Performance evaluation, safety assessment and risk analysis for dams

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After reviewing the present knowledge on incidents and failures of dams, the author considers what should be regarded as 'abnormal' behaviour in a dam, requiring remedial action, and he discusses the use of risk analysis in dam engineering.

There are several informative ICOLD publications relating to dam incidents and failure statistics [for example, ICOLD, 1995[1]] and some very good specialized conference proceedings [for example, Serafim, 1984[2]; Kreuzer, Taylor and Dungar, 1993[3]].

Dam incidents and failure statistics

The recent ICOLD Bulletin [ICOLD, 1995[1]) summarizes a world-wide review (except China) including 181 concrete and embankment dam failures and their causes, see Fig 1. This figure shows clearly that failures, when they occur, mainly take place very early in a dam's life. Fig. 2 provides specific information on the causes of failure for the 129 embankment dams included in the study, and Table 1 summarizes the primary causes. The information presented in Table 1 corresponds with the results in Table 2 from a previous study of incidents and failures registered for 240 embankment dams in the USA [US NRC, 1983[4]]. As can be seen from both Tables, internal erosion constitutes a primary cause of concern.

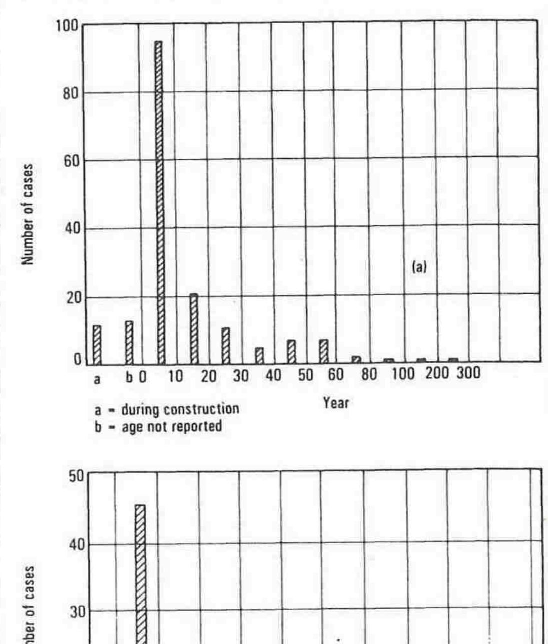


Fig. 1. Failures by age of failed dams, from an ICOLD study [1995[1]], where (a) shows failures by age (all ages), and (b) shows failures by age of failed dams less than 10 years old.

Table 1: Primary causes of embankment failures (according to ICOLD, 1995[1])

Causes of failure	TE (%)	ER (%)	ER/TE (%)
Overtopping (of crown)	23	45	31
Internal erosion (piping) - in dam body	20	8	23
Internal erosion (piping) - in foundation	13	13	15
Other structural causes	29	25	23
Other causes	15	9	8
Total (percentage)	100	100	100
Total number of dams	92	24	13

TE = Earthfill; ER = Rockfill

Fig. 3 gives a graphical representation of the experience with large embankment dams in Norway [Høeg et al., 1993[5]]. Most of these are rockfill dams with moraine cores. In Norway there has been no failure of an embankment dam higher than 15 m, but of the total of 174 built, 32 have been repaired. Twenty of these had sustained damage to their upstream slope protection (as a result of wave or ice action), six had suffered internal erosion, in some cases preceded by hydraulic fracturing, and six had been affected by excessive or differential settlement.

While internal erosion is a primary cause for repair and upgrading, the main cause has been damage to the upstream slope protection resulting from wave and ice action. (A 12 m-high earthfill dam built in 1975 failed in 1976 as a result of erosion around a concrete culvert going through the moraine core.)

Remedial work in Norway has consisted of improved riprap design and construction, and internal erosion and leakage have been controlled with water-cement-bentonite-sand mixture grout injection. Of all the dams com-

Table 2: Causes of embankment incidents and failures according to USCOLD

Causes of incident or failure	% of 240 dams
External erosion (overtopping/wave action)	29
Internal erosion (in dam body/foundation)	38
Foundation instability	14
Excessive dam deformations	13
Deterioration (chemical/physical)	2
Malfunction of gate	2
Earthquake effects	1
Construction error	1

Based on USNRC, 1983[4]

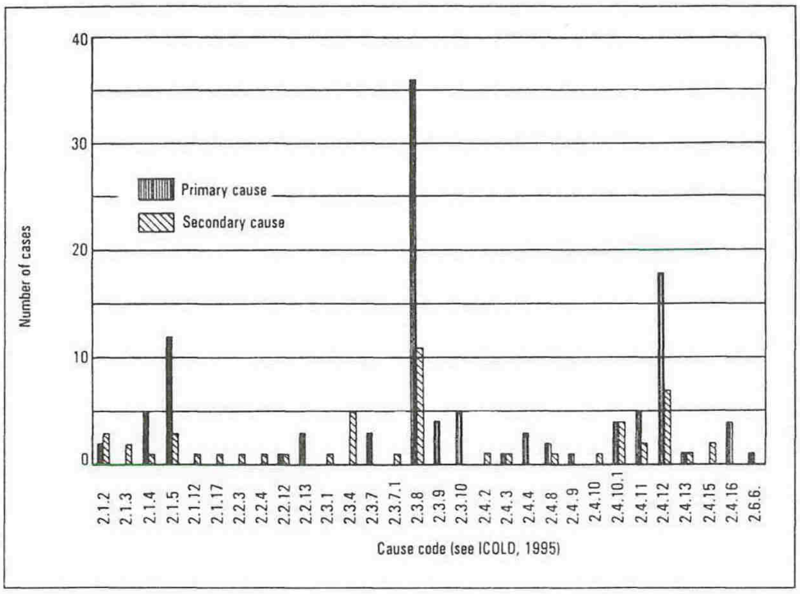


Fig. 2. Causes of failure in embankment dams, from an ICOLD study [1995[1]].

pleted since 1976 in Norway, only one has required repair, demonstrating that lessons have been learnt from the experience gained, and that there have been improvements in design criteria and construction and control procedures.

In summary, a review of the available statistics from different parts of the world and accompanying evaluations and analyses [for example, Peck, 1986[6]; Londe, 1993[7]; ICOLD, 1995[1]] shows that the causes of dam incidents and the frequency of failures vary from one region to another. However, the average probability of failure per dam-year has been reduced from around 10^{-4} towards 10^{-5} over the last 30 to 40 years. Furthermore, a dam built some 30 years ago and still performing well is as safe as a dam built with today's state-of-the-art technology and quality control.

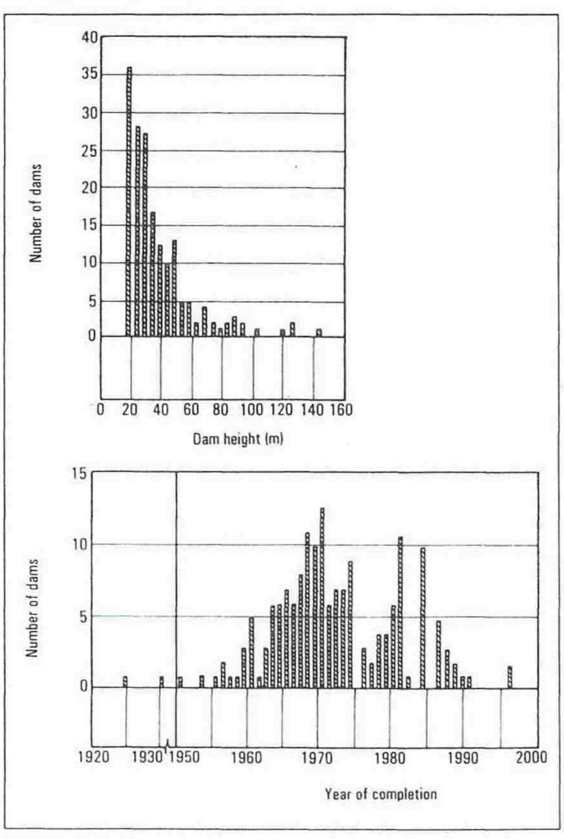


Fig. 3. Norwegian embankment dams higher than 15 m [Høeg et al, 1993[5]].

Dam performance evaluation and safety assessment

In many cases the reasons for excessive leakage, internal erosion, excessive displacements and differential settlements or cracking in an embankment dam may not be obvious. Careful interpretation of observations, visual and/or through field instruments, must be carried out to assess the situation before methods of repair or upgrading are decided upon. Unexpected observations may not give any reason for concern if a logical explanation can be found. There are three fundamental questions:

- What is 'abnormal' embankment behaviour? The observed behaviour may deviate from what the designer expected, but tested against the experience accumulated by the profession, it may prove not to be so abnormal.
- Does the deviation from predicted behaviour indicate lower safety than aimed for in the design?
- Does the deviation require remedial action, such as repair, upgrading or strengthening?

In the following sections, two aspects of embankment dam behaviour are used to illustrate the need for correct interpretation of observations before decisions are made.

Prediction and measurement of post-construction displacements

When post-construction dam crest displacements are recorded and evaluated, it is important to specify the location of each measurement point (bolt) and the time of the first measurement after the end of construction (that is, the reference value for subsequent measurements). It is not uncommon for the vertical displacement (settlement) of the upstream crest bolt in an embankment dam to be significantly larger than the settlement of the corresponding downstream crest bolt. This is primarily caused by the raising of the reservoir, especially the last 30 per cent up to the maximum level.

In several cases, the increased compressibility and subsidence of the upstream shoulder on submergence has caused a relative slip between the fill and the core in the upper part of the embankment (Fig. 4). This results in differential settlements across the crest and possibly longitudinal cracks along it. The phenomenon is of no real concern with respect to dam safety, as the movements stabilize after the first reservoir filling, lowering and refilling cycle, and cause no transverse cracks. The observation does not call for a general flattening of the upstream slope, but possibly some repair to the riprap protection [for example, Paré, 1984[8]; Høeg et al., 1996[9]).

Table 3 gives the measured post-construction crest settlements (upstream, top of the core and downstream) for some large Norwegian rockfill dams built since 1972 on stiff foundations. The dams above the horizontal line in Table 3 have central or gently sloping clay moraine cores, while the dams below have slender asphaltic concrete cores. It can be seen that the Vatnedalen and Deg dams have very small core and downstream settlements, but fairly large upstream settlements over a limited length of the crest. This is because of the rockfill saturation effect described above, resulting in local subsidence in the top part of the upstream slope. The upstream slope for the Vatnedalen dam is 1V:1.5H and for Deg dam it is 1V:1.6H.

Fig. 5 gives a simplified overview of the measured post-construction crest settlements for rockfill dams worldwide, built on stiff foundations. The data have been collected from several authors (Soydemir and Kjærnsli, 1975[10]; Clements, 1984[11]; Dascal, 1987[12]; and, Høeg et al., 1996[9]), and the scatter is substantial. In the literature

Table 3: Post-construction settlements for Norwegian rockfill dams on stiff foundations

Dam	Year	*H* (m)	Vertical settlements (%*H*)		
			Crown upstream	Top core	Crown downtream
Oddatjørn	1987	145	0.26	0.20	0.22
Svartevann	1976	129	0.75	0.75	0.65
Vatnedalen I	1983	125	0.80	0.15	0.20
Deg I	1972	90	0.70	0.15	0.15
Nyhelleren	1979	85	0.12	0.12	-
Vatnedalen II	1983	60	0.10	0.10	0.10
Jukla I	1973	59	0.60	0.65	0.33
Storvatn	1987	90	-	0.20	-
Berdalsvatn	1988	65	0.18	0.14	0.10
Styggevatn	1989	52	0.12	0.20	0.10
Riskallvatn	1986	45	0.20	0.15	0.10

it is not indicated for each case where across the crest the settlements are measured (upstream or downstream). Most rockfill dams built since 1970 show a trend that gives a post-construction crest settlement in the range of 0.1 to 0.3 per cent of the maximum dam height after 25 years of operation. The trend is approximately linear with the logarithm of time, and there is no clear indication that high dams give a higher normalized settlement than lower dams (that is, there is no indication of an increase with the second power of dam height). Some modern dams are reported to have settlements considerably larger than those indicated above, but no satisfactory explanations have been reported.

For older dams, built before 1970 mostly without rockfill compaction, the normalized post-construction settlements are considerably larger and the scatter in the results is greater (Fig. 5).

Post-construction displacements depend on many factors, including:

- properties of rockfill and core materials;
- layer (lift) thickness and compaction effort;
- sluicing of rockfill during placement;
- steepness of exterior slopes;
- upstream impervious face or central core;
- zoning of the embankment; and,
- foundation stiffness.

However, some important factors are sometimes overlooked when displacements are evaluated and compared with corresponding measurements from other embankment dams. These are:

- the location of points on the crest where measurements are taken;

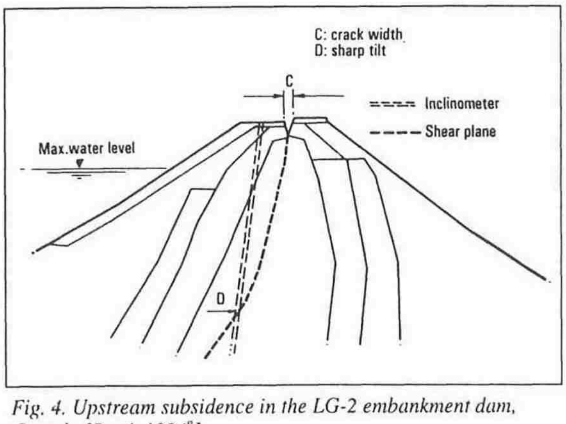


Fig. 4. Upstream subsidence in the LG-2 embankment dam, Canada [Paré, 1984[8]].

- time of first crest displacement measurement after the completion of construction (a significant part of the long-term settlement takes place during the first few months);
- water elevation at the end of construction when crest settlement bolts were installed;
- rates of dam construction and reservoir filling; and,
- magnitude of reservoir fluctuations.

These aspects make it difficult to make 'fair' comparisons such as those proposed in Table 3 and Fig. 5, and these factors must be considered before one judges whether the recorded post-construction displacements are 'abnormal' or not.

Prediction and measurement of pore pressures in earth cores

In several embankment dams with earth cores, the measured pore pressures in the downstream portion of the core have been much higher than predicted during design. Two well documented case studies will be used to illustrate the phenomenon.

Fig. 6 shows the maximum cross section of the Svartevann rockfill dam in Norway, with a slender and inclined broadly graded moraine core, founded on bedrock. Fig. 7 shows the measured piezometric heads versus reservoir level in three piezometers located in the core upstream, in the centre and downstream, respectively. In total 24 electric closed-system piezometers

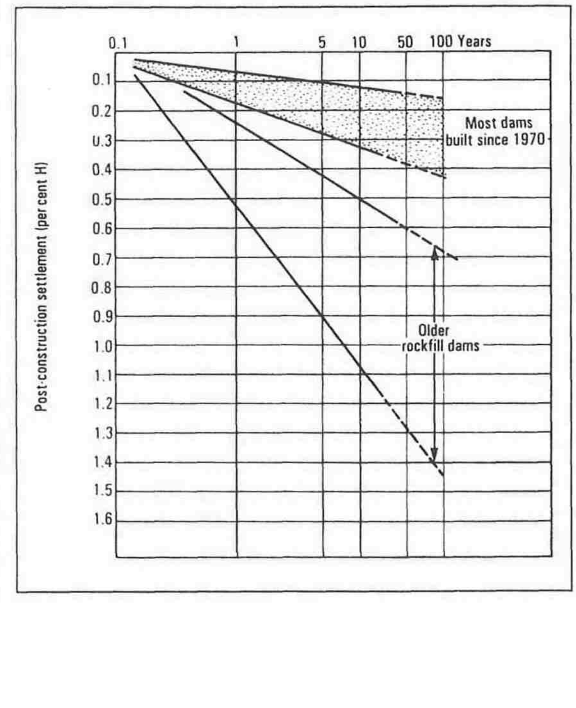


Fig. 5. Measured post-construction crest settlements for rockfill dams built on stiff foundations.

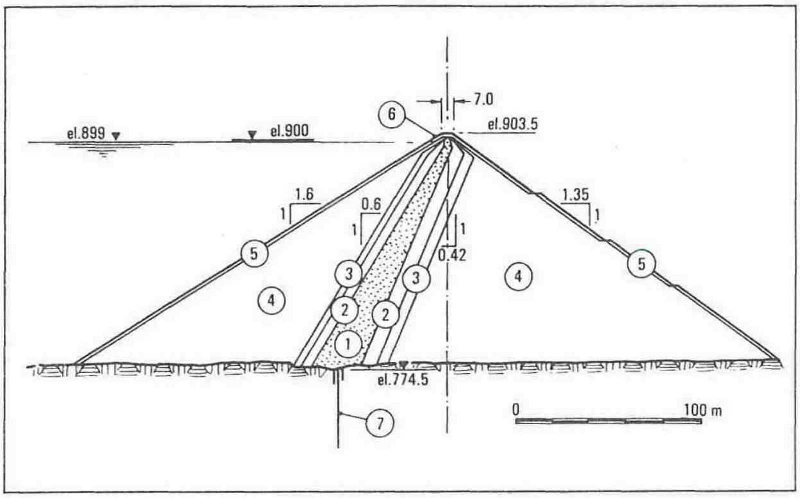


Fig. 6. Cross section of the Svartevann dam, Norway, where:
1 = moraine core;
2 = sandy gravel filter;
3 = transition rockfill;
4 = supporting rockfill;
5 = slope protection;
6 = protective cap of selected blocks; and,
7 = grout curtain..

(vibrating wire) were located in this same core cross-section. Piezometer P12 gave readings that consistently fall on the 45° line in Fig. 7, showing that the measured pore pressure close to the upstream face of the core was equal to the water pressure in the reservoir at that level. During the first year, piezometer P14 showed much higher pore pressures than those predicted based on the conventional theory of saturated steady-state flow. However, after two full cycles of seasonal lowering and raising of the reservoir, the piezometric head in P14 had decreased significantly. Fig. 8 shows the piezometric head versus time for P12, P13 and P14 up until 1989, 13 years after end of construction. The reservoir level at any given time is that recorded by piezometer P12. Equilibrium has been reached, and there is no further reduction in the pore pressures on the downstream side, as P14 only reflects the changes in reservoir level. Vatnedalen dam in Norway (Table 3) shows similar trends for the measured pore pressures (Myrvoll et al., 1989[13]).

Figs. 9 and 10 present the cross section and the results of piezometer readings in the core of WAC Bennett dam in Canada. Stewart and Imrie [1993[14]] carefully reviewed and reported the performance of this 183 m-high embankment dam at the international workshop on dam safety evaluation [Kreuzer, Taylor and Dungar, 1993[3]]. This very large dam has a thick broadly graded glacial till core and rests on a complex foundation which complicates the performance evaluation.

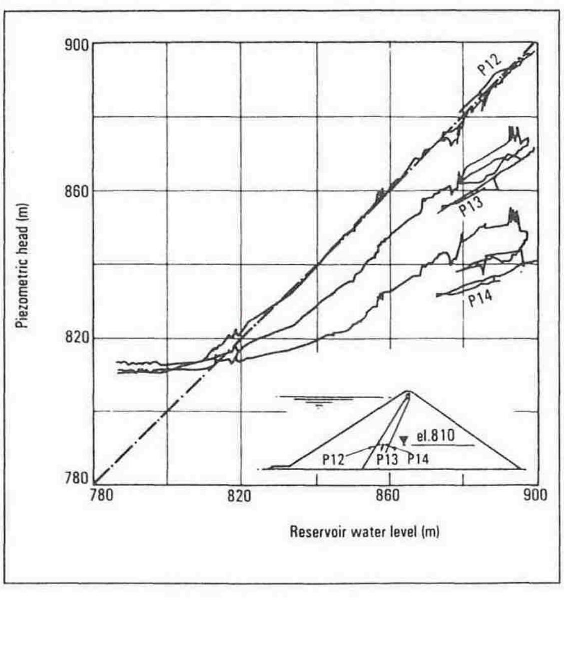
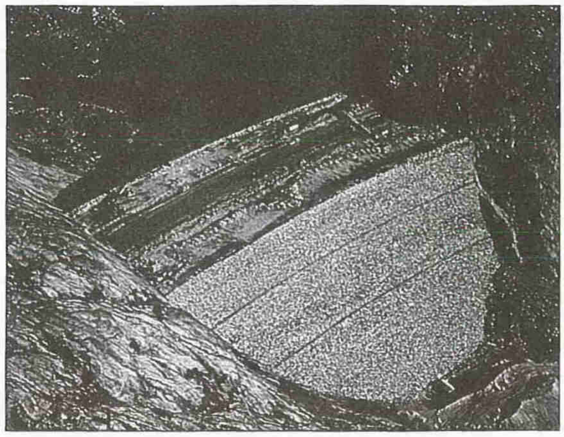


Fig. 7. Measured piezometric heads versus reservoir level from 1974-1979 for Svartevann dam.



The Svartevann rockfill dam during its construction, showing the placement of the broadly graded moraine core.

As can be seen in Fig. 10, the measured piezometric head in location EP04 increased between 1970 and 1976 to values much higher than those predicted. However, since that time the measured pore pressure has continuously decreased and is now not much higher than that predicted based on saturated steady flow. As discussed below, Stewart and Imrie [1993[14]] propose an explanation for this unexpected behaviour, and they refer to the experience from the Svartevann dam discussed above (Figs. 6-8).

Several other dams described in the international literature have shown pore pressures in the downstream portion of the core which were higher than expected, and many hypotheses have been proposed over the years. The following are some of the explanations proposed:

1. Heterogeneities, construction imperfections and larger horizontal than vertical permeability are not properly considered in the prediction analyses. Therefore the downstream pore pressures are underestimated.

2. Excess construction pore pressures remain in the core for a longer time than predicted.

3. A lower permeability coefficient on the downstream side as a result of higher effective stresses than on the upstream side (de Mello, 1980[15]; Vaughan, 1989[16]).

4. Cracks initiated by hydraulic fracturing caused by arching effects and locally lowered effective stresses in the core [for example, Sherard, 1984[17]; 1986[18]]. Pore pressure communication through the cracks gives high pore pressures downstream.

5. Migration of fines from the core to the downstream filter, which becomes 'clogged'. There will then be overpressures in the downstream portion compared with those predicted in a conventional analysis [Peck, 1990[19]].

6. Cracks opened in the core (hydraulic fracturing), and fines migrated and plugged the core-filter interface. The fractures turned 90° along the core-filter interface and propagated along the back side of the core which became clogged over a large area. Stewart and Imrie [1993[14]] offer this explanation for the high pore pressures in the downstream portion of the core of WAC Bennett dam (Fig. 11).

7. A satisfactory hypothesis or combination of hypotheses must not only be able to explain the high pore pressures downstream, but also the decrease with time. Since the presentation of pore pressure observations from the Svartevann and WAC Bennett dams, Hydro-Québec (Canada) has reported several similar cases, and St-Arnaud [1995[20]] and Dascal [1995[21]] offer the following explanation:

- After compaction, the material in an earth core is usually not saturated (typically 50-90 per cent).
- During reservoir impoundment, air (gas) is trapped in the submerged zones (bubbles). The pressures measured by closed-system piezometers in partly saturated soils are measuring the air pore pressure which, in general, is higher than the porewater pressure.
- The pore pressures decrease, and the volume of free gases increases in the direction of flow. Furthermore, dissolved gases in the water upstream come out of solution towards downstream. Thus the degree of saturation and core permeability decrease towards downstream.
- This decrease in permeability, which can be quite significant, causes pore pressures downstream which are much higher than those predicted from a seepage analysis for a homogeneous core.
- Some blockage of the flow will exist as long as free gases are present. Reaching full saturation and steady state flow may take a long time. Until then, there will be pore overpressures in the downstream portion of the core.

There will not be one hypothesis which satisfactorily explains all cases and situations encountered or reported in the literature. However, explanation No. 7 on p54 seems well founded, and may apply to many dams, for instance Svartevann and WAC Bennett, where cracking, erosion and transportation of fines are probably not the reasons for the unexpected pore pressure behaviour. At the time the pore pressures were so much higher than predicted, and before they started to decrease, there was considerable concern. At Svartevann the decrease in measured pore pressure started after only a short time, while at WAC Bennett it took several years before the trend turned. Further theoretical and experimental studies should be undertaken to understand more fully and to quantify the different phenomena entailed in hypothesis No. 7.

Unfortunately, no record of seepage (leakage) measurements with time seems to have been presented for Bennett dam. The increase in the coefficient of permeability with increasing degree of saturation should be reflected in the quantity of flow measured*.

Application of probabilistic risk analysis in dam engineering

There has been, and is, considerable reluctance to use so-called 'probabilistic risk analysis' (PRA) in dam engineering. There may be several reasons for this; one is the term itself, which is not well defined, and is used in different ways by various authors in the literature. Another is the notion that its use requires the prior existence or collection of extensive statistics and the application of difficult mathematics. What is not generally recognized is that the concept and the approach may be

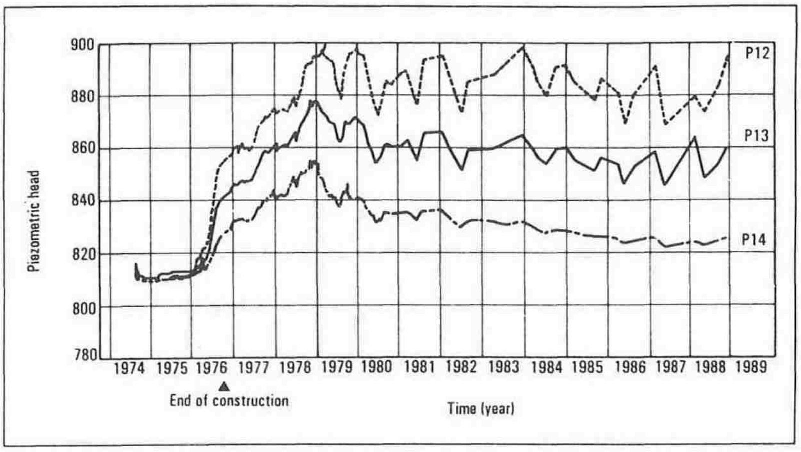


Fig. 8. Measured piezometric heads versus time from 1973 to 1989 for Svartevann dam.

used for different purposes and at different levels, for example:

- at the dam design stage, to achieve a balanced design and to place the main design efforts where the uncertainties and the consequences seem the greatest;
- as a basis for decision-making when selecting among different remedial actions and upgrading for old dams (which in a realistic situation must be done within time and financial restraints);
- to relate dam engineering risk levels to acceptable (tolerable) risk levels established by society for other activities.

The scepticism and the reluctance to use PRA may result from too much emphasis on the third item above, which indeed is a complex one, while the benefits from applications such as the first two may be overlooked.

Systematic application of engineering judgement

The concepts of probabilistic risk analyses have been around for a long time in other branches of engineering and have been discussed, for example, by Whitman [1984[22]] and at the international conference on the safety of dams in 1984 [Serafim, 1984[2]]. In Norway a simplified probabilistic risk analysis is currently being applied in the re-evaluation and recertification of existing rockfill dams and for putting priority on remedial measures. In the form used, the analysis could more appropriately be called 'systematic application of engineering judgement'. The procedure is the same as that practised by BC Hydro, Canada, which has taken a pioneering role in seeking to achieve greater consistency in risk reduction among their dams (for example, Vick and Stewart, 1996[23]). The steps in the procedure are:

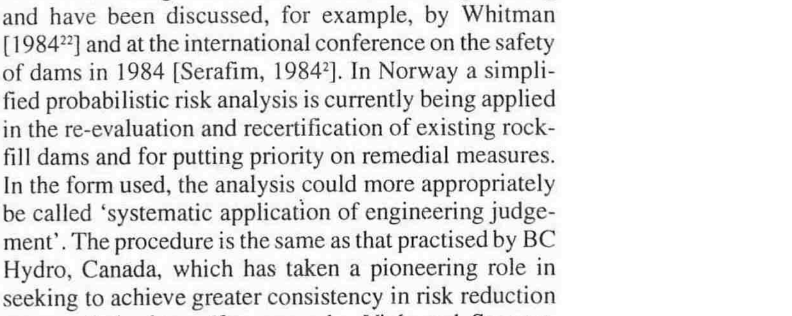


Fig. 9. Cross section of the W.A.C. Bennett dam in Canada [Stewart and Imrie, 1993[14]], where: 1 = core; 2 = transition; 3 = filter; 4 = drain; 5 = free-draining gravel; 6 = random shell; 7 = slope protection; 8 = normal max. reservoir level; 9 = grouting culvert; and 10 = grout curtain.

**On 14 June, after this paper had been prepared, a sinkhole was discovered at the crest of W.A.C. Bennett dam. Subsequent inspections below the paved roadway revealed a depression about 1 m deep and 1.5 m in diameter. A 50 mm-diameter steel pipe within a 150 mm steel casing, used as a survey benchmark founded on bedrock some 110 m below, was found at the bottom of the sinkhole. A possible mechanism of sinkhole formation is thought to be wetting-induced collapse of backfill around the benchmark casing and/or erosion around the casing. An intensive investigation has been started to explore the extent of the sinkhole and the loosened zone. In the author's opinion, the formation of the local sinkhole does not invalidate the reasoning behind the pore pressure development in the core of the dam (increase and subsequent decrease) presented earlier in this article.*

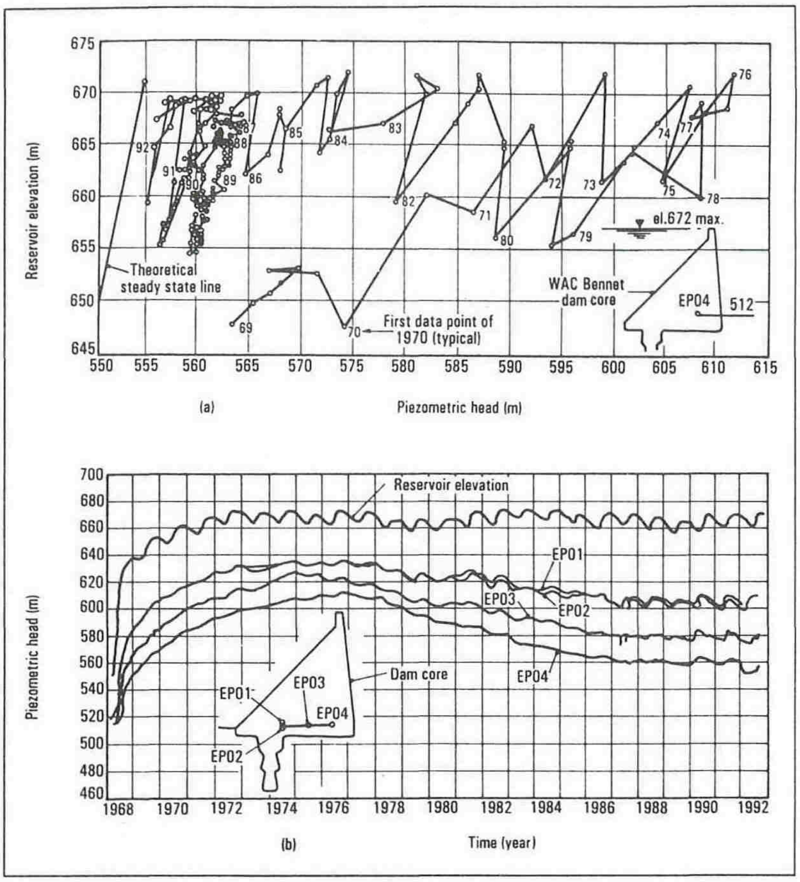


Fig. 10. Measured piezometric head versus time from 1969-1992 for the W.A.C. Bennett dam [Stewart and Imrie, 1993[14]], where: (a) shows piezometric head versus reservoir level and time in piezometer EP04, and (b) shows the variation of piezometric head with time.

Step 1: Dam and dam site inspection to refamiliarise review team with structure and site conditions

- Office review of documents alone is usually insufficient.

Step 2: Failure mode screening

- Review available incident and failure statistics for similar dams.
- Visualise and define all potential failure modes. No probabilities are assigned at this stage.
- Some failure modes are eliminated by physical reasoning, and the arguments for elimination are recorded.

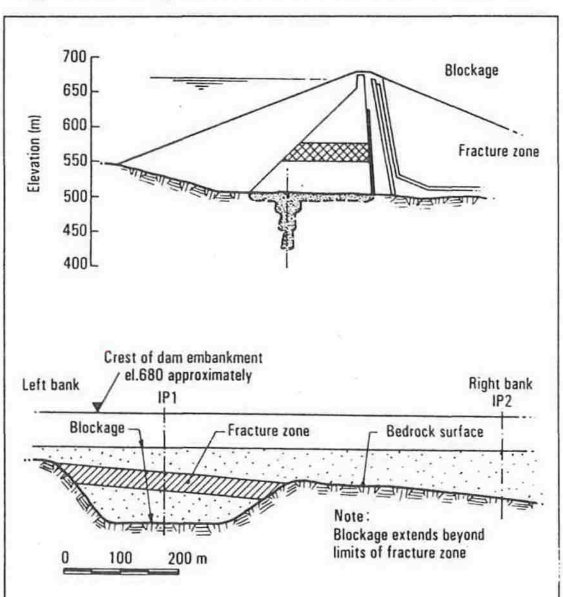


Fig. 11. Proposed explanation for the high pore pressures in the downstream portion of the core of W.A.C. Bennett dam [Stewart and Imrie, 1993[14]].

Table 4: Examples of verbal descriptors of uncertainty

Virtually impossible	0.01	(0.001)
Very unlikely	0.10	
Completely uncertain	0.50	↓
Very likely	0.90	
Virtually certain	0.99	(0.999)

Step 3: Construction of event tree

- Failure sequences are developed and detailed, displaying the interrelationships among events.
- Only those tree branch sets propagating to uncontrolled reservoir release are developed at this stage.
- Decomposition of failure sequences into their component events is essential for the next step in the procedure.

Step 4: Probability assessment

- Although helpful, this step does not require statistical information.
- Component event probabilities may be assessed using a subjective degree-of-belief approach (judgemental probability assessment) [Folayan, Høeg and Benjamin, 1970[24]].

To achieve internal consistency among team members and from one dam to another, conventions are adopted for anchoring probabilities [Vick, 1995[25]; Einstein, 1996[26]], as shown in Table 4.

Step 5: Evaluation of results

- The overall failure probability, p_f, is calculated from the component event probabilities.
- The event tree is examined to determine why certain failure modes give larger contributions than others, and possible reasons are carefully examined and reviewed.
- Where the mechanisms of failure are not well understood, the uncertainties are the largest.

Step 6: Iteration

- The initial run-through of the procedure almost always identifies some failure modes unlikely to contribute much to the overall p_f. It identifies the dam's vulnerability as well as its strengths.
- Other modes may have been overlooked during initial screening, and some may require further data or analysis.
- Following this first run-through, one refines the PRA in subsequent iterations.

Risk analysis may be considered as a diagnostic approach. The procedure provides a framework for the systematic use of engineering judgement in decision making in conditions of uncertainty.

Furthermore, the process brings together professionals from different areas of speciality in a very fruitful interaction and discussion about the dam under consideration.

Establishing 'acceptable risk levels'

The term risk implies a combination (product) of the probability of an event occurring and the consequences of the event should it occur. In the section above, risk was only discussed in terms of probability of failure (p_f) because it was assumed that all failure modes resulted in the same consequences. This is not generally valid, as different occurrences may lead to different failure events and time sequences which have an impact on the resulting consequences downstream.

Establishing 'acceptable risk levels' is a complex, difficult and controversial task, and the author does not claim to have an overview of all the learned literature published on the subject. However, with increasing frequency, society requires that analyses be performed of activities involving risks imposed on the public (as opposed to voluntary risks). At the same time, society accepts or at least tolerates some risk in terms of human life loss, damage to the environment and financial losses. Otherwise the public would not accept, for instance, the transportation systems, the chemical and nuclear plants and the health care systems we have today.

In the field of dam engineering, the first so-called F-N diagrams (frequency of occurrence versus severity of occurrence) have started to appear. Whitman [1984[22]] presented such a diagram in which he compares observations from different fields of activity, and also includes the historical statistics regarding the occurrence and consequences of natural hazards like earthquakes and hurricanes. Fig. 12 shows preliminary proposals for F-N curves for dam engineering, but these do not represent official risk criteria in any of the countries referred to. They rather serve as a starting point for further discussion.

A structure can usually be made safer by using more resources. The real challenge is through clever engineering to improve the reliability (reduce the failure probability) without spending more of society's resources. Fig. 12 presents the consequences of failure in terms of human life loss. For failure events with no potential fatalities or irreparable damage to the environment (or to professional reputation), the target annual failure probability may be decided exclusively based on economic considerations and corresponding risk analyses. A target level of 10^{-3} to 10^{-2} rather than 10^{-6} to 10^{-5} may then be a reasonable criterion.

The duty of the dam engineering profession is to explain and present the uncertainties involved, and the conventional use of a factor of safety does not do that. The profession should no longer hide behind it, and concepts from probability theory and reliability analyses should be applied [for example, Høeg and Murarka, 1974[27]; Londe, 1993[7]; Lacasse and Nadim, 1996[28]].

Summary and conclusions

Statistics presented from different parts of the world indicate the most common causes of dam incidents and failures. The causes vary somewhat from one region to another, but the statistics show that the average probability of failure per dam-year has been reduced from about 10^{-4} towards 10^{-5} over the last 30 to 40 years.

How can we best achieve a further reduction in failure probability?

- By interpretation and analyses of incidents and failures that have occurred to better understand the physical phenomena controlling the behaviour. (Further improvement in finite element/difference codes and analyses is probably not the answer.)
- By instrumentation and monitoring, which are essential elements in further dam safety enhancement. This does not require extensive research-oriented instrumentation programmes, but a limited number of carefully selected types of monitoring sensors and their locations. Data may now be automatically recorded, transferred and interpreted in real time. They should continuously be compared with predicted dam behaviour. (Monitoring requires more than a mere collection of data.)

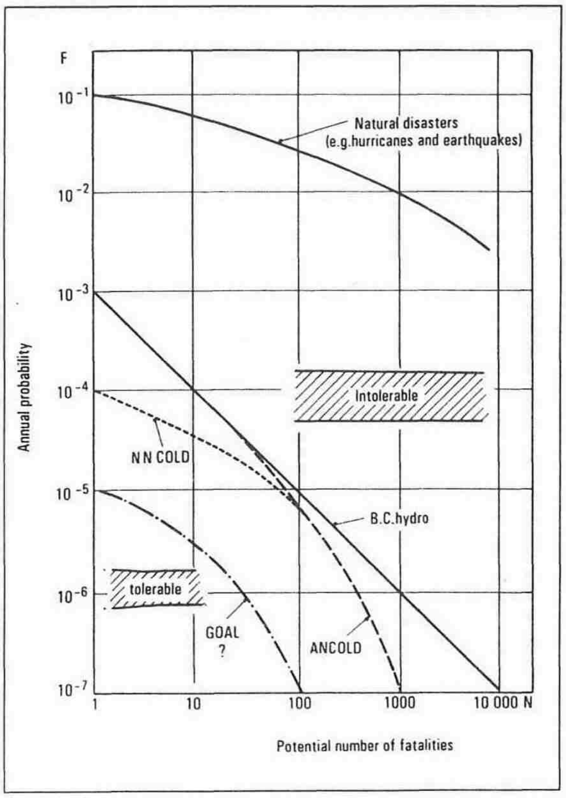


Fig. 12. Preliminary proposals for F-N curves in dam engineering.

- By systematic implementation of quality assurance and control during dam design, construction and operation. (For example, it is not sufficient to specify satisfactory filter criteria for an embankment dam if segregation of materials is overlooked during placement.)

What is 'abnormal' dam behaviour? Does the deviation from predicted behaviour indicate lowered safety?

Examples of interpretation of field measurements have been presented, focusing on:

- recorded post-construction crest settlements for embankment dams; and,
- recorded pore pressures in earth cores.

Erroneous interpretation may lead to inappropriate upgrading and repair measures.

Recorded pore pressures in the downstream portion of some earth cores, much higher than those predicted by conventional theories, may be explained by the fact that most cores are initially only partly saturated. This makes the permeability coefficient downstream significantly lower than the coefficient upstream, and therefore pore pressures downstream much higher than in a homogeneous core. As the degree of saturation increases with time, the pore pressures downstream decrease and approach those predicted from steady-state saturated flow. The process may take several years. For the cases discussed, this explanation for the 'abnormal' behaviour seems more reasonable than other explanations proposed, for example, occurrence of cracks, hydraulic fracturing, internal erosion and clogging of the downstream interface between the core and filter.

The term 'probabilistic risk analysis' (PRA) is not well defined and is used in different ways by various professionals and in the literature. This has hampered its application in engineering practice.

PRA does not require the prior existence of extensive statistics, or the application of complex mathematics, to be put to very meaningful use in a systematic application of engineering judgement and subjective probabilities.

There are three main applications of PRA in dam engineering:

- at the design stage;
- as a basis for decision-making when selecting among different remedial actions and upgrading (as a diagnostic tool); and,
- when comparing dam-related risks to acceptable/tolerable risk levels established or practised by society (F-N curves).

With increasing frequency, society demands that some form of risk analysis be carried out for activities involving risks imposed on the public (as opposed to voluntary risks). At the same time, society accepts or tolerates risks in terms of human life loss, damage to the environment and financial losses in a trade-off between extra safety and increased use of society's resources.

The duty of the dam engineering profession is to explain the uncertainties involved in the construction and operation of dams and to present the likelihood of incidents and failure in informative and meaningful terms. The conventional use of a factor of safety just does not do that, and concepts from probability theory and reliability analyses should be applied. ◊

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References

1. "Dam failures: a statistical analysis", Bulletin 99, International Commission on Large Dams, Paris, France; 1995.
2. **Serafim, J.L.**, editor, *Proceedings*, International Conference on Safety of Dams, Coimbra, Portugal (published by Balkema); 1984.
3. **Kreuzer, H., Dungar, R. and Taylor, R.**, editors, *Proceedings*, International Workshop on Dam Safety Evaluation, Grindelwald, Switzerland; 1993.
4. "Safety of existing dams: evaluation and improvement", US National Research Council, National Academy Press, Washington DC, USA; 1983.
5. **Høeg, K., Valstad, T., Kjærnsli, B. and Johansen, P.M.**, "Norwegian experience with embankment dams", *Proceedings*, Vol. 1, Symposium on Maintenance of Older Dams, Chambéry, France; 1993.
6. **Peck, R.B.**, "Where has all the judgement gone?" *Canadian Geotechnical Journal*, Vol. 17, No. 4; 1980.
7. **Londe, P.**,"Safety evaluation using reliability analaysis", *Proceedings*, Vol. 4, International Workshop on Dam Safety Evaluation, Grindelwald, Switzerland; 1993.
8. **Paré, J.J.**, "Earth dam behaviour at the La Grande project", *Proceedings*, International Conference on Safety of Dams, Coimbra, Portugal; 1994.
9. **Höeg, K., Johansen, P.M., and Torblaa, I.**, "Safety of embankment dams - causes for repair", Report 535001, Norwegian Geotechnical Institute, Oslo, Norway; 1996.
10. **Soydemir, C. and Kjaernsli, B.**, " A treatise on the performance of rockfill dams with unyielding foundations in relation to the design of Storvatn Dam. Report 53203, Norwegian Geotechnical Institute, Oslo, Norway; 1975.
11. **Clements, R.P.** "Post-construction deformation of rockfill dams", *Journal of Geotechnical Engineering*, ASCE, Vol. 110, No. 7, 1984.
12. **Dascal, O.**, "Post-construction deformations of rockfill dams", *Journal of Geotechnical Engineering*, ASCE, Vol. 113, No. 1; 1987.
13. **Myrvoll, F., Larsen, S., Sande, A. and Romslo, N.B.**, "Field instrumentation and performance observations for the Vatnedalen dams", *Proceedings* Vol. 1, 15th ICOLD Congress, Lausanne, Switzerland; 1985.
14. **Stewart, R.A. and Imrie, A.S.**, "A new perspective based on the 25 year performance of WAC Bennett Dam", *Proceedings* Vol. 1, International Workshop on Dam Safety Evaluation, Grindewald, Switzerland; 1993.
15. **de Mello, V.**, "Practice, precedents, principles, problems and prudence in embankment dam engineering". Symposium on Problems and Practice of Dam Engineering, Bangkok, Thailand; 1980.
16. **Vaughan, P.R.**, "Non-linearity in seepage problems - theory and field observations", de Mello Volume, BiTech Publisher Ltd., Vancouver, Canada; 1989.
17. **Sherard, J.L.**, Discussion of Dascal's article in: *Proceedings*, International Conference on Case Histories in Geotechnical Engineering, Vol. IV, University of Missouri-Rolla, USA; 1984.
18. **Sherard, J.L.**, "Hydraulic fracturing in embankment dams", *Journal of Geotechnical Engineering*, ASCE, Vol. 112, No. 10; 1986.
19. **Peck, R.B.**, "Interface between core and downstream filter", H. Bolton Seed Memorial Symposium, Vol. 2, USA; 1990.
20. **St-Arnaud, G.**, "The high pore pressures within embankment dams: an unsaturated soil approach", *Canadian Geotechnical Journal*, Vol. 32, No. 5; 1995.
21. **Dascal, O.**, "Pore pressure development in moraine (till) cores of embankment dams", Preliminary Report, Hydro-Québec, Canada; 1995.
22. **Whitman, R.V.**, "Evaluating calculated risk in geotechnical engineering", *Journal of Geotechnical Engineering*, ASCE, Vol. 110, No. 2; 1984.
23. **Vick, S.G. and Stewart, R.A.**, "Risk analysis in dam safety practice", ASCE Geotechnical Special Publication No. 58, Uncertainty in the Geologic Environment: From Theory to Practice; 1996.
24. **Folayan, J.I., Høeg, K., and Benjamin, J.R.**, "Decision Theory Applied to Settlement Predictions", *ASCE Journal of Soil Mechanics and Foundations Division*, Vol.96, No. SM4; July 1970.
25. **Vick, S.G.**, "Geotechnical risk and reliability - from theory to practice in dam safety", *Proceedings*, The Earth, Engineers, and Education (Whitman Symposium, Massachusetts Institute of Technology, Cambridge, USA); 1995.
26. **Einstein, H.H.**, "Risk and risk analysis in rock engineering". *Tunnelling and Underground Space Technology*, Vol. 11, No. 2; 1996.
27. **Høeg, K. and Murarka, R.P.**, "Probabilistic analysis and design of a retaining wall", *Journal of Geotechnical Engineering*, ASCE, Vol. 100, No. 3; 1974.
28. **Lacasse, S. and Nadim, F.**, "Uncertainties in characterising soil properties", ASCE Geotechnical Special Publication No. 58, Uncertainty in the Geologic Environment: From Theory to Practice; 1996.

Bibliography

Lafitte, R., "Probabilistic risk analysis of large dams: its value and limits", *Water Power & Dam Construction; March, 1993.*

Lombardi, G., "On the limits of structural analysis of dams", *Proceedings*, International Conference on Research and Development in the Field of Dams, Crans-Montana, Switzerland; 1995. (Also: *Hydropower & Dams*, Issue 5, 1996.)

Salmon, G.M. and Hartford, D.N.D., "Risk analysis for dam safety", *Water Power & Dam Construction*, March and April, 1995.

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