 m high at the location of the incident and has a three-metre wide puddle clay filled cut-off trench excavated in highly fissured sandstone terminating 35 m below the crest level.
Incident description	In January 1997, a one cubic metre hole was discovered on the upstream slope close to the crest of the embankment. As internal erosion of the clay core and the clay filled cut-off trench had occurred at the two dams immediately downstream, there was concern that the hole in this dam was also due to internal erosion.
Response	The reservoir was drawn down two metres. A lengthy investigation involving cone penetrometer, continuous sampling, piezometers, pressure cells and a trial pit at the location of the sinkhole was undertaken. No cause for the formation of the hole could be found.
Lessons	Holes, presumed to be sinkholes, have occurred at a number of dams. Although the initial concern is that they are due to internal erosion, especially when they are located near the crest, other mechanisms can create holes such as wave action, drainage works or localised construction defects.
Reference: Robertshaw <i>et al.</i> , 1998.	

38. Anglezarke (Heapey)	
Incident date: November 1997	
Construction details	Heapey is one of three dams forming Anglezarke reservoir which is part of the Rivington Reservoir group constructed in the 1860s.
Incident history	The dam has suffered a history of leakage from the downstream eastern abutment. A 0.45-m diameter cast iron pipe to supply White

Incident description	Brook to the north of the dam was grouted in 1970 to eliminate a possible cause of leakage. Routine surveillance of the dam on 26 November 1997 during refilling of the reservoir revealed a significant issue of clean water in the east mitre.
Response	An emergency drawdown was instigated to reduce the level by three metres. Emergency pumps were delivered to site and used for the first 48 hours of the drawdown.
Remedial works	A ground investigation was done and piezometers were installed. The leak was thought to be associated with the draw-off pipe and grouting was concentrated in this area. However, leakage reappeared on refilling the reservoir to within 0.5m of top water level.
Lessons	Frequent surveillance of reservoirs being refilled is vital. Although the leakage was thought to be associated with the draw-off pipe, grouting in the vicinity was not effective; grouting of the rock abutment probably cured the leakage.
References: Page, 2002; Parks and Walthall, 2002.	

40. Carno Lower	Incident date: January 2005
Construction details	The 27-m high puddle clay core embankment dam is located close to Ebbw Vale in South Wales. It was completed in 1911 to the designs of G F Deacon who died before its completion. The supply outlet was permanently sealed in 1933. A new draw-off system was constructed through the hillside upstream of the right abutment. The original scour and supply conduits were laid in a dry culvert with control valves at the base of the shaft just upstream of the core. The valve shaft was converted from a dry well to a wet well in 1933 because of serious leakage into the shaft. The original scour remains in use. The dam is founded mainly on mudstones and siltstones of the Lower Coal measures over which lies a glacial till.
Incident history	The dam has a history of leakage through the abutment and into the draw-off structures since first filling. Remedial works have included grouting on a number of occasions, provision of additional drainage at the left mitre and construction of new outlet works. Hydraulic fracture is a possible mechanism that initiated the leakage.
Incident description	On 20 January 2005 leakage flows at the toe accelerated rapidly and became turbid. The leakage flow had an estimated silt content of one per cent
Response	A controlled emergency drawdown was done which resulted in a rapid increase in crest settlement as the reservoir was emptied. As much flow as possible was diverted from Carno Upper through an existing adit. The rate of drawdown was increased following a rise in the leakage flow during investigations into the source of the leak and again when crest settlement occurred. Investigations indicated that the leakage and erosion were associated with the outlet culvert.
Remedial works	A cement bentonite slurry wall was constructed for the full depth of the core and extensive grouting was undertaken in the foundation, at the core/foundation interface, around the culvert and adjacent to the overflow on the left abutment. During construction of the wall, loss of slurry occurred at locations which would be consistent with voids that had formed at zones of hydraulic fracture caused by the presence of the draw-off culvert which protrudes into the clay core.
Lessons	Internal erosion of the core is often associated with a culvert passing through the core.
Reference: Rowland and Powell, 2006.	

5.3.4 Group 4: Incidents due to pipe or valve failure

41. Knypersley	Incident date: June 1828
Construction details	The 16-m high embankment was constructed in 1827. The shoulders consist of broken and weathered mudstone fill. A section is given on page 126 in Binnie (1987). The dam is founded on alluvium overlying glacial till in the valley bottom. Underlying these deposits are Lower Coal Measures of the Upper Carboniferous. The cut-off was eight to ten metres deep. In 1829 the depth of the cut-off at the west end of the embankment was increased to seal the core into the solid shale through the sand and gravel bed.
Incident description	In June 1828, the masonry wall at the core end of the upstream culvert fractured and joints in cast-iron pipes through the core pulled apart due to settlement of the core.
Remedial measures	The discharge pipes were replaced with thicker pipes where they had been pulled apart at joints. The masonry walls of the culvert were made thicker close to the core.
Lessons	Pipes passing through clay cores are vulnerable to embankment movement.
Reference: Binnie, 1987	

42. Torside	Incident date: November 1854
Construction details	Torside is one of the Longdendale reservoirs engineered by J F Bateman with construction beginning in 1849. The cast iron outlet pipes were sunk a considerable depth into solid ground below the base of the embankment and embedded in concrete except at the centre where they were supported by puddle.
Incident description	During first filling, when the reservoir was within three metres of being full, on the 17 November 1854, considerable quantities of water emerged on the downstream slope resulting from the fracture of both discharge pipes.
Remedial works	Investigations showed that the base of embankment had stretched such that elongation was up to 1.5 metres in the south range of pipes. About 4.5 m of pipe were crushed into an elliptical form. Some joints were pulled apart. The probable cause of movement was the existence of a bed of hard clay 1.5 to eight metres thick in the valley which allowed movement of the embankment. Repairs were made to the pipes for temporary use. To avoid future problems, a tunnel through the abutment with two pipes was constructed. A new puddle trench was sunk into the clay near the foot of the upstream slope and the slope was lined with a clay blanket. In 1889, new siphon valves were installed and repairs were made to the pitching.
Lessons	Unprotected outlet pipes within the foundation can move with the embankments.
References: Bateman, 1884; Binnie, 1981.	

43. Longwood Compensation	Incident date: November 1988
Construction details	The 13-m high embankment was completed in 1828 and enlarged about 1845. It is reported to have a puddle clay core. The two draw-off and scour pipes are laid directly through the embankment and according to Hallas and Kennard (1982) were to be investigated.
Incident description	A localised hole some 300 mm deep was found on the downstream slope. Water was entering through the brickwork of the valve

Response	chamber on the line of the draw-off pipe, downstream of the hole. The reservoir was drawn down. An excavation was undertaken to investigate the source of water. A complete circumferential crack was found in a draw-off pipe on the downstream side near to the spillway, about 2.5 m below the surface.
Remedial works	Repairs were carried out to the pipe which was subsequently lined.
Lessons	The fracture of an unprotected pipe in an embankment dam gave the potential for erosion of fill and slope instability if no action had been taken. The pipe had, however, performed satisfactorily for 160 years. However, the cause of the fracture may have been other remedial works on the dam.
Reference: Hallas and Kennard, 1982.	

5.3.5 Group 5: Slope instability during construction

45. Spade Mill No.1	Incident date: 1860
Construction details	Spade Mill Reservoir No. 1 was completed for the Preston and District Water Board in 1862. Spade Mill No. 2 was built between 1952 and 1959.
Incident description	An upstream slip occurred during construction at the south-east corner of the embankment due to the unstable nature of the fill. An upstream slip also occurred during service in 1964 at Spade Mill No. 2. The slip passed through the fill and boulder clay below.
Response	Two berms were added to the upstream side and the slope was reduced from 1:3 to 1:4.
Lessons	This is a typical example of a slope instability failure during construction that was remedied by the addition of berms.
References: Ayres, 1994; Bowtell, 1988; Hopkins <i>et al.</i> , 1991	

46. Alston No 1	Incident date: 1860
Construction details	Alston No. 1 was the third of a group of three reservoirs to be constructed in 1922-33 near Longridge outside Preston. The fill for the bank and puddle clay core were dug out of the bed of the reservoir, supplemented by some fill from an adjacent site.
Incident description	Slips which occurred during construction when the embankment was eight metres high caused collapse of the discharge pipe tunnel and fractured the pipe. Lateral movement of the southern embankment at the same time caused total collapse of the tunnel and fractured the 30-inch pipe.
Response	The works were completed in 1931 and satisfactory reservoir filling was achieved by 1933.
Lessons	Embankments at Spade Mill and Alston reservoirs suffered slips during construction and have continued to show signs of instability.
Reference: Bowtell, 1988.	

47. Bartley	Incident date: 1927
Construction details	Bartley reservoir is south west of Birmingham and was built for standby storage as part of the Elan Valley supply project. The 20-m high embankment was constructed between 1925 and 1930. It had a thin vertical lightly reinforced concrete core. The embankment shoulders are described as clay fill from glacial and post-glacial deposits. The specification required the fill to be deposited in layers and well-rammed and consolidated using heaving wooden pounders and rollers, although there was no evidence of this in construction

	<p>photographs. At intermediate construction stages the surface of the fill sloped towards the core, resulting in water pounding on the surface of the fill.</p>
Incident description	<p>Three slips of the upstream slope occurred during construction of the embankment at heights of 10 m, 15 m and at 18 m.</p>
Response	<p>Remedial works were carried out after each slip and involved the installation of ferro-concrete piles and placement of a substantial amount of stone on the upstream slope to form a benched profile with an overall slope of 1:4.7 (12°) compared to the designed slope of 1:3.</p>
Lessons	<p>Lack of compaction and drainage of the clay fill during construction influenced stability. The methods employed to stabilise the upstream slope were typical of those used at other dams. The reservoir has performed satisfactorily since completion but has rarely been drawn down except for a few occasions of less than two metres. Concerns about stability following an inspection in 1986, two years after the failure of Carsington, prompted a detailed investigation of the upstream fill to assess its stability on reservoir drawdown.</p>
<p>Reference: Binnie, 1992.</p>	

48. Abberton	Incident date: 20 July 1937
Construction details	<p>The 16-m high dam was built south of Colchester between March 1936 and August 1938. The embankment is of traditional design with a narrow puddle clay core and puddle filled cut-off down to the underlying London Clay. The original upstream slope was 1:4. The central 250 m of the dam is founded on six metres of recent alluvial deposits (clay, sands, gravels and a thin layer of peat) overlying London Clay. The inner shoulder next to the core is clay and outer shoulders of the dam are sands and gravels.</p>
Incident description	<p>A major deep-seated slip of the upstream slope took place during construction on 20 July 1937 with the embankment within two metres of the planned height. The dam crest dropped by 3.5 m and the upstream toe moved outward by 15 m. A survey of the slip indicates that it could have passed through the alluvial foundation.</p>
Response and reconstruction	<p>The slip took place nine days before the well-documented failure at Chingford which was investigated by Karl Terzaghi. As far as is known, Terzaghi did not become involved in Abberton and there is no record of soil mechanics principles being applied in the original design of the dam or its reconstruction with much shallower slopes. The original upstream slope of 1:4 was changed to between 1:7 and 1:11. The amount of disturbed material that was removed is uncertain. A sheet piled concrete toe was constructed on the upstream slope. Since completion, the dam has performed satisfactorily. Geotechnical investigations between 1995 and 1997 were carried out to provide data to design the raising of the dam.</p>
Lessons	<p>Back analysis of the construction slip (French <i>et al.</i>, 2000) indicates that failure was caused by high pore pressures in the foundation and like Chingford, this was one of the first dams to be built using modern earth-moving equipment, where rapid construction did not allow sufficient time for pore pressures to dissipate.</p>
<p>Reference: French <i>et al.</i>, 2000.</p>	

52. Tittesworth	Incident date: May 1960
Construction details	<p>The 15-m high dam just north of Leek in Staffordshire was completed in 1858 and raised slightly in 1870. The puddle clay core is supported by shoulders of compacted clay and shale, and was</p>

Incident description	believed to be founded on soft blue shale. The proposed construction of a new 30-m high dam with a rolled clay core immediately downstream involved the new dam having its centre line at the downstream toe of the old dam and incorporating the old dam into the upstream shoulder of the new dam.
Investigations	Construction of the new dam, which began in 1960, involved removal of a berm on the downstream side of the old dam and trimming the downstream side to uniform 1:2.5 slope. Before completing the excavation, the old embankment suffered a wedge failure comprising shearing through the puddle clay core, dropping of the crest 4.9 m and pushing outwards.
Redesign of new dam	Using values of the angle of shearing resistance derived from the original investigation in a back analysis of the stability did not account for the instability. A search of the construction records showed that slips had occurred during construction of the old dam. Further investigation of the slip revealed that the dam was founded on a black organic silt with a high moisture content and low strength that could account for the slip.
Lessons	As a result of the incident, the new dam was redesigned with a concrete wall instead of a rolled clay core which required less excavation of the toe of the old dam.
	The first site investigation failed to identify foundation weaknesses of the original dam. This case shows the importance of knowing the history of problems and remedial works. Despite the marginal stability of the embankment it performed satisfactorily for 100 years, until excavations disturbed the fragile equilibrium.
References: Little, 1997; Twort, 1964.	

5.3.6 Group 6: Slope instability in service

54. Earlsburn	Incident date: 24 October 1839
Construction details	The dam was built for a consortium of mill owners south west of Stirling, probably not long before 1839. It was formed of peat and earth with a narrow clay core of silty clay. The core extended down to rock but most of the dam was founded on peat. A survey of the dam pronounced the dam to be " <i>in a proper state of repair</i> ".
Incident description	The dam collapsed some eight hours after an earthquake of magnitude 4.8 M_L (Richter) which damaged many houses in Comrie. The resulting flood from the dam caused damage in excess of several thousands pounds, but no lives were lost.
Response Lessons	None known. The age of the incident makes it difficult to be certain that the incident was due to slope instability associated with the earthquake. A notable feature of the earthquake was the relatively large number of instances of ground slips and similar effects. It seems likely that the Earl's Burn collapse was an accident waiting to happen to a poorly built six-metre high embankment dam waterlogged after two days of heavy rain. Musson (1991) concludes it is reasonable to suppose the earthquake triggered the dam burst, even if it was not the principal cause.
Reference: Musson, 1991.	

56. Roddlesworth Upper	Incident date: 21 January 1954
Construction details	The 21-m high dam completed in 1865 is part of the Rivington scheme. The dam had suffered significant internal erosion of the deep cut-off which is described in Incident No. 28. The toe had been built of sand.

Incident description	On the night of 20-21 January 1954, heavy rains resulted in large amounts of run-off from the high ground from both sides of the valley onto the berm on the downstream slope. The sand was said to turn into a "running sand" when inundated by water. The sand toe was partially washed away and a slip occurred. The incident is somewhat similar to that at Lambieytham (Incident No. 61)
Remedial works	An effective drainage system was installed both on the downstream slope of the embankment and on the high valley sides to convey the water away from the embankment.
Lessons	An erodible sand toe provided unstable conditions. Surprisingly, the toe of the embankment was apparently stable for nearly 90 years.
Reference: Binnie, 1981.	

57. Auchendores	Incident date: January 1968
Construction details	The 10-m high earth dam is in the Strathclyde region.
Incident description	Heavy spray from waves caused downstream slope instability. The incident was very similar to Blithfield (Incident No. 84) but there was no report of damage to the upstream pitching.
Response	Urgent action was taken to draw down the reservoir and increase the toe weight of embankment. Drawing down the reservoir was impeded because the draw-off was closed with a timber bung.
Lessons	The incident illustrates the need for an effective wave wall, particularly where the slope stability could be marginal.
Reference: Hamilton, 1975.	

58. Buckieburn	Incident date: 1 November 1970
Construction details	The 23-m high embankment was built in 1905 and is situated north of the Carron Valley in Stirlingshire. The upstream slope is 1:3 and the downstream slope is 1:2.6. The drawings show the dam to have a puddle clay core but this was not identified in a later borehole investigation and the embankment appears to be of moraine fill.
Incident description	A substantial shallow slip of the downstream slope occurred during a period of heavy rain and high winds. The slide appears to have been initiated by heavy rain combined with flow of surface water in the mitre and water from wave action overtopping the parapet wall at the crest of the dam. There was no reported damage to the upstream pitching from the wave action.
Response	The reservoir was immediately drawn down. Flows from the two aqueducts that supplied the reservoir from the extended catchment areas were diverted. The reservoir was maintained three metres below top water level during the remedial works.
Remedial works	A rockfill toe was added after removing 2.5 m depth of peat. The soft slide material was removed and a berm of material was added to reduce the slope of the lower two-thirds of the downstream slope from 1:2.6 to 1:4.4. Piezometer observations near the crest indicated a safety factor of 1.3. Drains were installed in the embankment above the top of the berm to reduce the water table. The overflow sill level was reduced by 0.3 m to increase the freeboard and reduce the risk of future overtopping by waves.
Lessons	Failure occurred because the downstream slope was only marginally stable under normal operating conditions. Also, it was found during excavation of the slide area that agricultural drains were completely blocked. Their deterioration may also have contributed to the slide. Doubt was cast on the value of wave walls with concave faces which turn waves back on themselves as they transfer the wave from a horizontal travel to a vertical travel, which then allows the wind to blow the wave over the crest.

Reference: Osborne, 1975.	
59. Aldenham	Incident date: November 1984
Construction details	The seven-metre high homogeneous earthfill embankment was completed in 1795 and has a reservoir capacity of $78 \times 10^3 \text{ m}^3$. It was built to supply compensation water to mill owners and the Grand Junction Canal. The embankment was built of random fill of London Clay and was raised soon after construction.
Incident description	The clay fill embankment has been prone to instability since construction in 1795 and raising in 1802. Recommendations under the Reservoirs (Safety Provision) Act 1930 in the early 1970s included removal of all trees and undergrowth from the crest and upper levels of the downstream slope and marginal raising of the crest to increase the freeboard. Following reservoir drawdown in January 1975, two areas of the upstream slope slipped and subsequently two minor slips occurred on the 1:5 downstream slope which appears to have been marginally stable. The ill-considered removal of the vegetation, possibly assisted by the placement of fill on the crest was sufficient to allow instability to develop. The vegetation appears to have had a two-fold function in assisting stability: pore water pressures would have been reduced whilst the roots would have helped to hold the superficial layers of the slope together.
Investigations	Investigation showed that a shear surface was present at about one metre depth.
Remedial works	Despite remedial works in the form of two-metre drainage trenches at five metres in the 1980, works were unsuccessful in controlling stability. A limited length of sheet piling was installed in the 1990s where one of the areas of movement extended through the wave wall onto the upstream slope. Movement has continued and the installation of soil nailing has had little effect.
Lessons	The effects of vegetation must be considered as an integral part of the design of a new dam or a continuing facet of operation and maintenance of existing structures. Trees such as willow, poplar and alder which have high water demand and extensive root systems should not be allowed to grow on embankments. Trees can have a stabilising effect when surface stability of the embankment is marginal. Changes in seasonable moisture content are thought to affect down-slope creep with time.
References: Kennard, 1975; Hoskins and Rice, 1982.	

5.3.7 Group 7: External erosion due to flood flow

62. Tunnel End	Incident date: 1799
Construction details	The reservoir was one of three summit reservoirs built for the Huddersfield canal which was designed by Benjamin Outram. The nine-metre high dam was constructed in 1798 and is reputed to have a clay core. It is now largely filled with silt.
Incident description	The dam was overtopped in 1799 and suffered partial collapse, resulting in one death.
Response	The dam was rebuilt and at least four minor incidents of overtopping of the dam are reported.
Remedial works	The reservoir has a flushing shaft and a main spillway which is likely to have been added after the incident. Despite regular inspections, it was not until 1968 that a panel engineer reported that overflow arrangements were inadequate to pass the design flood. In 1994, it was reinforced to allow overtopping in the event of a major flood.
Lessons	This is one of the earliest documented cases of overtopping in

Great Britain.

References: Binnie, 1987; Dutton, 2001; Schofield, 1979.

63. Diggle Moss (Black Moss)

Incident date: 29 November 1810

Construction details The reservoir had two dams. The subsidiary dam which failed appears to have been made only of peat and was founded on moss and "ling" that covered the soil.

Incident description On the 29 November 1810, after continual rain, failure of the embankment dam caused a sudden release of reservoir water known as the "Black Flood" into the River Colne. The flood resulted in the loss of five lives and damage of £450.

Response None known.

Lessons It seems that the dam had been poorly constructed without a proper foundation. It is not known if the dam was overtopped or was just washed away.

References: Binnie, 1987; Smith, 1985.

64. Brent (Welsh Harp)

Incident date: 17 January 1841

Construction details The reservoir was built for the Regent's Canal Company. The seven-metre high earth dam had a puddle clay core 1.8 m wide.

Incident description Freezing conditions had existed since early December 1840 and the Thames had frozen over to a thickness of two metres. Following a thaw and seven-day period of non-stop heavy rain, the embankment breached on 17 January 1841. The resultant surge of water from the reservoir wrecked 113 barges, caused substantial property damage and some loss of life.

Response The dam was rebuilt and in 1852 it was raised by two metres.

Lessons A reservoir attendant's cottage was built after the repair so that a watchful eye could be maintained on the reservoir level.

References: Binnie, 1987; Makin, 1986.

65. Bold Venture (Darwen)

Incident date: 23 August 1848

Construction details The 10-m high embankment was constructed in 1844 west of Darwen in Lancashire for a local mill owner. It had an upstream slope of 1:3 and downstream slope of 1:2.5, and had a puddle clay core 0.9 m at the top and 1.5 m at the base. The overflow channel was 1.8 m wide by 0.9 m deep.

Incident description The embankment dam was overtopped during a heavy rainstorm and the released reservoir water caused 12 deaths in Darwen.

Response An inquest was held on the Friday following the incident on the Wednesday. The jury returned a verdict of accidental death in all cases. The verdict of the jury at the inquest was that "*all the deaths inquired into occurred by an accidental cause*".

Lessons The jury recommended that "*in case that the reservoir be re-constructed, to enlarge the byewash (overflow)*".

References: Aighton, 2003; Charles, 2002.

66. Woodhead No. 1

Incident date: 7 October 1849

Construction details Woodhead No. 1 is one of the dams in the Longdendale valley engineered by J F Bateman to a height of 29 m. It is of conventional design with a puddle clay core and clay filled cut-off trench. The dam leaked badly resulting in Woodhead No. 2 being

Incident description	built immediately upstream. The overtopping incident at Woodhead No. 1 occurred during construction when the embankment was only 7.3 m high. It is the only recorded incident of overtopping during construction.
Response Lessons	The weir erected for the purposes of diverting the floodwater during construction of the embankment along a new watercourse gave way under an extraordinarily large flood. The reservoir basin started to impound and within three hours the dam overtopped and breached. Crops, bridges and buildings were destroyed downstream over a distance of five miles. The contractors raised the dam by another metre during the flood. Despite making allowance to bypass the flood based on the Darwen flood in Blackburn in 1848, the flood at Woodhead was twice the estimated value. Additional provisions were made to bypass future floods.
References: Bateman, 1884; Binnie, 1981.	

67. Cwm Carne	Incident date: 14 July 1875
Construction details	The 12-m high embankment dam was constructed in 1792 some ten miles north of Newport in South Wales to supply the Monmouthshire canal. The centre of the dam was described as consisting of puddle clay of inferior quality and different from the retentive alluvium used to repair the dam. For many years, the reservoir was little used and fell into comparative neglect and disrepair.
Incident description	Following several days of heavy rain in mid-July 1875, the neglected embankment dam was overtopped at 17:30 on 14 July 1875. Failure occurred at 23:00 and the flood resulted in the loss of 12 lives.
Response Lessons	An inquest was opened on the 19 July 1875. From investigations of the failure, Jee (1877) concluded “ <i>no structure of its kind, however long it may have been in existence, could be considered safe without constant attention and supervision.</i> ” There had been considerable subsidence of the centre of the embankment occasioned by leakage which had existed for a number of years in the form of two permanent leaks, one of which supplied sufficient water to fill a three-inch pipe at all times. The incident is in many ways similar to Bilberry incident, with the dam having suffered from neglect over a period of many years, but Cwm Carne had lasted nearly 80 years before its failure.
References: Jee, 1877; Smith, 1992.	

70. Dunblane	Incident date: 1943
Construction details	The 12-m high dam was built in 1933 near Stirling.
Incident description	The dam overtopped, causing erosion of the downstream slope which was washed into the spillway.
Response Remedial works	Remedial works undertaken. The twin pipes leading into the spillway were replaced with an open channel. A low wave wall was built along the crest.
Lessons	The pipes had restricted flow into the spillway.

71. Bilberry	Incident date: 29 July 1944
Construction details	The new Bilberry dam was engineered by Bateman. It was 16 m high and completed in 1853 with the centre line nine metres upstream of the old one. The dam was made watertight by a layer of

Incident description	puddle clay on the upstream face of the embankment and on the floor and sides of the reservoir. During a catastrophic storm in 1944, parts of the embankment were washed away by the run-off from adjacent valley sides and waves breaking over the crest. The extensive damage to the downstream face is shown on page 170 of Binnie (1981). Three lives were lost in the Holme Valley during the storm.
Remedial works	The dam was modified so it could be overtopped. The top of the overflow shaft was lowered by 0.3 m. During the construction of Digley dam in 1954 (immediately downstream of Bilberry), a large berm was placed on the downstream slope which is stone-faced. At top water level, Digley reservoir submerges the lower part of the Bilberry embankment.
Lessons	An additional spillway was constructed to the waste pit, as without it the embankment would certainly have failed a second time.
Reference: Binnie, 1981.	

72. Tumbleton Lake	Incident date: 12 August 1946
Construction details	The 11-m high embankment was constructed in 1885 near Rothbury, Northumberland.
Incident description	Torrential rain swept across Northumberland on 12 August 1946. The dam was overtopped and came close to failure. The spillway channel was destroyed and a bridge over it was severely damaged. The downstream face was scoured. The dam had been overtopped to a lesser extent on 21 July 1927.
Response	The structure was so impaired that it was kept empty for many years.
Lessons	This small dam had an undersized spillway.
Reference: Archer, 1992.	

73. Doxford Lake	Incident date: 12 August 1948
Construction details	The two-metre high homogeneous embankment has a volume of only $29 \times 10^3 \text{ m}^3$.
Incident description	The embankment was overtopped by at least 0.3 m. It is believed to have overtopped on several occasions since. Minor erosion of the downstream slope has occurred at a point where the crest is lowest.
Response	None
Lessons	Some smaller dams are capable of being overtopped without serious damage. There are many such unreported incidents where small dams have been overtopped.
Reference: Archer, 1992.	

74. Thorters	Incident date: 12 August 1948
Construction details	The 15-m high embankment was built in 1900.
Incident description	The dam was overtopped during an abnormal flood in South East Scotland on 12 August 1948.
Lessons	The dam had an inadequate sized spillway.
Reference: Moffat, 1975.	

75. Trewitt Lake	Incident date: March 1963
Construction details	The five-metre high homogenous dam is located north-east of Kielder reservoir in Northumberland. The overflow was a drop shaft

Incident description	spillway built into the upstream fill. After a prolonged freeze-up in the first two months of 1963, reservoirs and even rivers had a solid cover of ice more than 0.3 m in thickness and there were deep accumulations of snow. A rapid thaw in early March delivered nearly two months of accumulated precipitation to rivers in a couple of days. With rising flow the reservoir filled and the cover of ice broke into large ice flows, some of which became wedged in the spillway shaft, reducing its capacity. When the dam overtopped a triangular breach grew rapidly to the full five-metre depth of the dam in less than 20 m. Fortunately, the flood dissipated before reaching the village downstream.
Response Lessons	None, the reservoir was abandoned. This is the only overtopping incident entirely due to snowmelt. The contribution of the blocked spillway to overtopping is not known.
References: Archer, 1992; Archer, 1984.	

76. Toddbrook	Incident date: December 1964
Construction details	The reservoir was constructed in 1840-41 to supply water to the Peak Forest canal. It is on the north-west edge of the Peak District National Park near Whaley Bridge. The embankment is 24 m high with 1:2 upstream and downstream slopes. Further details of the dam construction are given in Incident No. 23.
Incident description	The water level was one metre above the spillway crest for a period of 24 hours following heavy rain and it took another two days for the level to fall to normal top water level. Damage was caused to the lower part of the spillway channel. Some parts of the side walls were washed out and some erosion took place on the right bank adjacent to the downstream toe of the dam. The main deterioration was caused by excessive flow down the spillway.
Response	The 1964 flood damage was repaired in 1965 and subsequent flood studies confirm the spillway was inadequate to take the design flood. An additional spillway was built in 1969 with a 75-m weir built over the southern section of the embankment discharging over a concrete-protected portion of the downstream face. The sill level is above the original spillweir level.
Lessons	The incident showed that despite the dam being in existence since 1840, the spillway was inadequate. The incident instigated a flood study of the reservoir resulting in an additional spillway constructed.
Reference: Charles, 1986.	

77. Chew Magna	Incident date: 10 July 1968
Construction details	The 12-m high embankment dam was built for Bristol Waterworks between 1848 and 1850. The original overflow sill was 19.43 m long. This was extended to 22.9 m in 1936.
Incident description	A series of severe thunderstorms crossed the Mendips during the evening of 10 July 1968 giving rise to devastating flooding in the region. The eye of the storm was centred over Chew Magna with over 150 mm of rain falling between 20:00 and midnight. During the storms the embankment overtopped to an estimated depth of 90 mm. Extensive erosion took place at the end of the spillway channel and a four-metre deep hole developed in the floor of the stilling pool. The discharge down the spillway was calculated to be 40.5 m ³ /s, with 2.8 m ³ /s discharging down the face of the embankment.
Response	Following the storms, an auxiliary overflow was constructed and provided with a sill 0.3 m higher than the original sill. The embankment was raised by one metre.
Lessons	Despite enlargement of the spillway in 1936, the spillway proved to

be too small for floods resulting from the 1968 storm.

Reference: Heaton-Armstrong, 1984.

78. Corsham Lake	Incident date: 1968
Construction details	The reservoir is impounded by a three-metre high embankment.
Incident description	During a storm in 1968, 150 mm of rain fell in 10 hours. The spillway could not cope and became partially blocked by a collapsed retaining wall. The embankment then overtopped by 0.3 m over a 100 m length. This caused erosion of a one-metre deep trench in one place.
Remedial works	The spillway has been enlarged.
Lessons	The spillway was not large enough to take the flood.

79. Kype	Incident date: October 1977
Construction details	The 17-m high embankment was constructed for Hamilton Town Council in 1898. The overflow is 35 m wide and located on natural ground.
Incident description	Following heavy rainfall in October 1977, the walls of the spillway channel were overtopped and considerable erosion of the ground adjacent to the spillway took place, estimated to be 60 m ³ of rock. There was some minor damage at the mitre. The incident occurred during the night. It was estimated that the flood had a return period of one in 200-400 years.
Response	A flood study was undertaken and a hydraulic model of the overflow and spillway channel was constructed to examine the hydraulics of the existing channel. The design flow, based on the 1978 ICE Floods Guide, was three times the flow responsible for the overtopping of the spillway walls.
Remedial works	In 1982, works were carried out to the upper part of the spillway to increase its capacity. A proprietary mattress was applied to one side of the channel. In addition, a pre-cast concrete 650-mm high wave wall was provided along the upstream edge of the crest.
Lessons	The original spillway design of 1898 was inadequate despite having operated satisfactorily for nearly 80 years.

Reference: Little, 1984.

80. Walshaw Dean Lower	Incident date: 19 May 1989
Construction details	The three Walshaw Dean reservoirs are situated in the Pennines 15 km north west of Halifax. The dams were designed by G H Hill and were constructed between 1901 and 1915. They are typical puddle clay core embankments with deep puddle clay filled cut-off trenches. Each dam has a masonry stepped spillway close to the mitre and there is a bywash channel around the three reservoirs.
Incident description	On 19 May 1989 extreme rainfall occurred, with 193 mm rainfall recorded beside Walshaw Dean Middle reservoir in two hours. Much damage was done to the bywash channels and spillways.
Response	A subsequent statutory inspection recommended that the owners commission a flood study and physical modelling of the three reservoirs. Model tests showed that all the spillways could not pass the design flood. At Walshaw Dean Lower, less than a quarter of the design flood could be passed safely.
Remedial works	Modifications were carried out to all existing spillways and a new spillway was constructed on the right bank of Walshaw Dean Lower.
Lessons	The original spillway design, dating from the start of the twentieth century, was inadequate despite having operated satisfactorily for

nearly 90 years.

Reference: Harrison and Drabble, 1996.

81. Boltby	Incident date: 19 June 2005
Construction details	The 19-m high embankment was built around 1880 to supply water to North Yorkshire. It had a puddle clay core supported by earthfill shoulders. The stepped masonry spillway channel passed down the left mitre and adjacent to the downstream shoulder of the dam.
Incident description	In June 2005, an extreme rainfall flood event with a return period in excess of one in 10,000 years caused extensive damage to the spillway: half of it was damaged and there were deep erosion channels adjacent to the spillway. Damage also occurred at the toe of the dam. The embankment was not overtopped, the freeboard during the event being 600 mm. From an assessment of the flood event, it was concluded that the equivalent runoff depth was in excess of 200 mm and that the rainfall event was localised.
Response	An emergency drawdown was initiated using the scour facility, the draw-off and by pumping over the crest. Temporary stabilisation works at the toe were undertaken.
Remedial works	Work has been carried out to discontinue the reservoir by excavating a notch in the embankment and by constructing a new overflow at the base of the notch.
Lessons	The original spillway design was inadequate; however, the spillway performed satisfactorily for over 100 years. A number of incidents involving severe damage to masonry stepped spillways have occurred at British dams and have been highlighted by Boltby and Ulley. Guidance on the performance and behaviour of stepped masonry spillways has been issued by the Environment Agency.
References: Mason and Hinks, 2008; Walker, 2008.	

5.3.8 Group 8: Wave damage to upstream protection

86. Kielder	Incident date: January 1984
Construction details	Kielder dam is a 55-m high rolled clay core embankment dam built on the river North Tyne that was completed in 1982. The maximum direct fetch from the west is 4.9 km. The top 6.3 m of the 1:2.5 upstream slope is protected by pre-cast concrete blocks, 300 mm thick, with a simulated rockface finish. Below the concrete blocks, the slope is protected with riprap. Blocks were of two sizes: 1,065 mm by 400 mm weighing 310 kg and 705 mm by 500 mm weighing 260 kg. The blocks were laid on two 300-mm layers of crushed whinstone. The joints between blocks, which varied between 12 and 17.5 mm wide, were filled with pea gravel. There are three <i>in situ</i> concrete retaining beams set into the shoulder.
Incident description	Storm force winds in January 1984 generated waves up to two metres high for about three days. Wind speeds locally were 26 m/s with gusts up to 42 m/s. The pre-cast concrete block protection was disturbed and single blocks weighing 300 kg were lifted out of position. This enabled waves to break into the holes and remove stone filters overlying the glacial clay embankment.
Response	Blocks were rapidly lifted and filter material was replaced.
Remedial works	In 1984, the gravel layer was replaced with concrete slabs; the blocks were bedded in mortar and spaces between the blocks were grouted. In 1986, all the blocks were removed in a progressive operation across the whole length of the dam and re-set on 200-mm thick mesh reinforced concrete slabbing cast in panels. The joints

	between the blocks were pointed with mortar.
Lessons	The instability problems of the upstream slope protection can be attributed to an underestimation of the wave height, the introduction of positive gaps at the joints and inadequately graded bedding material. A re-appraisal of the exposure of the Kielder site led to the more conservative design.
References: Carlyle, 1988; Herbert <i>et al.</i> , 1995; Rocke, 1985.	

87. Bewl Bridge	Incident date: 16 October 1987
Construction details	The 31-m high rolled clay core dam near Tunbridge Wells, Kent, was completed in 1975. The upstream slope is protected by concrete slabbing of butt-jointed panels, four metres by four metres by 127 mm thick. The vertical joints are partly open incorporating tapered toothed slots. Horizontal joints are sealed with bitumen. The slabbing is laid on a two-layer filter consisting of 150-mm graded gravel (50 to 5 mm) over a 225-mm thick layer of gravel and sand.
Incident description	During the 'hurricane' that hit South East England on 16 October 1987, a 76-m length of concrete slabbing was damaged and the underlying bedding was washed out. Wind direction during the storm varied between south and south west, the directions likely to produce the largest waves at the dam. In places, up to 0.5 m of shoulder fill had been scoured out. The reservoir level was one-third the way up the third row of slabs. The postulated mode of failure was that the third row of slabs was lifted by water pressure under the slabs generated by run-up not dissipating quickly enough through the drainage underlayer. This resulted in the slabs being broken and undercutting of upper slabs which slid down the slope.
Response	Temporary repairs were done to get through the winter and the reservoir level was raised to take any further storm waves away from the area of worst damage. The temporary repairs involved placing sandbags in all the large gaps and securing them with two-metre long steel rods driven into the embankment.
Remedial works	The permanent repair was undertaken in the summer of 1988 and involved relaying the bedding and casting new 500-mm thick panels of concrete, with reinforcement, over rows 2 to 4 over the affected length. The revised design was based on a maximum wave height of two metres. Changes were also made to the joint details.
Lessons	The original slab thickness of 127 mm was based on wave heights significantly less than those that occurred in the storm. The position of the draw-off and overflow towers relative to the damaged area is also believed to have been significant. Diffraction around the towers could have increased the wave height over the damaged area.
References: Herbert <i>et al.</i> , 1995; Shave, 1988.	

5.3.9 Group 9: Reservoir basin leakage

88. Ainsworth Mill Lodge	Incident date: 11 December 1860
Construction details	John Rylands constructed a reservoir to supply water to his steam-powered textile mill in Lancashire. Thomas Fletcher operated mines on nearby land and had tunneled up to old disused mines which were under the land where Rylands' reservoir was located. Both parties rented the lands from Lord Wilton. While excavating the reservoir site, contractors came across some disused mine shafts which had been loosely filled with marl and soil. No attempt was made to seal these shafts. These shafts actually led, via a series of interconnected shafts and tunnels, into Fletcher's mines and land.

Incident description	Water from Rylands' reservoir flooded into Fletcher's mines on 11 December 1860, just days after completion of the reservoir and after it had been partially filled.
Lessons	<p>Fletcher sued Rylands for the recovery of £5,000 damages based on allegations of negligence. Although the local court of Liverpool Assizes in 1861 found in favour of Fletcher on the basis of trespass and nuisance, Rylands successfully appealed on the basis that he was involved in a reasonable and lawful act, with no ill-intent or negligence. Fletcher then appealed to the Exchequer Chamber of six judges in 1866 who ruled:</p> <p><i>“We think that the rule of law is, that the person who for his own purposes brings on his lands and collects and keeps there anything likely to do mischief if it escapes, must keep it at his peril and if he does not do so, is prima facie answerable for all damage which is the natural consequence of its escape.”</i></p> <p>The apparent outcome of legal action between Rylands and Fletcher which went to the House of Lords in 17 July 1868 was <i>“that the common law now imposed strict liability without proof of negligence on those who constructed or operated reservoirs that caused damage by the escape of water”</i>.</p> <p>However, the only individual who seems actually to have employed the rule in Rylands versus Fletcher to recover damages for a burst reservoir is Thomas Fletcher.</p>
Reference: Simpson, 1984.	

89. Colt Crag	Incident date: 1888
Construction details	Colt Crag is one of a number of dams built for the Newcastle and Gateshead Water Company in Northumberland. It was designed by Bateman. The reservoir is formed by two dams, the larger being 17 m high. The dam and the reservoir are on fissured limestone. The cut-off trench which was taken down deeper than expected had a maximum depth of 12 m and was filled with puddle clay except where the outlet tunnel crosses the cut-off. The minimum width of the trench is 1.8 m.
Incident description	<p>Movements of the embankments were reported between October 1880 and March 1881. Slips on the upstream and downstream shoulders were reported during construction or soon after. To counteract the slips, berms were built of dry rubble on both faces. During construction in 1881, it was found that water was being lost through sink holes at the upper water level. The reservoir was brought into use in 1884. In 1888, a six-metre square area had fallen in, forming a sinkhole from which a good proportion of the water escaped. Leakage had occurred due to the outcrop of limestone passing beneath the dam.</p>
Remedial works	The sinkhole was filled with boulders and concrete and finished with a layer of puddle clay. Watertightness was not achieved until 1912.
Lessons	This case illustrates the problems of achieving watertightness on limestone strata.
References. Binnie, 1987a; Rennison, 1982.	

91. Pen-y-Rheol	Incident date: 1985
Construction details	The nine-metre high embankment was built near Pontypool in South Wales in 1912. The enlarged pond was formed by an embankment

Incident description	with a puddle clay core and a lining to the floor of the reservoir. The collapse of disused iron ore workings beneath the reservoir following a minor earthquake in 1985 caused depressions in the reservoir floor. Sinkholes were around two metres in diameter but no leakage was seen in the mine workings. Earlier records of depressions in the reservoir floor were made in 1983 when the reservoir was drawn down for valve maintenance.
Remedial works	The reservoir was discontinued in 1986.
Lessons	The incident appears minor in nature but illustrates the problems of mine workings beneath reservoirs.

92. Larksheath	Incident date: 8 February 1996
Construction details	This farm reservoir with a capacity of 325,000 m ³ was formed by a continuous embankment 5.5 m high, formed from material within the reservoir area. The inner slopes were 1:4.3 and the outer slopes were 1:2.8. The excavated base of the reservoir consisted of chalk, sand and boulder clay overlying chalk. The watertight element consisted of a welded 0.75-mm thick HDPE membrane laid on 25-75 mm of sand. The membrane was placed in February and March 1995 in wet conditions. The reason for the mid-winter construction was to maximise licensed water abstraction. The reservoir was half-filled in March 1995 by pumping and drawn down in the summer.
Incident description	In January 1996 refilling of the reservoir began, but by 8 February the level started to drop. Major collapses and leaks had occurred. The failure did not constitute a threat to public safety.
Post failure investigation	The post-failure investigation identified numerous collapse features, some many metres across, tears in the membrane and faults with welds. Frequent large loosely filled sinkholes in the foundation had collapsed due to the initial flow of water through minor deficiencies in the lining and hence removed the underlying support to the lining.
Lessons	Shortcomings in design and construction were identified. Design and construction was carried out on the basis of a sound foundation as the presence of sinkholes was not picked up prior to construction. Contributing factors to the HDPE lining failure were its thickness of only 0.75 mm, the unsuitable weather conditions during its placement, poor welding and high frequency of repairs. It was concluded that the presence of loosely filled sinkholes susceptible to collapse following infiltration rendered the site unsuitable for construction of a membrane-lined reservoir. Successful use of HDPE membranes as the watertight element has been achieved on other dams such as Elvington and has been used at landfill sites. Generally, a two-mm thick membrane is used and quality control involves extensive testing of welded joints.
Remedial works	The dam was demolished.
References:	Tedd, 1999; Robertshaw and MacDonald, 2004.

5.3.10 Group 10: Concrete and masonry dams

96. Blackbrook	Incident date: 11 February 1957
Construction details	Built in 1906 on the site of the failed embankment dam in Charnwood Forest, near Loughborough, this was the first dam to be constructed of mass concrete with rock displacers. It was faced with blue brick and masonry. The downstream side consists of concrete with plums faced with local rock. A cross-section and description are given in Walters (1964). It is founded on pre-Cambrian rock which forms part of the Charnwood Forest inlier and consists of slates, conglomerates and grits. This is described as much disturbed and

<p>Incident description</p>	<p>fractured but nonetheless provides an excellent formation for a massive concrete dam. The cut-off trench is five metres below foundation level and 1.8 to 2.4 m wide. There is no grout curtain. On 11 February 1957, the dam was affected by an earthquake with a local magnitude of 5.3. The dam suffered fairly superficial damage which included displacement of 0.75-tonne copings, manhole covers sheared and displaced up to 20 mm, and cracks in the drainage gallery and on the upstream and downstream faces. Level and drainage monitoring established that the dam settled back to its original foundation but displacements of up to 30 mm had occurred at the abutments and cracks had occurred in the drainage gallery. Following the earthquake, drainage flow on the downstream side increased from 1.6 l/m to 631 l/m and after four weeks had fallen to 95 l/m. Subsequently the flow reverted to normal conditions. It is thought that the cause was disturbance of strata rather than the dam. If the line of the tremor had been 90 degrees displaced, it could have been much more serious. The epicentre was four miles north of the dam.</p>
<p>Response Lessons</p>	<p>The structure was inspected and monitored. This is the only dam in Great Britain where damage has been definitely attributed to an earthquake. A seismic tremor was also noticed at Newtown Powys on 15 April 1984 lasting 15 seconds with a severity of 3.5 on the Richter scale. Both Clywedog, a gravity buttress dam, and Nanty Geifr were within four miles of the epicentre, but no damage was reported.</p>
<p>References: Kennard and Mackey, 1984; Walters, 1964.</p>	

<p>98. Val de la Mare</p>	<p>Incident date: January 1971</p>
<p>Construction details</p>	<p>The dam is a 29-m high concrete gravity structure built on Jersey between 1957 and 1962. The dam was designed using the middle third rule allowing for an internal uplift pressure of 50 per cent of the reservoir head on the line of the upstream face, decreasing linearly to zero on the downstream face. The dam was constructed of 26 monoliths each 6.7 m wide in lifts of 1.22 m.</p>
<p>Incident description</p>	<p>In January 1971, it was noticed that the hand rails on the bridge were no longer aligned, with four of the monoliths displaced upstream by a maximum of 12 mm. The downstream face showed damp patches with random hairline cracking.</p>
<p>Response</p>	<p>A period of intensive investigation and research led to the conclusion that alkali-silica reaction (ASR) was occurring due to the combination of alkali-reactive silica (chalcedony and associated apaline vein material) in the local coarse aggregate and the high alkali cement imported from south of the Thames. The evidence indicated that deterioration would not progress to the point where the concrete could not withstand the applied compressive loads, but that expansive cracking could lead to higher internal uplift pressures and result in instability.</p>
<p>Remedial works</p>	<p>Remedial works included the provision of drainage into the gallery, grouting and the installation of anchors in the section of the dam most adversely affected. Instrumentation was installed to monitor the loads on the anchors and movements at this section.</p>
<p>Lessons</p>	<p>ASR only occurs where there is sufficient moisture in the concrete, high alkalinity in the cement and a critical amount of reactive silica in the aggregate. Water-retaining structures are most vulnerable. After the diagnosis of ASR, the main concern was that cracking of the concrete would lead to higher internal pore pressures resulting in unacceptably reduced stability. This was mitigated by remedial works and monitoring has allowed the dam to continue in service. ASR has occurred on relatively few dams in the Britain. Much</p>

literature now exists on the identification, performance and operation of concrete with ASR in dams.

References: Cole *et al.*, 1988; Coombes *et al.*, 1975; Haws *et al.*, 1994; Horswill *et al.*, 1994.

5.3.11 Group 11: Other incidents

100. Beggars Hall Lake	Incident date: 22 December 1999
Construction details	None known.
Incident description	On 22 December 1999, a Korean Air Boeing 747 cargo plane crashed into a small embankment dam three minutes after take-off from Stansted airport: all four crew were killed. Although the main impact crater was on the embankment, no impounded water was lost, but substantial remedial work was required.
Lessons	Although unlikely, such an event could produce catastrophic consequences on a larger dam.

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Bristol, BS32 4UD
Tel: 0870 8506506
Email: enquiries@environment-agency.gov.uk
www.environment-agency.gov.uk

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