INTERNAL EROSION OF EXISTING DAMS, LEVEES AND DIKES, AND THEIR FOUNDATIONS

BULLETIN 1XX

Volume 1: INTERNAL EROSION PROCESSES AND ENGINEERING ASSESSMENT

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CIGB ICOLD

22 January 2013

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FOREWORD AND ACKNOWLEDGEMENTS

Internal erosion and piping in embankments and their foundations is the main cause of failures and accidents at embankment dams. For new dams, the potential for internal erosion can be controlled by good design and construction of the core of the dam and provision of filters to intercept seepage through the embankment and the foundations.

However many existing dams, dikes and levees were inadequately zoned and not provided with filters or have filters or transitions which do not satisfy modern filter design criteria, and are susceptible to internal erosion failure, with a risk of internal erosion increasing with ageing. Accordingly, the reassessment of the safety of these dams is an important issue.

This has been recognized by the European Club of ICOLD which devoted a Working Group to the problem and much research has been carried out in recent years to better understand the physical processes and mechanics of internal erosion. This has built upon the knowledge gained from many years of earlier research and successful design of new dams.

The ICOLD committee on Embankment Dams believe that there are considerable benefits in gathering together international experts and researchers on this topic. Thus the chairman committed the members Jean-Jacques Fry (France) and Rodney Bridle (UK) to work with the Working Group of the European Club and other experts on internal erosion to write a bulletin and give guidance for practitioners responsible for assessing the safety of water retaining structures.

We expect that the people in charge of dam safety will have free access to this publication so it can be used effectively for the safety assessment of existing dams and dikes. It will also be useful for those designing new dams and dikes.

As the state of the art of internal erosion of dams, dikes and their foundations is an evolving science readers should look in the literature for new developments following the production of this Bulletin.

I also wish to record special thanks to Dr Jean-Jacques Fry, Mr Rodney Bridle, and Emeritus Professor Robin Fell (Australia) for undertaking much of the drafting of this Bulletin, to members of the European Club Working Group for their inputs in developing the Bulletin, and to members of the ICOLD committee for their contributions.

Jean-Pierre TOURNIER,

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Committee on Embankment Dams

EXECUTIVE SUMMARY

Internal erosion is one of the major causes of embankment dam failure. When constructing new dams, protection against internal erosion is provided by zoning and by providing filters (ICOLD Bulletin No. 95, ICOLD, 1994). However, many existing dams are not adequately zoned and do not have filters and may therefore be vulnerable to internal erosion. Others have filters not designed and/or constructed to modern standards, they too may be vulnerable to internal erosion.

Internal erosion occurs when soil particles within an embankment dam or its foundation are carried downstream by seepage flow. It starts when the erosive forces imposed by the hydraulic loads exceed the resistance of the materials in the dam to erosion. The erosive forces are directly related to reservoir water level.

Many internal erosion problems become apparent on first filling but most occur many years after first filling. This is almost certainly because the dam is subject for the first time to high water levels which may initiate erosion in the upper parts of the dam. After long periods of flood flows, for example, which cause the reservoir to rise higher on the dam and/or to subject it to greater gradients and velocities of flow than it has experienced before. It may be because ageing causes deterioration of the dam, particularly of the pipes and culverts through dams but also due to on-going settlements under repeated reservoir draw downs, or from the effect of earthquakes. Another factor may be that some internal erosion mechanisms progress slowly in an episodic fashion under cyclical reservoir loading, taking years and even decades before the effects of the episodes of erosion show any of the symptoms of internal erosion.

The Bulletin is in two volumes. Volume 1 deals predominantly with internal erosion processes and the engineering assessment of the vulnerability of a dam to failure or damage by internal erosion, with a brief oversight of monitoring for and detection of internal erosion and remediation to protect dams against internal erosion. It includes a comprehensive listing of the Terminology used in internal erosion. Many references are also given, including links to many from an ICOLD internal erosion webpage.

Volume 2 gives more details of internal erosion investigations, and appropriate testing, monitoring and detection, and remediation, and gives case histories.

This Volume 1 gives a statement of the problem, explaining why internal erosion is a threat to existing dams and the importance of assessing the vulnerability of individual dams to it. It then goes through the overall process of erosion from initiation, through continuation (or arrest) of erosion, through progression, and on to breach, unless erosion is detected early enough by appropriate monitoring systems, to allow timely intervention to halt or slow the development of a breach and failure.

The four mechanisms through which internal erosion is initiated are described: erosion in concentrated leaks, backward erosion, contact erosion, and suffusion. The Bulletin then describes the circumstances in which erosion, once initiated, will continue or be arrested by the filtering capability of the materials in the dam and foundations. Filtering processes, filter properties and filter rules are explained in order that the filtering capability can be assessed. The rate at which erosion continues, if it is not arrested, is discussed. In many cases, the rate is such that failure could occur too rapidly to give time to protect lives and property downstream, and some positive action should be taken, usually retrofitting filters, to prevent erosion.

An engineering assessment process is proposed. Assessment by the deterministic approach, by engineering judgment augmented by qualitative and semi-quantitative risk assessment; or by quantitative risk assessment, is described. Critical design loads relate to critical water levels. Systematic consideration of potential failure modes is required, eliminating those in which internal erosion could not continue to failure, because of fill and foundation soil properties, the zoning of the dam and other factors, as detailed in the Bulletin.

The remaining potential failure modes must then be examined. Information is given in the Bulletin to determine the erosion resistance, the hydraulic gradients and the soil properties at which erosion will initiate. Information is also given on how to assess the filtering capability of the materials in the dam and foundation and their ability to stop erosion, allow a limited amount of erosion which would then stop, or allow erosion to continue unchecked. Some information on remediation methods and monitoring is given to assist in decision making where the assessment has not given a definite conclusion on whether damaging erosion will or will not occur at the dam. Quantitative risk assessment methods are briefly described. Such methods may also be used to resolve uncertainty, and would be advisable in the case of high hazard dams and high remediation costs to confirm and justify the need for and the scale of the proposed remediation.

1. THE IMPORTANCE OF INTERNAL EROSION FOR DAM SAFETY, STATISTICS OF FAILURE, AND THE OUTLINE OF THIS BULLETIN

1.1 THE PURPOSE, SCOPE AND OUTLINE OF THIS BULLETIN

Internal erosion occurs when soil particles within an embankment dam or its foundation are carried downstream by seepage flow.

Internal erosion is a major cause of dam failures and of incidents that threaten the safety of dams. Most internal erosion failures have occurred on first filling as weaknesses in the dam or its foundation were 'discovered' by the rising water. However, failures do occur after first filling and internal erosion remains a threat to existing dams because:

- They have not been subjected to, or designed to resist, extreme loads such as extreme water level and earthquakes (which can cause settlement cracking),Cracking occurs as a result of cycling of water level, differential settlement and desiccation,
- Ageing causes deterioration, particularly deterioration of conduits, spillways and other structures through dams, at which internal erosion may be initiated
- They may not be protected against internal erosion by filters, or if filters or transition zones are present, they may not have been designed to modern standards and may be ineffective.

The implications of internal erosion and assessment of the vulnerability of dams to it have been addressed in recent years by risk assessment methods pioneered in Australia, North America and Europe (e.g. ICOLD, 2005; Fell & Fry, 2007; Hartford & Baecher, 2004; Brown & Gosden, 2004; Bowles et al, 2003). The end product of the methods is an estimate of the probability of failure of the subject dam. When coupled with a dambreak study from which likely loss of life and cost of damage from a failure can be estimated, an assessment of whether the subject dam is safe enough can be made based on assessments of acceptable societal risk. If it is not, the effects of safety works and improved monitoring can be assessed and implemented as necessary.

In developing the risk assessment approach researchers have improved understanding of the mechanisms that can result in dam failure by internal erosion. This improved understanding, when added to existing knowledge of seepage forces and hydraulic conditions in an embankment dam and its foundations, makes it possible to assess the ability of a dam to resist the forces imposed on it that may cause internal erosion to initiate, continue and progress to failure. This bulletin is based on the application of this improved understanding of the mechanics of internal erosion to assess whether existing dams will resist erosion, whether safety works are required to resist it or whether improved monitoring will be a sufficient safeguard against it.

The objective of the bulletin is to give practical guidance for dam owners, dam engineers and regulators on dealing with the threat of internal erosion at existing dams. It explains the mechanics of internal erosion and how to apply them to make an engineering analysis of the ability of dam to withstand extreme internal erosion loads. It recommends a staged iterative approach to the analysis, gradually improving data inputs if necessary to reach safe conclusions. It advises broadly on safety works should they be needed and on appropriate monitoring regimes.

It advises on the selection of critical internal erosion loads and notes that these occur when water level is highest, the same as that occurring during the passage of the critical flood.

The engineering analyses are intended to be used independently, but quantitative risk assessments will complement them, and for important dams it is recommended that both approaches be used.

The Bulletin was written largely for assessing existing dams, but the principles may be applied to new dams. The Bulletin was written mainly for large dams, but is applicable to small dams, dikes, levees and their foundations.

The Bulletin includes:

Statistics on dam failures and accidents caused by internal erosion

An overview of the internal erosion process, including the need to consider the complete process from application of the load, to initiation of erosion and through to the potential for continuation and progression of erosion and breach of the dam.

Details of methods which are currently available for assessing if concentrated leak erosion, backward erosion, contact erosion and suffusion will initiate and the ability of dams to resist erosion if it has initiated, and a summary of on-going research into these processes.

Advice on engineering assessment of the vulnerability of dams to internal erosion by three approaches: deterministic; engineering judgment backed by qualitative and semi-quantitative risk assessment; and quantitative risk assessment. Advice is also given on identifying Potential Failure Modes and screening out improbable ones.

Advice on remedial measures which may be taken if needed to make the dam resistant to internal erosion; and on methods for monitoring and surveillance of internal erosion to assist in decision-making if the results of the engineering analyses are not clear.

Brief descriptions of quantitative risk assessment methods are given and it is suggested that such methods may also be used to resolve uncertainty and would be advisable to justify the need for and scale of remediation at high hazard dams.

Terminology commonly used in the internal erosion process, and in risk assessment of internal erosion.

The sound assessment of potential internal erosion mechanisms has to be based on adequate subsurface exploration, assessment of data from site investigations prior to the construction of the dam, construction records, and analysis of the data from instruments within the dam and its foundation, and include field and laboratory testing. A staged approach is recommended, collecting and applying more data gradually if needed to decide if safety works are needed to make the dam adequately safe.

Volume 2 of the Bulletin provides details of laboratory tests which measure properties of erosion and filter capability of soils; monitoring methods including several newly developed techniques, and information on case studies relevant to the subject.

1.2 THE IMPORTANCE OF INTERNAL EROSION TO DAM SAFETY

Internal erosion is an important safety issue for large and small dams, dikes and levees as shown by the statistics of historic of failures and incidents which are discussed below.

Internal erosion occurs when soil particles are carried by seepage in embankments, foundations, from embankment to foundation, around and into conduits through embankments and adjacent walls supporting embankments.

The statistics of embankment dam incidents show that internal erosion is a significant cause of incidents and to a lesser extent failure for older dams. Incidents include new or increased seepage and leakage, sinkholes and accelerating settlements of the dam. There is also evidence that incidents and failures are more likely along the interface between the embankment fill and structures such as culverts and spillway walls. An understanding of internal erosion is important to dam owners as this:

- Is necessary to understand and interpret the behavior of the dam
- Is necessary to assess the safety of the dam, and whether it is safe enough or whether safety works are needed to make it so
- Should govern the dam safety surveillance and monitoring regimes

The cause of these incidents can include poor compaction and incorrectly designed filters which even for many relatively modern dams may not be adequately designed or constructed to control internal erosion. This is particularly important in the upper parts of dams where cracking is most likely to occur and filters are often omitted or were not constructed to modern standards, or were not included at the interface with structures. For many dams revised flood forecasts indicate the upper parts may now be required to hold the reservoir under conditions for which they were not designed.

Four mechanisms of initiation have been identified and are described in the Bulletin. The assessment of filters and transition zones and downstream zones to act as filters is well understood and in many dams it can be demonstrated that these will provide adequate filter protection even if they do not meet modern filter design criteria. Tools to carry out an assessment of the ability of the dam to resist internal erosion are also included in this Bulletin.

Many new and potentially useful methods for monitoring and detecting internal erosion and seepage have been developed in recent years, and are described in the accompanying Bulletin on Detection and Monitoring of Internal Erosion. These and long established methods such as visual inspection, leakage measurement and monitoring of pore pressures with piezometers are valuable tools in keeping dams safe by detecting internal erosion early in the process.

1.3 STATISTICS OF EMBANKMENT DAM FAILURES

Based on the records of dam incidents and the dam register in ICOLD (1974, 1983, 1995), Foster et al, (1998, 2000a) analyzed the statistics of dam failures for large dams constructed between 1800 and 1986, excluding dams constructed in Japan pre 1930 and in China (adapted from Foster et al, 1998, 2000a). This is summarized in Table 1.1 below:

Table 1.1 Statistics of embankment dam failures

Failure Mechanism	Erosion		Embankment Sliding	
Mode of failure	External erosion (overtopping)	Internal erosion	Static Instability	Seismic instability
% over the world	48%	46%	4%	2%
% over the world	94%		6%	

It can be seen that internal erosion has been responsible for about half of embankment dam failures where the mode of failure is known. It is approximately equal in importance to failures by overtopping in floods due to inadequate spillway capacity and malfunction of gates and other outlets leading to overtopping.

By comparison embankment slides and failures due to earthquakes account for only 4% and 1.7% of embankment dam failures in operation respectively.

Table 1.2.Historical frequencies of failures and accidents in embankments of large dams constructed from 1800 to 1986, excluding dams constructed in Japan pre 1930, and in China (adapted from Foster et al, 1998, 2000a)

CASE	TOTAL	IN DAM	AROUND CONDUITS OR WALLS
Internal erosion failures	36	19	17
Internal erosion accidents	75	52	23
Seepage accidents with no detected erosion	36	30	6
Total no. of failures and accidents	146	101	46
Population of dams	11192	11192	5,596
Historical frequency for failures and accidents	0.013	0.009	0.0082
Proportion of failures and accidents on first fill	36%		
Proportion of failures and accidents after first fill	64%		
Historical frequency for first fill		0.0032	0.0030
Historical frequency after first fill		0.0058	0.0052
Historical annual frequency after first fill		2.2×10^{-4}	2.0×10^{-4}

As Table 1.2 shows, overall about 1 in 80 embankment dams have failed or experienced an accident by internal erosion. There are about 4 failures per 10,000 dams per year from all causes about 2 of the failures are from internal erosion.

The largest number of failures has occurred in the embankment. Nearly half of these are associated with conduits which penetrate through the embankment, or walls which support the embankment.

There are many more accidents than failures. As defined by ICOLD (see Terminology), accidents are incidents which have been prevented from becoming failures by immediate remedial measures, including possibly drawing down the water in the reservoir. There are a similar number of accidents relating to internal erosion in the foundation as in the embankment.

For all internal erosion modes about two-thirds of all failures and about half of accidents occur on first filling or in the first 5 years of operation. This means however that around half of all incidents have occurred in dams after 5 years of operation.

Foster et al (1998, 2000a) found that nearly all internal erosion failures in the embankment occurred when the reservoir level was at the highest level ever, or within one meter of that level. For internal erosion in the foundation the reservoir level was not as important.

There is evidence from the ICOLD data in Table 1.3 below that the later statistics are better reflecting improved design and construction methods. However the problems are on-going.

Table 1.3: Failure statistics for embankment dams excluding China from 1970 to 1989 (adapted from Foster et al, 1998, 2000a)

Modes of failure	Ratio of failures to the number of dams 1970-1979	Ratio of failures to the number of dams 1980-1989
Internal erosion	0.0020	0.0016
Overtopping and appurtenant structure failure resulting in overtopping	0.0026	0.0019
Sliding	0.0004	0.0001

There has not been a systematic study of international failure and accident statistics since the ICOLD (1995) study. However there have been studies for individual countries. Li (2012) provides statistics of dam failures in China.

Brown and Gosden (2008) estimated that in the United Kingdom's population of about 2,500 dams registered under the Reservoirs Act, there are about 1,600 incidents for every recorded failure. About 60% of the incidents appear to be related to internal erosion, with typically two serious erosion incidents a year. It should be noted that the definition of incident used in this study is not the same as in ICOLD (see Terminology). It includes incidents which are unlikely to have resulted in failure. The "severe incidents" however would classify as ICOLD accidents.

Engemoen and Redlinger (2009) and Engemoen (2011, 2012a) carried out detailed analyses of internal erosion incidents for the Bureau of Reclamation's 220 embankment dams. They found that the successful performance of a dam does not preclude it from having an internal erosion incident; and that 99 dams had experienced incidents, of which only one was a failure; 53 involved definite particle transport, and the remainder were excessive seepage or sand boils. Of the 99 incidents, 9 were in the embankment, 70 in the foundation, 6 from embankment into a conduit, 5 were into or along a conduit and 11 were into toe or under drains. The dams store water for irrigation purposes and are usually filled to 'normal high pool level' (to with 7-12 feet of crest) at the beginning of each year. Thus, "full" or "high" pool levels occur in most years. They found that internal erosion can occur at any time in a dam's life; but it is likely that most incidents occurred when the pool was relatively close to normal high level. They developed annualized statistics of failures and incidents from these data. The definition of "incident" used by these authors is not the same as the ICOLD definition. Schaefer et al (2011) describe some recent internal erosion incidents in USACE dams.

2. AN OVERVIEW OF INTERNAL EROSION MECHANISMS

2.1 A DESCRIPTION OF THE OVERALL PROCESS

Failures and incidents by internal erosion of embankment dams and their foundations are categorized into three general failure modes, which are;

- Internal erosion through the embankment, which includes internal erosion associated with through-penetrating structures, such as conduits associated with outlet works, spillway walls or adjoining a concrete gravity structure supporting the embankment.
- Internal erosion through the foundation, and
- Internal erosion of the embankment into or at the foundation. Including (a) seepage through the embankment eroding material into the foundation, or (b) seepage in the foundation at the embankment contact eroding the embankment material.

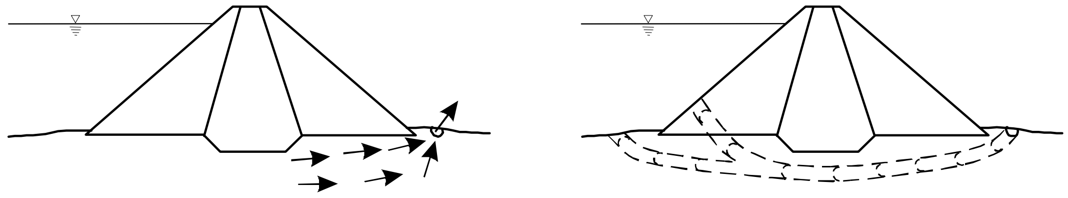
The process of internal erosion may be broadly broken into four phases:

- initiation of erosion,
- continuation of erosion,
- progression to form a pipe or occasionally cause surface instability (sloughing), and
- Initiation of a breach.

This is shown in Figure 2.1(A) for internal erosion through the embankment initiated by a concentrated leak. Similar processes apply for piping through the foundation, and from the embankment to the foundation and are shown in Figure 2.1(B) and (C).

INITIATION	CONTINUATION	PROGRESSION	BREACH
Concentrated leak forms, erosion initiates along walls of crack	Continuation of erosion	Enlargement of concentrated leak	Breach mechanism forms

(A) INTERNAL EROSION IN THE EMBANKMENT INITIATED BY EROSION IN A CONCENTRATED LEAK



INITIATION → CONTINUATION → PROGRESSION → BREACH

Leakage exits from the foundation and backward erosion initiates	Continuation of erosion	Backward erosion in progresses to form a pipe	Breach mechanism forms

(B) INTERNAL EROSION IN THE FOUNDATION INITIATED BY BACKWARD EROSION

INITIATION CONTINUATION PROGRESSION BREACH

Leakage exits the core into the foundation and backward erosion initiates as core erodes into the foundation	Continuation of erosion	Backward erosion progresses to form a pipe. Eroded soil is transported in the foundation	Breach mechanism forms

(C) INTERNAL EROSION FROM THE EMBANKMENT TO FOUNDATION INITIATED BY BACKWARD EROSION

Figure 2.1 - Models for the development of failure by internal erosion (Foster and Fell 1999b)

2.2 THE MECHANICS OF EROSION – PARTICLE DETACHMENT

The first condition for internal erosion to occur is particle detachment. Water seeping through the dam or flowing in cracks must provide sufficient energy to detach particles from the soil structure.

The nature of the soil in the dam determines its vulnerability to erosion. Three classes have to be distinguished:

- Non plastic (cohesionless) soils such as silts, sands and gravels. These may collapse when saturated under flooding, will not sustain a crack when saturated, and are relatively easily eroded. As non-plastic soils become coarser they progressively require more energy to initiate erosion. These soils are subject to backward erosion, contact erosion or suffusion depending on their particle size distribution.

- Plastic soils, such as clays, clayey sands, and clayey sandy gravels are generally more resistant to erosion than cohesionless soils. These soils are subject to concentrated leak erosion and contact erosion. Clay soils will hold a crack even when saturated. Higher energy is generally required to detach particles from cracks within a cohesive fill but the particles removed are small and easily carried through the crack. Backward erosion and suffusion cannot occur in plastic soils under the gradients normally experienced in dams and their foundations.

- Dispersive soils are plastic (clay) soils in which the chemistry of the seeping water causes dispersion (de-flocculation) of the clay flocs, breaking them down into smaller, easily eroded, particles.

More details are given in Section 9.7.2, including the influence of the degree of saturation on soils when used as fill materials in dams.

2.3 THE FOUR MECHANISMS OF INITIATION OF EROSION

Initiation of erosion occurs in four mechanisms:

- Concentrated leaks
- Backward erosion
- Contact erosion
- Suffusion

2.3.1 Concentrated leaks

Where there is an opening in plastic soil, and sometimes in unsaturated silt, silty sand or silty sandy gravel, through which concentrated leakage occurs, the sides of the opening may be eroded by the leaking water.

Such concentrated leaks may occur through a crack caused by differential settlement during construction of the dam or in operation, by hydraulic fracture due to low stresses around conduits or in the upper parts of the dam due to differential settlement, or through desiccation at high levels in the fill. Frost action also creates cracks in dam crests. Figures 2.2, 2.3 and 2.4 show different situations where cracks and gaps may occur.

Concentrated leaks may also occur because of collapse settlement of poorly compacted fill in the embankment, around conduits, and adjacent to walls. They may also occur due to the action of animals burrowing into levees and small dams and tree roots rotting in dams and forming holes.

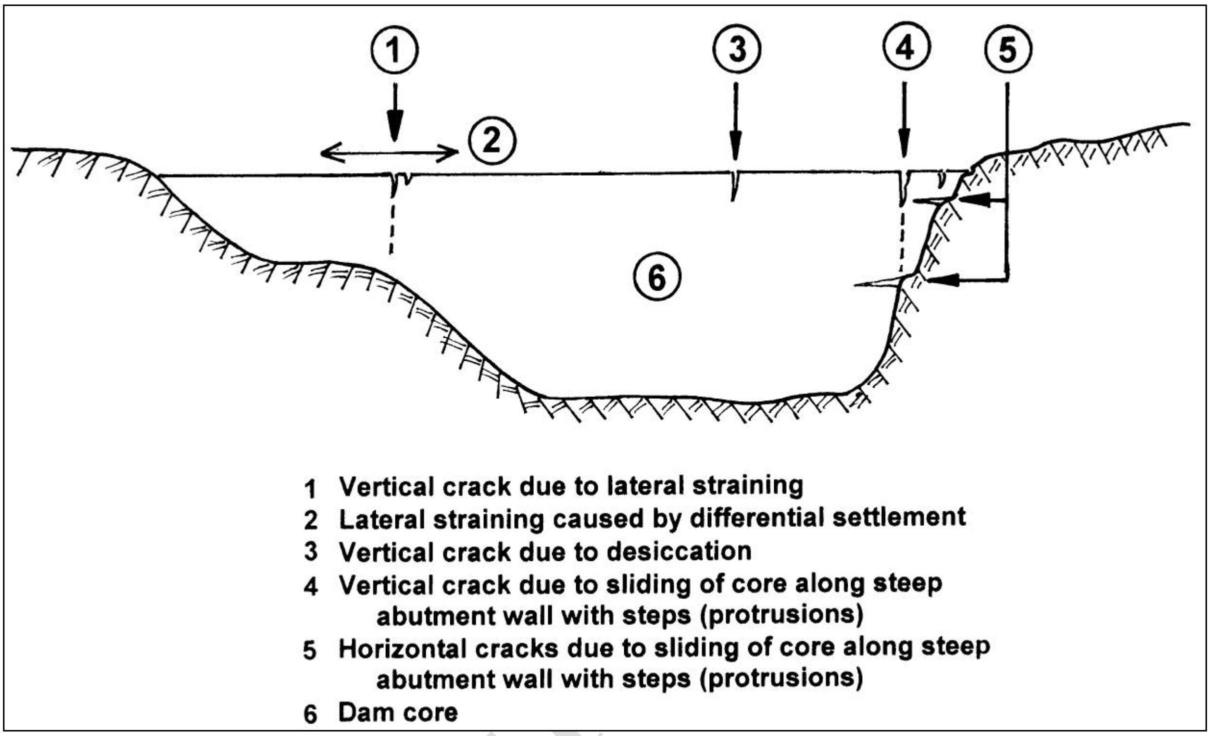


Figure 2.2 Situations in which cracks may form from desiccation or from cross valley differential settlement. Such settlement may lead to tension cracks, or tearing at steep abutment slopes or steps; or the lateral strain resulting from the settlement may result in low stress zones in which hydraulic fracture may occur.

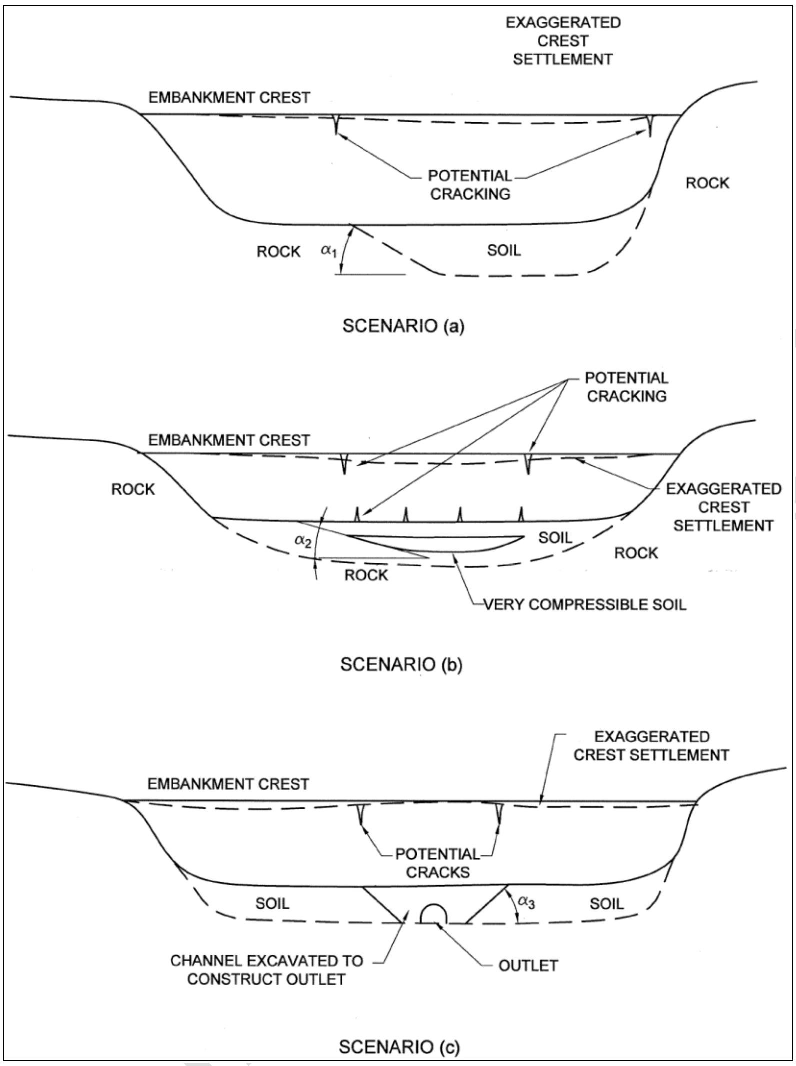


Figure 2.3 Situations which may cause differential settlement in the foundation of dams leading to cracking, lateral strains and low stress zones subject to hydraulic fracture (Fell et al 2008, based on Sherard et al, 1963)

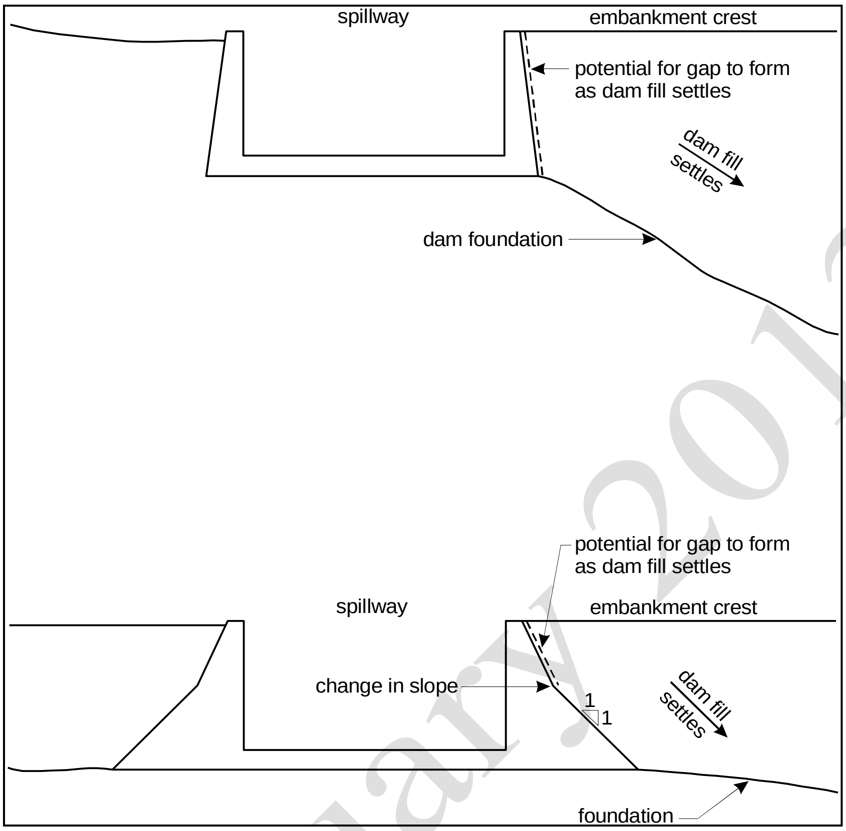


Figure 2.4 Situations where a gap may form between the dam fill and spillway wall (a) Steep foundation adjacent to spillway wall; (b) Change in slope of the retaining wall. (Fell et al, 2005)

2.3.2 Backward erosion

There are two kinds of backward erosion:

- Backward erosion piping, at the back (upstream) end of a very thin pipe below a "roof". It mainly occurs in foundations, as in Figure 2.5, but may occur within embankments.

- Global backward erosion, leading to development of a near-vertical pipe in the core of an embankment as in Figure 2.7.

Both occur in non-plastic soils.

2.3.2.1 Backward erosion piping

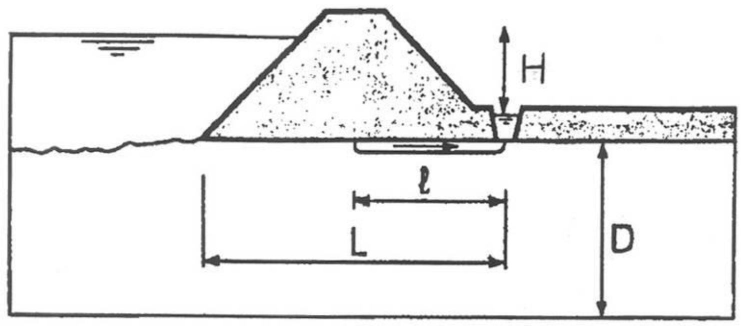


Figure 2.5 Backward erosion piping at back (upstream) end of a thin pipe below a "roof" (Koenders and Sellmeijer 1992)

The erosion process begins at a free surface on the downstream side of a dam or dike. This free surface may be in a ditch or other excavation penetrating into the eroding non-plastic (cohesionless) soil or may form by first heaving of the plastic (cohesive) strata overlying the non-plastic soil.

The process progresses beneath the dike or dam. For this to occur, the dike or dam, or the cohesive strata must form a roof for the eroding "pipe". The presence of backward erosion piping is often exhibited by the presence of sand boils at the downstream side of the dam or dike. Figure 2.6 shows an example of a sand boil.

Figure 2.6 Sand boil on the land side of a dike.

Backward erosion piping occurs where critically high hydraulic gradients at the toe of a dam erode particles upwards and backwards below the dam through small erosion channels and flow velocity can transport the eroded particles downstream.

2.3.2.2 Global backward erosion

There is a potential for global backward erosion in narrow or even reasonably wide sloping or partially sloping core dams, such as that shown in Figure 2.7, if they are not properly protected by filters or a transition zone.

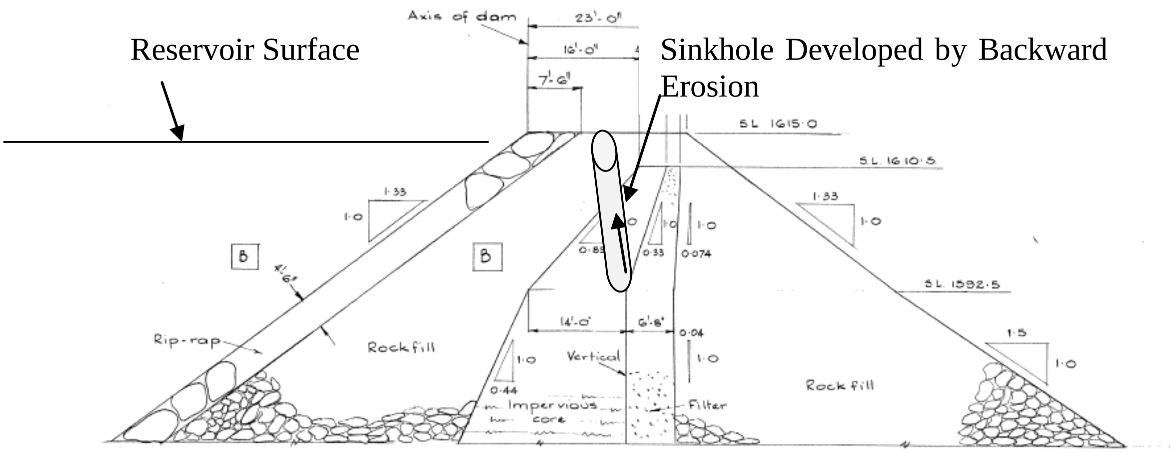


Figure 2.7 Potential mode of ‘global backward erosion’ leading to formation of a sinkhole in a sloping core.

Particles are detached at the downstream surface of the core which is inadequately protected by the filter or transition zone. The progression of the erosion process is assisted by gravity, and there is no need for a cohesive soil layer to form the roof for the pipe. It may be one of the causes of sinkholes in dams constructed of glacial tills.

The possibility of global backward erosion occurring in central core dams is discussed in Section 4.5 and the possibility of global backward erosion leading to unraveling of downstream slopes is dealt with in Section 4.6.

2.3.3 Contact erosion

Contact erosion occurs where a coarse soil such as a gravel is in contact with a fine soil, and flow parallel to the contact in the coarse soil erodes the fine soil. For example, flow through gravel alluvium in the foundations of dam or dike may erode the base of an overlying silt layer, or erosion of the finer layers of soil in a core may occur into a coarse gravelly layer formed by segregation during construction. Figure 2.8 shows examples.

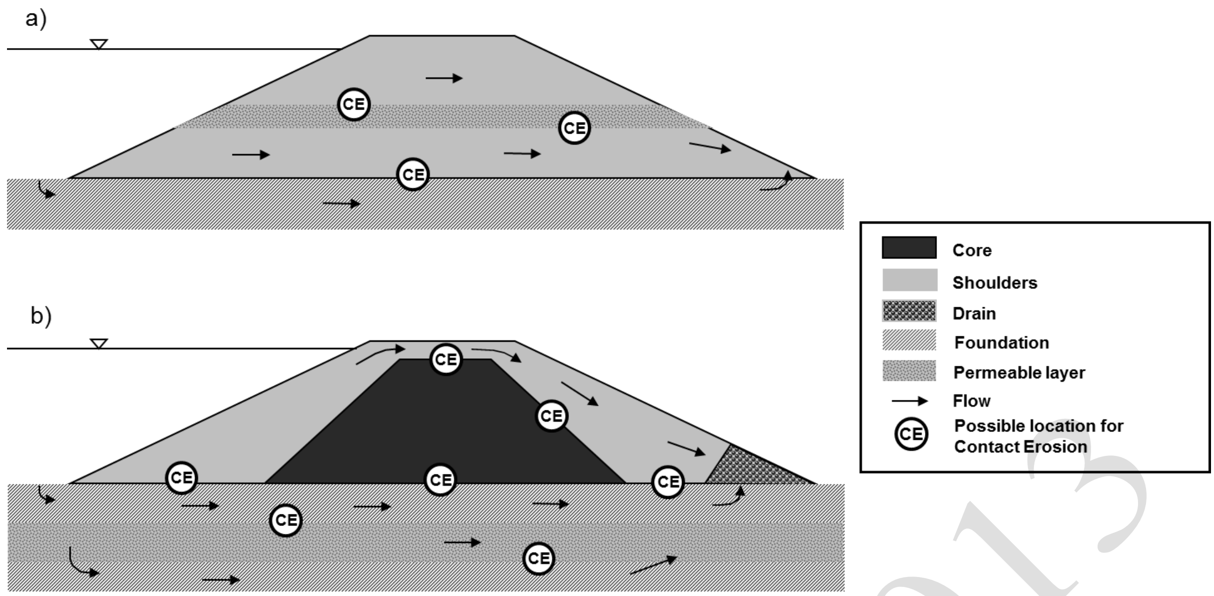


Figure 2.8: Possible location of contact erosion initiation. a) Homogeneous dam with layered fill due to segregation during construction and a coarse foundation soil. b) Zoned dam with potential for contact erosion at high reservoir levels above the core and for erosion into coarse layers in the foundation (Beguin et al, 2009).

2.3.4 Suffusion

Suffusion occurs when water flows through internally unstable widely graded or gap graded non plastic soils. Glacial tills are sometimes vulnerable to suffusion. Some fills and filters in dams also have very broad or gap-gradings, and contain excessive fines content.

The small particles of soil are transported by the seepage flow through the pores of the coarser particles. The coarser particles are not transported and the effective stresses are largely transferred through the matrix of the coarser particles.

Figure 13.3 (see Internal Instability in Terminology) shows how the "finer soil" fraction is defined.

Suffusion results in an increase in permeability, greater seepage velocities, and potentially higher hydraulic gradients, possibly accelerating the rate of suffusion. Stable situations may be reached with some of the finer fraction remaining in equilibrium with the seepage stresses. Suffusion may re-commence during cycling periods of water loads or during higher reservoir or river water level.

Suffusion occurring within an embankment core or the foundation of a dam may also lead to some settlement of the embankment.

A filter constructed of internally unstable materials will have a potential for erosion of the finer particles in the filter, rendering the filter coarser and less effective in protecting the core materials from erosion.

It should be noted that segregation of broadly graded or gap graded non-plastic soils during placement in the dam may create layers which are internally unstable even though the average grading of the soil is internally stable.

2.4 CONTINUATION AND FILTER ACTION

Erosion once initiated will continue unless the eroding forces are reduced or the passage of the eroded particles is impeded in some way.

Dam engineers have known since the 1950s and 1960s that the most efficient way of stopping the erosion process is to zone the dam and to provide filters. Filters were then mainly designed with the Terzaghi criteria (Terzaghi and Peck, 1948).

Figure 2.9 shows the types of embankment dam zoning.

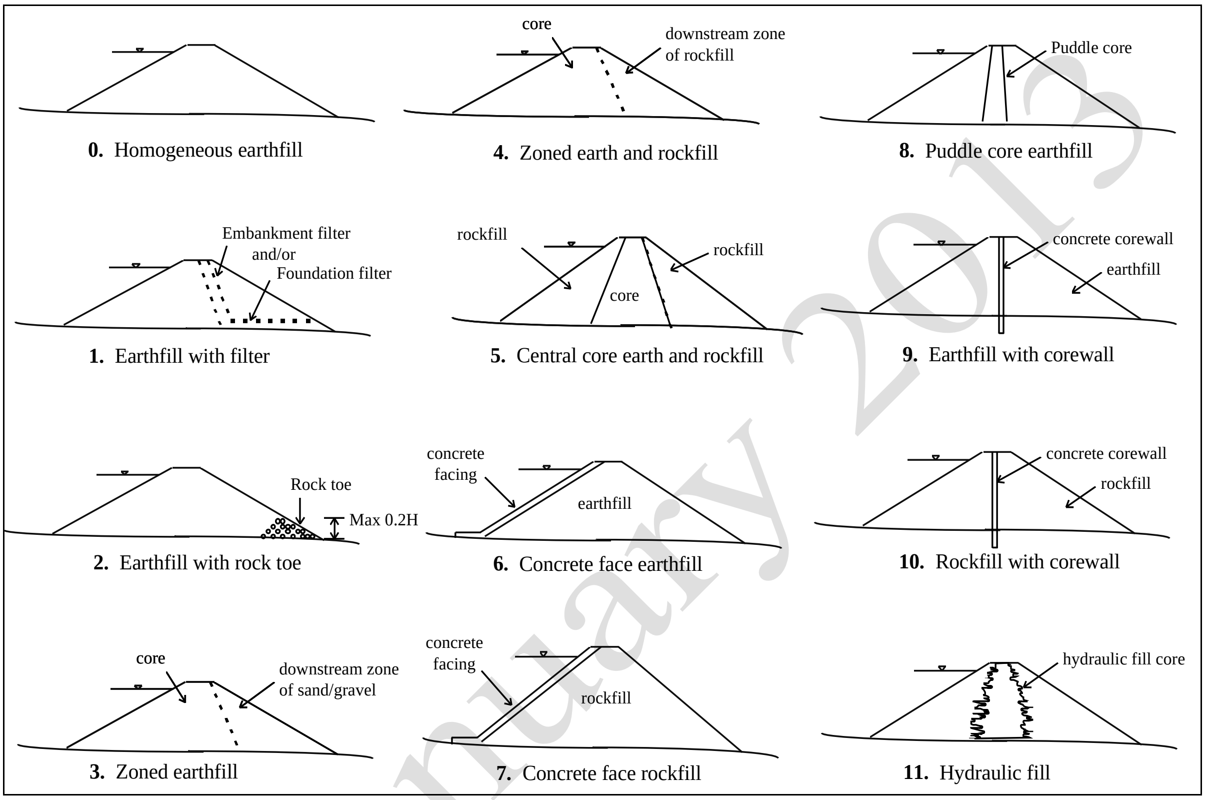


Figure 2.9 - Dam zoning categories. Some of these are only found in older dams.

These can be grouped in regards to their capability of providing control for internal erosion in the embankment (Table 2.2). This is based on the provision for or lack of filters, and the statistics of failure and incidents in Foster et al (2000a). It does not take account of the details of the dam zoning, design and construction so is only a general guide.

Table 2.2: Classification of likelihood of internal erosion associated with the dam zoning

Likelihood of Internal Erosion	Control for internal erosion	Dam zoning and category number
A Large	Little or no control	Homogeneous earth fill (category 0); Earth fill with rock toe (category 2).
B Moderate	Some control of internal erosion depending on detail of zoning and filter capability.	Zoned earth fill (3); Zoned earth and rock fill (4); Puddle core(8); Hydraulic fill (11).
C Low	Moderate control of internal erosion depending on the filter capacity and details of the core wall or face slab.	Concrete face earth fill (6); Concrete face rock fill (7); Concrete core earth fill (9); Concrete core rock fill (10).
D Very Low	Good control of internal erosion subject to good details of zoning and filter design.	Earth fill with filters (1); Central core earth and rock fill (5).

What has been recognized (Foster (1999a) and Foster and Fell (2001)) is that filters and transition zones which are coarser than required by design methods based on particle size will often be quite effective in controlling erosion. Even downstream rock fill and sand/gravel zones which were not designed as filters may provide some protection against erosion continuing.

Depending on the ratio of particle and pore sizes, the erosion will either:

- not continue, (no-erosion); or
- stop after only minor erosion, (some erosion); or
- stop only after a significant amount of erosion, (excessive erosion); or
- will continue (continuing erosion)

2.5 PROGRESSION

Progression is the phase of internal erosion where:

a) For concentrated leak erosion, the erosion in the crack or concentrated leak leads to development of a pipe.

b) For backward erosion the erosion process extends upstream from the point of initiation, and a network of small erosion channels forms beneath the soil or embankment providing the roof to the erosion pipes. If these small erosion channels reach the reservoir or river, then a pipe forms. In global backward erosion, sinkholes (vertical pipes) form.

c) For contact erosion the erosion of the finer soil into the coarser soil continues. This may in particular cases lead to development of a pipe in the finer soil.

d) For suffusion, some of the finer fraction is eroded leaving the coarse matrix of the soil. No pipe is formed but the permeability of the soil may be increased significantly.

2.6 DETECTION

The likelihood that a particular failure path can be detected, and if so, whether it is possible to intervene (e.g. by lowering the reservoir level), or carry out repairs to prevent the dam breaching is usually best considered as two questions:

1. Will this failure path be detected?

2. Will intervention and repair be possible in the time available?

The likelihood of detection and successful intervention and repair is dependent on a number of factors including:

a) The category of internal erosion i.e. internal erosion in the embankment, the foundation or embankment to foundation.

b) The mechanism of initiation of internal erosion – concentrated leak, backward erosion, contact erosion or suffusion.

c) The breach mode – gross enlargement of a pipe, instability of the downstream slope, slope instability from static liquefaction (which may include increase of pore pressure and sudden collapse in an eroded zone), unraveling or sloughing of the downstream slope, overtopping (e.g.

due to settlement of the crest from suffusion and/or due to the formation of a sinkhole from a pipe in the embankment).

d) The nature of and the geometry of the materials in the foundation.

e) The zoning of the embankment, and the materials in the embankment.

f) The reservoir level at the time of the internal erosion incident, and how rapidly it can be drawn down.

g) The type and frequency of monitoring and surveillance at the dam and the training of the staff to recognize a developing internal erosion and piping incident.

h) The ability to get trained personnel out to the site in the event of an internal erosion incident.

i) The ability of those responsible to be able to direct emergency release of the reservoir

j) The availability of materials and equipment to intervene and carry out repair works.

The signs of internal erosion detected by visual inspection related to the type of initiation phenomena are:

- Internal erosion by concentrated leak is in most cases detected by the emergence of leakage or water containing eroded particles on the downstream face, around conduits or at the downstream toe of the dam.

- Backward erosion, contact erosion and suffusion are usually detected by the presence of sand boils at the toe of the dam, or other signs of eroded soil. Suffusion may also be evidenced by emergence of leakage or water on the downstream slope rather than sand boils, unless the suffusion is in the foundation materials.

- For contact erosion and suffusion, the common evidence is settlement of the crest of the dam or levee.

Sinkholes may be present in all types of internal erosion. If sinkholes are the observed signs, the erosion has already progressed and the time to failure may be small. It is far preferable to detect the initiation of erosion earlier by regular surveillance, monitoring including leakage, pore pressure measurement, temperature measurement and acoustics.

Anomalies monitored by pore pressure or seepage measurements may be evidence of internal erosion under constant water load. Such anomalies include:

- random changes of piezometric head or seepage rate,

- irrecoverable trends.

These classical methods are more likely to be effective for backward erosion, suffusion and contact erosion. Concentrated leak erosion may progress rapidly in many situations.

Many less direct means of detecting seepage are now available. The most promising is temperature measurement which can be used to infer localized flow. Fiber optic cables facilitate data collection and make it possible to cover large parts of the dam. Remote sensing options also offer great potential in detecting whether the seepage has caused erosion. These will be discussed in detail in Volume 2 of the Bulletin.

In many situations it is necessary to consider the capability of monitoring to manage the risks. In some situations it may be necessary to carry out remedial works to achieve tolerable risks rather than rely on monitoring and surveillance.

2.7 INTERVENTION

Intervention to prevent the progression of internal erosion and piping and breach can take several forms including:

(i) Drawing down the reservoir level using spillway gates or outlet valves.

(ii) Installing pressure relief wells in the foundation of the embankment.

(iii) Building reverse filters over "boils" or areas where eroding material is emerging from the foundation of the embankment.

(iv) Building a weighting berm to reduce the likelihood of heave, or slope instability, or unraveling.

(v) Dumping granular material (sand/gravel/rockfill) into the upstream side of sinkholes to try to block them.

More than one of these measures may be used together. Which is applicable or feasible will depend on the particular circumstances of the dam.

It should be recognized that there may be reluctance on the part of the reservoir owner or operator to release water given the lost revenue that may result, or if release of reservoir water is likely to result in property damage and loss of life, for example if levee banks downstream of the dam are likely to be overtopped by the flood resulting from release of the water.

Table 2.3 can be used to assess the likelihood that, given the concentrated leak is detected, intervention is not successful. This can be done for various reservoir levels. It is not practical to cover all the possible scenarios. In making this assessment consideration should be made for the failure mode and location of the developing pipe.

If erosion is occurring without becoming catastrophic or if a dam appears vulnerable to internal erosion, it may be necessary to carry out works to inhibit its development.

Emergency works to interrupt progress to a breach are commonly done by providing filtered drainage at seepage points. This allows the seeping water to flow safely whilst trapping eroded particles and gradually inhibiting the development of erosion. Sometimes the width of the dam is increased by adding a filtered berm. This has the effect of decreasing the hydraulic gradient and the seepage energy available. It also loads the materials through which seepage and erosion is occurring thereby increasing the effective stress and reducing the permeability and consequently reducing the energy available to sustain erosion.

Backward erosion may be counteracted by adding filtered toe weights to increase the downward load to be greater than the upward water pressure, thereby reducing gradient below critical and preventing the development of the backward erosion pipes and sand boils.

In some cases, a small amount of coarse materials is put into and over the leak before the proper filter materials are added as the flows velocities can be too high for stable placement of filter sized particles. In many cases, multi-staged filter/drains as well as other weighting materials are needed to achieve stability.

Other interventions seek to block seepage, diaphragm walls and clay cores, for example, but these have the disadvantage that if they are not continuous, concentrated seepage and leakage can occur at gaps in the impermeable membrane and this may cause erosion locally and worsen the situation.

In the short to medium term increased surveillance and/or increased monitoring can be used as an intervention, until additional data is collected. For old dams where little is known about the internal geometry and soils and risk to the public is low to medium this may be the most effective means of managing risk.

Table 2.3 –Likelihood that given the concentrated leak is detected, intervention is not successful (after Fell et al, 2008)

Time for Development of Concentrated Leak to Initial Breach	What can be done	Likelihood of intervention not succeeding
< 3 hrs	There is too little time to successfully intervene regardless of the failure mode	0.99
3 to 12 hrs	In most cases it will be impractical to intervene successfully in this amount of time. Only in cases where there is a straight forward method of intervention, and there are personnel, equipment and materials available will intervention be successful.	0.9 to 0.99
12 to 24 hrs	In many cases it will be impractical to intervene successfully in this time. Only in cases where there is a straight forward method of intervention, and there are personnel, equipment and materials available will intervention be successful; or it is a small storage which can be drawn down to stop the failure mode.	0.85 to 0.99
1 to 2 days	In many cases it will be impractical to intervene successfully in this time. Only in cases where there is a straight forward method of intervention, and there are personnel, equipment and materials available will intervention be successful; or it is a small storage or medium storage with large gate discharge capacity so that the reservoir can be drawn down to stop the failure mode.	0.7 to 0.95
2 to 7 days	In some cases it will be practical to intervene successfully in this time. In cases where there is a straight forward method of intervention, and there are personnel, equipment and materials available; or it is a small storage or medium storage with large gate discharge capacity allowing the reservoir to be drawn down to stop the failure mode.	0.2 to 0.9 [a]
Weeks or months	In some cases it will be practical to intervene successfully in this time. Where there is a straight forward method of intervention, and there are personnel, equipment and materials available and large resources intervention has a fair chance of being successful; or it is a small or medium storage with large gate discharge capacity allowing the reservoir to be drawn down to stop the failure mode	0.1 to 0.8 [a]

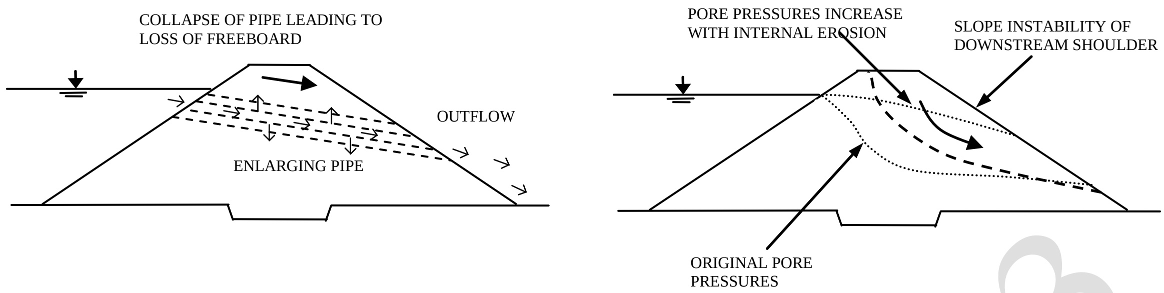
Note: (a) Use values less than 0.5 only if there is a high degree of confidence in the assessed value.

2.8 BREACH

In these circumstances, the entire process of internal erosion has been concluded, detection and intervention has failed, and the question is whether the dam will breach by one of the five mechanisms listed below, or whether the internal erosion process will stabilize. The breach phenomena are listed in order of their observed frequency of occurrence. The first four are shown schematically in Figures 2.10 (a, b).

- Gross enlargement of the pipe
- Overtopping (e.g. due to settlement of the crest from suffusion and/or due to the formation of a sinkhole from a pipe in the embankment).
- Slope instability of the downstream slope
- Unraveling of the downstream face.

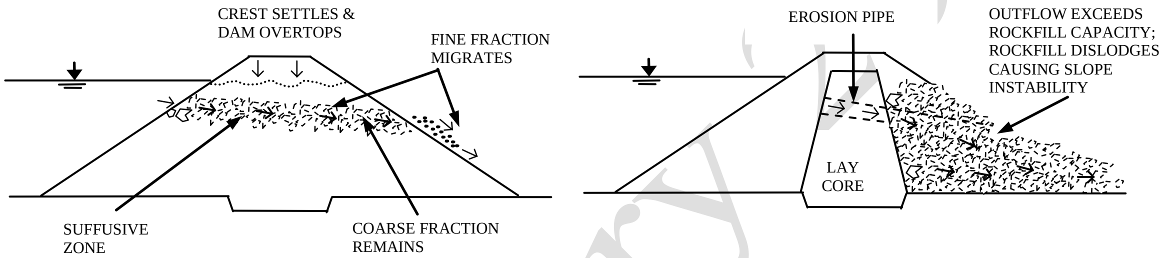
- Static liquefaction which is a form of slope instability which may include increase of pore pressure and sudden collapse in eroded zone.



Breach by pipe enlargement

Breach by slope instability

Figure 2.10(a) Potential breach (failure) phenomena-pipe enlargement and slope instability



Breach by overtopping by settlement

Breach by unraveling of the downstream face

Figure 2.10(b) Potential breach (failure) phenomena-overtopping by settlement and unraveling of the downstream face, Fell and Fry (2007)

The mechanics of breaching and the vulnerabilities of various dam types to breaching are not dealt with in detail elsewhere in this bulletin, and the following paragraphs summarize the situation.

An important aspect of breaching is the time it takes from the first observable concentrated leak to when the breach forms because that determines the time available to warn people downstream and to take action to prevent breach or alleviate the downstream impacts of a breach. More is said about this in Fell et al (2001, 2008).

Breach initiation is almost certain to occur if continuing erosion is expected or there are no filters and detection and intervention have failed. Exceptions to this will be if the reservoir has small capacity and the reservoir level drops below the inlet of the developing pipe before a breach mechanism has time to develop.

Breach initiation is less likely if some or excessive erosion is expected. For these cases, the seepage flows that develop through the dam or foundation will be limited by the filtering material.

For most dam types and failure modes, the likelihood for breach development will be dominated by one or two of the potential breach mechanisms. Breach mechanisms will not necessarily be applicable to some dam zoning types or modes of piping and can be ignored.

Table 2.4 lists breach mechanisms depending on the dam zoning type for internal erosion in the embankment due to a crack or poorly compacted zone. Table 2.4 also applies to internal erosion in a soil foundation. Figure 2.9 in Section 2.4 shows the dam zoning types.

Table 2.4 does not apply to failure modes involving open or in filled defects and solution features in rock foundations because the leakage flows may exceed the capacity of even free draining rockfill.

Table 2.4 Screening of breach mechanisms for internal erosion through the embankment, internal erosion in soil foundations, and from embankment into foundation (Fell et al, 2008)

Dam Zoning Type (refer to Figure 2.9)	Breach Mechanisms			
	Gross Enlargement	Slope Instability	Sloughing or Unraveling	Sinkhole Development
0 Homogeneous earthfill	✓*	✓	Exclude, except if downstream fill is cohesionless	✓
1 Earthfill with filters	✓*	✓	Exclude, except if downstream fill is cohesionless	✓
2 Earthfill with rockfill toe	✓*	✓	Exclude, except if downstream fill is cohesionless	✓
3 Zoned earthfill	Exclude, except if downstream fill can support a roof	✓	Exclude, except if downstream fill is cohesionless	✓
4 Zoned earthfill and rockfill	Exclude, except if downstream fill can support a roof	✓	✓*	✓
5 Central core earth and rockfill (or gravel shells)	Exclude, except if downstream fill can support a roof	Exclude, except if existing dam has marginal stability	✓*	✓
6 Concrete face earthfill	✓	✓*	Exclude, except if downstream fill is cohesionless	✓
7 Concrete face rockfill (including gravel fill)	Exclude	Exclude, except if dam is gravel of low permeability	✓*	Exclude
8 Puddle core earthfill	✓*	✓	Exclude, except if downstream fill is cohesionless	✓
9 Earthfill with corewall	Exclude	✓*	Exclude, except if downstream fill is cohesionless	✓
10 Rockfill with corewall	Exclude	Exclude, except if existing dam has marginal stability	✓*	✓
11 Hydraulic fill	Exclude, except if downstream fill can support a roof	✓	✓*	✓

Key: Exclude: Not a potential breach mechanism
✓ Potential breach mechanism
✓* Potential breach mechanism: usually the more critical one

Internal erosion by the process of suffusion is usually unlikely to lead to the formation of a pipe through the dam or its foundation, and hence the probability of breach by gross enlargement is low.

However there are situations where suffusion can lead to high pore pressures at the toe of a dike and this can cause hydraulic fracture or heave of an overlying berm. Breach by slope instability or sloughing/unraveling are usually the more critical mechanisms for suffusion, although the estimated probabilities for breach are usually relatively low for this mode of internal erosion.

3. INITIATION OF CONCENTRATED LEAK EROSION

3.1 CONCENTRATED LEAK EROSION: THE OVERALL PROCESS

Whether erosion will initiate in concentrated leaks in a dam or its foundation depends on whether the stresses in the dam will result in a crack or low stress zone subject to hydraulic fracture and geometrical and hydraulic criteria within the crack:

- Whether there is a crack (or flaw) below the reservoir level formed by differential settlement during or after construction, hydraulic fracture, desiccation, or collapse of a poorly compacted layer of soil in the embankment or around a conduit sited to pass through the embankment.

- Given there is a crack, whether the forces imposed on the sides of the crack by the water flowing through the crack are sufficient to initiate erosion.

Whether in the absence of filters or where filters are too coarse, the erosion will progress to form a pipe depends on:

- Whether the water flowing through the crack will cause the soil on the sides of the crack to swell, closing the crack, or reducing the width so the forces imposed on the sides of the crack by the water flowing through the crack are insufficient for erosion to progress.

- Whether the soil will hold open the pipe ("will it hold a roof").

- Whether upstream zones will limit the erosion process or crack stopping occurs.

Also important is the rate of erosion and enlargement of the pipe because this influences whether the leak can be detected and intervention taken to lower the reservoir or stop the erosion process before breach occurs.

3.2 INITIATION OF CONCENTRATED LEAK EROSION IN SITUATIONS WHERE CRACKS, OPENINGS AND LOW STRESS ZONES MAY BE PRESENT IN AN EMBANKMENT OR FOUNDATION

3.2.1 Cracking and hydraulic fracture due to cross valley differential settlement of the core

As an embankment dam is constructed the partially saturated compacted soil in the embankment consolidates and settlement occurs. Where the valley sides are steep and/or have steps in the profile, such as shown in Figure 2.2 differential settlements occur due to the variations in the height of the embankment, and these can lead to tensile or low stress zones in which cracks may form. This has been recognized for some time by Sherard (1973, 1985, 1986), Høeg et al (1998), Kjaernsli et al (1992) and others. Hydraulic fracture through these low stressed zones as the dam is filled or under flood conditions is an associated phenomenon as discussed by Sherard et al (1972), and Sherard (1985, 1986). For most dams, about 80% to 90% of the total settlement occurs during construction (Hunter, 2003, Hunter and Fell, 2003), so the stresses set up in the construction phase largely control the likelihood of low stress zones and cracking.

It should be recognized that these stresses are inevitable even in well designed and constructed dams and are not caused by poor design or construction practice.

Embankments with abutments steeper than about 45 degrees and particularly steeper than 60 degrees are likely to be susceptible to cracking, particularly for high dams. Embankments with a step in the foundation profile where the height of the embankment above the step is less than half the maximum embankment height and the width of the step is greater than the lower embankment are also susceptible to cracking.

3.2.2 Cracking and hydraulic fracture due to cross valley arching

If the valley in which the dam is constructed is narrow and steep, cross valley arching can occur and the vertical stresses are shed onto the sides of the valley. This can lead to a situation where hydraulic fracture can occur. Fell et al (2008) suggest that cross valley arching is most likely to occur if the width of the valley base is less than a quarter of the dam height, and the valley sides are steeper than about 60 degrees. It is unlikely to be an issue if the width of the valley base is greater than three quarters the dam height and the valley sides are flatter than 45 degrees.

3.2.3 Cracking and hydraulic fracture due to differential settlement in the foundation under the core

Figure 2.3 which is adapted from Sherard et al (1963) shows foundation conditions which are likely to lead to differential settlement and cracking or low stress zones conducive to hydraulic fracture. Zones which may lead to differential settlement greater than 0.5% of the dam height, with steep changes in the foundation profile are most likely to suffer cracking and hydraulic fracture. Differential settlements of less than 0.2% of the dam height spread over some distance are unlikely to lead to cracking and low stress zones.

3.2.4 Cracking and hydraulic fracture due to small scale irregularities in the foundation profile under the core

Small scale irregularities in the foundation of the core can lead to cracking or low stresses conducive to hydraulic fracture, e.g. NGI (1984). For cracking or low stresses to occur the small scale irregularities need to be persistent over all or most of the distance across the core, and have steps greater than about 3% to 5% of the embankment height.

There are examples of these irregularities being formed by constructing haul roads across the core (Newman and Foster, 2006), and steps in slope correction concrete (Gillon, 2007).

3.2.5 Cracking and hydraulic fracture due to arching of the core onto the shoulders of the embankment

As detailed in Bui et al (2004, 2005) and Fell et al (2008) this is most likely to be a problem for cores which are very narrow - core width less than 0.25 the embankment height, and for soils subject to collapse compression on saturation (poorly compacted soil placed dry of optimum moisture content). It is unlikely to be a problem for cores which are wider than 0.5 to 1.0 the embankment height and the core is well compacted at around optimum moisture content. It is not sufficient to cause hydraulic fracture to just have stiff shoulders and a less stiff core. There needs to be differential movement of the core after construction.

Differential expansion between the core and shoulders as a reservoir refills after being drawn down may result in hydraulic fracture in cores (e.g. Johnston et al, 1999).

3.2.6 Crack or gap adjacent to a spillway or abutment walls and where concrete dams abut embankment dams

Cracking or a gap may form adjacent to walls due to the earthfill settling away from the wall during and after construction. Cracks or gaps may also form at dam abutments. Figure 2.4 shows some situations where this is likely to occur.

Well designed walls with uniform contact slopes flatter than about 0.25H:1V are unlikely to have gaps form. Vertical or over-hanging walls are likely to have gaps.

Cracking and gaps may form due to deformations of flexible and / or under-designed retaining walls (e.g. designed for active rather than at-rest earth pressures.

Where there is a wrap-around junction between a concrete gravity dam and an embankment dam, differential settlements similar to those described above may occur. Fell et al (2008) give details.

3.2.7 Internal erosion associated with conduits embedded in the embankment

Many internal erosion and piping failures and incidents occur where conduits are embedded in the embankment.

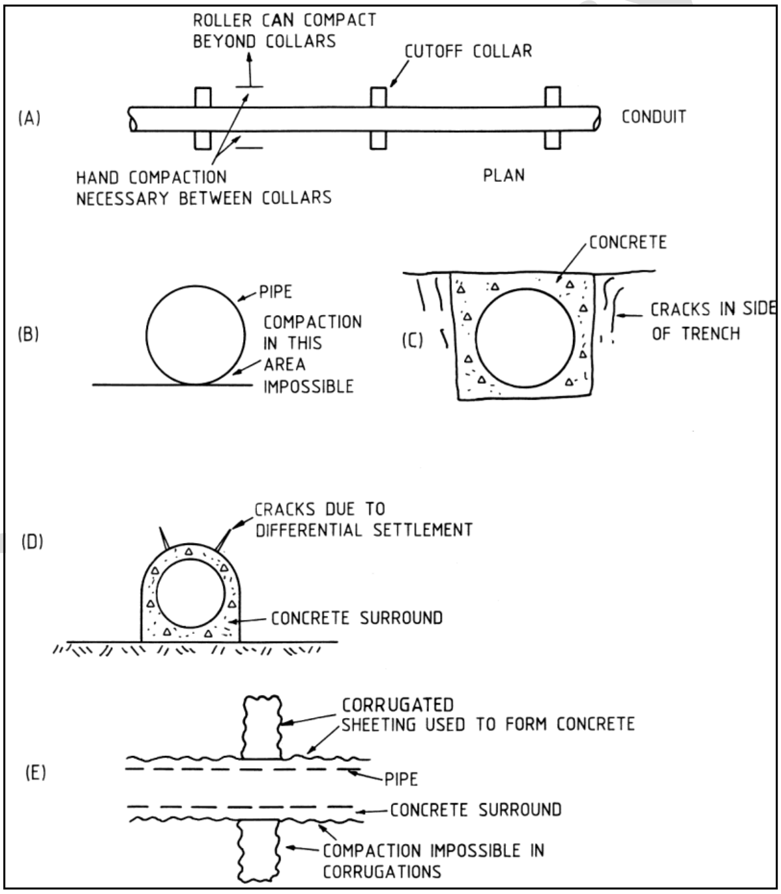


Figure 3.1 Some causes of internal erosion around conduits: (A) Inadequate compaction due to the presence of cut-off collars; (B) Inadequate compaction under pipe; (C) Cracking in soil or extremely weathered rock in the sides of a trench; (D) Cracks due to differential settlement; (E) Use of corrugations or other roughening of the surface of cut-off collars or concrete surround. (Fell et al 2005).

Figure 3.1 shows some of these features. The conduit facilitates the initiation of internal erosion by:

- Causing stress distributions due to the stiff conduit and its less stiff surrounding soil which lead to low principal stresses and hydraulic fracture. This is discussed by Sherard et al. (1972) and Charles (1997). This can occur on the sides of culverts which are constructed in a trench. Sherard et al. (1972) point out that it can also occur where the concrete culvert, or the concrete surround to a pipe, has a sharp corner. In this case piping can be expected above the culvert.

- Drying of the soil in sides of the trench in which the culvert is placed during construction can also cause cracks which allow initiation of piping.

- Making compaction of soil difficult, particularly if collars are provided at close intervals or the concrete is formed with corrugated steel sheet or other non-smooth formwork, preventing compaction of the soil adjacent to the conduit. It can also be difficult to compact the soil surrounding the conduit if the space between the conduit and the sides of the trench in which it is placed is small and compaction by rollers is not possible.

- Poor compaction is likely to lead to collapse settlement of the soil on saturation forming a gap adjacent to the conduit. Note that it is the continuity of the potential defect caused by a conduit and the excavation through the dam which is so critical to the initiation of internal erosion.

Internal erosion may also occur into open joints or other defects in conduits or into conduits which have deteriorated due to corrosion.

FEMA (2006) “Conduits through Embankment Dams, best practices for design, construction, problem identification and evaluation, inspection, maintenance, renovation and repair” provides a comprehensive coverage of the issues relating to conduits in embankment dams. Fell et al (2008) provide some guidance on how to assess the likelihood of these features leading to initiation of erosion.

3.2.8 Cracking or hydraulic fracture in poorly compacted layers in cohesive and broadly graded till soils in embankments

It is well documented, e.g. Sherard (1973), Foster et al (2000a), that internal erosion and piping occurs in poorly compacted cohesive soils. This is particularly so for dispersive soils. The mechanism is potentially of two types:

- The soil behaves as a series of clods with openings between the clods in which water passes.

- The soil collapses on saturation forming a flaw (open pathway) in which the water flows. The flaws may be the result of collapse deformation alone or be caused by hydraulic fracture when the minimum principal stress is reduced locally by the deformation and becomes less than the pore pressure.

This is most likely where there is poorly compacted soil against a conduit or abutment but is possible within layers of soil, particularly if they were too thick to be effectively compacted by the compaction equipment used. Sherard (1973) gives examples of this. Lawton et al (1992) consider the collapse settlement issue.

The large number of piping and sinkhole incidents experienced in Swedish central core earth and rock fill dams (Nilsson, 2007 a, b) constructed with broadly graded glacial till cores may have been primarily initiated by collapse settlement of the core which was lightly compacted in layers up to 800mm thick, some of which were close to structures and sheet piled cut-offs.

Segregation in broadly graded fills may also lead to cracking through local collapses or hydraulic fracture. More is said about soils subject to segregation in Section 6.5.

3.2.9 Cracking due to desiccation

Desiccation cracking is most likely to be an issue in climates with less than 250mm annual rainfall, in high plasticity cores, and where there is no surface layer over the core. Experience in excavating into the crest of embankments is that if there is a road pavement of granular road base or a rockfill or other non-plastic layer at least 300mm thick cracking is not generally observed even in seasonal climates with extended dry periods. The likelihood of cracking is further reduced if the road pavement is sealed with asphalt, concrete or bitumen seal.

Desiccation cracking does not commonly persist to a great depth so only becomes an issue for reservoir levels nearing the crest level.

Desiccation cracking may also occur on seasonal shut down surfaces during construction, or on the surface of the first stage of dams which are built in two stages. Good construction practice would be to remove the desiccated soil but this was not always done.

It should be noted that in some situations the vertical desiccation cracks in the crests of embankments form hexagonal columns, which are cut at frequent intervals by horizontal cracks, to form ‘blocks’. Such circumstances have led to failures, as reported by Marsland and Cooling (1958) and by Dyer, Utili and Zielinski (2007), because the desiccated blocks are easily eroded and overtopping may occur when the reservoir water level rises too rapidly for re-saturation and swelling to take place.

3.2.10 Transverse cracking caused by settlement during earthquakes

When embankment dams experience a large earthquake they often settle and spread in the upstream-downstream direction. Many exhibit longitudinal cracking, and some transverse cracking. Some examples are Austrian Dam (Harder et al, 1991; Forster and MacDonald, 1998); Guadeloupe Dam (Harder et al, 1991) and Lexington Dam (Fong and Bennett, 1995). Pells and Fell (2002, 2003) analyzed case data for dams not subject to liquefaction which showed that dams which settled more than about 1.5 % of their height were almost certain to exhibit transverse cracks; those which settled between 0.5% and 1.5% had about a 20% chance of exhibiting transverse cracks; and dams which settled between 0.2% and 0.5% had about a 5% chance of transverse cracking.

These cracks are in the upper part of the embankment and whether they are a potential initiator of internal erosion depends on the reservoir level and depth of cracking.

If liquefaction occurs the deformations are likely to be large and the likelihood of cracking greater. These may be estimated by numerical analyses methods.

3.2.11 Cracking or high permeability layers due to freezing

The effects of frost on embankment dams are described in Vuola et al (2007). The effects of frost are to cause extra water to be drawn into the soil by capillarity action. This causes ice lenses to form with associated heave and potentially cracking and / or loosening of the soil. The cracking is most likely to be longitudinal but may also be transverse at the crest of the dam. Figures 3.2, 3.3 and 3.4 show the potential effects.

Vuola et al (2007) provide an equation for estimating the depth of frost penetration.

There are a number of references which provide information on the susceptibility of soils to frost penetration. These include Wallace (1987), Holtz and Kovacs (1981), Vuola et al (2007) and USACE (1984) which gives the most specific data. Core materials which are most susceptible to freezing and ice lens formation include silts, clayey silts, silty sands, silty gravels and clayey sands and gravels with a plasticity index < 12.

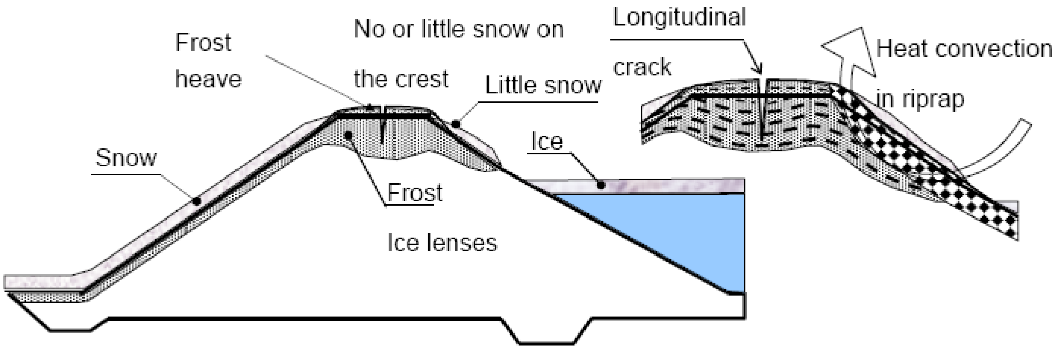


Figure 3.2 Effects of frost on an embankment dam (Vuola et al, 2007)

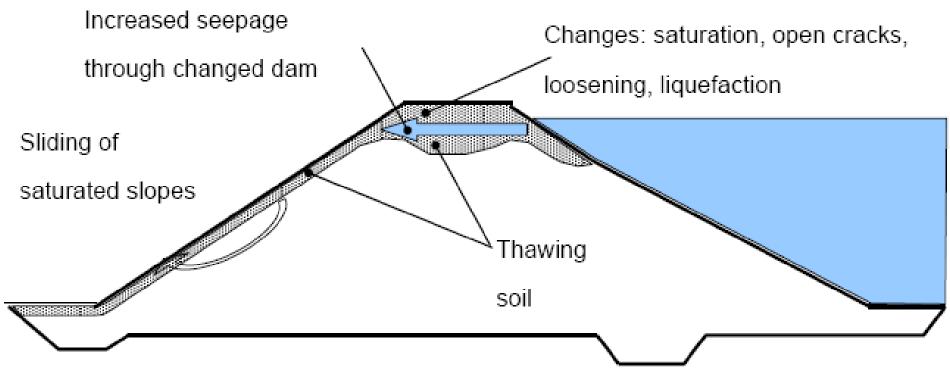


Figure 3.3 Damage phenomena occurring in embankment dams from frost thaw (Vuola et al, 2007)

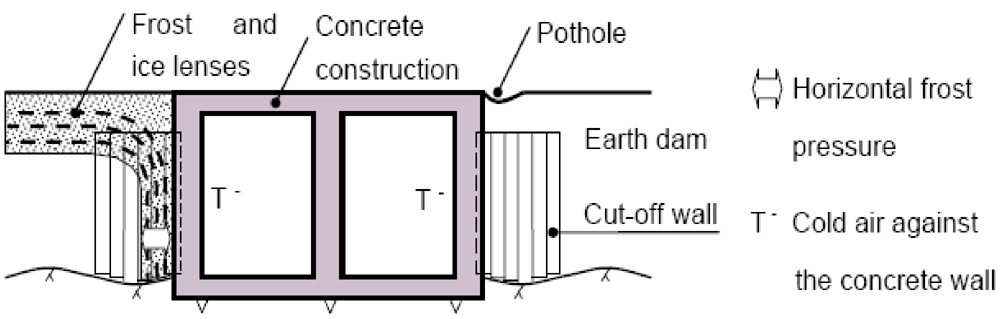


Figure 3.4 Effect of frost against a spillway structure (Vuola et al, 2007)

3.2.12 Internal erosion initiated by the effects of animal burrows and vegetation

Animal burrows in the embankment or levee can lead to a situation where there are nearly continuous holes through the embankment or situations like those shown in Figure 3.5 where high gradients between holes may result in initiation of erosion.

FEMA (2005a) "Impacts of animals on Earthen Dams, FEMA Report 473" provides a comprehensive coverage of the issues.

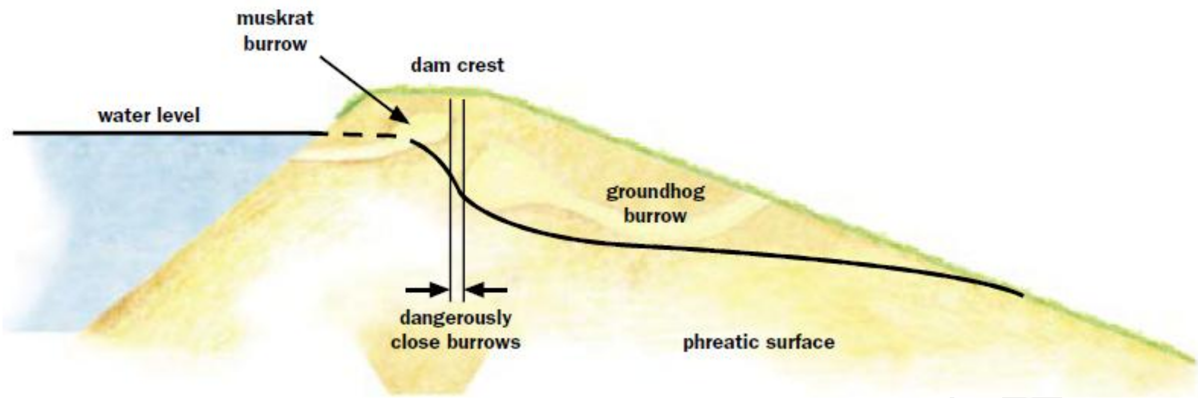


Figure 3.5 potential impacts of burrowing animals on internal erosion in embankments and levees (FEMA 2005a).

Vegetation growing on dams and levees can lead to situations which may lead to a potential concentrated leak and initiation of erosion. The effects include:

- Decaying roots that create seepage paths.
- Roots penetrating into open joint and cracks in foundation rock, potentially creating seepage paths.
- Root penetration of conduit joints and joints in concrete structures and opening the joints to allow erosion into or out of the conduit or wall.
- Clogging embankment under-drain systems.
- Interfering with effective visual inspection.

FEMA (2005b) Technical Manual for Dam Owners, Impacts of Plants on Earthen dams, FEMA Report 534 provides information and guidance on how to manage this problem. Wray (2006) prepared a CIRIA guide to management of the European Rabbit.

3.2.13 Relative importance of conduits, spillway walls cracking mechanisms, and poorly compacted zones

As detailed in Table 1.2, about half of internal erosion incidents in embankments have historically involved conduits through the embankment and spillway walls supporting the embankment. These are related to poor compaction, differential settlement and arching effects.

Excluding conduits and spillway walls the relative proportion of incidents in the Foster et al (1998, 2000a) ICOLD database caused by cracking and poorly compacted zones is:

- Differential settlement cracking – 36/57 = 63%.
- Poorly compacted and high permeability zones – 21/57 = 37%.

A further assessment of the case study information results is given in Table 3.1:

Table 3.1 Relative proportion of incidents from the different causes of crack formation (Fell et al, 2008, based on Foster and Fell, 1998, 2000a)

		No of cases	% of cases
Cracking mechanisms	Differential settlement, cross valley	20	35%
	Differential settlement, cross section	6	11%
	Differential settlement, foundation	4	7%
	Differential settlement, embankment staging	0	0%
	Desiccation cracking	3	5%
	Closure section	3	5%
Total for cracking mechanisms		36	63%
Poorly Compacted and High permeability zones	Foundation contact	5	9%
	In embankment	16	28%
Total for Poorly Compacted and High Permeability Zones		21	37%

3.3 ESTIMATION OF CRACK WIDTH AND DEPTH OF CRACKING

As described above, cracks form in the crests of embankments from desiccation, from settlement resulting from consolidation, creep or earthquake shaking or in poorly compacted and high permeability zones.

Settlement causes spreading of the embankment; this results in longitudinal cracks, parallel to the centerline of the dam, but settlement resulting in transverse cracks passing through the crest of embankments from the upstream face to the downstream face are sites where concentrated leak erosion may occur. It is necessary to estimate the width and depth of such cracks (and the hydraulic gradients in them) to ascertain, using the methods in Section 3.4, if the hydraulic shear force will or will not exceed the hydraulic shear strength of the soil in the walls of the crack, thereby initiating, or not initiating, internal erosion.

Transverse cracks form as a consequence of vertical settlement, which has the effect of elongating the crest. The elongation will be uniform, giving rise to numerous fine cracks, if the foundation profile is such that sharp local differences in settlement will not occur. If the foundation profile is irregular (see Figures 2.2 and 2.3, for example) differential settlement will occur, giving rise to locally greater elongation of the crest, and locally wider cracks in the crest, above the irregularities in the foundation profile. Estimates of crack width may be made by applying these principles, but there will be many approximations and uncertainties.

The width and depth of cracks which may be present in the embankment may be more reliably estimated using methods detailed in Fell et al (2008) and Fell et al (2007). The methods require assessing the likelihood of cracks forming, including by hydraulic fracture, and are briefly described in Section 12.3.2. However, they are based on a review of the literature on observed cracking including Sherard (1973), Talbot (1994) and Lawrence (2002), and the results of numerical modeling by Bui et al (2004, 2005) which assists in assessing the likely depth of cracking. The values estimated by these methods are also approximate and this uncertainty should be allowed for in assessing internal erosion.

Assessment of crack widths and depths requires careful consideration and analysis, probably by more than one method. Table 9.7 gives information on situations in which cracks or concentrated leaks are unlikely to form in embankments. If cracks seem likely to form, the methods and references mentioned above should be used to estimate crack dimensions. To assist in checking such estimates, Table 3.2 gives some examples of the likely maximum crack depths and widths formed as a result of cross valley differential settlement or differential settlement in the foundation using these methods. It should be noted that it includes examples of situations where both narrow cracks and wide cracks may be formed, including referring to Figures 2.2 and 2.3 to show how different foundation profiles and different foundation conditions cause different amounts of settlement and lateral strain, resulting in cracks of different widths and depths.

Table 3.2 Examples to assist in assessing the likely maximum crack depths and widths formed as a result of cross valley differential settlement or differential settlement due to soil in the foundation

Description	Situations similar to those shown in parts of:	Maximum Likely Depth of Cracking Below Dam Crest	Maximum Likely Crack Width at Depth Shown Below Dam Crest		
			1.5m	3m	6m
Uniform abutment without benches, gentle abutment slopes < 30^o, dam less than 15m high		3m	2mm	1mm	
Wide bench low in abutment, moderate abutment slopes 30^o < to < 45^o; dam 15m to 30m high	Figure 2.2	5m	20mm	5mm	
Wide bench high in abutment, steep abutment slopes 45^o < to < 60^o ; dam 30m to 60m high	Figure 2.2	7m	40mm	20mm	2mm
Wide bench near crest in abutment, very steep abutment slopes > 60^o; dam >60m high	Figure 2.2	10m	60mm	35mm	7mm
Rock or uniform soil foundation, gentle slopes < 30^o on sides of compressible zone, dam less than 15m high		3m	2mm	1mm	
Shallow soils, differential settlements of less than 0.2% of embankment height, moderate slopes 30^o < to < 45^o on sides of compressible zone ; dam 15m to 30m high	Figure 2.3	5m	20mm	5mm	
Moderate depth of soils, differential settlements of 0.2% to 0.5% of embankment height; steep slopes 45^o < to < 60^o on sides of compressible zone; dam 30m to 60m high	Figure 2.3	10m	60mm	35mm	7mm
Deep soils, differential settlements > 0.5% of embankment height, very steep slopes > 60^o on sides of compressible zone; dam >60m high	Figure 2.3	12m	90mm	50mm	10mm

3.4 THE MECHANICS OF EROSION IN CONCENTRATED LEAKS

3.4.1 The procedure

The procedure for assessing whether erosion will initiate in a crack or flaw is to:

Estimate the hydraulic shear stresses in the crack for the reservoir level under consideration, taking account of the geometry of the core of the embankment and the assumed crack dimensions and location relative to the reservoir surface so the flow gradient can be determined.

Compare this hydraulic shear stress to the critical shear stress which will initiate erosion for the soil in the core of the embankment (τ_c) at the degree of saturation of the soil on the sides of the crack. In doing this take account of the dispersion properties of the soil and the chemistry of the seepage water.

There is always some uncertainty regarding the input parameters so the analysis should check the sensitivity to the assumptions made.

The following sections give an outline of this process.

3.4.2 The estimation of hydraulic shear stresses in cracks and erosion pipes

Wan (2006), Wan and Fell (2002, 2004, a, b) and Fell et al (2008) give details on how to estimate the hydraulic shear stresses in cracks and cylinders.

They give the following equations for estimating the hydraulic shear stress on the surface of a cylindrical pipe, or parallel sided transverse crack in an embankment. The assumptions are:

- Linear head loss from upstream to downstream
- Steady uniform flow along the crack
- Zero pressure head at the downstream end
- Uniform frictional resistance along the surface of the crack or cylindrical pipe
- Driving force = frictional resistance.

(a) Cylindrical pipe:

$$\tau = \rho_w \frac{gH_f d}{4L}$$

(b) Vertical transverse crack

$$\tau = \frac{\rho_w g H_f^{2} W}{2(H_f + W)L}$$

where τ = Hydraulic shear stress in N/m^2

ρ_w = Density of water in kg/m^3

g = Acceleration due to gravity = 9.8m/s^2

H_f = Head loss in pipe or crack due to friction in meters

L = Length of pipe or crack base in meters

d = Diameter of the pipe in meters

W = Width of crack in meters

Bonelli and Brivois (2008) have developed a generalized equation for the rate of enlargement.

3.4.3 Erosion properties of soils in the core of embankment dams - basic principles

It is important to reiterate that the resistance to initiation of erosion of the core to concentrated leak erosion is characterized by the critical shear stress, and the rate of enlargement of the pipe in the progression phase is characterized by the erosion coefficient.

The erosion properties of soils for concentrated leak erosion can be determined by rotating cylinder tests e.g. Arulanandan and Perry (1983), Chapius (1986a, b), Lim (2006), Lim and Khalili (2010); Slot Erosion Tests (SET), and the Hole Erosion Tests (HET) e.g. Wan (2006), Wan and Fell (2002, 2004a, b), Bonelli et al (2007) and Jet Erosion Tests, e.g. Hanson (1990, 1991) Hanson and Cook (2004). These tests allow determination of the critical shear stress for initiation of erosion, and the erosion rate. At this time the most widely used tests are the Hole Erosion Test and the Jet Erosion Test. More details on these tests are given in Volume 2 of the Bulletin.

Wan and Fell (2002, 2004a, b) and Wan (2006) expressed the erosion rate in the form of an Erosion Rate Index, I_{HET} defined by:

$I_{HET}=-\log(C_e)$ (3)

C_e is the Coefficient of Soil Erosion, and is derived from the Hole Erosion Test in which:

$\varepsilon^{\bullet} = C_e (\tau - \tau_c)$ where: $\varepsilon^{\bullet}$ is the erosion rate per unit area

τ is the shear stress

τ_c is the Critical Shear Stress for initiation of erosion

The representative erosion rate index $\tilde{I}_{HET}$ is the hole erosion index I_{HET} for soil compacted to a density ratio of 95% of standard maximum dry density at optimum moisture content.

Soils can be classified into 6 groups according to their Representative Erosion Rate Index, $\tilde{I}_{HET}$. The 6 groups are as shown in Table 3.3. Note that this is a logarithmic scale and the rate of erosion of soils varies by up to five orders of magnitude.

Table 3.3 Descriptors for erosion rates of soils (Wan 2006)

Group No.	Representative Erosion Rate Index	Description
1	<2	Extremely rapid
2	2 – 3	Very rapid
3	3 – 4	Moderately rapid
4	4 – 5	Moderately slow
5	5 – 6	Very slow
6	>6	Extremely slow

In the absence of laboratory test values the Representative Erosion Rate Index ($\tilde{I}_{HET}$) can be related approximately to soil properties. Table 3.4 has been developed from test data to give a first approximation to the likely range of $\tilde{I}_{HET}$ for different classifications of non-dispersive soils.

Table 3.4 Representative erosion rate index (I_{HET}) versus soil classification for non-dispersive soils based on Wan and Fell (2002)

Unified Soil Classification	Representative Erosion Rate Index (I_{HET})		
	Likely Minimum	Best Estimate	Likely Maximum
SM with < 30% fines	1	<2	2.5
SM with > 30% fines	<2	2 to 3	3.5
SC with < 30% fines	<2	2 to 3	3.5
SC with >30% fines	2	3	4
ML	2	2 to 3	3
CL-ML	2	3	4
CL	3	3 to 4	4.5
CL-CH	3	4	5
MH	3	3 to 4	4.5
CH with Liquid Limit < 65%	3	4	5
CH with Liquid Limit > 65%	4	5	6

Notes: (1) Use best estimate value for best estimate probabilities. Check sensitivity if the outcome is strongly dependent on the results.
(2) For important decisions carry out Hole Erosion Tests, rather than relying on this table which is approximate.

The Critical Shear Stress is related to the Erosion Rate Index. The approximate estimates and likely range of Critical Shear Stress (τ_c) in Table 3.5 have to be used, with caution, when Hole Erosion Test values are not available.

Table 3.5 Approximate estimates and likely range of Critical Shear Stress (τ_c) versus Hole Erosion Index (I_{HET}) (Fell et al 2008)

Hole Erosion Index (I_{HET})	Critical Shear Stress for initiation of erosion (τ_c) Pa			
	Non Dispersive Soil Behavior		Dispersive Soil Behavior	
	Best Estimate	Likely Range	Best Estimate	Likely Range
<2	2	1 to 5	1	0.5 to 2
2 to 3	2	1 to 5	1	0.5 to 2
3.5	5	2 to 20	2	1 to 5
4	25	10 to 50	5	2 to 10
5	60	25 to 100	5	2 to 10
6	100	60 to 140	5	2 to 10

Note. To be used with caution. For important decisions carry out Hole Erosion Tests to determine the Critical Shear Stress (τ_c)

It is emphasized that it is better to carry out a series of Hole Erosion Tests at varying heads or to use the method of Bonelli et al (2007); Bonelli and Brivois (2008) to define the critical shear stress (τ_c) than to rely on these relationships.

Marot et al (2011) performed a series of Hole Erosion Tests and Jet Erosion Tests on different fine-grained soils, covering a large range of erodibility. It is shown that with commonly used methods erosion coefficient and average critical shear stress are different with the JET and with the HET. Moreover the relative soils classifications yielded by the two erodimeters are not exactly the same. Based on the energy method, an erosion resistance index is determined for both the JET and HET apparatus and a classification of surface erosion resistance is proposed. For both apparatus, values of erosion resistance index are roughly the same for each soil and a single classification of soil erodibility is obtained.

Other points to note are the effect of degree of saturation of the soil and the effect of dispersion. By distinguishing dispersive behavior from non-dispersive behavior, Regazzoni and Marot (2011) proposed an expression of the erosion resistance index as a function of three physical parameters: compaction, saturation ratio and difference between clay water content and liquid limit.

3.4.4 Comparison of the hydraulic shear stress in the crack (τ) to the critical shear stress which will initiate erosion for the soil in the core of the embankment (τ_c)

To assess the likelihood erosion will initiate in a crack the estimated hydraulic shear stress is compared to the initial shear stress for the soil taking account of the moisture content of the soil in the core, and the chemistry of the water in the reservoir. When doing this assessment allowance should be made for the uncertainty in the calculations and properties.

For an average gradient of 0.5 in a 2mm wide crack the hydraulic shear stress is 5Pa, for a 5mm wide crack 12Pa, and for a 20mm wide crack 50Pa. Hence from Table 3.4 for dispersive and other highly erodible soils even narrow cracks will be sufficient for erosion to initiate.

It is important to recognize that erosion in cracks or flaws is not a result of establishment of a flow net, but is due to flow in open cracks. This occurs quickly as the reservoir rises into the cracked zone. Steady state and transient flow net analyses are irrelevant to this process.

3.5 DISPERSIVE SOILS AND THEIR IMPORTANCE IN CONCENTRATED LEAK EROSION

3.5.1 What are dispersive soils?

Soils in which the clay particles will detach from each other and from the soil structure without a flow of water, and go into suspension are termed dispersive clays.

The clay particles in many clay soils exist in accumulations known as flocs. In these soils changes in pore water chemistry brought about by seepage through a dam, for example, can cause the flocs to break down, de-flocculate, and effectively become smaller, more easily eroded, particles.

The dispersivity of a soil is directly related to its clay mineralogy. In particular soils with a high exchangeable sodium percentage such as Na or Ca montmorillonite present, tend to be dispersive, while kaolinite and related minerals (e.g. halloysite) are non dispersive. Soils with illite present tend to be moderately dispersive.

The dispersivity depends also on the pore water chemistry. Low pore water salt concentrations lead to greater dispersivity and high salt concentrations can suppress dispersion in susceptible soils. Hence percolation of a saline soil with fresh water can lead to dispersion.

There have been many failures of dams constructed of dispersive clays. Mitchell (1993) indicates these have been mostly low to medium plasticity (CL and CL-CH) clays that contained montmorillonite. Fell et al (2005) indicate that their experience is similar, that is the most susceptible soils are not those with a high plasticity, but those with limited clay size fractions sufficient only to give low to medium plasticity.

Sherard et al. (1976) however tested some soils as dispersive in the pinhole test with % passing 0.005 mm greater than 50%. The better performance of the higher plasticity clays probably relates to greater resistance to erosion and greater likelihood for cracks to close as the soil swells.

Sherard et al. (1976) indicate that based on their tests, soils with less than 10% finer than 0.005 mm may not have enough clay to support dispersive piping.

3.5.2 How can dispersive soils be identified?

There are several laboratory tests which can be used to determine the dispersivity of a soil:

a) Emerson class number; Standards Australia (1997), test AS 1289, 3.81 and USBR (1979). The test is carried out in distilled water, but may be repeated in water from the dam, or groundwater. This often gives significantly different results due to the presence of dissolved salts in the water (higher salt content gives less dispersive results). The soils are graded according to class, with Class 1 being the highly dispersive, Class 8 non dispersive. Soils with Emerson Class 1 to 4 needs to be treated with extra caution in dam construction

b) Soil Conservation Service test, also known as the double hydrometer test, or percent dispersion test; ASTM 2001 test D42291-99). Sherard et al. (1976) indicate that soils with a percent dispersion greater than 50% are susceptible to dispersion failure in dams, and those with a percent dispersion less than 15% are not susceptible. This test, and the 'triple' hydrometer test in which the soil is also dispersed in reservoir water, compares the dispersion in water to the dispersion that occurs when a chemical dispersant is used purposely to deflocculate the soil as it is in standard laboratory particle size distribution procedures.

c) Pinhole dispersion classification, also known as the pinhole test, or Sherard pinhole test (ASTM 1998, test D4647-93). Soils which tested as D1 and D2 were found by Sherard et al. (1976) to have suffered piping failure in earth dams, and severe erosion damage by rainfall in embankments and natural deposits while those with ND1 and ND2 classification had not. As for the Emerson class number, the results are dependent on the chemistry of the water used for the test (the standard test uses distilled water).

d) Chemical tests. Based on correlation with many dam failures and soil from dams which have leaked continuously (without any filters to control erosion) and not failed (Sherard et al., 1976) proposed to determine the dispersivity of soil based on the percent sodium and sodium adsorption ratio. These are defined as:

$$\text{Percent sodium} = \frac{Na^{+}}{\text{Total dissolved salts}} \times 100$$

$$\text{or} \qquad = \frac{Na^{+}}{Ca^{++} + Mg^{++} + Na^{+} + K^{+}} \times 100$$

$$\text{Sodium adsorption ratio (SAR)} = \frac{Na^{+}}{\left[\frac{1}{2}\left(Ca^{++} + Mg^{++}\right)\right]^{1/2}}$$

in which Na^{+}, Ca^{++}, and Mg^{++} are measured in milli-equivalents per liter of saturation extract. Details are given in Sherard et al. (1976).

Most authors consider that it is necessary to use more than one test to ascertain the dispersivity of a soil. Sherard and Decker (1977) suggest that four tests should be used: Soil Conservation Service, pinhole, Emerson and chemical test. They were of the opinion that the pinhole test was best.

More details on these tests and the properties of dispersive soils can be found in Fell et al (2005).

While laboratory tests are a useful way of identifying dispersive soils, much can be determined by observing the behavior of the soils in the field, e.g.:

- The presence of deep erosion gullies and piping failure in existing small farm dams usually indicates the presence of dispersive soils.
- Erosion of road cuttings, tunnel erosion along gully lines and erosion of weathered or clay infilled rock joints may indicate potentially dispersive soils.
- The presence of cloudy water in farm dams and puddles of water after rain indicates dispersive soils.

3.5.3 Effect of dispersion on erosion properties

See Section 3.4.3.

3.5.4 Effect of dispersion on filter design and assessment

See Section 7.5

4. INITIATION OF BACKWARD EROSION

4.1 BACKWARD EROSION PIPING LEADING TO FORMATION OF A PIPE

4.1.1 The overall process

Backward erosion piping occurs in non-plastic soils. It mainly occurs in foundations but may occur within embankments.

The erosion process begins at a free surface on the downstream side of a dam or dike as shown in Figures 2.5 and 4.1.

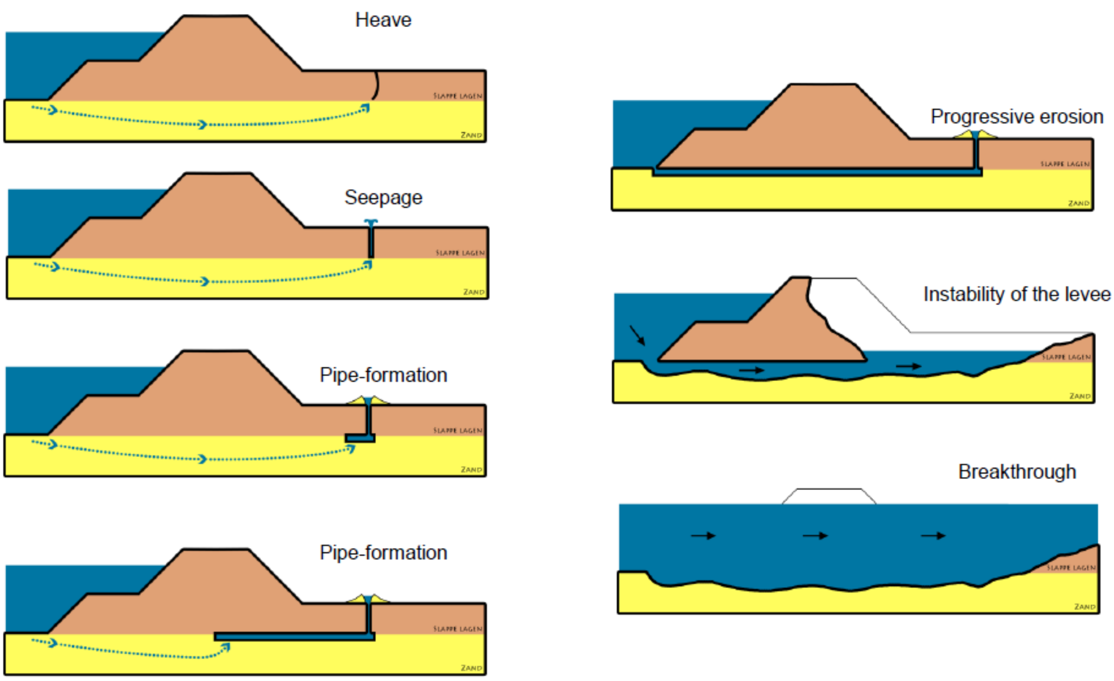


Figure 4.1 Geometry of Delft backward erosion piping model (Sellmeijer et al. 2012)

This free surface may be in a ditch or may form by cracking due to heaving of the cohesive strata overlying the cohesionless soil as shown in Figure 4.1, or in some cases may be the seeping surface on the downstream face of the dam, or in other cases the stream bed downstream of the dike.

4.1.2 The process in sandy dam or dike foundations

The backward erosion process progresses beneath the dike or dam. For this to occur the dike or dam or cohesive strata must form a roof for the eroding “pipe”. The presence of backward erosion piping is often exhibited by the presence of sand boils at the downstream side of the dam or dike.

Sellmeijer and co-workers from Delft Hydraulics and Delft Geotechnics Laboratories in The Netherlands carried out more than 70 backward erosion piping tests. The first were in flumes and are reported in de Witt et al (1981), Silvis (1991), Weijers and Sellmeijer (1993) and Technical Advisory Committee (1999). The tests were mostly on fine to medium sands, with a few tests on medium to coarse sands. The sands were uniform with uniformity coefficients Cu = 1.58 to 3.53. Early tests were on small scale models (base length 0.8 m), but later tests were on very large models. These experiments and those carried out by Townsend et al (1988) at University of Florida model backward erosion in the foundation of dikes, levees and dams.

These experiments showed that backward erosion initiates in the slot through the strata overlying the eroding soil representing a crack or drainage ditch excavated through the strata, and progresses in multiple small "channels" rather than a single "pipe". The channels are quite small. The height of the channels is typically 4 to 10 (d_{15}); that is often less than 2mm. For any head less than a critical head, the development of the channels stops. If the head is increased, erosion begins again. Figure 4.2 shows some examples. The critical head occurs when the length of the channel (l) is about 0.3 to 0.5 of the flow path length L. For heads less than this the progression of the pipe reaches a stable condition. For heads greater than the critical head the piping channel extends upstream and breaks through to the reservoir. The erosion then progresses rapidly as erosion in an open pipe. For these experiments the rate of progression of the pipe was relatively uniform until the length approaches about 40% of the total seepage path. It then accelerates. The piping progressed about 6 meters in an hour in the largest of the experiments.

In 2009 and 2010 further experiments were carried out at Deltares. These are reported in Van Beek et al (2010, a, b), Sellmeijer et al (2012) and Van Beek et al (2012b). The largest of these was virtually full scale. From these experiments Van Beek et al (2012b) refined the description of the backward erosion process as follows:

1. *Phase 1 Seepage occurs in the permeable strata*. In the experiments there were no confining strata.

2. *Phase 2.Backward Erosion.* At the beginning of the process there is rearrangement of grains, individual grain movements, and formation of small channels. The process reaches equilibrium for the hydraulic head applied. Very small amounts of sand are transported, in the order of cubic centimeters in this phase of the process. With an increase in the hydraulic head to the critical head sand is transported continuously. A variety of erosion patterns is observed in the small and medium scale experiments. In the large scale experiment sand boils are formed, and in the small and medium scale experiments craters are formed. These characterize the reaching of the critical head. The flow barely increases in this phase of the process. At a head greater than the critical head the erosion does not cease and the erosion rate is in the order of cubic decimeters per hour. The rate of erosion increases with increasing head.

3. *Phase 3. Widening of the channel*. As soon as the pipe reaches the upstream side a pressure surge occurs in the pipe. In the small scale experiments this in turn causes a large amount of sand to be eroded rapidly. In the medium and large scale experiments blockages caused by local collapse of the roof of the pipe take place and the widening process takes longer. The widening pipe develops from the upstream to downstream. The flow and sand transport to the exit point do not increase significantly. The widening process took up to a few days in the large scale experiments. When the widening pipe has almost reached the downstream side the sand transport and flow increases suddenly. The situation can change from sand boils to this condition without warning.

4. *Phase 4 Failure and Breakthrough*. Failure occurs soon after the widening phase is complete, but can be delayed due to collapse of the levee causing the first pipes to close.

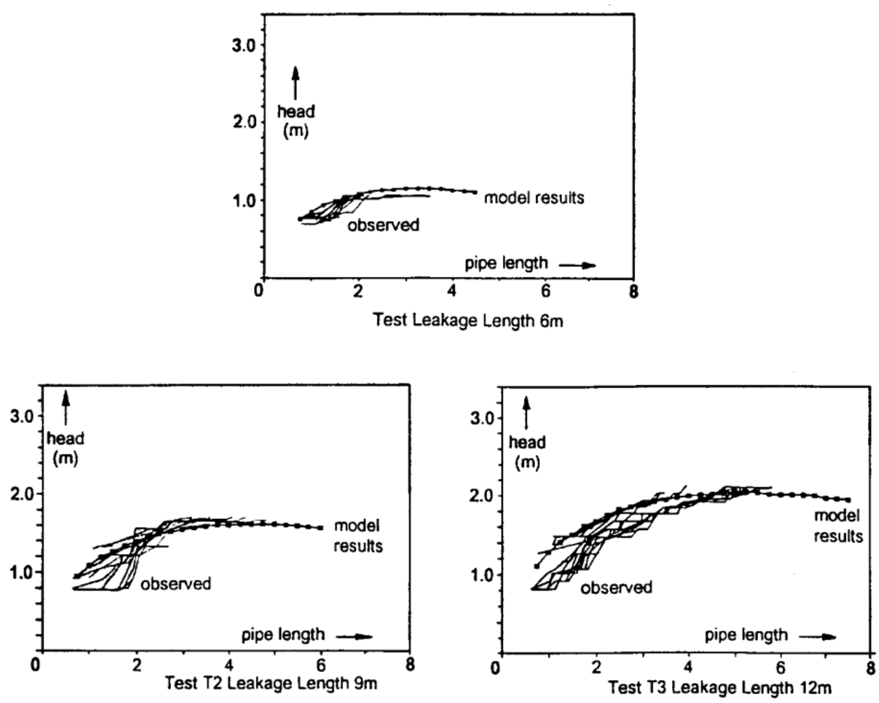


Figure 4.2 Delft Hydraulics laboratory piping flume test model: typical test results showing the development of the length of the tunnel as the head increases (Weijers and Sellmeijer (1993).

4.1.3 Backward erosion piping within a dam, levee or dike

For backward erosion piping to occur within a dam, levee or dike embankment the following criteria would have to be met:

1. The soil where pipes develop would need to be non-plastic.

2. There would have to be a free surface at the downstream of the dam core at which erosion could initiate; e.g. if there is no filter or an inadequately filtered contact between the core and a transition zone.

3. The core or some layer within the core would have to form a roof for the pipe to progress. This might occur at the phreatic surface if the partially saturated soil above the surface was silty, or if more cohesive strata were layered in the core.

4.1.4 Soils which are subject to backward erosion piping leading to formation of a pipe

The experience in USA and Europe is that backward erosion piping mostly occurs in the foundations of levees, dikes and dams where the eroding soil is fine to medium grain size sand, with a uniformity coefficient Cu < 3.

The participants at the Aussois Workshop (Fell and Fry 2007) considered that at gradients likely to occur within a dam or its foundation backward erosion is probably restricted to non plastic soils or soils with only limited plasticity. For practical purposes, Fell et al (2008) have concluded that based on the available data, the results of Wan and Fell (2004c, 2007, 2008) tests on internal instability and their experience and judgment, soils with Plasticity Index > 7 should be considered not subject to backward erosion at the gradients experienced in dams and their foundations.

4.2 METHODS FOR PREDICTING WHETHER BACKWARD EROSION PIPING WILL INITIATE AND PROGRESS

4.2.1 Terzaghi and Peck (1948)

Terzaghi and Peck (1948) show that backward erosion piping will initiate when a heave or zero effective stress condition occurs in cohesionless soils at the downstream toe of a dike, levee or dam. They do not consider the conditions under which the backward erosion will progress to form a pipe except to recognize that it is necessary for a roof to be formed in an overlying stratum or by the levee.

The Terzaghi and Peck (1948) philosophy has been the dominating influence in design of levees (dikes) and dams, particularly in the USA (e.g. USACE, 2000, 2005, Wolff, 2002). Design methods have concentrated on avoiding the heave or blow-out condition. It has however commonly been assumed, at least implicitly, that if backward erosion initiates to form a sand boil it will progress to form a pipe, at least under repeated loading from successive floods.

The Delft and University of Florida experiments (see below) show that this is not necessarily the case and that the progression of the pipe may stabilize. However the latest experiments by Deltares indicate that once sand boils form the critical gradient at which progression continues has probably been exceeded and backward erosion will progress unless the gradient is reduced by the river level dropping or the tail water level being raised, e.g. by building sand bag levees around the sand boil.

4.2.2 Method of Sellmeijer and co-workers at Deltares

Sellmeijer (1988), Sellmeijer and Koenders (1991), Koenders and Sellmeijer (1992) developed a mathematical model for backward erosion piping based on the experiments carried out at Delft Laboratories in the 1980's.

This model was refined using the results of the Deltares testing and is presented in Van Beek et al (2010) and Sellmeijer et al (2012).

The critical gradient is determined as a product of three contributions: resistance factor, scale factor and geometrical shape factor. The improvement is realized by the outcome of 38 small-scale tests, applying the multivariate method. The refined equations for the critical gradient at which backward erosion will progress are:

$$\frac{H}{L} = \frac{1}{c} = F_R F_S F_G$$

$$F_R = \eta \frac{\gamma'_p}{\gamma_w} \tan \vartheta \left(\frac{RD}{RD_m}\right)^{0.35} \left(\frac{U}{U_m}\right)^{0.13} \left(\frac{KAS}{KAS_m}\right)^{-0.02}$$

$$F_S = \frac{d_{70}}{\sqrt[3]{\kappa L}} \left(\frac{d_{70m}}{d_{70}}\right)^{0.6}$$

$$F_G = 0.91 \left(\frac{D}{L}\right)^{\frac{0.28}{\left(\frac{D}{L}\right)^{2.8} - 1} + 0.04}$$

Where:

H	[m] :	hydraulic head across structure
L	[m] :	seepage length (= base length of the embankment)
D	[m] :	thickness of sand layer under the embankment
c	[-] :	erosion coefficient
F_R	[-] :	resistance factor
F_S	[-] :	scale factor
F_G	[-] :	geometrical shape factor
RD	[%] :	relative density
U	[-] :	uniformity coefficient $Cu = d_{60} / d_{10}$
KAS	[%] :	roundness
d_{70}	[m] :	soil particle diameter for which 70% by weight of the soil is finer
γ'_p	[kN/m^3]:	submerged unit weight of soil particles 9.8(G-1)
G	[t/m^3]:	soil particle density
γ_w	[kN/m^3]:	unit weight of water
η	[-] :	Whites drag coefficient
ϑ	[DEG]:	bedding angle (angle of repose) of sand
κ	[m^2] :	intrinsic permeability

Where: $\kappa = \frac{\upsilon}{g} k$

υ [m^2 /sec]:	kinematic viscosity
g [m/sec^2]:	gravity
k [m/sec]:	hydraulic permeability

H, D and L are defined in Figure 2.5 in Chapter 2. In these equations the variables are normalized by the mean values in the data set. For the data set used by those authors the mean values are as detailed in Table 4.1.

Figure 4.3 can be used to assess critical gradients for soils within the limits in Table 4.1. Note that the effect of the F_G term is included in the curves on Figure 4.3.

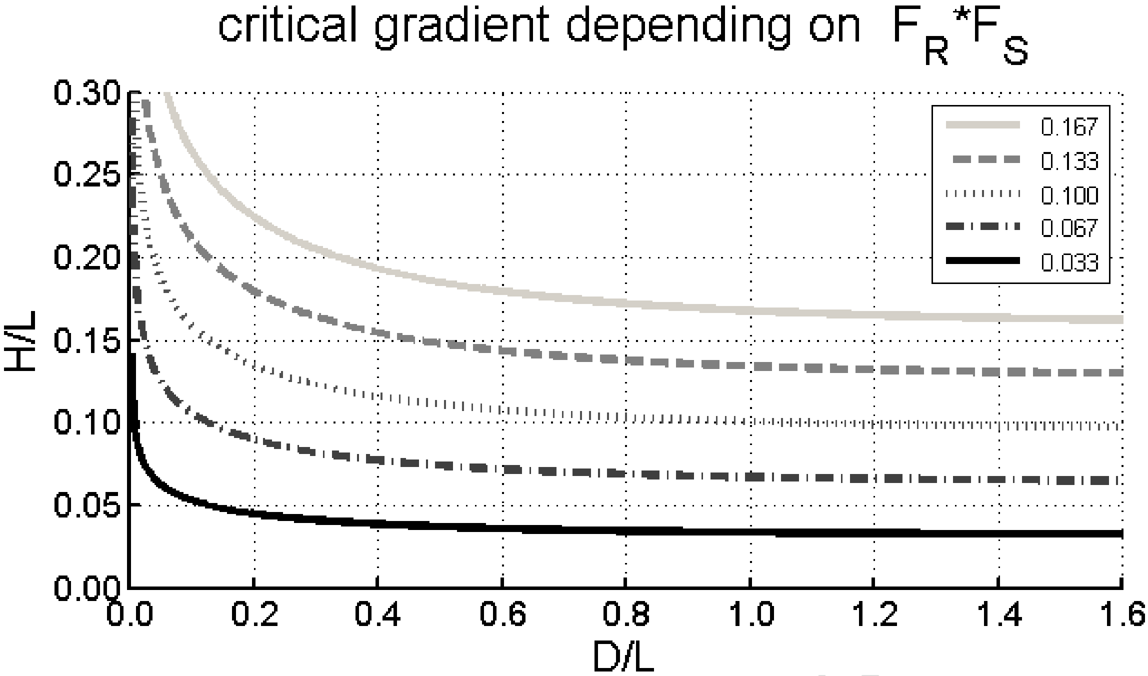


Figure 4.3 Critical gradient for various $F_R * F_S$ values and embankment dimensions. H, D and L are defined in Figure 2.5. As an example, for $F_R * F_S = 0.100$, D/L = 1.0, critical gradient at which backward erosion will progress to form a pipe back to reservoir, is H/L = 0.10. (Courtesy of Dr Hans Sellmeijer and Vera van Beek).

Sellmeijer et al (2012) indicate that their data set assumes a bedding angle of 37°, and a Whites coefficient of 0.25. For water at 20^o Celsius, $\kappa = 1.02 \times 10^{-7}$ (k) where κ is m^2, k is in m/sec. They indicate that they ignore the roundness and Uniformity Coefficient terms as they do not contribute significantly.

They indicate that the refinements have been determined from the small scale tests and it is not altogether clear if there may be a scale effect, so that the outcome for large structures may not be properly modeled. They also indicate that the equations should only apply within the limits of the parameters during testing. These limits are given in Table 4.1.

Table 4.1 Limits of the Sellmeijer et al (2012) method

Parameter	Minimum	maximum	mean
RD	34 %	100 %	72.5 %
U	1.3	2.6	1.81
KAS	35 %	70 %	49.8 %
d_{70}	150 µm or 1.5E-04m	430 µm or 4.3E-04m	207 µm or 2.07E-04m

They found that the equations predicted the large scale experiment behavior quite well for the fine grained soil, but that it was not so accurate for the soil with a d_{70} = 260 µm.

There are very significant scale effects. The smaller the structure, the higher the critical gradient. In the tests the value of the scale factor F_S ranged from 0.134 for the large scale IJKdijk tests, to 0.192 for the medium-scale tests and 0.421 for the small-scale tests.

It should be noted that Sellmeijer et al (2012) emphasize that the above equations do not include any margin of safety and that for design rules they are likely to apply factors of safety.

Van Beek et al (2012a) extended the earlier work to make it applicable to the prediction of piping in multi-layer foundations.

4.2.3 Hoffman's Hydraulic Approach

Dr Gijs Hoffmans, from Deltares, Delft, Netherlands re-analyzed the results of the Delft experiments (Hoffmans, 2013). He, like Sellmeijer, recognizes that for backward erosion to progress, the particles being released from the initiating zone at the upstream end of the piping channel must be transported down the channel. Hence it is the interaction of the gradients at the backward eroding head of the piping channel, and the flow gradient in the piping channels which controls the process. The latter is controlled by the gradients of flow into the pipes but also the permeability of the soil as this affects the flow quantity in the pipes.

Table 4.2 gives Hoffmans' equations and lists the variables. The most important variables in the Hoffmans approach are hydraulic conductivity (k), particle sizes d_{50}, d_{15} and the critical pipe height ($\ell_{z,c}$), and a coefficient α_H. The latter two are determined from the experiments.

Table 4.2 Overview of Hoffmans' piping equation (from Hoffmans 2013)

$$\frac{\left(H_1 - H_2\right)_c}{L} = \frac{\sqrt{g}\left(\Psi_{\ell am,c}\Delta d_{15}\right)^{3/2}}{\nu\sqrt{\alpha_{\mathrm{Re},\ell}}} + \left(1 - \frac{\ell_c}{L}\right)\frac{d_{50}\nu}{\ell_{\mathrm{Re}}KD}$$

with

$$\frac{\ell_c}{L} = \exp\left(-\left(\frac{\alpha_f D}{L}\right)^2 \frac{\sqrt{g}\left(\Psi_{\ell am,c}\Delta d_{15}\right)^{3/2}}{\nu\sqrt{\alpha_{\mathrm{Re},\ell}}}\right)$$

Symbol	Indicative range	Comment
d_{15}	0.08 mm < d_{15} < 0.32 mm	d_{15} for which 15% of the particles is finer than d_{15}
d_{50}	0.13 mm < d_{50} < 0.75 mm	mean particle diameter
D	0.1 m < D < 50 m	thickness of the sand layer
$D_* = d_{50}\left(\Delta g / \nu^2\right)^{1/3}$	4 < D_* < 16	dimensionless particle diameter
g	g = 9.81 m/s^2	acceleration of gravity
$(H_1 - H_2)_c/L$	0 < $(H_1 - H_2)_c/L$ < 1	critical hydraulic gradient
H_1	1 m to 10 m	sea or river level
H_2	usually taken as 0 m	water level in ditch or at soil surface
K	10^{-5} m/s < K < 10^{-3} m/s	hydraulic conductivity (see also Eq. 2.37)
ℓ_c	0 < ℓ_c/L < 1	ratio of critical pipe length and seepage length
ℓ_{Re}	$\ell_{\mathrm{Re}} = 18.2\cdot10^{-6}$ m	geometrical length scale (see also Section A17)
L	0.3 m < L < 200 m	seepage length
α_f	α_f = 5	geometrical groundwater coefficient (Section A17)

$\alpha_{\mathrm{Re},\ell}$	$\alpha_{\mathrm{Re},\ell} = 6.8$	geometrical pipe coefficient (see also Section A17)
$\Delta = \rho_s/\rho - 1$	$\Delta = 1.65$ (sand)	relative density
ν	10^{-6} m^2/s $< \nu < 1.4 \cdot 10^{-6}$ m^2/s	kinematic viscosity
$\Psi_{\ell am,c} = 0.2(D_*)^{-0.3}$	$0.08 < \Psi_{\ell am,c} < 0.16$	critical Shields parameter (for laminar flow)

if $\ell = 0$ then the equilibrium is stable (or no erosion occurs);

if $0 < \ell < \ell_c$ then the equilibrium is unstable (or occasional erosion occurs);

if $\ell \geq \ell_c$ then there is no equilibrium anymore (or erosion occurs until the dike fails).

4.2.4 Russian Design Methods

Belkova et al (2011), Zhilenkov et al (2011) and Rumyantsev et al (2012) give summaries of Russian standards for design for seepage flow in the embankment and foundations of dams. Radchenko et al (2012) give more details. They indicate that these standards require that numerical analyses of seepage in the dam body, foundation and river banks are made, including accounting for non-uniform and anisotropic geological conditions.

The standards require that the following parameters of seepage flow should be determined in studies and analyses:

- position of phreatic surface in the dam body and banks;
- seepage flow rate through the dam body, foundation and banks;
- heads (or hydraulic gradients) of seepage flow in the dam body, foundation and zones of outflow into the drainage or downstream of the downstream toe, in the zones of contact of soils of different soil parameters and at the boundaries of impervious elements.

In seepage analyses of thawing earth dams constructed in the northern construction and climatic zone, parameters of seepage flow should be determined.

Possible variation of seepage parameters with time should also be taken into account.

To determine what is termed the “seepage strength” of the system as a whole and its separate elements, two concepts, those of overall and local seepage strength, are used:

The “seepage strength” of the dam body and impervious elements is estimated on the basis of numerical analyses and laboratory testing of soils under the hydraulic gradients expected to be acting in the structural elements of the dam. The analyses are made using the highest value of head (reservoir water level) expected to occur at the dam. The stress-strain state of the dam and its foundation are also taken into account.

Using the results the seepage strength is checked, and the dam layout, in particular the location of drainage, is adjusted if necessary to satisfy the design criteria.

For the dam body, the following are determined:

- permeability factor k;
- unit water absorption q, for soils in the freezing zone

- parameters of soil seepage strength (local and averaged critical hydraulic gradients J_{cr} and $J_{cr,m}$ and critical seepage rates V_{cr})

If necessary, structural solutions are adjusted after numerical checks of the foundation strength.

The standards require that in the foundations the following should be prevented:

- loss of overall seepage strength of non-rock foundations;
- damage to, or ineffective operation of, impervious elements in the foundation, which causes unacceptable loss of water, flooding, saturation of slopes, etc.;
- non-uniform displacements of foundation sections, which might cause failure of impervious elements;
- increase of pore pressures or seepage flow rate.

As a result, the following should be determined:

- overall seepage strength of non-rock foundations;
- local seepage strength of rock and non-rock foundations in cases when piping channels may occur.

The main parameters used for the estimation of seepage strength are hydraulic gradient (average or local), or seepage flow rate.

The criterion of seepage strength is the following inequality:

$$J_{est,m} \leq \frac{1}{\gamma_n} J_{cr,m}$$

Where:

$J_{est,m}$ = average hydraulic gradient acting in the analyzed seepage zone;

γ_n = reliability factor which depends on the social and economic consequences of failure of the dam;

$J_{cr,m}$ = average critical hydraulic gradient taken on the basis of soil testing in conditions representing operational conditions at the dam.

In preliminary analyses, values of $J_{cr,m}$ may be taken using available similar results or from Table 4.3 (for soils in dam zones) and Table 4.4 (for soils embedded in structure foundation).

Table 4.3. Values of average critical hydraulic gradients $J_{cr,m}$

Soil	Apron	Diaphragm / core	Dam body
Clay, clay concrete	15	12	8-2
Sandy clay	10	8	4-1.5
Clayey sand	3	2	2-1
Medium sand	-	-	1
Fine sand	-	-	0.75

Table 4.4. Values of $J_{cr,m}$ for soils embedded in structure foundation

Soil	Calculated averaged critical hydraulic gradient
Fine sand	0.32
Average coarse sand	0.42
Coarse sand	0.48
Sandy clay	0.60
Loam	0.80
Clay	1.35

The values of average critical gradient are determined depending on the physical and mechanical properties of soils and the methods of soil placement, higher values of $J_{cr,m}$ are set for denser soils.

In the estimation of local seepage strength, the following conditions are used:

$$J_{est} \le \frac{1}{\gamma_n} J_{cr}, \qquad \text{or} \qquad V \le n \frac{V_{cr}^{o}}{\gamma_n}$$

Where:

J_{est} – actual hydraulic gradient determined by numerical analyses, modeling or from field observation data;

J_{cr}, V_{cr}^{o} – local critical hydraulic gradients and seepage flow rates in pores determined in target zones by numerical methods or by testing piping strength of soils in laboratory or field conditions;

J, V – actual local hydraulic gradients and seepage flow rates determined in the same zones by seepage analysis;

n – porosity.

4.2.5 Other Methods

Schmertmann (2000) developed a method based on backward erosion piping tests in flumes at University of Florida. These were carried out in a number of sets, including those by Townsend et al (1988) who carried out 15 tests. Those tests were carried out on a range of soils from fine to medium sands, up to coarse sand/fine gravel mixes.

The tests were different to the Delft tests in that a starter pipe was formed at the downstream end. The head differential was increased progressively to keep the tunnel progressing upstream until the critical head was reached causing the tunnel to form a pipe to the upstream source.

Most of the tests (32 out of 39) which Schmertmann (2000) relied upon were carried out on sands with a uniformity coefficient less than 3.2. The other 7 tests were carried out on gap-graded soils and a well graded soil with uniformity coefficients around 6.

Schmertmann (2000) found that the critical average gradient $\bar{i}_{pmt}$ above which backward erosion would occur was related to the uniformity coefficient Cu (d_{60}/d_{10}) of the soils tested, as follows:

$$\bar{i}_{pmt} = 0.05+0.183(C_u-1)$$

He also plotted the Delft tests and found a similar correlation. However there were few tests on soils with a Cu >3.

The method involves quite large corrections for scale effects. Given it is based on little data in the larger uniformity coefficient range, and some of these may be affected by internal instability, caution should be exercised in using the method for soils with Cu > 3.

4.3 SOME FIELD OBSERVATIONS

The US Army Corps of Engineers (USACE) have a shared responsibility with state and local governments and local levee boards for managing the levee systems along major USA Rivers including the Mississippi. USACE carried out extensive underseepage investigations from the 1930s to the 1960s covering the middle and lower Mississippi River, from Alton, Illinois to New Orleans, Louisiana. As recorded by Sills and Vroman (2007), these early studies resulted in the development of current USACE levee design methods which were validated during decades of field observations and laboratory tests.

More recent investigations (Wolff 1974 and 2002, Shannon and Wilson Inc, 1995, Wilson 2003, and Glyn and Kuszmaul, 2010) have observed the ageing levee systems' performance. Some observations from these include:

In any one flood the levees in one USACE District, e.g. St Louis, may experience many hundreds of sand boils in the foundations. However there are few documented cases of breaching of the levees from piping. This is influenced by the "flood fighting" efforts of the Districts and the Levee Board personnel. They build sand bag rings and in some cases sub-levees around the sand boils to stop the flow of sand (but not the flow of water). The few documented breaches may support the laboratory tests (see 4.2.1 above) which show that erosion initiates at lower average gradients than the higher critical gradients required to progress the erosion to form a continuous pipe from the land side to the river side. More analysis of the field data is required to investigate the conditions at which erosion causing sand boils progressed to erosion causing levee failure.

Sills and Vroman (2007), Wolff (2002) and Glynn and Kuszmaul (2004) report that there are some cases of levees which have sand boil activity occurring at successively lower average gradients (lower river stages). Glynn and Kuszmaul (2004) show that greater sand boil activity occurred in the 1995 flood than the 1993 flood along the levees studied, even though the river stage was lower in 1995. In 1995, 37% of the sand boils occurred at locations close to the 1993 ones. The phenomenon of possibly decreasing gradients at which successive backward erosion to form sand boils occurs was discussed in the early investigations by USACE, but not quantified. It was not investigated by the laboratory tests described in 4.2.1 above. Sills and Vroman (2007) suggest that it may arise because after sand boil episodes, the erosion pipes are not re-filled, and probably remain open, thereby increasing the permeability/porosity of the soil, and reducing the gradient at which backward erosion commences during subsequent high water level events.

USACE (1956) and Wolff (2002) also show that local geology has an important influence on the occurrence of sand boils. Sand boils are more likely to occur where swales from point bar deposits cross the levee at an angle and concentrate seepage at the toe. Glynn et al (2012) conducted statistical analyses of sand boil locations and found that the parameters most significant to sand boil locations are: geological conditions, landside confining layer thickness, effective grain size coefficients and previous sand boil locations. Using logistic regression of these significant parameters, empirical models were developed for calculating the probability of sand boils developing at selected levee segments. The logistic regression modeling method can be used to develop a unique statistical model for predicting sand boil activity for any levee system. More research is planned to apply the modeling method to rivers located in different geologic regions.

4.4 GUIDANCE ON WHETHER THE OVERLYING SOIL WILL FORM A ROOF TO THE PIPE

Section 8.2.1 and Table 8.1 give guidance on whether the overlying strata will form a roof for the backward erosion pipe.

4.5 THE PROCESS OF GLOBAL BACKWARD EROSION IN CENTRAL CORE DAMS

The available evidence on the occurrence of global backward erosion (see Figure 2.7 in Section 2.3.2.2) in central core dams is that this internal erosion mode will only occur at relatively high gradients except when the core consists of cohesionless, internally unstable soil subject to suffusion. This evidence includes:

a) Laboratory tests by Sun (1989), Marot et al (2007) and Bendahmane et al (2008) confirmed that backward erosion could occur in more cohesive soils, but initiated at very high gradients which were not likely to occur in dams or their foundations. The tests done at the VNII Vodgeo Laboratory and reported by Istomina (1957) showed that the critical hydraulic gradient of a soil with a very low plasticity and liquid limit equal to 14 is higher than 10 at moisture content close to the Optimum Proctor. They showed that the critical hydraulic gradient not only depends on the plasticity but on the soil consistency as well.

b) Bendahmane et al (2008) carried out tests which showed that soils consisting of 10% kaolin / 90% fine sand initiated backward erosion at gradients between 90 and 140 depending on confining pressure. For kaolin contents 20% and 30% no backward erosion occurred at gradients as high as 100. They observed what they called suffusion with gradients as low as 5, but noted that this value should not be taken as generally applicable.

c) Moffat and Fannin (2011) and Moffat et al (2011) indicate that there is an initial movement of finer particles at gradients of about 4 for the cohesionless soils they tested and more extensive movement of finer and some coarser fraction soils at higher gradients, from about 10 to 30 in the soils tested. These movements of particles are accompanied by a reduction in volume as particles are eroded from the soil through inadequate filters.

d) Marot et al (2007) and Bendahmane et al (2008) tested clay soils and Sail et al (2011) a non plastic gap graded soils with 40% finer fraction. They observed some particle movement at a gradient of about 5 and major movement at much higher gradients.

e) Some recent tests on glacial till soils from an Australian dam showed that for vertical downward flow to model the condition in Figure 2.7 what appears to be backward erosion occurred in broadly graded cohesionless soils at a gradient of 9. Erosion had progressed for about 40 days at a gradient of 5 and may have reached the failure condition without the increased gradient if the duration of the test had been longer. This raises questions as to time effects for this type of backward erosion as it seems the process may be very slow. More testing is required to explore this phenomenon.

There are no known reliable methods for predicting global backward erosion in broadly graded cohesionless soils such as glacial tills in the cores of dams, and for development of vertical piping as shown in Figure 2.7. For these situations it is recommended that specifically designed laboratory tests should be carried out.

4.6 GLOBAL BACKWARD EROSION, LEADING TO UNRAVELING, AT A SEEPING DOWNSTREAM SURFACE OF THE EMBANKMENT

This is quite a different mode which may occur in dikes or small dams constructed only of silty sand gravel. There is a potential for particles to be removed from the downstream slope by gravity and hydraulic seepage gradient under the through flow breakout line, where seepage emerges from the downstream face or toe. The failure mode is one of unraveling of the downstream slope rather than formation of a pipe through the embankment.

It is very likely that unraveling will occur if the particle diameter is under a critical size associated to the discharge rate. This can be assessed by the methods Solvik (1991, 1995), and the revised method by EBL Kompetanze AS (2005). The following equation is recommended to describe the initiation of erosion of dam toe subject to through flow:

$$d_{50} = 0{,}43 S_0^{0{,}43} q^{0{,}78}$$

Where d_{50} is rock size in meters, S_0 slope of rockfill and q unit discharge in m^3/s/m.

If suffusion occurs in the material used to construct the dike this process may be accelerated.

5. INITIATION OF CONTACT EROSION

5.1 THE OVERALL PROCESS

Contact erosion occurs where a coarse soil such as a gravel is in contact with a fine soil, and flow parallel to the contact in the coarse soil erodes the fine soil. For example flow through gravel alluvium in the foundations of dam, levee or dike may erode the base of an overlying silt layer, or erosion of the finer layers of soil in a core may occur into a coarse gravelly layer formed by segregation during construction. Figure 5.1 shows examples.

Particles of the finer layer may be destabilized by the water flow and transported through the pores of the coarser layer parallel to the interface.

This phenomenon requires two conditions. First, the coarse layer has to be geometrically open to the other layer, that is to say, to have pores sufficiently large so that fine particles can pass through them. Second, the hydraulic conditions must be such that the flow velocity is sufficient to detach particles and to transport them.

5.2 FAILURE MODES AND VULNERABLE DAM TYPES

All interfaces between different soils which exist in the dam or in the foundation may be a vulnerable to contact erosion (Figure 2.8). Nevertheless, locations with possible high velocities in the coarser layer and high particle size grading contrast between layers are the most likely to experience contact erosion. These characteristics often occur at the interface between the core and a gravelly foundation. For example, at the zoned dikes on the Rhone River, with clayey silt cores and gravel shoulders, some twenty cases of leakage associated with development of a sinkhole or subsidence have been reported (CFGB, 1997). The process starts with contact erosion at the interface between the silt and the gravel, often at the contact between the fill and the foundation or in the foundation. This causes a cavity within the fill, and then the pressure drops around that cavity, causing the roof collapse. Those materials fill in the lower part of the cavity and enlarge it at the top; then the roof of the new cavity is decompressed and, step by step, the cavity progresses along a practically vertical chimney towards the crest or the upstream face (Figure 5.1 a). Leakage flows decrease after the materials fall, and then progressively build up again, carrying away the fallen material until the next occurrence. In most cases the sinkhole volume is about 1 to 2 m^3.

Contact erosion may also appear between any granular layer (filter, drain, riprap) and a fine soil in contact with that layer. For example, in the case of abnormally high water level in the reservoir rising above the top of the core, flow can pass through a riprap layer or a gravelly shoulder zone over the top of the core and can erode the core.

Other consequences of contact erosion are possible (Figure 5.1). First, depending of the mechanical properties of the core, the cavity created by contact erosion may not collapse and can be a beginning for backward erosion piping (consequence b). This process has been identified in hydraulic flume tests at scale 1/1, where it appeared that the downstream part of the opening was held open by hydraulic fracture. Second, a weaker zone, less dense because of the development of contact erosion, can cause a loss of stability (consequence c). Finally, the eroded fine particles may clog the permeable layer and increase pore water pressure which may result in instability of the downstream slope (consequence d).

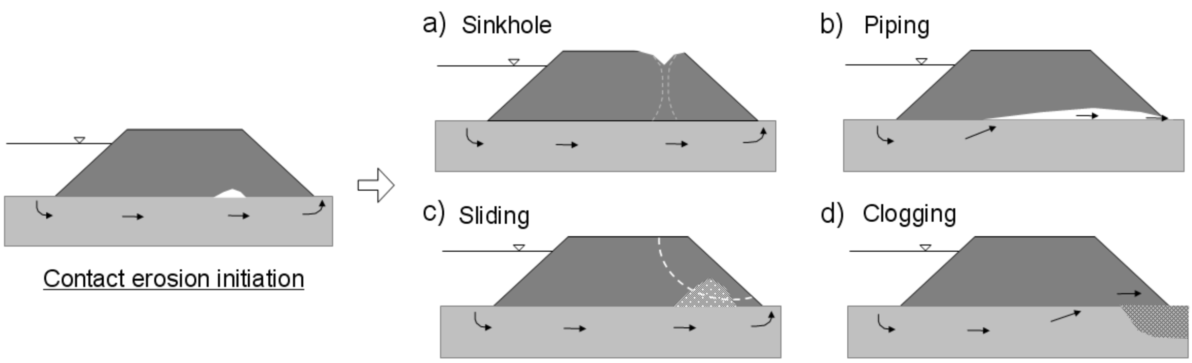


Figure 5.1 Consequences of contact erosion. Black arrows indicate a groundwater flow through a more permeable layer (light grey) under a less permeable dam (dark grey). a) sinkhole daylights b) beginning of backward erosion piping c) creation of a weaker zone initiating instability d) clogging of the permeable layer and increase of pore water pressure. (Beguin, 2011).

5.3 METHODS TO ESTIMATE LOADS AND RESISTANCE TO CONTACT EROSION

The two conditions needed for initiation of contact erosion are:

1. Geometrical condition: pores of the coarse layer have to be sufficiently large to allow particles to pass through. Some authors have proposed criteria in the specific context of contact erosion but classic filter criteria can also be used (Section 7.5, ICOLD 1994).

2. Hydraulic condition: the flow velocity has to be sufficient to detach the particles and also to transport them.

If both conditions are fulfilled, contact erosion is likely to occur.

Different authors who have studied contact erosion e.g. Brauns (1985), de Graauw et al (1983), Wörman et al. (1992), Den Adel et al (1994), Guidoux et al (2010), proposed expressions for the hydraulic conditions which imply the detachment, depending on each configuration, and the transport of particles. These laws have been established based on experimental results. These authors noticed that close to the geometrical limit, the hydraulic loading for erosion initiation is increasing. They noted a transition zone of combined influence where the hydraulic loading needed to initiate contact erosion is higher than in the domain of pure hydraulic influence as shown in Table 5.1.

Table 5.1 is derived from Brauns (1985), Wörman (1992) and Den Adel et al (1994). They found that for D_{15} / d_{85} ratios less than in the third column, there is geometrical filtration whatever the hydraulic loading, so contact erosion could not occur. For D_{15} / d_{85} ratios more than in the fifth column hydraulic loading controls erosion and there is no filtration effect. In between these two limits both geometric and hydraulic factors control erosion.

Table 5.1 Domain of geometrical and hydraulic influence. (courtesy of R.Beguin, from Brauns, 1985, Wörman, 1992 and Den Adel, 1994).

	Grading ratio D_{15} / d_{85} →				
Brauns (1985) soil with n=0.4	Geometrical condition	7.5	Geometrical and Hydraulic condition	25	Hydraulic condition
Wörman (1992) soil with D_{15}=0.88D_H				14.6	
Den Adel (1994) soil with d_{85}=d_{50}/0.9		8.1		11.7	

Note: D_{15} is the particle size of the coarser soil for which 15% is finer; d_{85} is the particle size of the finer soil for which 85% is finer. n is porosity. For D_H see 5.3.2 below.

The second boundary between “Geometrical and Hydraulic condition” and “Hydraulic condition”, defines if the coarse layer grading has, or has not, an influence on the hydraulic criteria for erosion initiation. For example, Brauns (1985) obtained that in the “Hydraulic condition” domain, the critical velocity for a fine soil can be calculated without taking into account the coarse soil grading. (This is valid in its experimental range $25<D_{15} / d_{85} <57$). In the “Geometrical and Hydraulic condition” domain, for $7.5<D_{15} / d_{85} <25$, the critical velocity will be also function of the coarse soil grading. In a similar manner, erosion laws proposed by Den Adel (1994) and Wörman (1992) are valid in their “Hydraulic condition” domain, where the influence of the coarse layer on the initiation of erosion can be neglected.

For one particular fine soil, the “critical” gradient, which corresponds to the gradient in the coarse layer parallel to the contact at which the erosion initiates, can vary by one order of magnitude, depending of the permeability of the coarse layer. However, in the same tests, the “critical” Darcy velocity for erosion initiation does not significantly depend on the coarse layer permeability, and is only related to the fine soil resistance to erosion. Therefore Darcy velocity has been chosen by the majority of authors as a good indicator of the hydraulic loading.

The hydraulic conditions for contact erosion depend on the configuration considered.

5.3.1 Fine cohesionless soil below a coarse soil layer

This configuration has been widely studied (Istomina, 1957; Pravedny, 1966, Brauns, 1985; Bezuijen et al., 1987).

In the case of sand, erosion particles are mainly transported as bed load and authors conclude that we can use classical river erosion criteria empirically adapted to the case of contact erosion. In consequence, they propose laws based on Shields (1936) criterion. One conclusion of these works is that the diameter of coarse layer particles has a weak influence on the critical velocity.

Experimental results (Beguin 2011) are shown in Figure 5.2 with other results based on the work of other researchers.

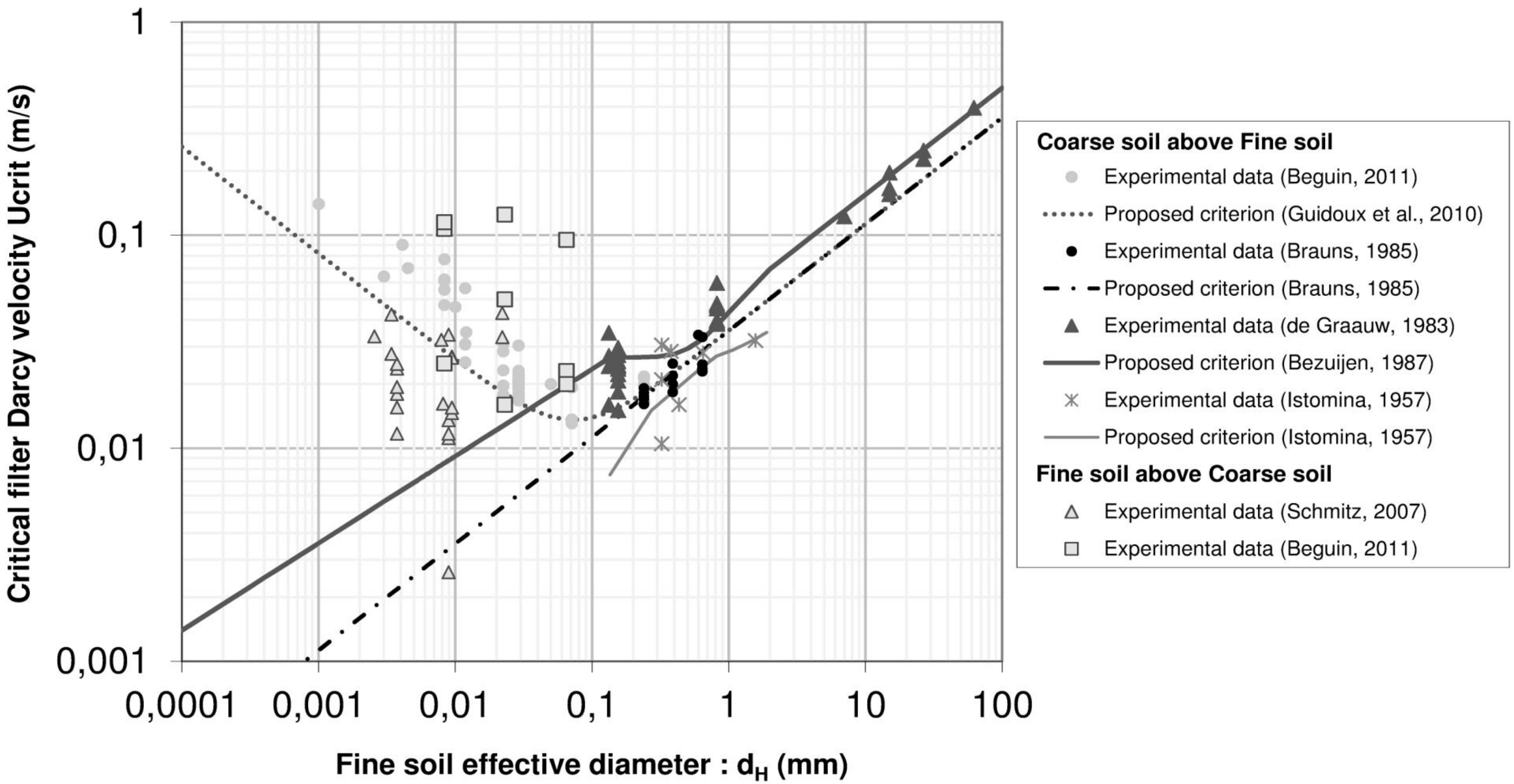


Figure 5.2 Critical velocities for contact erosion of sand above and below gravel (courtesy of Dr Remi Beguin)

Experimental results range between 0.01m/s and 1m/s for the critical Darcy velocity, and a minimum seems to appear for particles of diameter 10^{-4} m (0.1mm, 100µm).

Brauns' (1985) law is the simplest formula to use and gives a good approximation for sand:

$$U_{crit} = 0.65.n_D \sqrt{(\frac{\rho_s - \rho_w}{\rho_w})gd_{50}}$$

with n_D the porosity of the gravel layer, ρ_s (kg/m^3) the density of the sand particles, ρ_w (kg/m^3) the water density, d_{50} (m) the median diameter of sand grading curve.

For loose-packed materials (say $d > 10^{-4}$ m), Hoffmans (2013) proposed an equation, based on Darcy's law, the Poiseuille flow, the shear stress approach of Shields (1936), the Shields Number $\Psi_{\ell am,c}$ of sediment transport in laminar flow and some geometrical assumptions regarding the pipes, giving the critical Darcy velocity U_{crit}:

$$U_{crit} = \sqrt{\tfrac{1}{2}\mathrm{Re}_{m,c}\,\Psi_{\ell am,c}\Delta g d_{15}}$$

Thus U_{crit} depends on the maximum critical Reynolds number and the critical bed shear velocity. Considering the laminar flow conditions, where the Reynolds filter number (Re_f) is smaller than 10, it is noted that Hoffman's equation is comparable with the method as proposed by Guidoux et al (2010) (see 5.3.2 below).

To predict the rate of erosion evolution, three models of sand erosion have been developed which allow estimation of the amount of transported sand as a function of the hydraulic loading (Wörman, 1992; Den Adel, 1994; Scheuermann, 2002). However these models have been validated only for the soils tested by their authors, and they have to be used with caution.

5.3.2 Silt and clay erosion (particles < 75µm)

Guidoux et al (2010) carried out some experimental tests of contact erosion with silt and clay. He adapted Brauns' (1985) law with an empirical parameter to take into account the adhesive forces. To consider widely or gap graded soils, he also proposed to use the effective diameter d_H of the fine soil instead of the d_{50} (Kozeny, 1953). The effective diameter is defined by:

$$d_H = \left(\sum_{j=1}^{m}\frac{F_j}{d_j}\right)^{-1},$$

Where F_j is the percentage of the fraction of diameter d_j in the grading curve of the soil.

This diameter conserves the specific surface of the initial grain size distribution.

Guidoux et al (2010) proposed that the critical Darcy velocity U_{crit} can be determined from:

$$U_{crit} = 0.65.n_D \sqrt{\left(\frac{\rho_s - \rho_w}{\rho_w}\right) g d_H \left(1 + \frac{\beta}{{d_H}^2}\right)}$$

The empirical parameter β is a function of the cohesive properties of the soil, selected as 5.3×10^{-9}m^2 by Guidoux et al (2010). The other variables are as for Brauns (1985) above. This formula gives a reasonable fit to the experimental data for these finer soils. It cannot be applied for very cohesive soils which exhibit different erosion behavior linked not to particle diameter, but to other more relevant properties such as the clay mineralogy and degree of saturation.

5.3.3 Fine soil above a coarse soil layer

In this configuration experimental data is limited. The phenomenon is complex and cannot be directly linked to river erosion. In consequence, a safe hydraulic condition for contact erosion in this configuration should be based on the transport velocity for the particles, considering that detachments occur whatever the flow velocity is. For example, Goltz et al (2009) proposed a law for particle transport, based on Muckenthaler's approach (Muckenthaler, 1989; Perzlmaier, 2007) comparing sedimentation velocity of particles and pore velocity.

Schmitz (2007) carried out experimental tests of erosion of silt layers above coarse layers. In contrast to the previous configuration (fine soil below coarse soil) he noticed an influence of the confining stress on the critical velocity. For higher vertical stresses on the sample, he measured higher critical velocities. Except for one value, the critical velocities measured are of the same order of magnitude as in the other configuration, between 1cm/s to 10cm/s (Figure 5.2). He proposed a law for critical velocity depending on results of vane shear strength tests on the soil, but comparison of its model with results is not very convincing. This is to be expected as the critical hydraulic shear stress for clay soils is not related to the shear strength, as discussed in Section 3.4.

Beguin (2011) carried out six tests on silt embankments over gravel. The results are included on Figure 5.2. The critical velocities were more scattered than those for the gravel above silt configuration, but of a similar order of magnitude. It seems that, even in the silt above gravel configuration, where erosion might be expected to be initiated when silt particles 'fall' into the gravel, the initiation of erosion is dependent on the transport of particles, not by detachment.

5.3.4 Influence of uniformity

When contact erosion occurs in a widely graded fine soil, segregation processes usually influence the process. Fines particles are eroded preferentially because they are more sensitive to hydraulic loading. Coarser particles are also likely to clog the pores of the coarse layer and generate a filter layer at the surface of the fine layer. This can occur for a fine layer above or below the coarse soil and tends to result in a decrease of the erosion rate with the development of this filter layer. In consequence, the geometrical and the hydraulic condition for erosion may be fulfilled for contact erosion but after a certain amount of eroded soil which can be excessive for dam safety or not, erosion stops.

6. INITIATION OF SUFFUSION

6.1 GENERAL DESCRIPTION OF THE PROCESSES

Suffusion occurs when water flows through widely graded or gap graded cohesionless soils such as alluvium of a large river, colluvium in the bed of rivers in mountainous areas, embankment cores constructed of glacial origin soils, and in filters which have very broad or gap gradings or excessive fines content.

The small particles of soil are transported by the seepage flow through the pores of the coarser particles. The coarser particles are not transported and the effective stresses are largely transferred through the matrix of the coarser particles.

For suffusion to occur, the following three criteria, geometric criterion, stress criterion and hydraulic criterion, have to be satisfied:

- Criterion 1: The size of the finer soil particles must be smaller than the size of the constrictions between the coarser particles, which form the basic skeleton of the soil.
- Criterion 2: The amount of finer soil particles must be less than enough to fill the voids of the basic skeleton formed by the coarser particles. If there are more than enough finer soil particles for void filling, the coarser particles will be "floating" in the matrix of fine soil particles, instead of forming the basic soil skeleton.
- Criterion 3: The velocity of flow through the soil matrix must impose a high enough stress to overcome the stresses imposed on the particles by the surrounding soil and to move the finer soil particles through the constrictions between the larger soil particles.

Figure 13.3 (see Internal Instability in Terminology) shows how the "finer soil" fraction is defined, in this case as the point of inflection of the particle size distribution plot. This may also be determined by experiments to determine the percentage of finer fraction which gives the maximum density, that is which just fills the voids in the coarser fraction. Soils with more fine soil are over-filled, those with less are under-filled. ICOLD (2008) says more about under- and over-filled soils.

Suffusion results in an increase in permeability, greater seepage velocities, and potentially higher hydraulic gradients, possibly accelerating the rate of suffusion. Stable situations may be reached with some of the finer fraction remaining in equilibrium with the seepage stresses. Suffusion may re-commence during cycling periods of water loads or during higher reservoir or river water level.

Suffusion occurring within an embankment core or the foundation of a dam may also lead to some settlement of the embankment. A filter constructed of internally unstable materials will have a potential for erosion of the finer particles in the filter, rendering the filter coarser and less effective in protecting the core materials from erosion.

It should be noted that segregation of broadly graded or gap graded non-plastic soils during placement in the dam may create layers which are internally unstable even though the average grading of the soil is internally stable.

6.2 METHODS OF IDENTIFYING SOILS WHICH ARE INTERNALLY UNSTABLE AND POTENTIALLY SUBJECT TO SUFFUSION (CRITERION 1: GEOMETRIC AND CRITERION 2: FILLED OR UNDERFILLED VOIDS)

6.2.1 General requirements

Figures 6.1 and 6.2 show particle size distributions of some soils which have been found to be internally unstable in laboratory tests. It will be noted that there are gap graded and broadly graded soils. The Kenney and co-authors' soils are sandy gravels, whereas the Wan and Fell soils are silty sandy gravels. Soils 14A and 15 had 11% and 21% kaolin in them so were slightly plastic.

As pointed out by Kenney and Lau (1985, 1986) for a soil to be internally unstable and subject to suffusion the percentage of finer fraction (finer than the point of inflection of the particle size plot) must be smaller than the available void space. They suggested this lay between 20% of the total soil for well graded soils and 30% for narrow graded soils. Wan and Fell (2004c, 2007) showed that this could theoretically be as high as 40% but in their sample it was between 22% and 33% for broadly graded soils and 29% and 38% for the gap graded soils.

For a greater proportion of finer soil the coarse particles are surrounded by the finer particles. Such soils are not subject to suffusion but may be subject to global backward erosion.

For practical purposes Fell et al (2008) have concluded that based on the data described available soils with Plasticity Index > 7 should be considered not subject to suffusion at the gradients usually experienced in dams and their foundations. If for some particular reason the gradient is higher than about 4 then soils with a Plasticity Index $\leq$ 12 should be considered for suffusion. This was considered to be a somewhat conservative approach.

It must be recognized that the types of soils which are potentially susceptible to suffusion are also potentially subject to segregation during placement in the dam or dike. Therefore when assessing whether the material in the dam or dike is potentially suffusive, a range of gradations should be considered which covers the potential effects of segregation (Section 6.5).

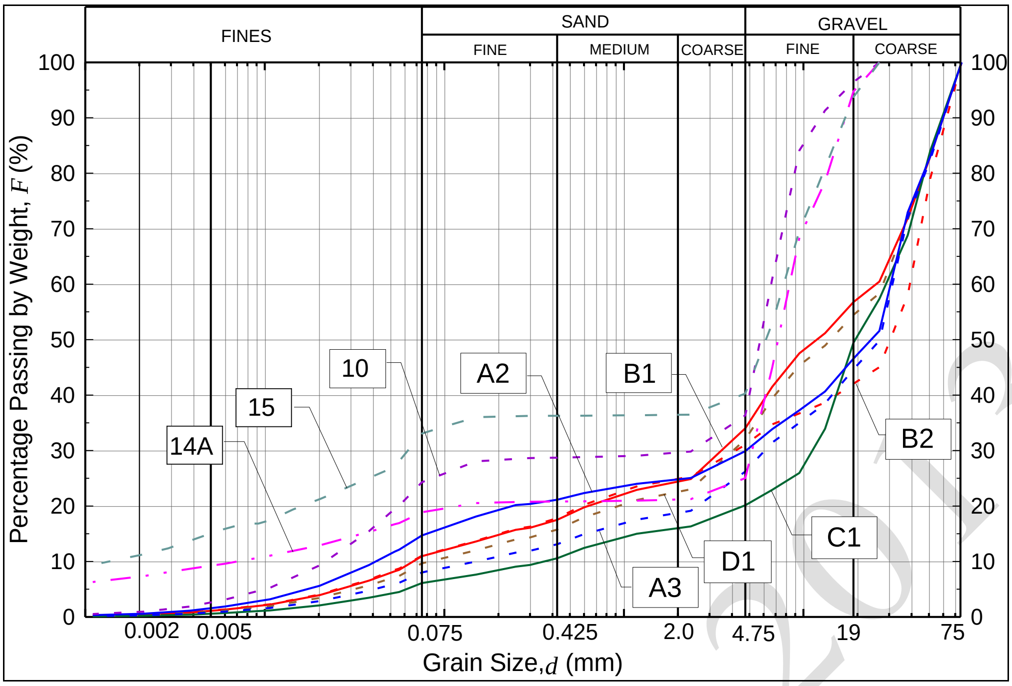


Figure 6.1 (a) Soil samples tested as being internally unstable (suffusive) by Wan and Fell (2004c, 2007)

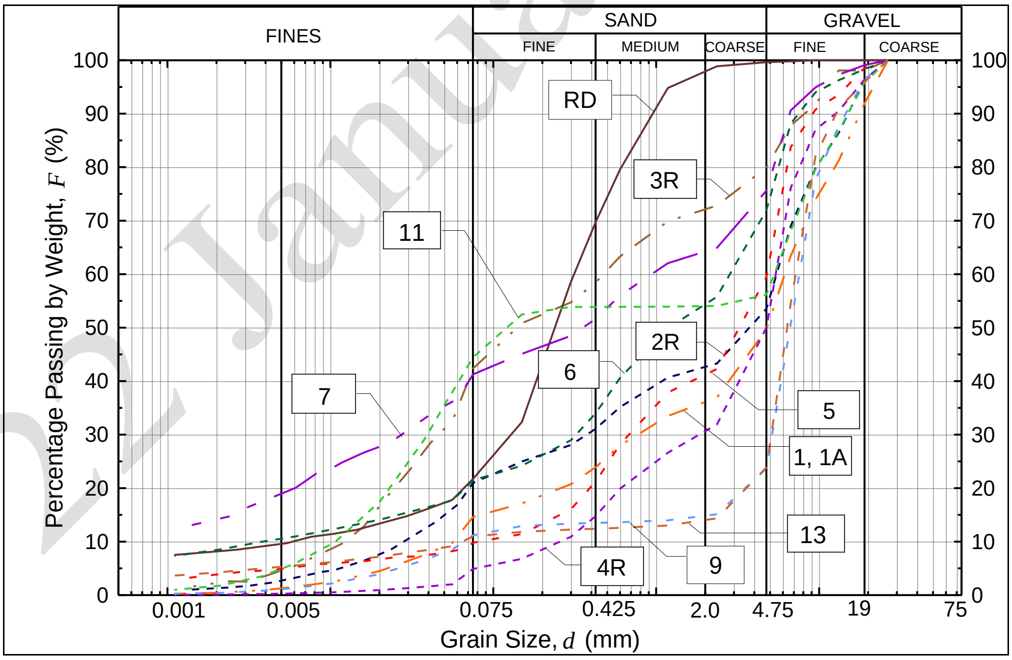


Figure 6.1 (b) Soil samples tested as being internally stable (not suffusive) by Wan and Fell (2004c, 2007)

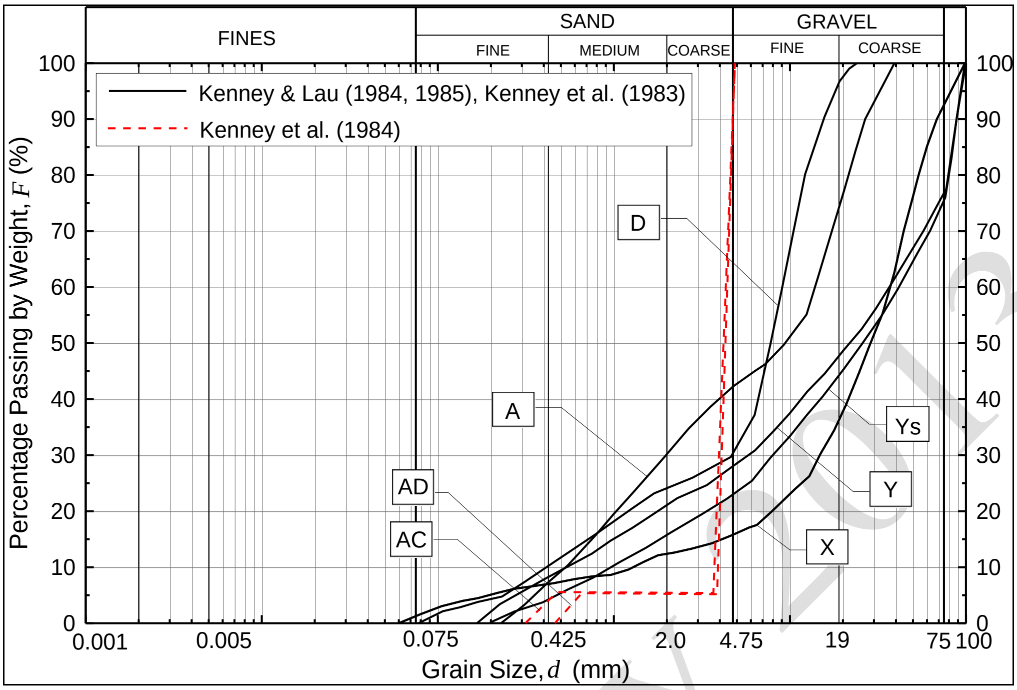


Figure 6.2 Soil samples tested as being internally unstable (suffusive) by Kenney & Lau (1984, 1986) and Kenney et al. (1985)

6.2.2 Some methods for assessing whether a soil is subject to suffusion.

There are a number of methods available to determine whether a soil is subject to suffusion. The following are some of the more widely used and / or later methods. It is suggested that the method or methods used are those which were developed for soils most closely matching the soil being assessed. For important decisions it may be necessary to carry out tests on the soils under consideration given the uncertainty in the methods currently available. Messerklinger and Straubhaar (2011) report on a series of such tests carried out on potential filter materials for Goescheneralp dam in Switzerland.

Kenney and Lau method

The Kenney and Lau (1985, 1986) method plots values of F, the mass fractions smaller than selected grain sizes D (taken from a conventional particle size distribution plot) for the soil against H, the mass fractions smaller than 4D. For particles to move there must be a deficiency in the mass of particles in the range D to 4D. Figures 6.3 and 6.4 show the method.

If the soil plots to the right of the boundary of Figure 6.4 (H<F), it is likely to be internally unstable provided that it satisfies the rules on limiting finer fraction; that is F, 0.2 for widely graded soils and F, 0.3 for narrow graded soils.

The method was developed for filter and transition zones with less than 5% fines passing 0.075mm.

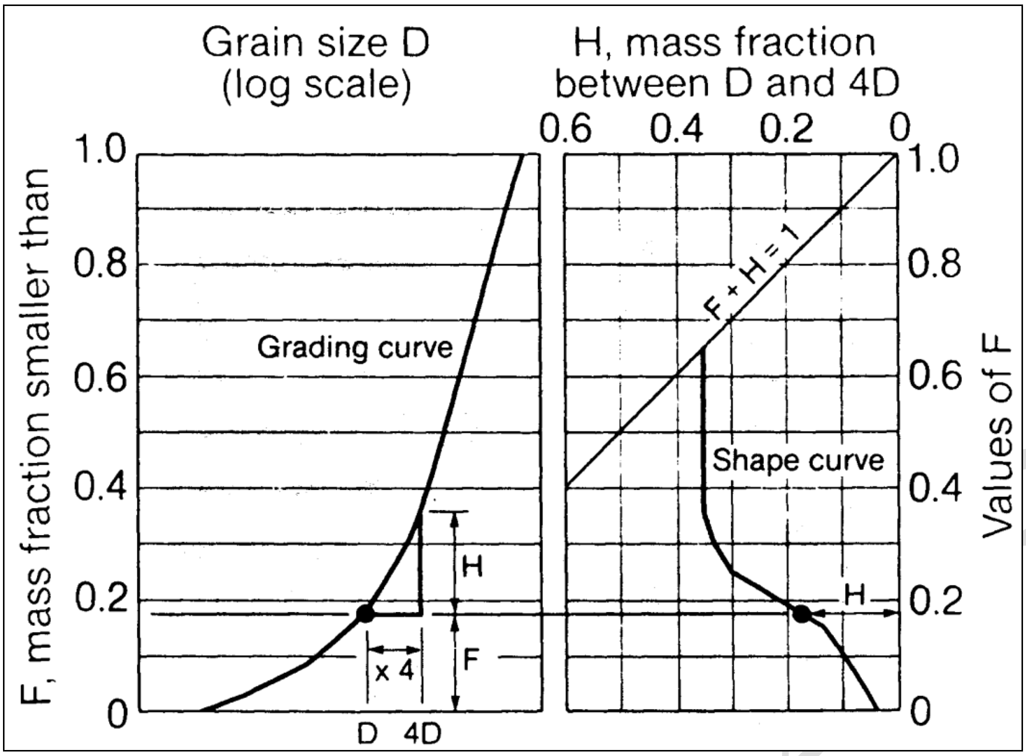


Figure 6.3 Method of characterizing the shape of a grading curve (Kenney and Lau, 1985).

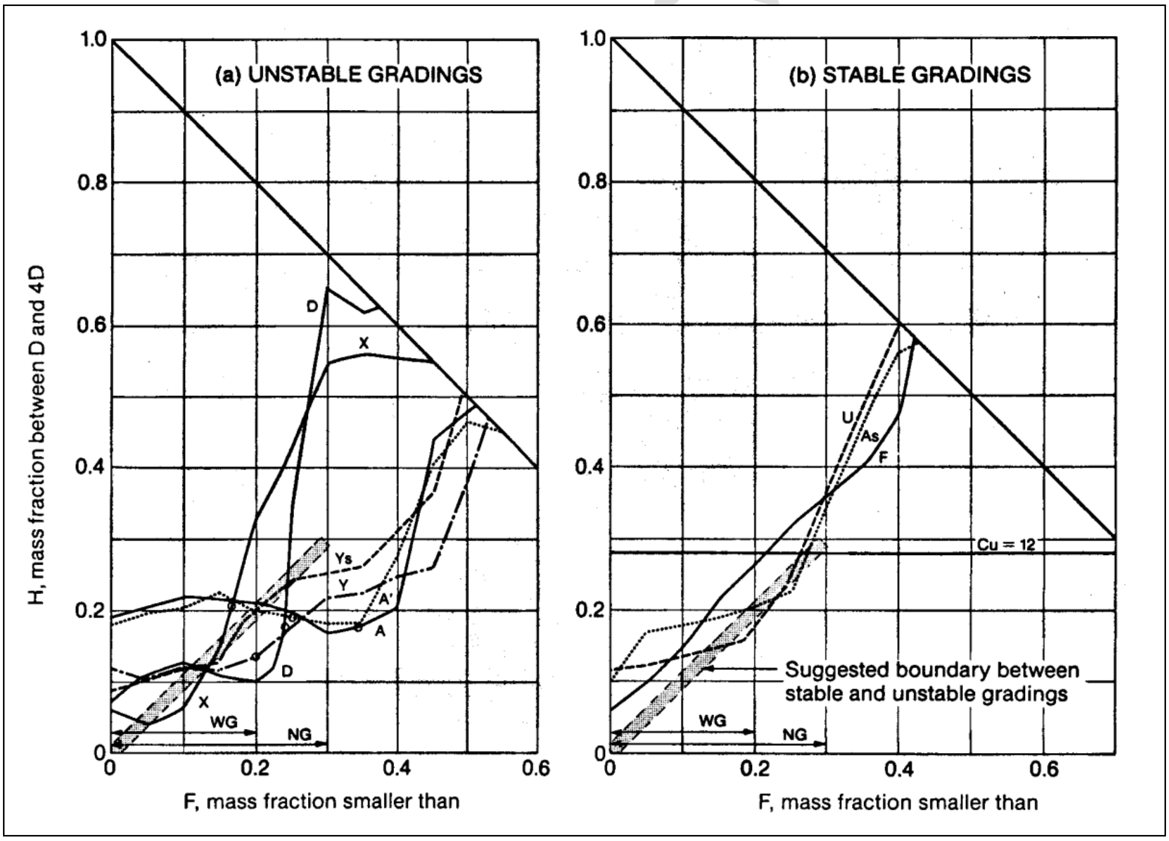


Figure 6.4 Method of assessing internal instability by Kenney and Lau (1985), with the revised criteria from Kenney and Lau (1986). Legend: WG, soils widely graded (Cu > 3) in the range F= 0.2-1.0; NG, soils narrowly graded (Cu < 3) in the range F= 0.3 to 1.0.

Wan and Fell adaptation of the Burenkova method

Wan and Fell (2004c, 2007) found that the Burenkova (1993) method gave reasonable assessments of whether a soil was internally unstable when used for the soils they had tested. However the method does not give a clear-cut boundary between internally stable and unstable soils in the data set. To model this logistic regression was used by Wan and Fell (2004c, 2007) to define contours of equal probability of internal instability. The contours and the logistic equations for silt-sand-gravel and clay-silt-sand-gravel mixtures are shown on Figure 6.5 and those for sand-gravel soils are shown on Figure 6.6.

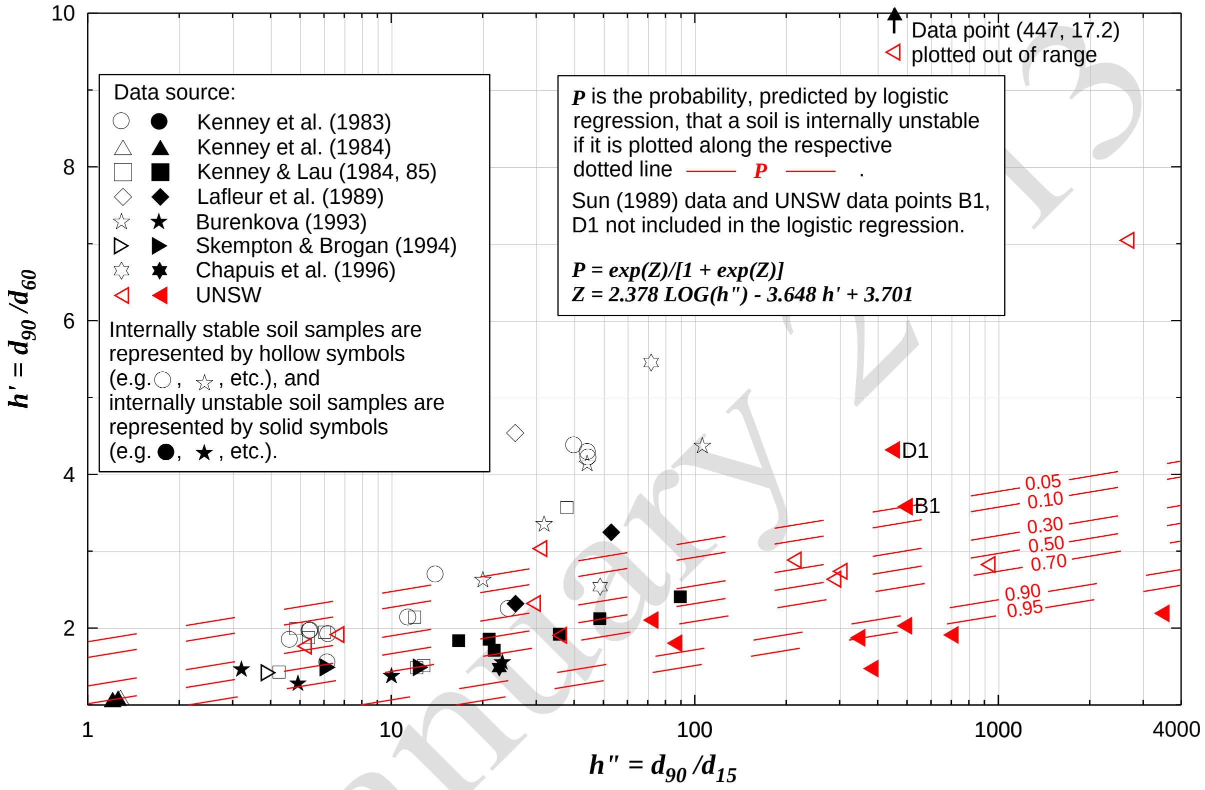


Figure 6.5 Contours of the probability of internal instability (potentially suffusive) for silt-sand-gravel and clay-silt-sand-gravel mixtures with a plasticity index less than 13% and less than 10% clay size fraction (% passing 0.002 mm) (Wan and Fell, 2004c, 2007).

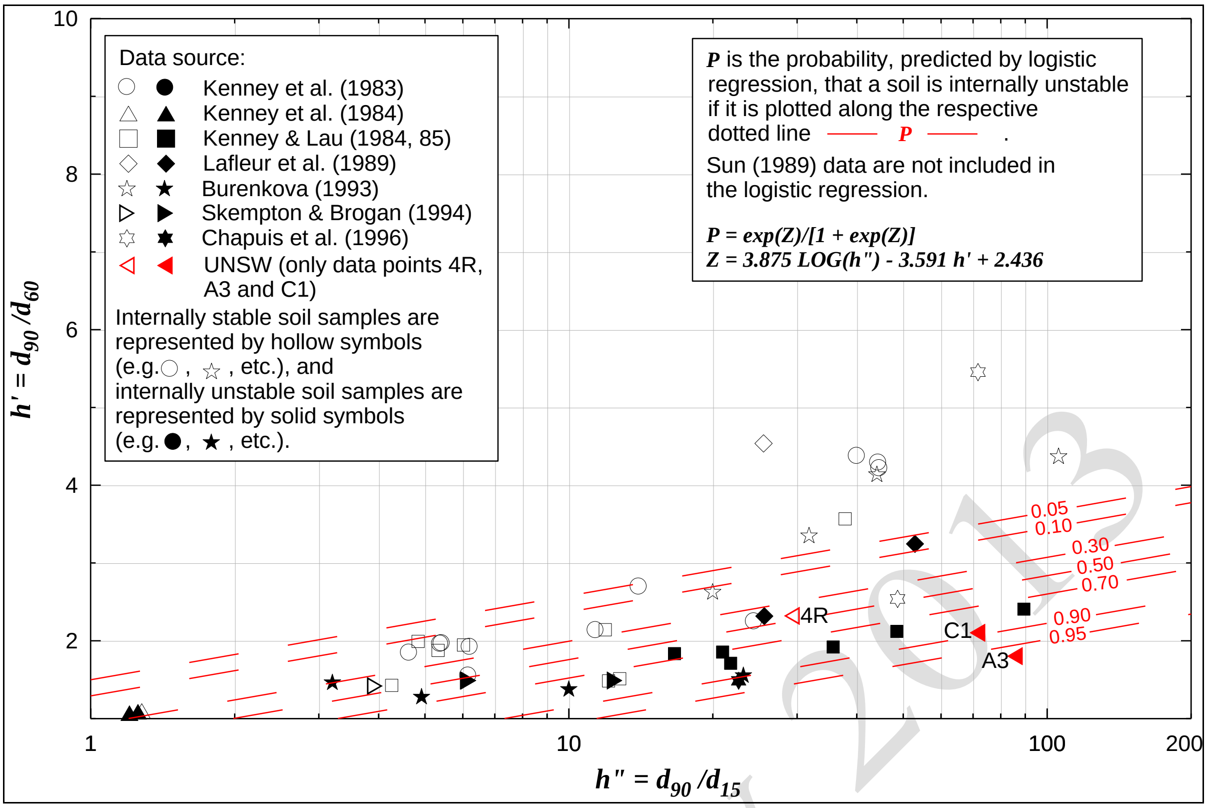


Figure 6.6 Contours of the probability of internal instability (potentially suffusive) for sand-gravel soils with less than 10% non-plastic fines passing 0.075mm (Wan and Fell, 2004c, 2007).

Some general comments

Wan and Fell (2004c, 2007) found that for the silty sandy gravel soils they tested, the Kenney and Lau (1985, 1986) method was too conservative, identifying the soils shown in Figure 6.4 as internally unstable when they tested as stable. In view of this it is apparent that the Kenney and Lau (1985, 1986) method is too conservative for silty sand gravel soils and it is suggested it only be applied to the sand gravel soils for which it was developed. Wan and Fell (2004c, 2007) also found that the Istomina (1957), Sherard (1979) and Sun (1989) methods were conservative for these silt sand gravel soils.

Rönnqvist (2007, 2008, 2009, 2010) applied Kenney and Lau (1985, 1986) to assess the internal stability of the filters and core grading of a number of existing moraine (glacial till) core dams, a number of which had exhibited signs of internal erosion, and others which had not. He found a correlation between dams with historic performance of internal erosion and internal instability of the filter and the core. However this was related to the fact that the filters of the dams showing internal erosion were coarser than the no-erosion filters. Most were in the "some erosion" or "excessive erosion" range as defined by Foster and Fell (2001).

6.3 ASSESSMENT OF THE LARGEST ERODIBLE PARTICLES IN SUFFUSION

(a) Geometric criteria

Wan (2006) and Wan and Fell (2004c) give details of a method to determine what fraction of the soil will be eroded. Fell et al (2008) have found that in practical terms it can be assumed that 50% of the finer fraction as defined by the point of inflection of broadly graded soils and the fine limit of the gap in gap-graded soils is eroded, and the particle size distribution re-plotted.

They suggest that if this becomes critical, laboratory tests should be carried out on the internally unstable soil.

Salehi Sadaghiani and Witt (2012) and Salehi Sadaghiani et al (2012) used a statistical approach to identify mobile grains and the supporting skeleton, and for considering the geometrical filtration response of widely gap graded soils. They used the method to re-assess the internal stability of 20 grain size distributions from the literature, agreeing with the findings of Wan and Fell (2008) in all but two cases.

(b) Hydraulic criteria.

Goltz et al (2009) are developing hydraulic criteria based on constriction opening size. Trials using these criteria have so far been carried out on limited test data.

6.4 ASSESSMENT OF SEEPAGE GRADIENT WHICH WILL CAUSE SUFFUSION (CRITERION 3)

Skempton and Brogan (1994) showed that erosion, which they described as 'segregation piping', will begin in internally unstable cohesionless soils at seepage gradients lower than the Terzaghi critical gradient (which is approximately one, depending on soil density or zero effective stress, when the pore pressure in a permeable soil below the interface equals the total pressure applied by an impermeable soil above and 'heave' or 'uplift' occurs).

Wan and Fell (2004c, 2007) found that internally unstable soils eroded with upward gradients of 0.8 or less, with several less than 0.3. There is a general trend that soils with a higher porosity begin to erode at lower hydraulic gradients. Loose, higher porosity soils began to erode at gradients less than 0.3. Soils with plastic fines required higher gradients to begin to erode. Gap-graded soils tended to begin to erode at lower gradients than non gap-graded soils with the same fines content.

Monnet (1998) correlated the hydraulic gradient initiating failure measured by Skempton and Brogan (1994) to the measured permeability, showing that the hydraulic criterion governs the value of hydraulic gradient moving finer particles.

Li and Fannin (2008) proposed a hydro-mechanical criterion deduced from tests on unstable materials. Starting from the fact that seepage failure occurs in stable soils under the Terzaghi critical hydraulic gradient, i_{CT}, Fannin and Li deduced the critical gradient, i_C, for any unstable soil, from the proportion of the effective stress α sustained by the fines:

$$i_C = \alpha . i_{CT} = \alpha \frac{\gamma'}{\gamma_W}$$

where γ_W and γ' are the water and buoyant soil specific weight and α is the reduction factor of the vertical effective stress σ'_V carried by the finer particles in the internally unstable soil as was first proposed by Skempton & Brogan (1994). The finer grains carry a reduced portion of the effective stress, σ'_f:

$$\sigma'_f = \alpha.\sigma'_V$$

The reduction stress factor α is related to the mobility of the finer particles and is dependent on the geometric criterion d'_{85}/O_{50}, where d'_{85} is the d_{85} of the finer fraction of soil and O_{50} is the effective constriction size of the coarse fraction, as follows:

$$\alpha = 3.85(d'_{85}/O_{50}) - 0.616$$

This method needs more test data on internally unstable soils which are subject to suffusion so the variables can be calibrated.

Moffat and Fannin (2011) have further developed these concepts but the test data is for soils which are not internally unstable as defined in this Bulletin.

With a specific centrifuge bench, Marot et al (2012) analyzed the influence of hydraulic gradient on initiation and development of suffusion. As for the backward erosion process, the characterization of suffusion depends on the length of the seepage path by using the hydraulic gradient concept. The difference in the results is in the magnitude of critical hydraulic gradient and magnitude of erosion rate. For an increase of tested specimen length by a factor 2, the value of critical hydraulic gradient can be multiplied by a factor of 0.6 and rate of erosion can double. This influence of seepage path length may be due to the probability for a detached particle to be filtrated or not. This filtration can induce clogging and thus a modification of the seepage. An energy analysis of interstitial fluid has been developed. This analysis leads to a linear correlation between expended power by fluid flow and rate of erosion. By integrating over the time, the eroded mass is linearly correlated to the energy dissipation. The suffusion characterization does not depend on specimen length by using the energy analysis.

Chang et al (2012) found that increases in deviatoric stress lowered the gradient at which suffusion initiated and skeleton deformation occurred when compared to one dimensional seepage tests.

As there are no generalized methods for accurately predicting the critical seepage gradient laboratory tests should be carried out which carefully simulate the field conditions. When selecting the particle size distribution of the soils to be tested allowance should be made for the potential effects of segregation of the materials during placement in the dam. It may be necessary to construct specific test equipment because of the large particles involved.

6.5 ALLOWING FOR THE EFFECTS OF SEGREGATION WHEN ASSESSING SUFFUSION

Soils which are subject to suffusion are also subject to segregation in stockpiles prior to placement and during placement on the embankment. Segregation is also a problem for broadly graded soils such as those in Figure 13.3 and in filters.

Characteristics which make it more likely that silt-sand-gravel soils will segregate on placement include:

- A broad grading, particularly with maximum particle size >75 mm.
- A low percentage of sand and fine gravel sizes (<40% finer than 4.75 mm).

- Poor construction practices e.g. end dumping from trucks, high lift heights and poor control of stockpiling operations.

- Soils placed dry, rather than moistened prior to placement.

These criteria are based on Sherard and Dunnigan (1989) and Kenney and Westland (1993). Milligan (2003) reports on the effect of wetting in preventing segregation, based on Sutherland (2002) who investigated the mechanisms of segregation.

Some information on segregation may be available from particle size distributions taken during construction, from photographs of placement during construction, or from samples taken from test pits in the embankment.

Segregation can become critical because it may result in layers of coarser soil which are internally unstable whereas the average gradation is not. A segregated filter can be coarser than what is reported in for example the dam records, and streaks or pockets of gravel/cobble-sized materials can exist.

7. CONTINUATION OR INTERRUPTION OF EROSION BY FILTER ACTION

7.1 INTRODUCTION

After the internal erosion process has been initiated, by one of the four mechanisms described in the previous chapters, it must continue if it is to become a threat to the dam.

Erosion may be interrupted or stopped by filter action in the soils in the fill and foundation materials downstream of the point where it initiated. Filtering occurs when the sizes of pore spaces in the soil through which seeping water containing eroded particles is passing are such that the eroded particles are trapped, thereby stopping the erosion from continuing. Filters are normally coarser and more permeable than the filtered soil (called the 'base soil') and the seeping water, no longer containing eroded particles, drains away downstream.

The zoning of dams is important in preventing or allowing erosion to continue and various scenarios are discussed below. In many existing dams filter zones are included downstream of the core zone, as filter blankets on foundations and in other vulnerable locations. In others, there may be no formal filter zones, but the zoning may include materials such as transition zones that may act as filters to vulnerable zones. If erosion has initiated in homogeneous dams, the seepage containing eroded particles will not pass through any filter zone, and erosion will therefore continue (although it may not progress, as discussed in the following chapter).

Formal filters in old dams may not have been designed to modern methods, and other potentially filtering zones will not have been designed as filters. Their ability to act as filters can be assessed using modern filter design methods, but modern methods (described below) lead to very safe filters, as is appropriate given the vital role of filters in protecting dams against internal erosion. However, soils which are coarser or more permeable than filters designed to modern criteria often provide a substantial filtering capability. The means of assessing such filtering potential is described below.

7.2 CONTINUATION SCENARIOS

7.2.1 Internal erosion in the embankment

There are a number of scenarios which may be applicable depending on the zoning of the dam and the failure mode under consideration. Example sketches are shown in Figure 7.1.

- Scenario 1: Homogeneous zoning with no fully intercepting filter.
- Scenario 2: Downstream shoulder of fine grained cohesive material which is capable of holding a crack/pipe. Soils which are capable of holding a crack or pipe are:
  - well compacted shoulder (shell), containing > 5% plastic fines; or
  - poorly compacted shoulder (shell), containing >15% plastic fines
  - well compacted shoulder, containing > 30% non plastic fines

  - poorly compacted shoulder, >30% non plastic

- Scenario 3: Filter/transition zone is present downstream of the core or a downstream shoulder zone which is not capable of holding a crack/pipe. This includes earthfill dams with a chimney filter.

- Scenario 4: Erosion into a crack or open joint (e.g. open joint or crack in a conduit or adjoining concrete structure).

- Scenario 5: Erosion into a toe drain and under-drains.

The manner in which the likelihood of continuation of erosion for these scenarios is considered differently:

Scenario 1: There is no potential for filtering. Continuing erosion is certain.

Scenario 2: The issue for this scenario is whether the crack/high permeability feature that is present through the core is continuous through the downstream shoulder, or if not, whether it can find an exit. This depends on the following factors:

- The mechanism causing the concentrated leak, in particular whether it also causes cracking in the shoulder.

- The material characteristics and width of the downstream shoulder zone.

Scenario 3: Assess the likelihood of the filter or transition zone being effective using filter design criteria as detailed in Section 7.3. For existing dams where many filters do not satisfy modern no-erosion design criteria it is suggested that a four way split for filtering behavior is considered (see Section 7.6):

- *Seals with No Erosion* – the filtering material stops erosion with no or very little erosion of the material it is protecting. The increase in leakage flows is so small that it is unlikely to be detectable.

- *Seals with Some Erosion* – the filtering material initially allow erosion from the soil it is protecting, but it eventually seals up and stops erosion. Leakage flows due to piping can be up to 100 l/s, but are self healing.

- *Seals with Excessive Erosion* – the filtering material allows erosion from the material it is protecting, and in the process permits large increases in leakage flow (up to 1000 l/s), but the flows are self healing. The extent of erosion is sufficient to cause sinkholes on the crest and erosion tunnels through the core.

- *Continuing Erosion* – the filtering material is too coarse to stop erosion of the material it is protecting and continuing erosion is permitted. Unlimited erosion and leakage flows are likely.

This approach often shows that filters which do not satisfy modern filter design criteria will after some erosion of the soil being filtered eventually seal with medium size particles eroded from the protected soil. Provided the embankment can accommodate the leakage that occurs up to the time the filters seal, the dam will not fail. In some cases erosion will initiate again adjacent to the original area due to changed leakage pathways, causing a second incident.

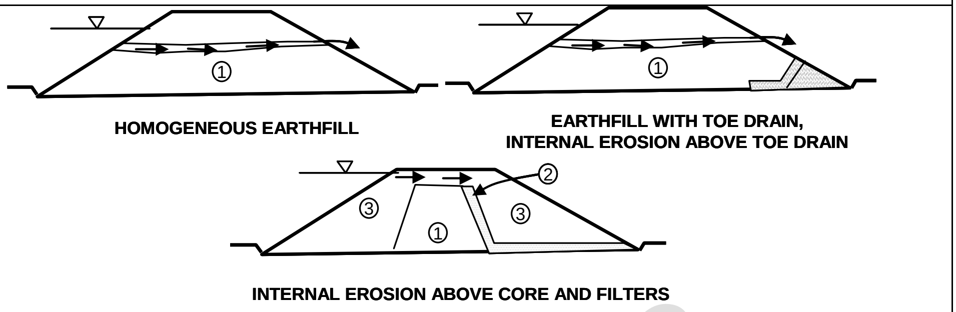
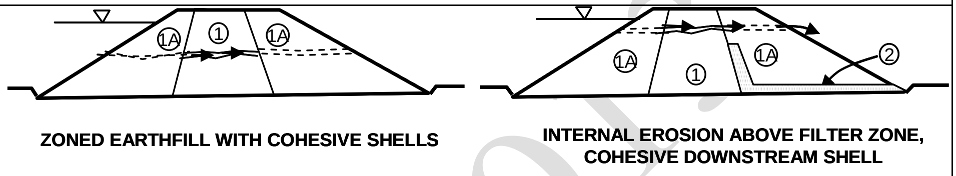
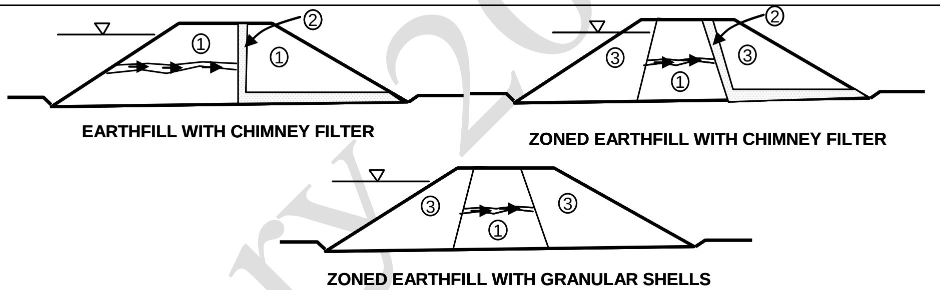
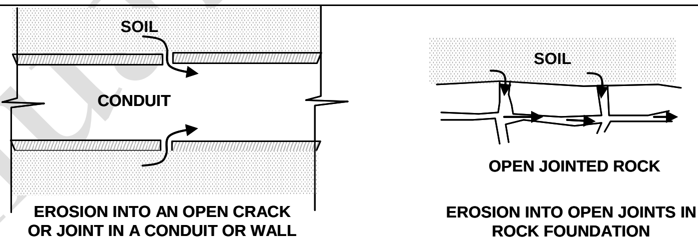
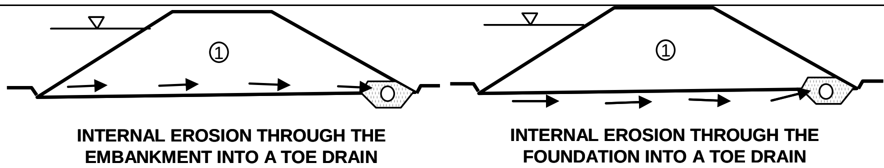
Scenarios	Examples
Scenario 1: Homogeneous zoning with no fully intercepting filter	
Scenario 2: Downstream shoulder of fine grained cohesive material which is capable of holding a crack/pipe.	
Scenario 3: Filter/transition zone is present downstream of the core or a downstream shoulder zone which is not capable of holding a crack/pipe.	
Scenario 4: Piping into an open defect, joint or crack.	
Scenario 5: Erosion into a toe drain	

Figure 7.1 Scenarios for continuation of internal erosion in an embankment (Fell et al, 2008)

***Scenario 4*:** For erosion to continue through an open defect, the defect needs to be sufficiently open to allow the soil particles surrounding the defect to pass through it. The suggested procedure for examining this is given in Section 7.7.

***Scenario 5*:** This scenario is applicable if the failure path under consideration involves a seepage path that exits into a toe drain which could lead to continuing erosion of the embankment or foundation materials. The assessment considers the design and construction details of the toe drain, whether filter criteria are met, and the observed condition of the toe drain (from video or external inspections).

7.2.2 Internal erosion in the foundation

The likelihood of continuation of erosion should be estimated by assessing the likelihood that the exit will be a filtered or unfiltered exit.

a) Given the exit is unfiltered the likelihood of continuing erosion will be 1.0.

b) Given the exit is filtered, estimate the likelihood for Continuing Erosion using the method described in Section 7.3. The probability of continuation will be the product of the likelihood of a filtered exit and the likelihood assessed considering the filters.

It needs to be recognized that low permeability strata beneath horizontal drains may prevent them working effectively, Figure 7.2 shows an alluvial foundation where the lower permeability strata (A and E) will prevent the seepage in the most permeable sand and gravel strata (B, and D) from flowing into a filtered exit in the horizontal drain.

7.2.3 Internal erosion of the embankment at or into the foundation

a) *Erosion into open joints in the foundation*. The likelihood of continuation depends on the joint openings being sufficiently open and continuous to allow erosion of the core materials.

b) *Erosion into coarse grained soil foundation*. The likelihood of continuation depends on the coarse grained soils being sufficiently coarse and continuous to allow erosion of the core materials

These assessments are often difficult to make because the amount of information on the foundations is often limited. If there are soils overlying the exits to the open joints or the coarse grained soils these may act as filters to the erosion.

The assessment of filtering should be carried out as detailed in Sections 7.3 and 7.7.

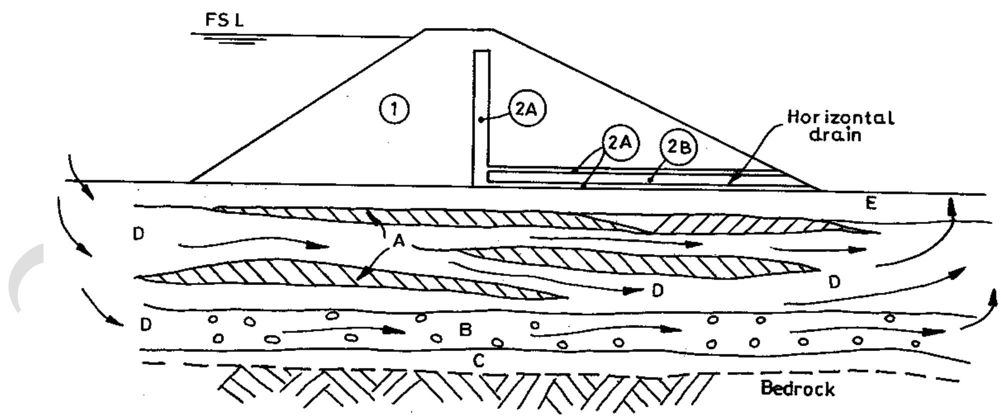


Figure 7.2 Schematic example of an embankment founded on a stratified alluvial soil where much of the seepage flow will be to an unfiltered exit (Fell et al, 2005)

7.3 ASSESSMENT OF THE LIKELIHOOD OF CONTINUATION WHERE A FILTER/TRANSITION ZONE IS PRESENT DOWNSTREAM OF THE CORE OR THERE IS A DOWNSTREAM SHOULDER ZONE WHICH IS NOT CAPABLE OF HOLDING A CRACK

The following steps should be followed when assessing the likelihood of continuation:

- Gather the available information on particle sizes of the core or foundation and the filter / transition materials. This may include data from construction control testing, testing on samples taken from the dam and its foundation, data from investigations of borrow areas used for core and filter materials.

- Plot the particle size distributions for the core material and the filters or transitions which are protecting the core. If the maximum particle size of the core material is >4.75 mm, then re-grade the core grading such that the maximum size is 4.75 mm. If the base soil is gap graded, then re-grade the base soil grading on the particle size that is missing (i.e. at the point of inflection of the grading curve).

- Check if the filter/transition zone will hold an open crack. Modern filter design rules require less than 5% non-plastic fines content and these will not hold a crack. Fell et al (2008) suggested Table 7.2 to assist in assessing the likelihood of holding a crack based on published information and field performance. Table 7.1 was developed based on laboratory tests carried out by Park, (2005) and case data from Foster (1999), Foster and Fell (1999b).

Table 7.1 Likelihood for Filters with Excessive Fines Holding a Crack (Fell et al 2008)

Fines Plasticity	Fines Content % Passing 0.075 mm	Probability of holding a crack	
		Compacted	Not compacted
Non plastic (and no cementing present)	5%	0.001	0.0002
	7%	0.005	0.001
	12%	0.05	0.01
	15%	0.1	0.02
	>30%	0.5	0.1
Plastic (or fines susceptible to cementing)	5%	0.05	0.02
	7%	0.1	0.05
	12%	0.5	0.3
	≥ 15%	0.9	0.7

Note: Fines susceptible to cementing for filters having a matrix predominately of sand sized particles (e.g. filters derived from crushed limestone).

- Assess also if the materials used in the filters may be subject to cementing; e.g. they are constructed of limestone or dolomite. Fell et al (2005), Schaefer et al (2011) and USBR (2011) discuss this and other potential mechanisms for cementation and crack holding.

- Assess if the filter / transition materials are likely to be subject to segregation taking account of the specified and/or measured particle size distribution, the method of placement, and the width of the filter/transition zone. Adjust the particle size distributions to allow for segregation if it is likely to occur. The methods of USDA SCS (1994), Kenney and Westland (1993) and Sherard and Dunnigan (1989) may be used to assist in this assessment. Milligan (2003) makes important observations on the occurrence and prevention of segregation in filters and fills.

- Assess if the filter / transition materials are internally unstable, and if so adjust the particle size distributions to account for this using the procedure outlined in Section 6.3.

- Use modern filter design criteria to assess whether the filter/transition will prevent continuation. There are a number of filter design criteria which may be used. These are outlined in Section 7.5. Which method is used will be a matter of personal choice. For filters or transitions which are coarser than required by the filter design criteria the Foster and Fell (1999a, 2001) method as described below may be used because it allows assessment of filters which are too coarse to satisfy modern no-erosion design criteria. Fry (2006, 2007) proposed a hydraulic approach of the concentrated leak development through the core-shoulders system.

- Check for a "blow-out" condition. In cases where there is limited depth of cover over the filter/transition zone, assess the potential for blow out by comparing the seepage head at the downstream face of the core to the weight of soil cover.

- Check whether the filter or transition and the zones downstream will be sufficiently permeable to perform their required drainage function. This is related to the particle size distribution of the filter and fines content and plasticity, but more importantly to whether there are two filters or another downstream zone which will act as a drain.

7.4 FILTER FUNCTIONS

Five filter functions govern the capability of providing control for internal erosion.

1. ***Retention.*** The filter limits the transport (erosion) of particles of the base soil it is protecting.
2. ***Self Filtration or stability.*** The filter is internally stable and not subject to finer particles eroding from within the filter, and will not hold a crack due to cementation or the presence of plastic fines.
3. ***No cohesion*** so the filter will not hold a crack.
4. ***Drainage.*** The filter is sufficiently permeable to transport the water flowing through the base soil into drainage layers or to the toe of the dam.
5. ***Strength.*** The filter transfers the stresses within the dam without being crushed and therefore becoming finer.

Filter Retention

The basic concept of filter retention design is to design the filter so that the voids in the filter are sufficiently small to prevent erosion of the base soil.

A further basic concept, inherent in filter design and explained by Sherard et al (1984), is that the base soil will provide a degree of 'self-filtering'. Hence in Figure 7.3 (a), in a well graded base soil, the coarser particles in the base soil are prevented from eroding into the filter and they in turn prevent the medium sized particles in the base soil from eroding and the medium sized particles in the base soil prevent the fine particles in the base soil from eroding. If the base soil is gap-graded or graded concave upwards, there is a deficiency of medium sized particles, as shown in Figure 7.3 (b). Then self filtering does not occur and the fine particles in the base soil will erode through the coarse particles. In these situations for the filter to be successful in controlling erosion, it must be able to control the erosion of these finer particles.

For dispersive soils the clay particles do not flocculate to form aggregated particles of clay so finer filters are required than for non-dispersive soils of the same particle size distribution.

For filters and transitions which are subject to suffusion the finer fraction may erode from the filter and result in a coarser filter less able to prevent continuation of erosion.

Four methods are used to achieve the retention function:

- **Geometric Criterion**

1. Methods based on particle size distribution: These were first developed by Terzaghi and later refined, e.g. methods based on Sherard & Dunnigan, (1989). For design purposes the D_{15F} is most commonly used to define this void size and D_{85B} the representative size of the base soil. This is determined on the fraction of the soil passing a 4.75mm sieve.

2. Methods based on the controlling constriction size or equivalent opening size: e.g. Brauns and Witt (1987); Schuler and Brauns (1993, 1997); Lafleur et al (1993); Giroud (2003); Indraratna and Raut (2006); Indraratna et al (2007) and Raut and Indraratna (2008). Giroud (2003) and Indraratna's research team's methods allow for the effects of relative density on the filter design. Generally, these methods relate the controlling constriction size to a % of fine passing sizes.

- **Hydraulic Criterion**

3. Methods that include an element of hydrodynamics such as the Russian standards (see 4.2.4 and Pavchich, 1961, ICOLD, 1994).

4. Methods based on permeability which is regarded as being correlated to the constriction size: e.g. Vaughan & Soares (1982), Vaughan and Bridle (2004), Delgado et al, (2006).

Which is used is a matter of personal preference and experience. The methods above are mainly used to design no-erosion filters in critical filter situations.

In some applications such as in long dikes it may be very costly to use such filters and filters which allow some erosion (or "open filters") may be used. These can be designed using the methods of Bakker (1987), Den-Adel et al. (1994) and Verheij et al (2012). However great care should be taken in applying this approach, particularly to situations where, if the filter fails and erosion occurs, it is not practical to repair the damage.

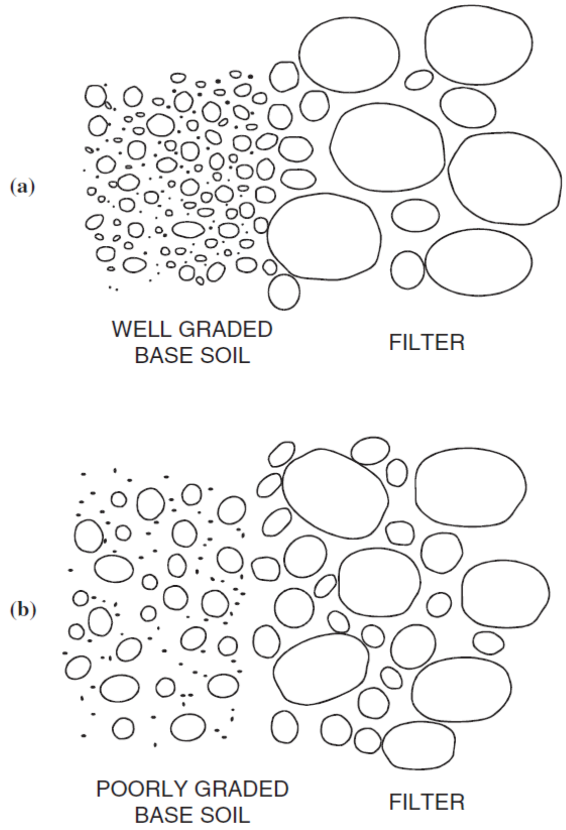


Figure 7.3 Filtering and self filtering concepts, (a) well graded base soil, (b) gap graded, or concave graded soil, deficient in medium sized particles (Fell et. al. 2005).

No cohesion (filter will not hold a crack)

This function is to confirm that the filter will collapse at locations where an erosion pipe or a crack has initiated. The ability not to hold a crack is commonly achieved by limiting the fines content (% passing 0.075mm) and requiring that the fines are non-plastic.

There is some evidence that even with these precautions very dense compacted filters may hold a crack (Redlinger et al, 2011).

It should be recognized that cementing of filters may occur if the filter is composed of carbonate rock particles such as limestone or dolomite particularly for the sand fraction (Fell et al, 2005), and that even small amounts of silt in broadly graded silty sandy gravel may result in the filter being able to crack. This is discussed further in Section 7.3.1 and in Table 7.2.

Self filtration or stability

Self filtration and stability are achieved by checking against internal instability as described in Section 6.2.2.

Drainage

This is achieved by limiting the fines content (% passing 0.075mm), and in some design manuals relying on relationships between D_{15F} / D_{15B},to ensure the filter is sufficiently more permeable than the base soil to perform the drainage function. This is also related to the zoning of the dam, and in particular whether drainage filter zones or rockfill zones are provided.

Strength

This is achieved by specifying durability and crushing strengths of the aggregates.

Some more details on filters are provided in Section 7.5. More information is also available in the ICOLD Bulletin on Embankment Dam Granular Filters and Drains, (ICOLD, 1994).

7.5 BRIEF DESCRIPTION OF SOME OF THE AVAILABLE FILTER DESIGN CRITERIA FOR CRITICAL FILTERS

Particle Size Based Criteria-Sherard and Dunnigan (1989)

Sherard and Dunnigan (1985, 1989) recommended the following steps:

For all soils with a gravel component (except Group 3 below), the filters should be designed on the grading of that part of the soil finer than 4.75 mm. This is an important step and designed to overcome problems with very broadly graded soils which will not self filter. Base soil gradations are determined by sieve tests and hydrometer tests using deflocculant such as sodium hexametaphosphate.

Impervious Soil Group 1 (fine silts and clays): For fine silts and clays that have more than 85% by weight (of the portion finer than the 4.75 mm sieve) finer than the 0.075 mm sieve, the allowable filter for design should have $D_{15F} < 9\ D_{85B}$.

Impervious Soil Group 2 (sandy silts and clays and silty and clayey sands): For sandy (and gravelly) impervious soils with 40% to 85% by weight (of the portion finer than the 4.75 mm sieve) finer than the 0.075 mm sieve, the allowable filter for design should have $D_{15F} \leq$ 0.7 mm.

Impervious Soil Group 3 (sands and sandy gravels with small content of fines): For silty and clayey sands and gravels with 15% or less by weight (of the portion finer than the 4.75 mm sieve) finer than the 0.075 mm sieve the allowable filter for design should have $D_{15F} < 4\ D_{85B}$ where D_{85B} can be the 85% finer size of the entire material including gravels.

Impervious Soil Group 4: For coarse impervious soils intermediate between Groups 2 and 3 above, with 15% to 40% passing the 0.075 mm sieve, the allowable filter for design is intermediate, inversely related linearly with the fines content and can be computed by straight line interpolation. As an example (Figure 7.4) for an impervious sandy soil with 30% of silty or clayey fines and D_{85B} = 2 mm, the allowable filter for design is in between the value of D_{15F} = 0.7 mm (for soils of Group 2) and D_{15F} = 4 x (2) = 8 mm (for soils of Group 3), and is calculated as follows:

$$D_{15F} = \frac{40\text{-}30}{40\text{-}15}(8\text{-}0.7) + 0.7 = 3.6\text{mm}$$

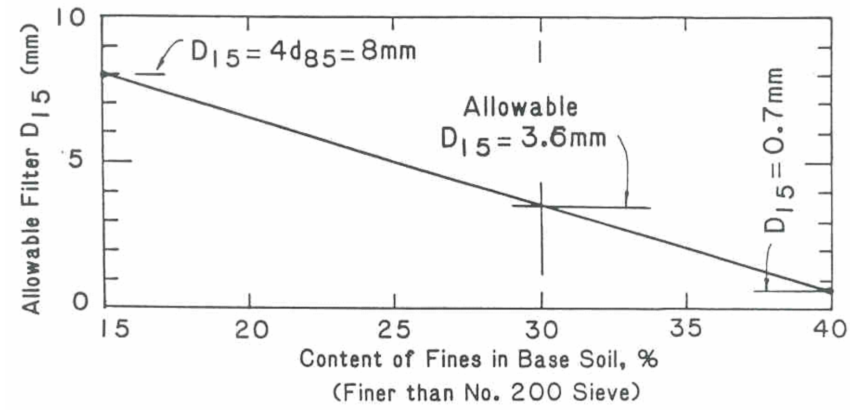


Figure 7.4 Determination of allowable D_{15F} for filters for impervious Soil Group 4 (having between 15% and 40% finer than 0.075 mm sieve). (Sherard and Dunnigan, 1985)

These criteria have been used to develop Design Guidelines such as USDA-SCS (1994) and USBR (2004). They supersede the methods of Terzaghi and Peck (1948) which are not conservative for coarse base soils and too conservative for fine soils.

Foster and Fell (1999a, 2001) tested more soils and added these to the Sherard and Dunnigan test data and found that the Sherard and Dunnigan criteria were not sufficiently conservative for dispersive soils. For soils testing as D1 or D2 in the Pinhole test in distilled water Foster and Fell (2001) suggested using $D_{15F} < 6\ D_{85B}$ for Group 1 soils, and $D_{15F} \leq$ 0.5 mm for Group 2 soils, and to move the boundary between Group 1 and 2 soils from 40% to 35% fines passing 0.075mm.

Methods based on constriction size or opening size

There are a number of these methods, which generally relate the controlling constriction size to a % of fine passing sizes. Some are described in ICOLD (1994): e.g. Brauns and Witt, (1987). Others include Schuler and Brauns, (1993, 1997); Lafleur et al (1993); Giroud (2003, Terzaghi lecture, 2010); Indraratna and Raut (2006); Indraratna et al (2007) and Raut and Indraratna (2008). The methods of Giroud and Indraratna's research team allow for the effects of relative density on the filter design.

Methods based on the permeability of the filter

There are a number of methods relating the permeability of a filter to its filtering capability. This is potentially useful in assessing the vulnerability of a dam to internal erosion because if the permeability of the zone adjacent to a core were known, its ability to filter and protect the core against erosion could be assessed. The difficulty with this approach is that permeability is very variable, both locally and widely even in one fill zone, and measuring it in-situ is also subject to some difficulty. However, as the discussion above on applying particle size filter design criteria shows, assessing filtering capability requires both analysis and judgment, some permeability based methods are included.

Vaughan and Soares (1982), Vaughan and Bridle (2004) describe the "perfect filter" method which was developed so that the filter itself would be able to prevent erosion of the finest particles in the base soil. It was pointed out that the finest 'particles' present in non-dispersive clay soils are often flocs, accumulations of finer particles. Flocs are usually dispersed (de-flocculated) when samples are prepared for the sedimentation stage of a particle size analysis, but the floc size for the base soil is determined by hydrometer sedimentation tests with no dispersant added (the 'double-dispersion' test). The water used for the test should be of the same chemistry as the water to be seeping through the dam since the floc size is related to the degree of dispersion which is controlled by the water chemistry. A more conservative approach is to prepare samples in distilled water also (the ‘triple dispersion’ test). Samples can be tested as a dilute suspension in filter experiments.

Vaughan & Soares (1982) carried out a series of tests to determine the sizes of floc trapped by filters of varying permeability. These are summarized on Figure 7.5. The open points show effective filters and the solid points show ineffective filters. For example, a floc of 100 microns (0.1 mm) effective diameter would be retained by a filter with a permeability of about 10^{-2} m/s or less.

The Vaughan & Soares (1982) equation linking permeability of filter to the particle size that the filter will trap is as follows:

$k = 6.7 * 10^{-6} * \delta_R^{\ 1.52}$

where: δ_R = size of smallest particle retained in microns (10^{-6} mm)

k = permeability of filter (m/s)

This can be transposed as follows:

$\delta_R = 2.54 * 10^3 * (k)^{0.658}$

The “perfect filter” method is very conservative because it does not rely on self filtering (Bridle 2007, 2008 and Bridle et al 2007). It will often give finer filters than particle size based methods, but conversely if the permeability of a potential filter zone downstream of the core, say, satisfies the Vaughan & Soares criterion, erosion will not continue.

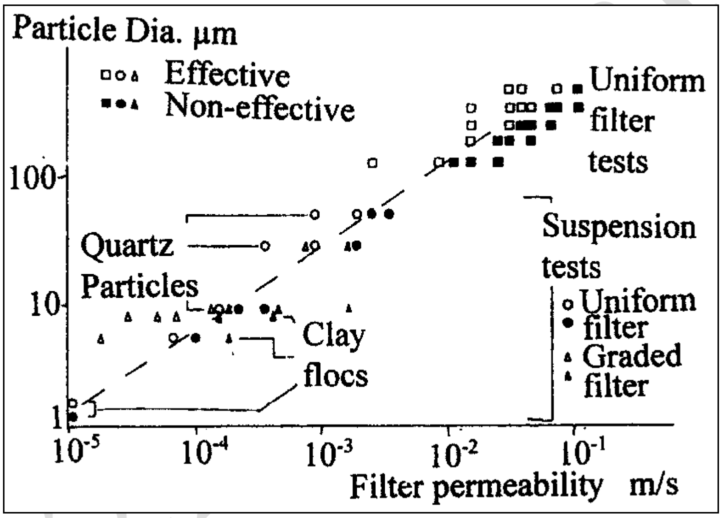


Figure 7.5 Vaughan & Soares (1982) relationship between filter permeability and size of floc trapped

Delgado et al (2006) carried out extensive filter tests to determine no-erosion filter (NEF) criteria based on the percentage of the base soil passing the 0.075mm sieve, and the permeability of the filter.

They combined these data with that of Sherard et al (1984), Foster and Fell (1999a) and others to produce Figure 7.6. For the tests done elsewhere Delgado et al (2006) determined the filter permeability using the relationship from Sherard and Dunnigan (1989) ($k = 0.35\ (D_{15})^2$ where (D_{15}) is in millimeters, and k is in cm / sec). For the University of Granada (UGR) tests Delgado et al (2006) determined the permeability by laboratory tests, thereby taking account of the degree of compaction of the filter.

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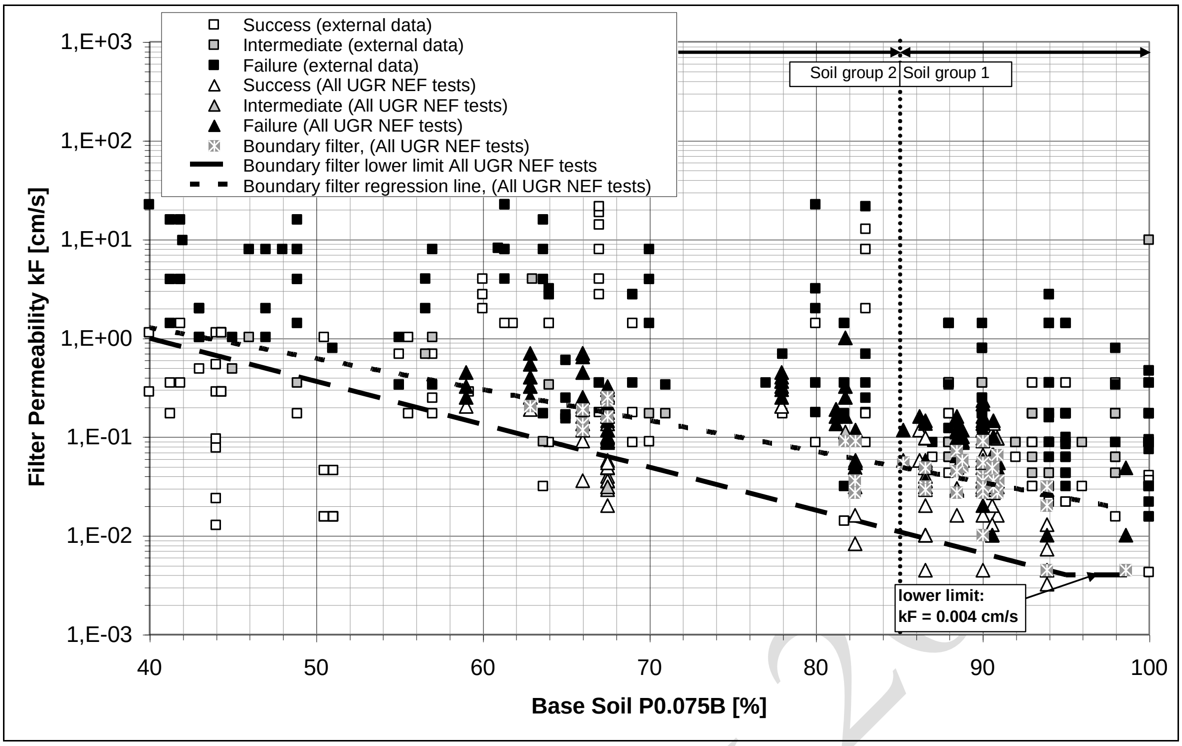


Figure 7.6 Delgado et al (2006) filter design criteria for non dispersive soils with 40% to 100% passing 0.075mm sieve relating percentage passing 0.075mm of the base soil to filter permeability in cm / sec.

Delgado et al (2006) show two design criteria, one the "Boundary Filter Lower Limit of all UGR NEF tests, and the second "Boundary Filter Regression Line" which is fitted to base-filter combinations characterizing the transition from no failure to failure. They do not specifically say which of these should be used but suggest that users take account of all the data in the Figure 7.6 so the uncertainty in the criteria is apparent.

Delgado et al (2012) used NEF tests to examine the influence of plasticity, mineralogy, water content, hydraulic gradient and additives including aluminum sulfate on internal erosion of clayey soils protected by granular filters. As expected the aluminum sulfate increased floc size and thereby increased filter size. Base soils at higher water content required finer filters but the other factors had no significant effect on filter requirements.

7.6 FOSTER AND FELL (1999a, 2001) METHOD FOR ASSESSING THE LIKELIHOOD OF CONTINUATION OF EROSION FOR FILTERS AND TRANSITIONS WHICH DO NOT MEET MODERN FILTER DESIGN CRITERIA

Foster and Fell (1999a, 2001) analyzed the tests in the NEF equipment with filters which were too coarse to satisfy no-erosion filter criteria but which did eventually seal as coarser particles from the base soil were eroded and collected on the surface of the filter to form a finer filter which stopped the erosion process. They also considered the tests to define the filter grading at which erosion would continue, called the continuing erosion boundary.

From these data and the test data from Sherard et al (1984) they proposed the criteria in Table 7.3 for the excessive and continuing erosion boundaries.

Information from case histories of poor filter performance suggests the potential maximum leakage flows that could develop due to piping are as follows:

- Filters falling into the Some Erosion category – up to100 l/sec:
- Filters falling into the Excessive Erosion category – 100 to 1000 l/sec.
- Filters falling into the Continuing Erosion category – flows of 1000 l/sec and increasing.

Figure 13.2 in Terminology presents the ‘no’, ‘some’, ‘excessive’ and ‘continuing’ erosion concept diagramatically.

Fell et al (2008) recommend that the maximum leakage flows listed above be used in the assessment of the probability of a breach mechanism developing. Lower leakage flows are likely if upstream flow limitation occurs, but the factors would need to be carefully considered and justified if they were to be relied on in the assessment.

Table 7.3 Excessive and Continuing erosion criteria (Foster and Fell 1999a, 2001).

Base Soil	Proposed Criteria for Excessive Erosion Boundary	Proposed Criteria for Continuing Erosion Boundary
Soils with DB95<0.3 mm	DF15 > 9 DB95	For all soils: DF15 > 9DB95
Soils with 0.3<DB95<2 mm	DF15 > 9 DB90	
Soils with DB95>2 mm and fines content >35%	DF15 > the DF15 value which gives an erosion loss of 0.25g/cm^2 in the CEF test (0.25g/cm^2 contour line in Figure 7.7)	
Soils with DB95>2 mm and fines content <15%	DF15 > 9 DB85	
Soils with DB95>2 mm and fines content 15-35%	DF15 > 2.5 DF15 design, where DF15 design is given by: DF15design=(35-pp%0.075 mm) (4DB85-0.7)/20+0.7	

Notes: (1) Criteria are directly applicable to soils with DB95 up to 4.75 mm. For soils with coarser particles determine DB85 and DB95 using grading curves adjusted to give a maximum size of 4.75 mm.(2). CEF is Continuing Erosion Filter test.

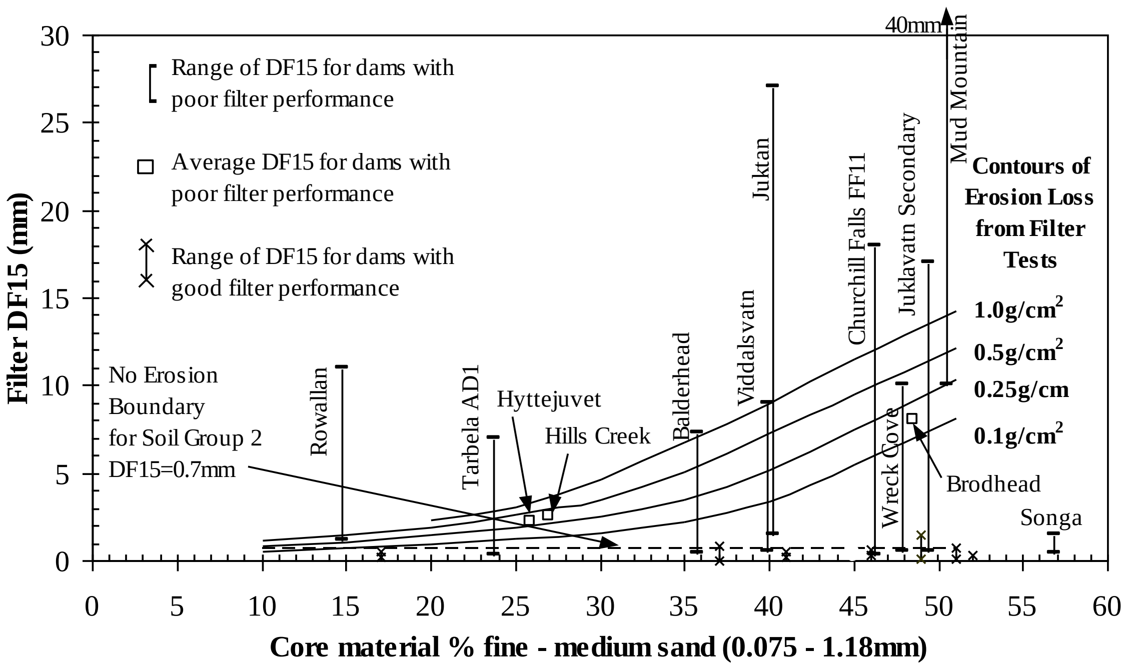


Figure 7.7 Comparison of erosion losses measured in filter tests to dams with poor and good filter performance (Foster 1999, Foster and Fell 2001).

This approach is particularly useful when assessing existing dams as many have a filter or transition which is coarser than required for no-erosion filters. Many of these dams have rockfill or permeable zones downstream which can cope with some leakage flow without failing before the filters eventually seal.

It should be noted that for soils with a small percentage of sand size particles (in Sherard and Dunnigan Group 1) there is often not a large difference between the no-erosion, excessive erosion and continuing erosion D_{15} sizes. There are insufficient sand size particles in the soils to be eroded and seal on the filter and form an effective filter for the base soil. Delgado et al (2006) found the same applies to their permeability based criteria.

In practice, the core and filter or transition in a dam have a range of particle size gradings, and it is important to consider the implications of this. To do this the steps listed below should be followed, producing the results of the type shown in Figure 7.8:

- Evaluate the DF15 values for the No Erosion (e.g. in accordance with Sherard & Dunnigan, 1989), Some Erosion (i.e. low end of Excessive), Excessive Erosion and Continuing Erosion boundaries using Table 7.3. Do this for the finest base soil grading, the average base soil grading and the coarsest base soil grading. Plot the DF15 values for these boundaries on the grading curve limits of the filter/transition material (see Figure 7.8 for an example). Use the grading curves for the filter/transition zone adjusted for segregation and suffusion if applicable;

- Estimate the probabilities for Some, Excessive and Continuing Erosion for each representative base soil grading. Estimate the proportion of the filter/transition grading that fall into each of the particular erosion categories based on the plot of filter/transition grading curves versus Filter Erosion Boundaries. More detailed guidance on how to do this is given in Fell et al (2008);

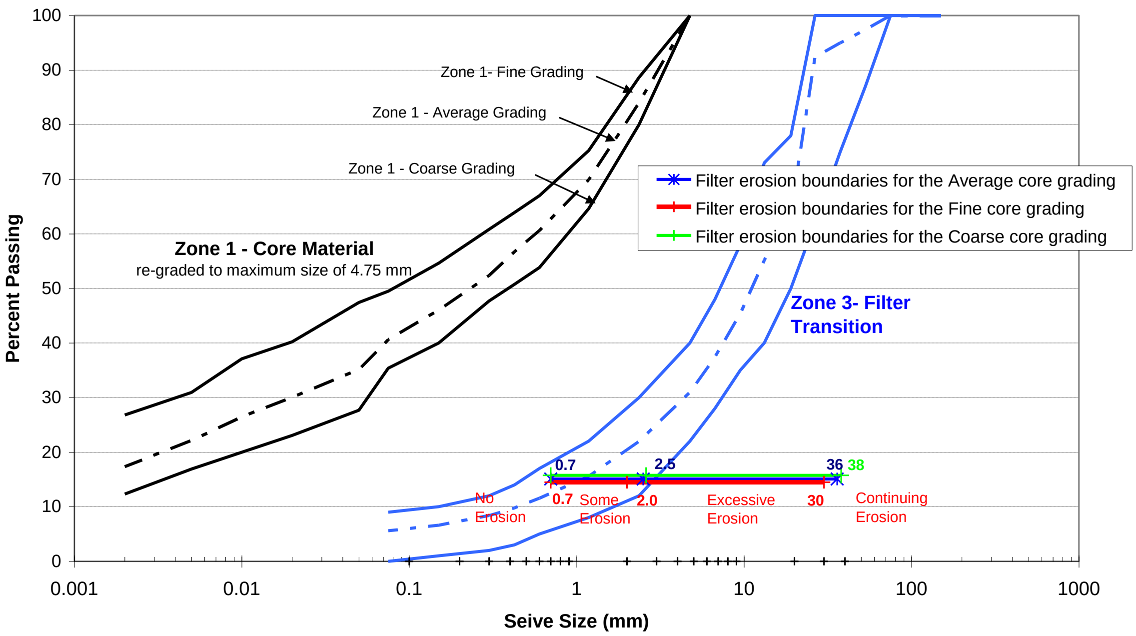
- Assess the potential leakage flows that could develop if piping were to initiate in the core based on the guidance given above;

- Assess the implications of this leakage on the stability of the downstream slope and whether unraveling might occur.

It should be noted that as for the example in Figure 7.8, the filters can be significantly coarser in dams than those required for no-erosion filters, and still eventually seal with only some erosion of the protected soil, and relatively small leakage flows.

The principles of this process are applicable regardless of which method is being used to assess the suitability of the filter or transition.

An example of the application of these principles is given in Foster (2007).



Assessment of Zone 1 core against no erosion, excessive erosion and continuing erosion criteria

Core Gradation	Base soil sizes (mm)				No Erosion	Excessive Erosion	Continuing Erosion
	DB85 (mm)	DB95 (mm)	% passing 0.075mm	% fine-medium sand (0.075 - 1.18mm)	DF15 (mm)	DF15 (mm)	DF15 (mm)
Fine Grading	1.9	3.3	50	25	0.7	2	30
Average	2.4	4	41	29	0.7	2.5	36
Coarse Grading	2.5	4.2	35	30	0.7	2.6	38

Figure 7.8 Example of plot showing filter/transition gradings compared to Filter Erosion Boundaries (Fell et al 2008)

7.7 ASSESSMENT OF THE LIKELIHOOD OF CONTINUATION FOR INTERNAL EROSION INTO AN OPEN DEFECT, JOINT OR CRACK IN THE FOUNDATION, IN A WALL OR CONDUIT

There are no commonly adopted criteria for assessing the likelihood of continuation for this scenario. Fell et al (2008) suggest use of the criteria in Table 7.4 which have been determined by assuming that the Foster and Fell (1999a, 2001), continuing erosion criteria apply to erosion into an open defect, joint or crack, and that the crack width is equivalent to the filter opening size of the voids between the particles in a filter. The filter opening size has been shown by Sherard et al (1984) to be D_{15} / 9.

Table 7.4 Continuing Erosion criterion for erosion into an open defect (Fell et al, 2008)

Erosion Condition	Comparison of Soil Gradation to Joint/Defect Opening Size (JOS)
Continuing erosion (CE)	$JOS_{CE} = D_{95}$ surrounding soil

Notes: JOS_{CE} = Joint/defect opening size that would allow continuing erosion of the surrounding soil.

D95 should be based on the average soil grading after re-grading on 4.75mm particle size.

8. PROGRESSION OF EROSION

8.1 THE TWO CONDITIONS FOR PROGRESSION

Erosion, once initiated and not arrested by filter action in the dam, will progress under two conditions: one a hydraulic condition and the other a mechanical condition:

8.1.1 Hydraulic condition

For erosion to progress, hydraulic conditions must be suitable. Water seeping through the dam must be doing so with sufficient velocity to provide sufficient energy or drag force to continue to transport particles along openings and off the external surfaces of the dam in a continuous process. This happens for the four initiating mechanisms when:

- For concentrated leak erosion, as the pipe enlarges the hydraulic shear stresses increase, so the erosion will progress unless the reservoir is drawn down to reduce the gradient.
- For backward erosion, whether the erosion progresses upstream towards the reservoir or river depends on the seepage gradient within the pipe. There is a critical gradient above which the particle will be detached and a critical flow velocity above which the soil particles will be transported in the pipe.
- For contact erosion, the situation may be as for concentrated leak erosion or erosion may not progress if for example the finer soil clogs the coarse soil.
- For suffusion, provided the eroded soil is carried away, the hydraulic condition will become increasingly able to erode the finer fraction as the permeability increases.

The progression can stop if the flow in the concentrated leak pipe or in the soil experiencing concentrated leak erosion, contact erosion or suffusion, is limited by head losses in the upstream or downstream zones. In this case an equilibrium situation may develop where the eroding forces become equal to or less than the resisting forces. In backward erosion, progression of the pipe stops if the gradient is lower than the critical gradient.

8.1.2 Mechanical condition

For erosion to progress, the mechanical conditions must be suitable. Water seeping through the dam must be doing so because the crack is sustained by hydraulic fracture or because the pipe or the cavity through which eroded particles are being transported does not collapse. Plastic (cohesive) soils, when both saturated and partially saturated, can sustain openings by the soil acting as a roof or wall to the pipe. Non-plastic silts, sands, and gravels will in general not “hold a roof” as the roof will collapse when they become saturated. However, partially saturated and high fines content non-plastic soils may hold a roof, along the phreatic surface, for example, but the roof, which in this situation is sustained by pore pressure suction, may collapse on saturation.

Progression may also stop if particles from a zone upstream of the core are transported in the developing pipe and eventually seal the filter. This action is sometimes called crack stopping. For it to occur there must be a filter or transition downstream of the core to trap the eroded particles. An example of this is Matahina dam (Gillon, 2007).

8.2 OVERALL APPROACH TO ASSESSING PROGRESSION OF CONCENTRATED LEAK EROSION

The likelihood of concentrated leak erosion progressing depends on whether:

- The soil will "hold a roof" over a pipe.
- "Crack filling" action will not stop the erosion process.
- Flow in the developing pipe will not be restricted by an upstream zone or hydraulic losses in upstream and downstream zones (or for example a concrete face slab)

It should be noted that in the absence of crack filling and flow limitation, or lowering of the reservoir to reduce the gradient of flow in the developing pipe, erosion will progress. This is because the hydraulic shear stresses in the pipe increase as the pipe enlarges.

8.2.1 Assessing whether the soil will hold a roof to a developing pipe

For internal erosion and piping through the dam or piping from the embankment into a rock foundation, the core must be capable of holding the roof of a pipe.

Based on case studies (Foster 1999), (Foster and Fell 1999b), the most important factors are:

- The fines content of the soil (% passing 0.075 mm) – ≥ 15% fines likely to be able to hold a roof regardless of whether the fines were non plastic or plastic.
- Whether the soil was partially saturated or saturated.

Other factors which were considered to be likely to have an influence were degree of compaction (loose soil would be less likely to support a roof to a pipe than dense), and reservoir operation (cyclic reservoir levels were more likely to cause collapse than steady).

Fell et al (2008) developed Table 8.1 based on these case data and taking into account the results of testing by Park (2005) which showed sandy gravels with 5 to 15% non-plastic fines collapsed quickly when saturated. Sandy gravels with 5% cohesive fines collapsed after some time, but very slowly with 15% cohesive fines. The table adds detail to the general rule that non-plastic silts, sands and gravels cannot support a roof, except when partially saturated, and plastic (cohesive) clays and silts can support a roof when saturated or partially saturated. In the table a likelihood of 1.0 means that the soil is certain to support a roof, very small likelihoods mean that the soil will not support a roof.

Table 8.1 Likelihood of a soil being able to support a roof to an erosion pipe (Fell et al 2008)

Soil Classification	Percentage Fines	Plasticity of Fines	Moisture Condition	Likelihood of Supporting a Roof
Clays, sandy clays (CL, CH, CL-CH)	> 50%	Plastic	Moist or saturated	1.0
ML or MH	>50%	Plastic or non-plastic	Moist or saturated	1.0
Sandy clays, Gravely clays, (SC, GC)	15% - 50%	Plastic	Moist or Saturated	1.0
Silty sands, Silty gravels, Silty sandy gravel (SM, GM)	> 15%	Non plastic	Moist Saturated	0.7 to 1.0 0.5 to 1.0
Granular soils with some cohesive fines (SC-SP, SC-SW, GC-GP, GC-GW)	5% to 15%	Plastic	Moist Saturated	0.5 to 1.0 0.2 to 0.5
Granular soils with some non plastic fines (SM-SP, SM-SW, GM-GP, GM-GW)	5% to 15%	Non plastic	Moist Saturated	0.05 to 0.1 0.02 to 0.05
Granular soils, (SP, SW, GP, GW)	< 5%	Non plastic Plastic	Moist and saturated Moist and saturated	0.0001 0.001 to 0.01

Notes: (1) Lower range of probabilities is for poorly compacted materials (i.e. not rolled), and upper bound for well compacted materials.

(2) Cemented materials give higher probabilities than indicated in the table. If soils are cemented, use the category that best describes the particular situation.

8.2.2 Assessing whether crack filling action will occur

For crack filling to occur requires a granular zone upstream of the core, with particles of a size which can be transported by the water flowing into the crack or pipe; and a downstream filter transition zone or rockfill which is sufficiently fine to act as a filter to these particles. Case studies where crack filling was evident include Matahina Dam, (Gillon, 2007), Swedish dams (Nilsson, 2007a, b), and others in Foster and Fell (1999b). These are typified by sinkholes appearing above the upstream filter zone which is taken as evidence of material being washed into the pipe.

Crack filling action is only possible for central and sloping core earth and rockfill (or gravel) shoulders. Whether it will be effective is dependent on compatibility of particle sizes of the granular soils upstream of the core and in the downstream filter transition, and then the compatibility of the downstream transition with in-filled material from upstream, and the core.

8.2.3 Assessing whether upstream flow limitation will occur

Upstream flow limitation may occur where there is a relatively fine grained granular material (fine rock fill, or sandy gravels) upstream of the core, or where there is a concrete face slab or concrete core wall.

Fell et al (2008) gives guidance on the assessment of the likelihood of upstream flow limitation being effective.

Both upstream and downstream zones can generate such high hydraulic losses according to the permeability of the filter that the erosion rate in the crack passing through the core can be stopped. Concentrated leak erosion through a 2 mm wide crack of the core of a 50 m high zoned dam without a filter was modeled by Fry (2007). This showed that there are three behaviors: erosion does not initiate, erosion initiates and stops, and erosion does not stop.

These behaviors are mainly dependant on the critical shear stress of the core which controls the initiation of erosion and the stabilization of erosion and the permeability of the upstream shoulder controlling the head loss at the borders of the core. Fry (2007) showed that for shoulder permeability lower than 10^{-3} m/s the final discharge rate stabilized below 10 l/s for the case modeled. This figure is an indication of the benefits of upstream flow limitation but the actual 'stabilized' discharge will vary in individual situations.

8.3 ASSESSING THE RATE OF DEVELOPMENT OF A CONCENTRATED LEAK EROSION PIPE

If analysis using the information in the paragraphs above shows that a concentrated leak will be sustained and the roof will not collapse or the flow rate drop because of upstream and downstream restrictions, the concentrated leak erosion pipe will enlarge.

From the Hole Erosion Index of the eroding soil and average hydraulic gradient along the pipe it is possible to estimate the rate at which a piping hole will enlarge. This is important because it is a significant factor in assessing the likelihood of successful intervention to stop the piping process should an emergency occur. The results are summarized in Figure 8.1 assuming (a) Unrestricted potential for erosion (i.e. no flow limitation, continuing erosion condition); (b) Initial pipe diameter of 25mm; (c) Zero critical shear stress which is conservative, particularly for I_{HET} >3.5; (d) Reservoir level remains constant. It will be seen from Figure 8.1 that the time for erosion to progress is very dependent on the soil erosion properties, expressed by the Erosion Rate Index, I_{HET}, see Tables 3.3 and 3.4.

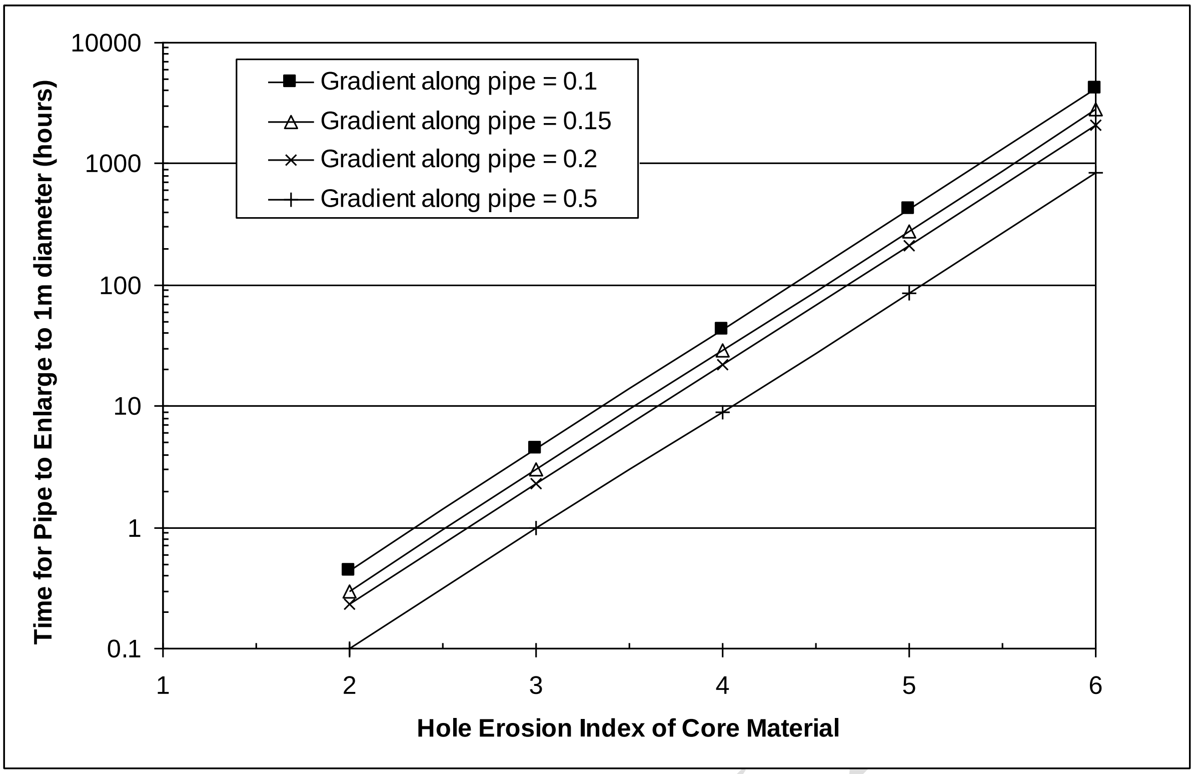


Figure 8.1 Approximate time for pipe to enlarge from 25mm to 1 m diameter The time to erode to 2 m diameter is about 20% greater (Courtesy of M Foster)

8.3.1 Rapid enlargement of concentrated leak erosion pipes and need for filters or other works to prevent continuation and progression

Note that Figure 8.1 shows that the time for the pipe to enlarge is only a matter of hours in most soils, and even in the most resistant of soils (Erosion Rate Index 5, best estimate, see table 3.4) enlargement occurs in only 100-500 hours, or 4 days to 3 weeks. These are short periods but would provide time to issue warnings and take precautions to alleviate some of the impacts of failure in an emergency. However, for the less erosion resistant soils the times to enlarge are so short that, other than at dams where the consequences of failure are minimal, some action, such as providing filter protection, should be taken to prevent erosion from continuing and progressing.

8.4 RATE OF PROGRESSION OF INTERNAL EROSION GENERALLY

Sections 11.2.2 and 11.2.3, particularly Tables 11.1, 11.2 and 11.3, give general information on the rate of progression of internal erosion. In many situations, the rate is rapid. Therefore, as with the progression of concentrated leak erosion discussed above, if erosion is intiated and continues, and is not arrested by the filtering capability of the dam, some remediation, such as providing filters, will be required.

9. ENGINEERING ASSESSMENT OF THE VULNERABILITY OF A DAM TO INTERNAL EROSION

9.1 EIGHT STEPS TO FAILURE BY INTERNAL EROSION

This chapter deals with the framework for carrying out an engineering analysis of the vulnerability or resistance of a dam to internal erosion. It follows the process described in Chapter 2 and examines the loads that may cause internal erosion and their effects at all locations where parts of the overall internal erosion process may occur. In effect it examines failure modes - the steps along routes through which the internal erosion process may lead to failure – and analyses whether the mechanical properties of the material in the dam will allow internal erosion to occur or whether they will resist it.

The framework for making the engineering analysis of the mechanics of internal erosion has been adapted from those developed for risk assessment which goes through the process from application of loads through to breach. The entire process must be examined because the probability of erosion proceeding or stopping must be estimated at each step in order to arrive at an overall probability of failure from internal erosion. Each step in the process is summarized in Chapter 2. The entire framework is summarized in Table 9.1:

Table 9.1: The eight steps of the framework for assessment of the probability of failure by internal erosion for a dam adapted from Fell and Fry (2007)

STEPS	MATTERS TO BE CONSIDERED
1 Loading	**Hydrostatic or reservoir loads**: Frequent water level, rare flood and dam safety flood. **Seismic**: a range of loading including OBE and MDE (ICOLD, 2011)
2 Location of initiation of erosion	**Embankment:** Upper portion; Lower portion; Along conduit; Adjacent to wall; Over changes in slope in the cross valley foundation profile; At construction features such as haul roads and river closure sections; **Foundation:** Valley; Abutment **Embankment to Foundation:** Valley; Abutment
3 Initiation	**Erosion Mechanism:** Concentrated leak; Backward Erosion; Contact Erosion; Suffusion Whether erosion will initiate under the seepage gradients :
4 Continuation (Filtration)	Assess whether filters, transition zones or downstream zones will prevent erosion continuing: Particle size methods: This can provide no erosion; excessive erosion; continuing erosion limits. If there is no filter erosion will continue. Equivalent opening size methods Permeability based methods.

STEPS	MATTERS TO BE CONSIDERED
5 Progression	For concentrated leak and contact erosion: Will a developing pipe stay open? Upstream and downstream flow limitation taking account of erosion properties. For backward erosion, and suffusion: Is the critical gradient or velocity reached for erosion to progress?
6 Detection	Piping process: Monitoring type and frequency: Surveillance frequency:
7 Intervention	Piping process: Personnel availability and training: Equipment and materials availability: Weather and the impacts of flooding on access: The erosion rate and its impact on the time to available intervene. Whether the reservoir can be drawn down in time to prevent breach.
8 Breach	Gross enlargement of the pipe Overtopping (e.g. due to settlement of the crest from suffusion and/or due to the formation of a sinkhole from a pipe in the embankment) Slope instability of the downstream slope Unraveling of the downstream face. Static liquefaction

9.2 APPROACHES TO ENGINEERING ASSESSMENT

To assess if internal erosion could progress to failure, it is necessary to assess each of the eight steps described in Table 9.1.

First, all the Potential Failure Modes (PFM) are identified, and those not likely to progress to failure of the dam screened out as detailed in Section 9.10 and 9.11.

The remaining Potential Failure Modes are then assessed. To assess the initiation of concentrated leak erosion, the width and depth of cracks, or the dimensions of openings, is required in addition to a knowledge of soil properties and estimates of hydraulic gradients. Guidance on the estimation of crack dimensions is given in Section 3.3. Initiation of failure by backward erosion can be assessed with a knowledge of soil properties and water level, because backward erosion is intiated when the hydraulic gradient exceeds some critical value (Figure 4.4). Initiation of contact erosion can be assessed from a knowledge (or estimate) of permeability and hydraulic gradient (to give Darcy velocity) (Figure 5.2). Suffusion may initiate in soils with unfavourable grading at low hydraulic gradients (Chapter 6).

Three approaches are proposed to define the applied hydraulic loads and the means of analysing whether or not internal erosion could progress to failure, as follows:

9.2.1 Deterministic or Standards-Based Approach

In the Deterministic or Standards-based approach the reservoir level is taken to be the spillway design flood level (or the level at which the hydraulic gradient would be greatest if the passage of the design flood raises tailwater levels). The maximum design seismic load is also assumed to apply.

It is assumed that concentrated leak erosion will initiate in the embankment under the design flood level and that cracks in which erosion would intiate would be formed under the maximum design seismic load. For backward erosion, suffusion and contact erosion calculations at spillway design flood level will determine whether or not initiation of erosion will occur. The engineering assessment then examines whether continuation, progression, and breach could occur.

9.2.2 Engineering Judgment

The Engineering Judgment Approach, which may include qualitative and semi-quantitative risk assessment, as the engineer may judge necessary, attempts to enumerate all the factors involved. This includes estimating crack widths and depths, using methods and knowledge from the engineer's experience and the guidance in Section 3.3. A range of crack dimensions should be considered. The information in Section 3.4 can then be used to assess whether or not the dam would resist the intiation of concentrated leak erosion. If the engineer judged, or analyses had indicated, that openings resulting from hydraulic fracture, in low stress zones, for example, would be present, the potential for erosion through them could also be examined.

As in the Deterministic Approach, the analysis is carried out assuming the reservoir level is the spillway design flood level, and that the maximum design seismic load applies. If the assessment showed that concentrated leak erosion could occur, the engineering assessment then examines whether continuation, progression, and breach could occur. Whether backward erosion, contact erosion or suffusion will initiate, continue and progress to breach will have been determined from the Deterministic Approach.

As a further aid to judgement, the situation at reservoir water levels lower than the spillway design flood could be examined, where the zoning of the embankment changes, or where seepage has been known to increase significantly, for example. The approach then becomes semi-quantitiative because the likelihoods of these reservoir levels being reached can be estimated from the hydrology and the spillway characteristics. Using these data engineering judgment can be used to qualitatively assess the safety of the dam. This may include assessing the likelihood of successful detection and intervention in the event that internal erosion initiates.

The method of engineering judgement augmented by qualitative and semi-quantitative risk assessments is also a preliminary to quantitative risk assessment, and may assist in deciding if the particular circumstances at a dam necessitate additional investigation by quantitative risk assessment.

9.2.3 Quantitative Risk Assessment

For the Quantitative Risk Assessment approach calculations are carried out as for the Engineering Judgment approach but estimates are made of the likelihoods of whether cracking or hydraulic fracture may occur, and the depth and width of cracks or other defects in which concentrated leak erosion may initiate estimated. The likelihood of initiation of erosion is estimated from these, the seepage flow gradients in the cracks, and the soil erosion properties. These likelihoods and the likelihood of cracking are estimated by engineering judgment by person experienced in such matters, aided by published information and manuals. In critical cases, these elements of quantitative risk assessment could be used to investigate concentrated leak erosion alone. The likelihood for all 8 steps in the internal erosion process are then combined to estimate the likelihood of failure of the dam. This is done for each PFM. The methods are introduced briefly in Chapter 12.

9.2.4 Use of the three approaches

Which approaches are used is for the Dam Engineer to decide in consultation with the Owner and Regulator.

As many dam engineers are not familiar with risk based methods they will prefer to use Deterministic or Engineering Judgement Approaches.

It should be recognized that the deterministic approach is inherently conservative because it assumes that cracks are present in which concentrated leak erosion will initiate.

A major objective of the bulletin is to provide information to support engineers in making analyses and judgments about the vulnerability of dams to internal erosion. The engineering judgment approach is intended to provide the means to make a reality check on what may be over-conservative conclusions from the deterministic approach, or to show that the uncertainties are such that quantitative risk assessment is also required to reach responsible conclusions as to the dam's resistance or vulnerability to internal erosion and the need for remediation.

The methods outlined recognize the difficulty and uncertainty in estimating crack depths and widths in concentrated leak erosion. Such uncertainties can best be examined in a risk based framework. There will be dams where a staged approach is warranted, with a progression from the Deterministic or Standards Based Method, to Engineering Judgement, and then to Quantitative Risk Assessment.

It is recommended that the engineering analyses and assessment be carried out iteratively in stages, making use initially of all readily available information about the dam, its geometry and the foundations, making further investigations only if more data would make it possible to make clearer and more definite decisions on the vulnerability or resistance of the dam to internal erosion.

9.3 ENGINEERING ANALYSIS PROCEDURE

The full procedure for an engineering analysis of the vulnerability of a dam and its foundations to internal erosion is:

(1) Assemble all the relevant information about the dam and its foundations as described in Section 9.8.
(2) Determine the loading conditions for the dam resulting from the reservoir and earthquakes as described in Sections 9.4, 9.5 and 9.6 and Section 3 in relation to concentrated leaks.
(3) Carry out a detailed Potential Failure Modes Analysis (PFMA) as described in Section 9.9.
(4) Screen out PFM which are not applicable to the dam. Some guidance on how this may be done is given in Section 9.10. Screening may be done on the zoning of the dam and the foundation conditions, or a more detailed understanding of the factors influencing the initiation and continuation of erosion.
(5) For the remaining PFM analyze the ability of the soils in the dam to resist the maximum applied loads (at the highest water level, and in the widest crack or opening in the case of concentrated leak erosion). Make staged assessments and collect and use more data as necessary to reach definite conclusions on the ability of the dam to resist erosion or not. Use engineering judgment and similarities of PFM as far as possible to reduce the number of PFM examined in detail. Some guidance on how this may be done is given in Section 9.11.
(6) For PFM where definite conclusions cannot be reached and in cases where extensive remedial works are necessary to make the dam resistant to erosion, use

qualitative or quantitative risk analysis methods to estimate the likelihood of failure taking account of the frequency of the applied reservoir and seismic loads and whether the risk is acceptable and expenditure should or should not be made. Some guidance on how this may be done is given in Section 9.12 and Chapter 12.

It can be seen that the scope of the analysis is defined by the number of realistic failure modes and the procedure therefore concentrates first on eliminating unrealistic failure modes and then examining realistic modes in detail, going through stages if the analyses show more data to be needed to reach reliable conclusions.

If the engineering analysis plainly shows that internal erosion could progress and the consequences of failure were severe, works to make the dam safe against internal erosion would be required. On completion of the works, a monitoring regime proportionate to the consequences of failure would be required.

If the properties of the dam and its foundation were such that the engineering analysis plainly shows that internal erosion could not initiate or progress, no works would be necessary, but a monitoring regime proportionate to the consequences of failure would be required.

In cases where the properties of the dam and its foundation were such that it was not clear from the engineering analyses, even after applying more data in several iterations, that internal erosion could or could not initiate and progress, decisions about safety works or no safety works and the stringency of monitoring regimes would have to be made on the basis of the assessed likelihood of failure and the severity of the consequences of failure. In such cases, decisions could be usefully supported by making quantitative risk assessments, as described in Chapter 12.

Remedial works to make dams safe against internal erosion are discussed in Chapter 10. Information on appropriate monitoring regimes is given in Chapter 11.

9.4 RESERVOIR AND SEISMIC LOADING CONDITIONS

9.4.1 The effect of reservoir level on internal erosion

The statistics about dam failures resulting from internal erosion in the embankment (Chapter 1) show that most failures of existing dams occurred when the reservoir water level was at or close to its highest ever level. This may be because the upper parts of dams are rarely tested by high water levels, consequently weaknesses at high levels in the dam are found only during rare periods of high water level.

However many incidents within the embankment have occurred at normal operating reservoir levels and it should also be noted that for internal erosion in the foundations most failures and accidents occur at other than maximum reservoir levels. Hence the assessment of the safety of the dam should not be only for “extreme” reservoir levels, but over the full range.

The upper parts of dams are often more vulnerable to internal erosion than elsewhere. They may be zoned without filters as shown in the examples in Figures 9.3 and 9.4. The upper parts are also more likely to be affected by cracking and hydraulic fracture due to differential settlements within the dam and its foundation during and after construction. Also, the upper parts of an embankment dam may dry out and desiccation cracks may provide routes in which erosion occurs.

It should also be noted that during periods of high water level the entire dam may be subjected to higher hydraulic loads than ever before. The high pore pressures may cause hydraulic fracture to occur leading to cracks and openings through the dam fill in places, such as beside culverts, which were previously intact. Note that hydraulic fractures may close and re-open as pore pressures change, seasonally, for example. Note also that because of cycles of loading (from wet and dry seasons, for example), the stress state in embankment dams is not constant over time. Consequently cracks or low stress zones through which erosion may initiate could form at any time in dams which have been resistant to erosion previously.

9.4.2 Reservoir levels to be considered in the engineering assessment of internal erosion

The reservoir levels to be considered depend on which approach is being followed:

Deterministic approach: The highest reservoir water level (or hydraulic gradient) expected during the passage of the spillway design flood.

Engineering Judgment: The highest reservoir water level expected during the passage of the spillway design flood; and reservoir levels at which there are changes in zoning, the level of geological features which occur in the foundation, levels at which there have been recorded anomalous seepage or piezometer readings.

Quantitative Risk Assessment: As for the Deterministic approach, plus pool of record (historic high reservoir level), and embankment crest level (if the spillway design flood will overtop the dam).

Depending on the geometry of the dam zoning and the characteristics of the floods and reservoir, the most vulnerable parts of the dam may be reached by relatively high likelihood floods (as in Figure 9.3(b) or lower likelihood as in Figure 9.3(a)). This should be taken account of in making the assessment of the safety of the dam.

9.4.3 Seismic loads to be considered

The seismic loads to be considered depend on which approach is being followed:

Deterministic approach: The maximum design earthquake.

Engineering Judgment approach: The maximum design earthquake, and seismic loads which may cause minor cracking, are very likely to cause transverse cracking, or likely to cause liquefaction in the dam foundation or within the embankment

Quantitative Risk Assessment approach: As for the Engineering Judgment approach.

For seismic loading, a coincident pool (i.e. reservoir level at the time of the earthquake) must be considered. The selection of this pool depends on several factors. For dams where the joint occurrence of earthquake and floods are high, the reservoir levels to be considered may be as described above. Otherwise, high reservoir levels may be screened out based on the low joint loading probability, and the evaluation of seismic failure modes can be based on a “sunny day” or “normal” operating level, often defined as a 90-percent duration exceedance pool elevation (i.e., the reservoir level that is exceeded 90 percent of a given year).

9.5 DESIGN STANDARDS AND THE LIKELIHOOD OF FAILURE FROM INTERNAL EROSION

It should be noted that if the spillway can safely pass the Probable Maximum Flood, and the dam can resist the internal erosion load applied by the peak water level during the passage of the PMF flood, it is safe to PMF standards from both floods and internal erosion.

If the dam could be shown to resist the earthquake of the same probability of occurrence as the PMF, it could be shown to be resistant to overtopping, internal erosion and seismic stability to PMF standards.

PMF is a deterministic criterion, similar to MCE (maximum credible earthquake). The probability of occurrence of deterministic criteria is unknown, although expected to be very low.

It should be recognized that it is inconsistent to design a spillway upgrade for an existing dam to PMF standards, and not provide a similarly low likelihood of failure by internal erosion; e.g. by operating the dam without filters or with inadequately controlled internal erosion in the foundation.

The above observations about co-incidence of probability of failure from earthquake, flood and internal erosion can be more plainly made by using probabilistic, not deterministic, criteria. Thus, if the spillway is designed to pass the 1 in 10,000-year flood safely, and found or improved to resist internal erosion during the peak water level or highest hydraulic gradient as the 1 in 10,000-year flood passes, and found or improved to be stable during the 1 in 10,000-year earthquake, the probability of failure from any of the three main threats to embankment dams will be at most 1 in 10,000.

9.6 SAFETY MARGINS AND APPROXIMATIONS

Engineers should recognize that estimates of the flood and earthquake characteristics are approximate, as are estimates of the erosion characteristics of the materials in the dam subject to seepage flows. They should exercise judgment when drawing conclusions based on the application of this Bulletin.

Estimates of crack widths could be critical in establishing whether or not the dam is safe or not from erosion. Engineers may judge that the resistance of the dam to erosion should also be examined for a range of crack widths and for varied soil resistance parameters.

The state of the art in estimating crack depths and widths is limited and there is considerable uncertainty in the estimates. Fell et al (2008) give guidance but the uncertainty remains. This is a reason why dam engineers may chose to use deterministic, engineering judgment and qualitative or semi quantitative risk assessment approaches as detailed in Section 9.2 rather than methods based on the physics of erosion and / or quantitative risk assessment methods.

Many internal erosion failures initiate through deterioration or shortcomings of conduits and spillways through dams. For example, if holes form in conduits, seepage velocities towards the hole increase and may be sufficient to erode the fill and erosion may continue and progress in the 'pipe' provided by the conduit. Analysis to check if deterioration of the conduits or spillways could lead to erosion can be made and in some circumstances may show that rapid erosion to failure could not go undetected. Generally however it is necessary to take precautions to avoid the ill-effects of deterioration or design shortcomings.

In many circumstances, knowledge of the soil parameters, soil behavior and the applied loads makes it possible for an analysis of the mechanics of internal erosion to show that erosion could not occur. In others, the approximations and unknowns in the analysis of the mechanics of internal erosion make it uncertain whether or not internal erosion could occur. In such cases, decisions on whether to err on the side of safety and install safety works or to trust to a monitoring regime to give adequately early warning of the onset of internal erosion must be a matter of engineering judgment. The judgment will have been supported by the analysis of the mechanics of internal erosion described in this bulletin. It could also be supported by quantitative risk assessment, using methods described in Chapter 12.

9.7 SOIL PROPERTIES IN RELATION TO INTERNAL EROSION

9.7.1 Material susceptibility, stress conditions and hydraulic load

As stated in the Terminology, internal erosion occurs when soil particles within an embankment dam or its foundation, are carried downstream by seepage flow.

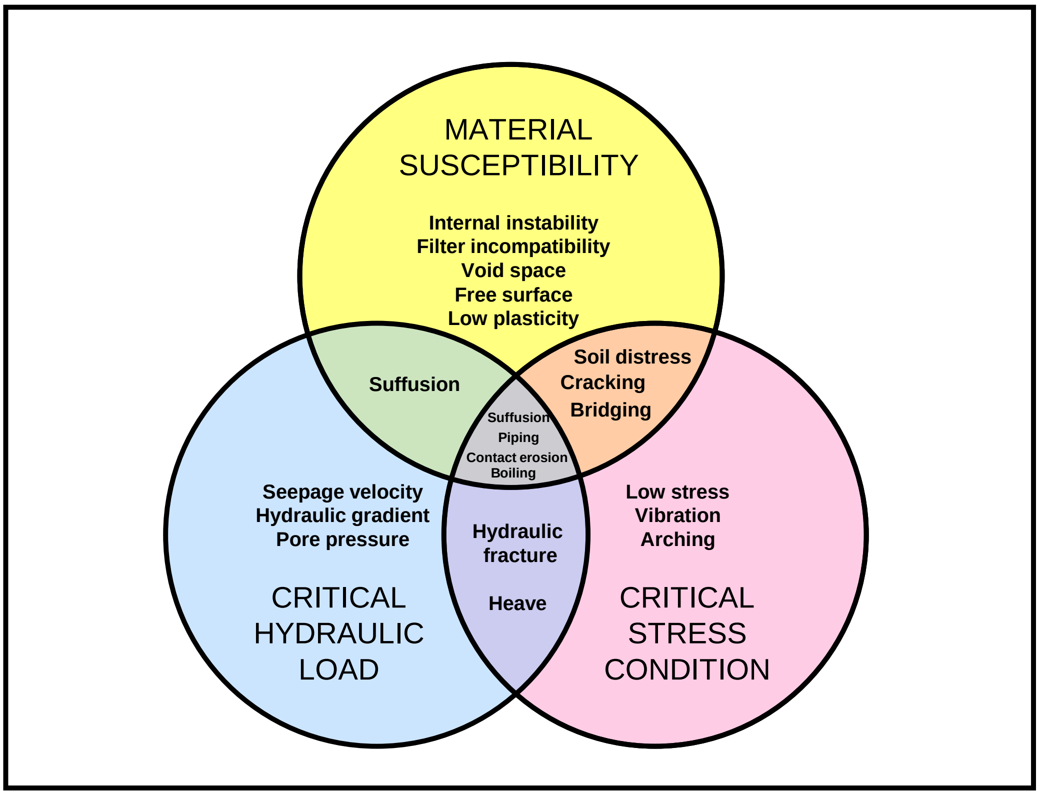


Figure 9.1 Venn diagram illustrating interaction of geometric, hydraulic and mechanical susceptibilities of soils to initiation of internal erosion (Garner and Fannin, 2010)

Garner and Fannin (2010) use the Venn diagram in Figure 9.1 to illustrate that erosion initiates when an unfavorable coincidence of material susceptibility, stress conditions and hydraulic load occurs. The three conditions are:

Material Susceptibility: the potential for soil to experience loss of a portion of its finer fraction, as a consequence primarily of grain size and also shape of the grain size distribution curve;

Critical Stress Condition: the inability to resist internal erosion due to the magnitude of effective stress, with recognition that stress varies spatially and/or temporally within the body of the dam; and,

Critical Hydraulic Load: the hydraulic energy required to invoke a mechanism of internal erosion, by means of seepage flow through the dam.

Some forms of erosion initiate when only two factors coincide, and the central subset describes a zone within the dam that is susceptible to all three factors. The combination of Material Susceptibility, Hydraulic Loading and Critical Stress gives rise to the release or detachment, and transport of soil grains.

Hydraulic Fracture

An important issue in internal erosion is the stress conditions resulting from cross valley differential settlement, differential settlement due to compressible soils in the foundation, or arching in narrow cores which results in hydraulic fracture. This occurs when the pore pressure (u) exceeds the minimum principal stress (σ_3) (strictly speaking the pore pressure should exceed the minimum principal stress plus the tensile strength of the soil). When the orientation of the minimum principal stress is unfavorable, the resulting fractures (cracks) provide sites for concentrated leak erosion. The fractures will remain open, in all soil types, until the pore pressure drops, as may occur as a result of drainage through the fractures, or the minimum stress increases, as may occur through settlement or creep. Fractures which close may re-open when the stress state or pore pressure changes. Such changes may occur when reservoir level rises, or seasonally, through rainfall on the downstream slopes in wet weather for example.

9.7.2 The physical process

The first condition for internal erosion to occur is particle detachment. Water seeping through the dam or flowing in cracks or concentrated leaks must be doing so with sufficient velocity to provide sufficient energy to detach particles from the soil structure. The nature of the soil in the dam determines its vulnerability to erosion. Three classes have to be distinguished:

Three classes of soil in relation to internal erosion

- Non plastic soils such as silts, sands, silty sands, and silt, sand, gravel soils. These collapse when saturated under flooding, will generally not sustain a crack when saturated, and are relatively easily eroded. As cohesionless soils become coarser through silts, sands, gravels and cobbles they progressively require more energy to initiate erosion. Erosion resistance is related to particle weight, and in some cases the stress state, and involves detachment of individual particles. These soils are subject to backward erosion, contact erosion or suffusion depending on their particle size distribution.

- Plastic soils, such as clays, clayey sands, and clayey sandy gravels are generally more resistant to erosion than cohesionless soils. These soils are subject to concentrated leak erosion and contact erosion. Clay soils will hold a crack even when saturated. Non plastic soils with a high silt content will sustain a crack when partially saturated and may do so when saturated. Higher energy is generally required to detach particles from cracks or concentrated leaks within a cohesive fill but the particles thus removed are small and easily carried through the crack. Erosion resistance is related to contact forces between the water flowing in the crack or concentrated leak, and the critical shear stress of the soil on the sides of the crack.

Backward erosion and suffusion cannot occur in these soils under the gradients normally experienced in dams and their foundations but may occur if local gradients are very high.

- Dispersive plastic soils are soils in which, because of their clay mineralogy and the water chemistry, erosion will initiate in cracks or concentrated leaks under very low hydraulic stresses and gradients.

Degree of saturation

- Wan and Fell (2002, 2004a, b), and Lim (2006), Lim and Khalili (2010) found that most clay soils tested have significantly higher erosion rate indices (slower erosion) and higher critical shear stresses when saturated than at the partially saturated compaction condition. There was however less dependence on the degree of saturation for silty soils. This is an important finding because it means that once the core of a dam constructed of clay soil is saturated and consolidated, it may have a slower rate of erosion, and a higher critical shear stress.
- Just as important is that this does not apply to silty sand cores such as decomposed and residual granites.
- From a practical point of view, it is therefore better to compact cohesive soils to the normal requirements for dam cores; e.g. to 98% standard maximum dry density, on the wet side of optimum, because the erosion resistance increases with the degree of saturation of the soil. However if filters are provided, the difference in behavior of soils compacted wet or dry of optimum is minor.
- It should be noted that clays in flood embankments and near the crest of dams can become desiccated. Vertical shrinkage cracks form hexagonal blocks cut at frequent intervals by horizontal cracks, and the resulting ‘blocks’ are readily eroded. More information is given in Section 3.2.9.

Dispersion properties of the soil

- Soils which show dispersive behavior; soils classifying as Emerson Crumb Class 1 or 2, and Pinhole Dispersion D1 and D2, will have a very low critical shear stress if the eroding fluid is sufficiently free of salts which might otherwise suppress dispersion (deflocculation). That is if the eroding water has low salts content. It should be noted that under flood conditions the salts content of the water in the reservoir is likely to drop, so tests done in reservoir water may be un-conservative. If in doubt with dispersive soils it is best to assume the reservoir water will not inhibit dispersion and rely on the results of tests using distilled water. The effect of reservoir water, distilled water and dispersants (e.g. sodium hexametaphosphate) on grain size distribution is most obviously illustrated by ‘triple dispersion’ particle size distribution tests as in Figure 3.6.
- Lim (2006) and Lim and Khalili (2010) showed that for rotating cylinder tests the Erosion Rate Index is not greatly affected by whether the soil is dispersive after the initially rapid part of the erosion process. So the major effect of dispersion is on the critical shear stress at which erosion initiates, not the rate of erosion.

Effect of soil structure

- Benahmed and Bonelli (2012), Lim (2006) and Wahl et al (2008) have all noted that soil structure has an important effect on the erosion properties. They find that erosion rates are significantly higher for the same soil if the soil is compacted dry of optimum moisture content and the soil forms aggregated particle, and / or micro-cracks. These allow erosion of blocks of the soil rather than of individual particles. This is one of the reasons why higher erosion rates are measured in JET than HET (see Volume 2), as the HET test is stopped with a relatively small hole diameter not allowing the "blocks" of soil to dislodge from the sides of the hole. This behavior was also noted in rotating cylinder tests by Lim (2006).
- Note also the effects of desiccation on soil structure, described in the paragraph above on degree of saturation and in Section 3.2.9.

Effect of testing method on erosion rate index

- The effect of test method is discussed in Bulletin 2.

Effect of shear strength of the soil

- Contrary to what many have assumed, the erosion rate and critical shear stress are unrelated to the effective stress or undrained shear strength of the soil. Soils with similar undrained strengths can have an erosion rate 100 or even 1000 times different; and critical shear stresses ranging from 1Pa to 100Pa depending on dispersion properties, clay mineralogy, and degree of saturation.

9.8 THE IMPORTANCE OF HAVING RELIABLE INFORMATION UPON WHICH TO MAKE THE ASSESSMENT OF INTERNAL EROSION

It is essential that the assessment of internal erosion of a dam be based on the best available information. It is best carried out in association with assessing other failure modes such as slope instability as there may be interactions between failure modes.

The data to be gathered includes:

Geometric model

The geometric model compiles the details of the internal geometry of the dam. Is it zoned, and what are the geometric details of the zones and the foundation contact of the fill within the various zones?

Geological model of the foundation

The geological model identifies the lithology and stratigraphy of the soil and rock, structural features (tectonic, position and orientation of discontinuities: joints, faults, etc) and geological background (earthquake, geomorphology). The susceptibility to erosion of the soils overlying rock; and in some cases the erosion characteristics of weathered or soft rock and width of joint openings, and the presence of features such as karst openings, are the key factors.

The hydrogeology of the foundation should also be understood.

Geotechnical model of the embankment and foundations

The geotechnical model is the compilation of the mechanical properties required for internal erosion assessment and stability analysis, including soil classification and particle size, permeability, bulk density, and preferably data on the erosion properties of soils based on laboratory tests. These are related to the geometric model of the embankment and the geological model of the foundation. The data is obtained from site investigations prior to construction, construction control testing, construction photographs, and site investigations of the existing dam.

Hydraulic or seepage model

It is fundamental to understand the hydrodynamic situation in the dam and its foundations. The routes of seepage paths through and beneath the dam can be determined from piezometric readings. Seepage and hydraulic gradients through the dam and its foundations in normal operating and high reservoir levels should be considered.

Stress state in the dam and its foundation

This may be required for detailed assessments of suffusion and global backward erosion, and for aiding in the prediction of the likelihood and depth of cracking and hydraulic fracture.

Staged approach and collection of data

In many dams not all this information is available so the assessment has to be made using what is available. If critical information, such as the particle size distributions of the core and the filter or transition, is missing it may be necessary to carry out investigations into the dam or its foundations to obtain it. If initial analysis does not give a definite result, further investigations to collect more data or carry out more tests may be required as part of the staged approach to assessing whether or not the dam can or cannot resist internal erosion.

9.9 FAILURE MODES ANALYSIS

A potential failure mode is a sequence of events starting from an initiating mechanism, such as a defect, flaw or seepage path in the dam or its foundation, which may lead to an uncontrolled release of the reservoir. After compiling the best available information, and considering the geotechnical model of the dam or dike and foundation, the dam safety assessment team should go through a discussion of all potential failure modes and develop a thorough understanding of the sequence of events and the potential location of seepage and erosion paths through the embankment and foundation. The sequence of events and seepage pathways should be documented by annotating cross sections and longitudinal sections of the embankment and its foundation to help visualize the failure path. The development of the failure modes should consider the following:

- Potential initiating mechanisms for each of the failure locations;
- Zoning of the embankment, including the configuration and properties of internal filter and drainage measures;
- Foundation geology and stratigraphy; and properties and
- Filtered and unfiltered exit points of seepage.

Failures and incidents by internal erosion of embankment dams and their foundations are categorized into four general failure paths, which are;

- Internal erosion through the embankment,
- Internal erosion alongside through-penetrating structures, such as conduits associated with outlet works, spillway walls or other adjoining a concrete gravity structure supporting the embankment.
- Internal erosion through the foundation, and
- Internal erosion of the embankment into or at the foundation. Including (a) seepage through the embankment eroding material into the foundation, or (b) seepage in the foundation at the embankment contact eroding the embankment material.

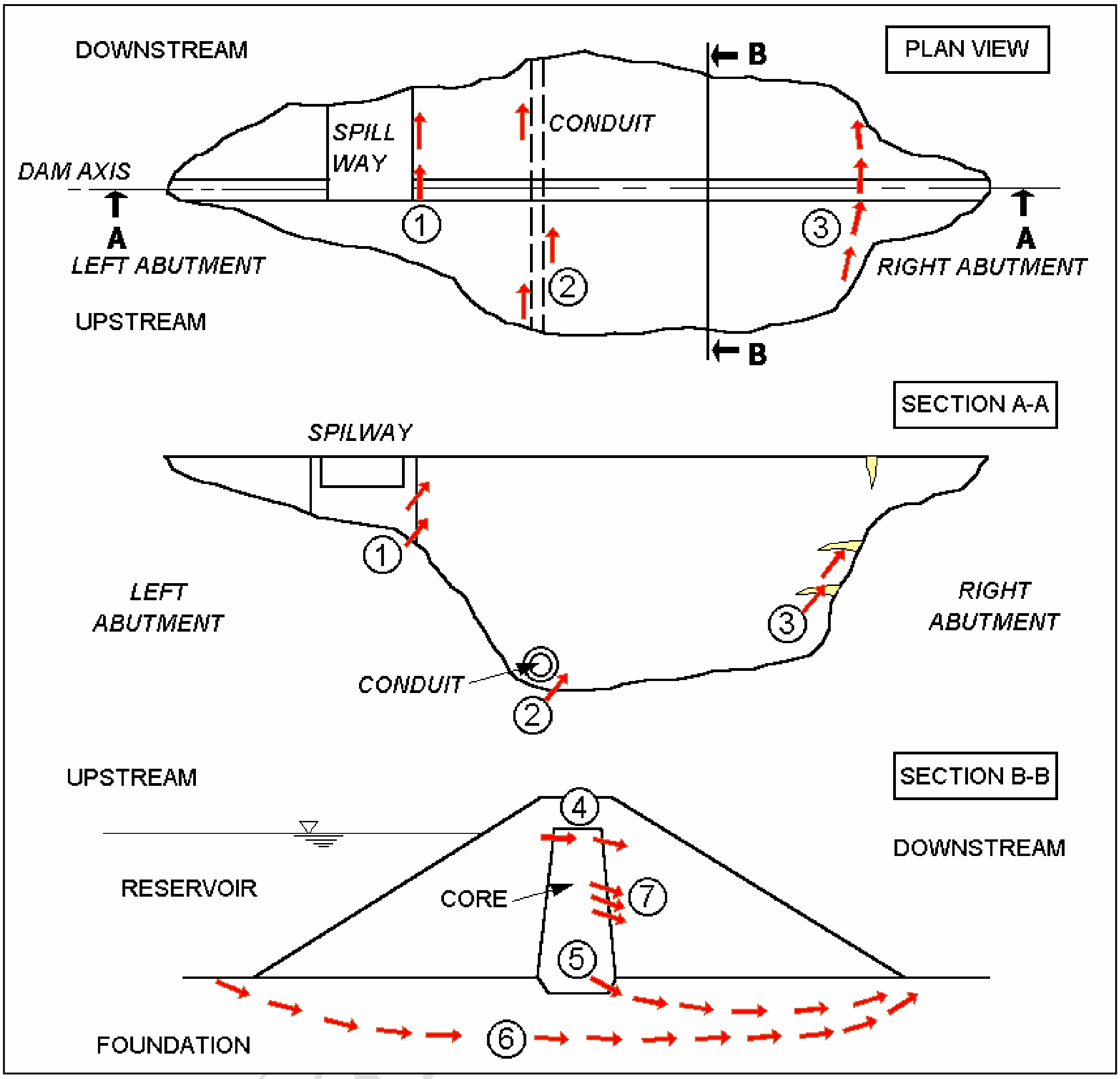
It can be seen that with four general failure paths, four types of initiation of internal erosion, and several zones and layers of differing materials in the dam and foundation, the number of potential failure paths at a single dam soon becomes large. Useful first steps therefore are to consider the soil types in the dam and their vulnerability to the four types of erosion and the zoning in the dam to identify where the types of erosion the dam to which the dam seems most vulnerable could occur. This often soon identifies the most likely locations and the most likely initiating mechanisms of erosion, leading promptly into Steps 2 & 3 of the framework for analysis of the mechanics of internal erosion, Table 9.1.

The possibility of overlooking potentially important failure modes is reduced by considering the particular details of the dam and its appurtenant structures, such as details of walls retaining the embankment, conduits through the embankment, by assembling construction photographs and reports and inspecting the dam as part of the failure modes assessment. It is also reduced by having the failure modes assessment done by a team which includes the engineer and geologist most familiar with the dam, dam operating and surveillance staff, and facilitated by a person experienced in failure modes analysis.

Figure 9.2 gives examples of possible locations of initiation of internal erosion.

For many dams, it is more likely that features likely to lead to initiation of internal erosion are in the upper part of the dam. This is because cracking due to differential settlement over large scale irregularities in the foundation profile is more likely to be present near the crest. Continuation is also more likely because often the detailing of the dam design, or as built, will give no or little filter protection. Figure 9.3 gives examples of this with increased likelihood of failure by internal erosion under flood loading near the crest of the dam because the earthfill core and filters were not taken to the crest of the dam.

Figure 9.4 is an example where there is a significantly different probability of internal erosion above the berm than below because the upper part is essentially a homogeneous dam while the sandy gravel in zone 2 may act as a filter. This is best managed in the analysis by considering internal erosion above the berm (the upper part of the embankment) separately to below the berm (the middle and lower parts of the embankment).



1. Spillway wall interface
2. Adjacent to conduit
3. Crack associated with steep abutment profile
4. Desiccation on top of core
5. Embankment to foundation
6. Foundation (if the foundation is soil or erodible rock)
7. Embankment through poorly compacted layer, crack, (or by backward erosion if the core is cohesionless)

Figure 9.2 Example of possible locations of initiation of internal erosion (Fell and Fry, 2007)

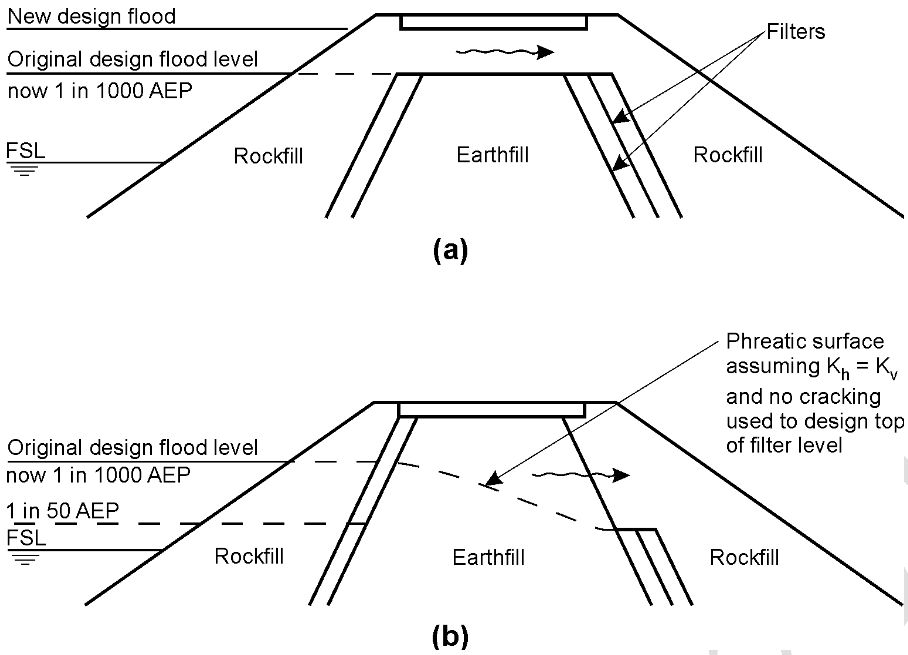


Figure 9.3 - Examples of embankment crest details which may result in relatively high likelihood of internal erosion (Fell et al, 2008)

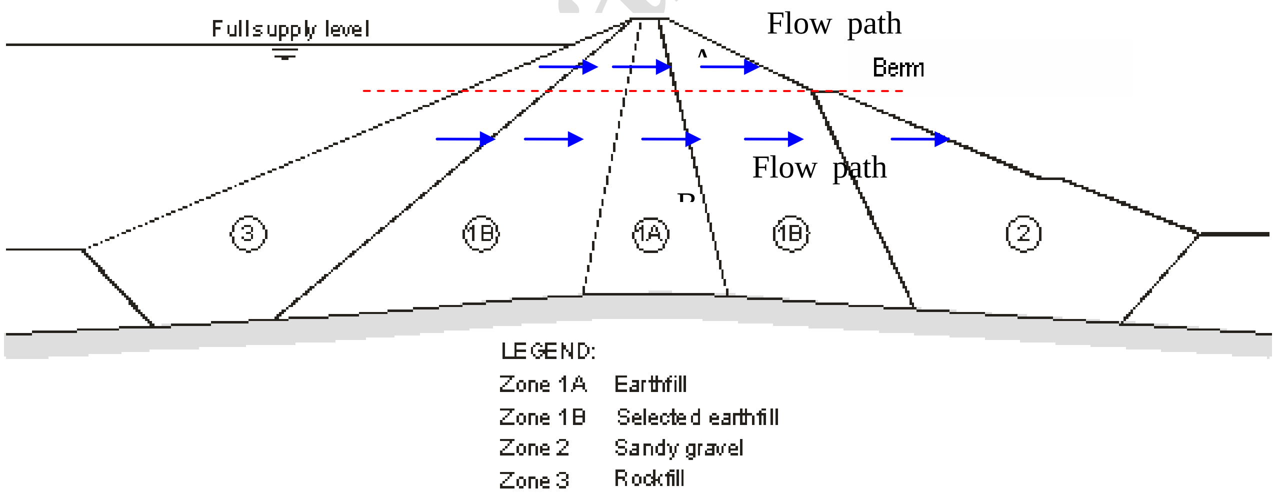


Figure 9.4 - Example of an embankment with significantly different likelihood of internal erosion above and below the top of the downstream berm. (Fell et al, 2008)

9.10 SCREENING OF POTENTIAL FAILURE MODES

9.10.1 Screening of PFM on the zoning of the dam and the properties of the core of the embankment.

(a) Zoning of the embankment

The zoning of an embankment and in particular the presence or absence of filters which satisfy modern design criteria, and are constructed to a high standard, has a significant effect on the likelihood of failure by internal erosion. Figure 2.9 and Table 2.2 in Section 2.4, summarized in Table 9.3 below, show typical embankment zoning and their relative vulnerability related to the degree of filtering provided. These are generalizations but it can be said that any dam in the first group A is very vulnerable because there is no filtering. Internal erosion in Groups B dams may be limited by the hydraulic head losses through the upstream and downstream fill and by the filtering capability of the fills. Group C dams are of low vulnerability because of the concrete upstream slab or core wall, subject to good detailing and no damage. Those in the fourth group D have in principle a very low vulnerability because filters are provided, but the detail of zoning is critical. However in reality in dams with formal and informal filters, the details of the filters are critical if internal erosion is to be controlled, and dams should not be regarded as safe from internal erosion just because their general zoning is favorable.

Table 9.3 Screening by dam zoning

Vulnerability to internal erosion	Potential for control of internal erosion	Dam zoning and category number
A Very vulnerable	Little or no control	Homogeneous (unzoned) earth fill (Category 0) Earthfill with rock toe (Category 2)
B Vulnerable	Some control of internal erosion depending on detail of zoning and filter capability	Zoned earthfill (3) Zoned earth and rock fill (4) Puddle core (8) Hydraulic fill (11)
C Low vulnerability	Moderate control of internal erosion depending on the filter capability and details of core wall or face slab	Concrete face earthfill (6) Concrete face rock fill (7) Concrete core earth fill (9) Concrete core rock fill (10)
D Very low vulnerability	Good control of internal erosion subject to good details of zoning and filter design	Earthfill with filters (1) Central earth core and rock fill (5)

(b) Screening of PFM on the properties of the core of the embankment

Table 9.4 shows PFM which can be screened out for internal erosion in the embankment. If these criteria are met the only PFM which have to be considered are concentrated leak and contact erosion PFM.

Table 9.4 Screening of PFMA for internal erosion in the core of the embankment

Initiating Mechanism	Exclude the Failure Mode if the Following Conditions are Satisfied
Backward erosion in the core	Exclude if the soil has a Plasticity Index ≥ 7.
Suffusion in the core	(1) Exclude if the soil has a Plasticity Index ≥ 7. OR (2) Exclude if the soil is not gap-graded. If the soil is gap-graded or broadly graded, exclude if the proportion of the finer fraction of a non-plastic soil is more than 40% of the total mass of the soil.

9.10.2 Screening of PFM on foundation geology and properties

Tables 9.5 and 9.6 show how PFM for internal erosion in the foundation, and from embankment to foundation can be screened based on the geology of the foundation and the cut-off provided for the core of the embankment.

Table 9.5 Screening of PFM for internal erosion in the foundation (adapted from Fell et al, 2008)

Initiating Mechanism	Exclude the Failure Mode if the Following Conditions are Satisfied
All modes of internal erosion of the foundation (backward erosion, suffusion, erosion in a crack)	Exclude if the soil layer beneath the dam is isolated by a cut-off trench founded in non-erodible rock.
Backward erosion in a soil in the foundation	Exclude if: (1) The foundation soil has a Plasticity Index ≥ 7. OR (2) If the soil layer with PI ≤ 7 layer is not continuous below the embankment (i.e. it terminates beneath the dam)
Suffusion in a soil in the foundation	Exclude if: (1) The foundation soil has a Plasticity Index ≥ 7. OR (2) The proportion of the finer fraction of a non-plastic soil is more than 40% of the total mass of the soil. OR (3) If the soil layer with a PI ≤ 7 is not continuous below the embankment (i.e. it terminates beneath the dam)
Erosion in a crack in soil in the foundation	Exclude if the foundation soil is non plastic.

Table 9.6 Screening of PFM for internal erosion from the embankment to the foundation and contact erosion (adapted from Fell et al, 2008)

Initiating Mechanism	Exclude the Failure Path if the Following Conditions are Satisfied
Internal erosion of the embankment into or at a rock foundation	Exclude if: (1) Rock foundation below the core is comprised of rock containing closed rock defects (<1 mm wide) or defects open less than $3D_{95}$,of the fine limit of the core OR (2) Rock foundation below the core has been adequately treated (e.g. shotcrete, slush grouting, mortar or concrete treatment)
Internal erosion of the embankment into or at a soil foundation	Exclude if: (1) Soil foundation below the core is comprised of fine grained soils with greater than 12% fines (fraction finer than No 200 sieve (0.075mm)), and the soil does not contain macrostructure such as root holes, relic joints or solution features. OR (2) Soil foundation below the core is comprised of sands (SP or SW) which are filter-compatible with the embankment materials (i.e. satisfy the No Erosion criteria).

9.10.3 Screening of PFM on details of the embankment and conduits and retaining walls.

The likelihood of an embankment experiencing initiation of concentrated leak erosion will be low provided that the conditions described in Table 9.7 are ALL satisfied, AND the soil in the embankment is in Erosion Group 3 or 4, moderately erodible or erosion resistant, as determined from Table 9.8.

The likelihood of an embankment experiencing initiation of concentrated leak erosion at conduits and retaining walls will be low provided that the conditions described in Table 9.9 are ALL satisfied, AND the soil in the embankment is in Erosion Group 3 or 4, moderately erodible or erosion resistant, as determined from Table 9.8.

These are based on quantitative assessments in Fell et al (2008).

This is not to say these PFM cannot occur and for high consequence of failure dams they should not be excluded without more detailed analysis.

Table 9.7 Screening of PFM for conditions in which a crack or concentrated leak in the embankment is unlikely (adapted from Fell et al, 2008)

Initiating Mechanism	Exclude the Failure Path if the Following Conditions are Satisfied
Transverse cracking due to cross valley differential settlement	Exclude if the embankment abutments are flatter than 15°, and the embankment height is uniform across the valley.
Transverse cracking due to cross valley arching	Exclude if the width of valley to dam height ratio $W_v/H > 2$.
Transverse cracking due to differential settlements in the foundation beneath the core	Exclude if there is no compressible soil in the foundation below the core
Cracking in the crest due to desiccation by drying	Exclude if the reservoir level is below the likely depth of desiccation cracking under all conditions including during extreme floods.
Cracking due to earthquake	Exclude if the MDE is below a peak ground acceleration of 0.2g, AND The embankment abutments are flatter than 15 degrees, and the height is uniform across the valley; AND The soils in the embankment and its foundation are not susceptible to liquefaction.
Transverse cracking at the foundation contact due to small scale irregularities in the foundation profile under the core	Exclude if the persistence of the irregularity across the width of the core is less than 50% of the core base width
Poorly compacted or high permeability layer in the embankment	Exclude if: All soils are very well compacted with lift thicknesses less than 200mm, with good documentation and records; This means: (1) For plastic soils (Plasticity Index > 7), ≥98% standard dry density ratio, moisture content 2% dry to 1% wet of OWC; (2) For non plastic soils and soils with PI ≤7, >75% relative density.

Initiating Mechanism	Exclude the Failure Path if the Following Conditions are Satisfied
Poorly compacted or high permeability layer on the core-foundation contact	Exclude if: (1) Contact soils are well compacted on a regular foundation surface with good documentation and records OR (2) Uniform or regular rock surface or surface treated with shotcrete or concrete to correct slope irregularities, and soils well compacted (contact soil compacted using special compaction methods (e.g. rubber tires, use more plastic material, compaction wet of OWC). OR (3) Uniform well compacted soil foundation, with good mixing, bonding and compaction of contact fill. OR (4) Compacted soil foundation
Poorly compacted or high permeability layers in the crest due to freezing	Exclude if; The climate is such that temperatures do not fall below freezing point except possibly overnight or for a day or two. OR If the reservoir stage being considered is below the likely depth of freezing

Table 9.8 Erosion resistance of soils related to classification and dispersivity

Erosion Soil Group	Soil Classification
1.Extremely erodible	All dispersive soils; Sherard pinhole classes D1 and D2; or Emerson Crumb Class 1 and 2. AND SM with <30% fines
2.Highly erodible	SM with > 30% fines; SC with < 30% fines; ML; SC with >30% fines; CL-ML;
3.Moderately Erodible	CL; CL-CH; MH; CH with Liquid Limit <65%
4.Erosion resistant	CH with Liquid Limit > 65%

Table 9.9 Screening of PFM for internal erosion around and into conduits through the embankment or adjacent a wall supporting the embankment core for which internal erosion is unlikely

Poorly compacted or high permeability zone around a conduit through the embankment	Exclude if: (1) There is no conduit passing through the embankment, OR (2) The conduit is totally embedded in a trench excavated in non-erodible rock, backfilled to the surface with concrete
Erosion into a (non-pressurized) conduit	Exclude if: (1) There is no conduit passing through the embankment, OR (2) Careful internal inspection of conduit showing no evidence of open joints or cracks.
Poorly compacted zone associated with a spillway or abutment wall	Exclude if there is no spillway or abutment wall in contact with the embankment
Crack/gap adjacent to a spillway or abutment wall	Exclude if there is no spillway or abutment wall in contact with the embankment

9.11 ANALYSIS OF POTENTIAL FAILURE MODES AT WHICH INTERNAL EROSION COULD INITIATE, CONTINUE AND PROGRESS

9.11.1 Some general principles

Many PFMs will have been excluded by use of the criteria in the tables above. Some iterations may have been necessary to collect the data needed to make full use of the tables. It is now necessary to examine the remaining PFMs and analyze if the erosive forces are sufficient to overcome the resistance of the soils in the dam and foundation and initiate erosion, and whether the erosion could continue because no filtering capability is present, and progress because cracks and openings could be sustained. If PFMs are identified through which the dam's properties are insufficient to resist erosion, it will be necessary to consider remedial works to prevent it. If no PFMs along which erosion occurs are identified, monitoring regimes will be required, focused on identification of the onset of erosion, particularly along what seem to be the most vulnerable PFMs.

There are some general principles which should be applied when assessing the potential for erosion leading to failure to occur along PFMs. These are:

(1) Consider the eight steps listed in Table 9.1 for the engineering analysis to establish whether the dam's properties are sufficient to resist erosion or whether remedial works to protect the dam against erosion are needed.

(2) Take account of the fact that some PFM may be through weaknesses at high levels in the dam, not yet tested by high water levels; e.g. concentrated leak erosion above the filters in Figure 9.3 (a) and (b). Other PFM at low levels in the dam will be most severely loaded when the water level in the reservoir is highest during the spillway design flood.

(3) For concentrated leak erosion in the embankment or in a plastic soil foundation consider whether a crack or low stress zone subject to hydraulic fracture may occur for the PFM, whether the crack will extend below the reservoir and the erodibility of the soil (Sections 9.9, 9.10.3 and 3.3 and 3.4, and Chapter 3 generally)

(4) For backward erosion piping in the foundation assess whether the reservoir level during the spillway design flood will cause heave of any layer overlying the erodible strata or if there is an open ditch or drain into which erosion can occur without heave. Assess if the hydraulic gradient at spillway design flood water level and other reservoir levels under consideration is at or above the critical gradient at which erosion will initiate and progress for the erodible soil layer. (Section 4.2, Chapter 4).

(5) For global backward erosion in the embankment estimate the likely critical gradient by comparison with published data, and compare this to the gradient in the embankment at spillway design flood water level and other reservoir levels under consideration. Carry out laboratory tests if the design gradient is approaching the critical gradient (Sections 4.5 and 4.6).

(6) For contact erosion at the embankment-foundation interface, estimate the Darcy velocity at spillway design water level and other reservoir levels under consideration and compare this to the velocity at which contact erosion initiates (Section 5.3, Chapter 5).

(7) For suffusion in the embankment core or filters, or in the foundation, assess the likelihood that the soil is internally unstable and subject to suffusion. If it is, assess the critical gradient at which erosion will initiate based on published data or laboratory tests. Then assess the annual probability that these critical gradients will be reached related to reservoir level. Assess the likely gradation of the soil after suffusion and the effect on permeability and seepage flows (Section 6.3, Chapter 6).

(8) Assess whether filters, transition zones or downstream zones will prevent erosion continuing. For filters and transitions which do not satisfy modern design criteria, assess the likelihood that the filters will eventually arrest erosion using the no-erosion, some erosion, and excessive erosion concepts (Chapter 7).

(9) Assess whether erosion will progress or whether upstream flow limitation will arrest the erosion (Chapter 8).

(10) Assess the time for development of erosion from when it can be detected to when a breach may develop. Then assess the likelihood of detection and successful intervention taking account of the mode of internal erosion, the zoning of the embankment, and the management factors which influence whether detection and intervention will be possible (Sections 2.6 and 11.2.3).

(11) Assess whether failure may be averted by the erosion process self limiting, e.g. by the reservoir draining, even if detection and intervention is unsuccessful (Section 2.8).

It should be noted that whether or not internal erosion leads to failure is mostly controlled by the resistance or vulnerability of the dam to initiation of erosion, (2) to (7), and by the effectiveness of filters and transitions to arrest erosion (8).

9.11.2 Concentrated leak erosion-some aids to judgment

Erosion in concentrated leaks occurs when the widths of cracks or the size of openings is large enough to allow seepage velocities to generate hydraulic shear stresses in excess of the hydraulic shear strength (see Sections 3.3 and 3.4). Estimating crack width and opening size and the hydraulic gradients in them is a vital part of the process of assessing the vulnerability to erosion, as discussed in Section 3.3 and throughout this Section 9, particularly 9.10.3 and Table 9.7.

9.12 DECISION MAKING: ARE ANTI-EROSION MEASURES NECESSARY OR WILL MONITORING BE SUFFICIENT?

If PFMs are identified through which the dam's properties are insufficient to resist erosion, it will be necessary to consider remedial works to prevent it. If no PFMs along which erosion occurs are identified, monitoring regimes will be required focused on identification of the onset of erosion, particularly along what seem to be the most vulnerable PFMs.

Before making decisions, the types, effectiveness and scope of anti-erosion measures and monitoring that could be installed should be considered using the comments in Chapters 10 and 11 as a basis.

10. REMEDIATION AND IMPROVEMENTS TO DAMS TO RESIST INTERNAL EROSION

10.1 METHODS AVAILABLE

This chapter is to give a broad overview of the methods available to remediate or improve dams to enable them to resist internal erosion. It does not deal with 'intervention' in an emergency, which is dealt with briefly in Section 2.7 above.

More information on the repairs and improvements are included in Volume 2 of the bulletin. The information here is intended to provide sufficient information to give a preliminary understanding of the alternatives and scope of works likely to be necessary to make a dam able to resist erosion, and to assist in making the decision whether or not to 'fix' a dam which the analyses carried out above have shown to be only marginally able to resist erosion when subject to critical loads in its present condition.

There are two basic approaches to making dams capable of resisting internal erosion:

- Barriers, to prevent seepage from passing through the dam or foundation
- Filters to trap (filter) any eroded particles from seeping water while allowing the water to drain away downstream

Engemoen (2012b, 2012c) describes the criteria used by USBR to decide if remediation is necessary and, if so, the types of remediation used. These include 'seepage control' (filters and filtered drains) and 'seepage reduction' (barriers) measures. Their advantages and disadvantages are listed. Other things being equal, Engemoen preferred seepage reduction approaches, principally because they involve standard earthworks construction and uncover the embankment and foundation, whereas seepage reduction measures, such as cut-off trenches, typically work 'blind'.

10.2 BARRIERS

Barriers can be provided in many different forms including sheet piles, diaphragm walls, secant piles, and grouting. Some forms can be installed to substantial depths; there are examples of cut-off walls greater than 100 m deep. It is vital that the barrier is complete. Any openings or gaps in barriers may concentrate flow, increase seepage velocities and initiate erosion. In extreme circumstances, locally strong seepage flows can erode concrete or grout from the barriers and enlarge gaps in them. Providing barriers only locally along part of a dam drives seepage towards the ends of the barrier, where the extra seepage may initiate erosion. Ideally the barrier should make a complete cut-off into impermeable strata at its base. Providing barriers to only part depth in permeable fill or foundation concentrates seepage below the barrier and may initiate erosion. Barriers must make a seal into impermeable strata at the abutments; in some circumstances considerable lengths of abutment cut-offs are needed to prevent seepage and loss of water through permeable strata above impermeable foundation strata. Rice (2007) reports on the long-term performance of seepage barriers.

10.3 FILTERS

10.3.1 Filter blankets and filtered berms

Filters, granular materials with gradings designed by methods listed in Section 7.5 to filter the smallest particles expected to be present in the dam fill or foundations, can also be used to protect dams against internal erosion. FEMA (2011) provides much information on design and construction of filters.

Filtered berms, filter blankets on the downstream face loaded by new fill in a berm downstream, are commonly used to retrofit filters to dams without filters or with inadequate filters. Filter blankets should also completely cover the entire seepage surface. This is because eroded particles could escape in water seeping out through any areas not protected by the filter and allow continuation and progression of erosion.

It is usually necessary to extend filter blankets at the downstream toe of embankments to intercept and filter seepage water from the foundation. Sometimes it is necessary to provide filters in trenches below the toe filter blankets to intercept and filter seepage which would otherwise pass below the filter. Filtered relief wells are also used to intercept and filter seepage flowing below the surface. It is also usually necessary to extend downstream filter blankets, including toe blankets, along the valley sides downstream of the dam to intercept and filter the surfaces where seepage flowing through the abutments will emerge.

10.3.2 Filter collars on conduits and spillway channels

Many internal erosion failures and incidents occur alongside conduits and spillways passing through dams. Filter collars should be installed around the downstream end of conduits, culverts and pipes and spillway channels through dams to prevent the continuation of erosion through fill or foundation materials around them. Provision of filter collars at the upstream end and if possible at intervals along the conduits, provides crack filler to reduce seepage velocities and thereby inhibit the initiation and continuation of erosion.

10.3. 3 Filter trenches

Filters can be installed in trenches to deal with shallow seepage routes near the surface of embankments and at the toe, as mentioned above. Bio-degradable slurry is available (e.g. Jairaj and Wesley, 1995) to support and form deep filter trenches, if necessary. The slurry is degraded by biological action, leaving the sand filter in-situ. If installed in the crest, they would cross transverse cracks through which concentrated leak erosion may occur.

10.3.4 Filtered relief wells

Filtered relief wells can be used to relieve high pore pressures at depth or under impermeable layers and to draw seepage waters upward into foundation filters. Filtered relief wells are simailr to water wells, with the filter pack graded as a filter to the soils through which the well passes, with screens designed to retain the filter.

10.3.5 Multiple filters to provide drainage capacity

As mentioned above, it is usually necessary to provide both a filter layer and a drainage layer in a filter blanket. This is because the permeability of filters capable of filtering the small particles that may be present in the dam, and therefore eroded from it, is low. The drainage capacity of such filters is also low and not likely to be able to drain away the quantities of seepage that may arise if a leak occurs. In extreme circumstances a third ‘filter’ coarser than the drainage filter, perhaps in the form of ‘finger drains’, may be needed to provide additional drainage capacity. The filter and drains must all comply with filter rules so that there is no erosion from one to another. Filter rules must also be applied between filters and adjoining fill or foundation materials to prevent erosion at the interfaces. This requirement sometimes makes it necessary to provide a filter ‘sandwich’ with say fine and coarse filter layers on both sides of a very coarse drain filter.

10.4 CONDITION OF CONDUITS

Many internal erosion failures initiate through deterioration or shortcomings of conduits and spillways through dams. For example, if holes form in conduits, seepage velocities towards the hole increase and may be sufficient to erode the fill and erosion may continue and progress in the ‘pipe’ provided by the conduit. New linings to conduits, and filter collars in the surround to the spillways and conduits are precautions to prevent erosion.

11. SURVEILLANCE AND MONITORING

11.1 OBJECTIVES AND CHALLENGES OF SURVEILLANCE AND MONITORING FOR TIMELY DETECTION OF INTERNAL EROSION

This chapter gives an overview of the needs and methods for surveillance and monitoring of the dam after its vulnerability to internal erosion has been reviewed and it has been repaired or improved if the review has shown this to be necessary. More information on details of monitoring methods and equipment are given in Volume 2.

Following the review and improvements, the dam should have a low likelihood of failure by internal erosion, but it is not possible to be absolutely certain that it is so, and its condition may change over time. Therefore the dam must be kept under surveillance (particularly visual inspection) and monitored (using instrumentation) to detect potential vulnerabilities. The intensity and frequency of surveillance and monitoring should be designed to detect the onset of failure by internal erosion early enough in the failure process to give sufficient time to warn and evacuate people from the dambreak floodway downstream of the dam or to empty or lower water level in the reservoir before breach occurs.

Before designing a monitoring and surveillance system to detect the onset of internal erosion early in the failure process, the following points about internal erosion failures and their detection should be noted:

(a) For internal erosion in embankment failures, initiation is most common in cracks or hydraulic fractures in the embankment or around a conduit. These mechanisms could be expected to initiate very rapidly or rapidly once the reservoir level reaches the critical level at which erosion begins in cracks or the critical level at which hydraulic fracture initiates. However this depends on the rate of erosion of the soil.

(b) Initiation of erosion by suffusion (internal instability) is likely to be a more slowly developing process, accompanied by more gradual increases in seepage and changes in pore pressure with time.

(c) 'Blow-out' or 'heave' in dam foundations subject to backward erosion occurs when seepage forces create a zero effective stress condition. This should be readily detected by carefully positioned and well monitored piezometers. Often these low effective stresses occur below lower permeability layers which act to confine the seepage flow and it is important that piezometers are installed to measure pore pressures in these areas. These pressures are usually directly related to reservoir levels and it is important to monitor the relationship between the pore pressures and reservoir levels. Such pressures can often be released in controlled circumstances through filtered relief wells.

(d) Failures from internal erosion in the foundation and from the embankment to the foundation are mostly from backward erosion, following 'blow-out' or 'heave'. Or erosion into a drain or ditch which penetrates into the erodible layer. These would not necessarily be expected to be preceded by large increases in seepage during the time the erosion is gradually working back from the downstream exit point. When the erosion has progressed to within a short distance of the reservoir/foundation interface, it breaks through rapidly or very rapidly.

(e) The ease of detection of internal erosion in the foundation by visual and seepage means is more readily achieved if the area downstream of the dam is not vegetated, or is at least cleared of larger vegetation. Detection is difficult if the area downstream is densely vegetated or if the dam toe area is covered by water.
(f) Detection by pore pressure measurement is more likely if there is extensive instrumentation read regularly and if the erosion is widespread. However it is unlikely the piezometers will detect concentrated leak erosion unless the concentrated leak is associated with installation of the piezometers.
(g) The frequency at which seepage is measured can be related to the likelihood that a dam may fail or experience an accident by internal erosion and piping, the consequences of failure and the likely time for internal erosion to progress to form a pipe and the dam breach. It might also be related to the reservoir level, with enhanced measurements at times of high reservoir level. There is strong evidence that failures and accidents from internal erosion occur above or near historic high reservoir levels. At such times the dam is also at greatest risk from a slope instability point of view. For dams with known deficiencies in internal erosion, there should be a significantly increased level of inspection and monitoring under these reservoir conditions.
(h) The seepage monitoring system needs to be capable of being calibrated to separate out the effects of rainfall or snowmelt e.g. by prior observation and monitoring and coupling seepage measurements to rainfall and snowmelt measurements at the dam. It should however be recognized that there are many situations where it will be unlikely that seepage will be detected, e.g. at night when visual surveillance is being relied upon; if the toe of the dam is submerged; if seepage occurs high on the dam abutments and bypass measuring weirs or in winter when the dam is covered by snow.
(i) There are many dams which may have progression and breach times of the order of only hours. These dams particularly include some older dams which have no filter or transitions, dams without downstream rockfill zones to stop or slow the erosion process, and/or those dams on erodible foundations without well designed and constructed cut-offs or filters to intercept the foundation seepage. For these dams, an effective seepage monitoring program would require virtually continuous monitoring. Daily, or even twice daily inspections, or measurements may be inadequate. The installation of anti-erosion measures should be seriously considered at such dams (Section 9.14 and Chapter 10).

Dams analyzed and shown, after improvement, if necessary, to resist internal erosion following the methods in this bulletin, should no longer be vulnerable to the more obvious internal erosion failure paths identified in the examples above. However, because conditions may change over time, and the possibility of the development of internal erosion by some unforeseen route cannot be ruled out, a surveillance and monitoring regime should be provided at an intensity and frequency proportionate to the consequences of failure, the rate of progression to breach and the rate at which the reservoir can be emptied.

11.2 GENERAL PRINCIPLES

Internal erosion is difficult to detect. Internal erosion and piping could suddenly occur without significant warning beforehand. For instance, a homogeneous long embankment can be breached in less than one hour by internal erosion triggered by animal holes or cracks. The frequency of visual inspections may be ineffective. Piezometers in the lower portion of a long embankment often indicate low water levels, whilst leakage is apparent at higher elevations. Piezometers spaced at 200m intervals will not be sufficient to guarantee detection of leakage.

Dirty water on the surface of a dike is a sign of ongoing internal erosion; however turbid water flowing in the foundation may not be seen.

11.2.1 The likelihood of detection and intervention

A method for assigning likelihoods to detection and successful intervention is given in Table 2.3 based on Fell et al (2008).

It is known that most internal erosion failures in the embankment occur at reservoir levels close to or above historic high, and the physical processes are driven by the reservoir water. Hence a good monitoring and inspection program will have a greatly increased frequency of inspections and reading of critical instruments under such reservoir conditions. There should also be increased inspection and monitoring following any significant seismic event.

Detection may be possible in the continuation or early progression phase, or more likely, in the advanced stages of progression and breach formation. Detection is likely to be by:

- Observation of increased seepage or turbid seepage out of the downstream face of the embankment or in the foundation. This may be by visual observation, or by seepage measurement, or more sophisticated methods such as thermal monitoring of the foundation or the downstream slope.
- Measured higher pore pressures in the foundation and/or embankment.
- Settlements, deformation and cracking in the embankment or area downstream of the dam.

Whether detection is likely depends on:

- The rate at which the internal erosion and associated processes, such as instability of the downstream face, occurs.
- The frequency of inspections, and measurement of monitoring equipment.
- The dam zoning and the location of the concentrated leak and whether the leak will be visible to those doing the inspection.
- Design, location and sensitivity of monitoring equipment

For example if a process goes from initiation, or first presence of a concentrated leak, to breach in say 6 hours, and the dam is only inspected or monitored weekly, it is very unlikely that a piping incident will be detected before breach occurs. However if the dam is visible by the general population, there is some chance the leak may be noticed nonetheless.

Detection early in the internal erosion process is usually difficult, particularly for erosion initiating along a crack, or by backwards erosion because the amount of leakage is very small at the start. Fell et al (2001, 2003) record that most piping incidents are first identified as a concentrated leak in the progression phase. Suffusion is more likely to be detected by piezometers because the process is slower to develop. The presence of conditions potentially leading to heave and backward erosion in the foundation may also be detected by piezometers provided they are correctly positioned and read as reservoir levels rise.

Visual inspection is a vital tool in detecting internal erosion, whether it is successful is dependent on the factors discussed above, but also on such practical issues as:

- Inspections are seldom practical at night, so there is 30% to 50% of the time (varying throughout the year) when detection will not be effective, particularly for rapidly developing piping mechanism. Many dams are not inspected on weekends, further reducing the likelihood of detection.

- Dense vegetation, runoff from rainfall, snow cover can all hide the presence of a concentrated leak. However it can be the case that melted snow is a good indicator of areas affected by seepage.

- For very long embankments, it is not practical to walk to inspect, so it is less likely small leaks are detected.

11.2.2 Some Information on the Rate of Internal Erosion

The likelihood of detection leading to successful intervention and action to avert the ill-effects of the failure of the dam depends on the time from when the internal erosion process may be detected to when breach begins.

Fell et al (2001, 2003) studied case histories of failures and accidents for piping in the embankment, foundation, and embankment to foundation. Based on the case histories and an understanding of the physical processes they provided guidance on the time for progression beyond when a concentrated leak is first observed, and development of a breach.

Most of the cases studied were for breach by gross enlargement, so the method is applicable to cases where the mechanism is gross enlargement. It is considered to be reasonably applicable to cases where the final breach is by slope instability, following development of a pipe. It will probably underestimate the time for breach by sloughing. Sloughing is a slowly developing breach mode which should take days or weeks to lead to breach.

Breach by sinkhole development is potentially a rapid process in the final stages when the sinkhole emerges into the reservoir. The breach would be expected to occur in a small number of hours but there is not case data to support a more refined estimate.

Note that the dispersivity of the soil does not significantly affect the rate of erosion.

Fell et al (2001, 2003) show that the method gives a reasonable estimate of the time for progression beyond where a concentrated leak is observed and breach, and the times are acceptably accurate for the purpose here which is to assess the likelihood of detection, intervention and repair. Fell et al (2001, 2003) caution however, against over-reliance of these figures for life loss estimates where the estimates are sensitive to the assumed warning times.

11.2.3 A review of methods of monitoring and detection and their applicability to internal erosion modes

Tables 11.1, 11.2, and 11.3 summarize the usual rate of development, based on Fell et al (2001, 2003) as described above, ease of detection, and methods of detection in generic terms for internal erosion in the embankment, foundation and from the embankment to foundation. In these tables the terms are defined as follows:

Rate of development: Slow = weeks or months, even years; Medium = days or weeks; Rapid = hours (>12 hours) or days; Very Rapid = <3 hours.

Ease of detection: Difficult = unlikely to be detected in most cases; Moderate = may be detected in some cases; Readily = readily detected in most cases.

Methods of detection: Visual (inspection), seepage (measurement either visual or by instruments), pore pressure (measurement); survey (survey of surface markers, to determine horizontal and vertical deformation); thermal (thermal measurements within or on the downstream surface of the embankment including use of fiber optics); geophysical (resistivity and self potential).

The classifications for ease of detection assume the dam is well instrumented to measure seepage and pore pressure, readings are frequent and the dam is subject to regular inspection by experienced personnel. These classifications will be too optimistic if the dam and the area downstream are covered in vegetation, or snow covered for part of the year or if the toe of the dam is drowned by the river or another reservoir downstream.

Table 11.1 Rate of development, ease of detection and methods of detection of internal erosion in the embankment (adapted from Fell et al, 2005)

Phase of Develop-ment	Mechanism	Usual Rate of Development	Ease of Detection	Method of Detection
Initiation	Concentrated leak in cracks or hydraulic fracture, associated with conduits or walls	Rapid or very rapid	Difficult	Seepage, visual.
	Concentrated leak in a crack or in a hydraulic fracture	Rapid or very rapid	Difficult	Seepage, visual if the crack emerges on the downstream face
	Global backward erosion in the core	Medium to slow, potentially rapid to very rapid at the end	Difficult (unless observed at downstream face)	Pore pressure, seepage, thermal, geophysical Visual if sinkhole emerges on the downstream face.
	Contact	Normally slow, rapid failure can occur only if another process like piping or sliding is initiated by contact erosion	Monitoring of contact erosion can be achieved by visual observation of fines particles into leakage or by identification of sinkholes.	Detection of cavities in the dam by appropriate methods such as geophysics may also indicate the existence of contact erosion. When contact erosion is identified, a decrease of the flow velocities where it takes place, can reduce or stop the phenomenon
	Suffusion /internal instability	Slow	Moderate to difficult	Pore pressure, seepage, thermal, geophysical, visual if emerging on the downstream face
Contin-	Filters satisfying	Erosion will		

Phase of Develop-ment	Mechanism	Usual Rate of Development	Ease of Detection	Method of Detection
uation	no-erosion criterion	cease		
	Filters satisfying excessive erosion criterion	Rapid to very rapid	Moderate to difficult	Seepage, pore pressure
	Filters not satisfying continuing erosion criterion, or no filters	Rapid to very rapid	Moderate to difficult	Seepage, pore pressure
Progression to form a pipe, and a breach		Assess time using method in Section 11.2.2.		
	Gross enlargement; and slope instability linked to development of a pipe.	Very rapid or rapid Medium or slow	Moderate. to readily Readily	Seepage, visual
	Crest settlement or/and sinkhole in embankment	Usually slow to medium.	Readily	Visual, survey, seepage
	Slope instability, unraveling or sloughing	Usually slow to medium unless linked to rapid development of pipe.	Readily to Moderate	Visual, survey, seepage

It should be noted that monitoring of seepage, either by visual surveillance or measurement, is the most common means of identifying whether internal erosion is occurring.

It is not common to have sufficient change in the seepage, or in other factors such as pore pressure changes or settlement, to identify conclusively that internal erosion has initiated and is continuing. It is more common to recognize when the erosion has progressed to the stage that a pipe has developed, or recognize that changes in pore pressures, seepage or settlement have occurred, the cause of which is not clear. Such changes may be a pre-cursor to internal erosion and piping, so it is important they are observed and, when they occur, investigated.

Table 11.2 Rate of development, ease of detection and methods of detection of internal erosion in the foundation and from embankment to foundation (adapted from Fell et al, 2005)

Phase of Development	Mechanism	Usual Rate of Development	Ease of Detection	Method of Detection
Initiation - Foundation	Backward erosion	Slow	Difficult	Visual, seepage, pore pressure, thermal, geophysical
	Backward erosion following “blow-out”	Rapid to very rapid	Readily to difficult	Visual, seepage, pore pressure, thermal, geophysical
	Concentrated leak erosion	Rapid or very rapid	Moderate to difficult	Visual, seepage
	Suffusion /internal instability	Slow	Moderate to difficult	Visual, seepage, pore pressure, thermal, geophysical
Initiation - embankment to foundation	Backward erosion	Slow, potentially rapid to very rapid at the end	Difficult	Pore pressure, seepage
Initiation – Embank-ment to foundation	Contact erosion	Slow	Difficult	Pore pressure, seepage, survey, thermal, geophysical
Continuation	Filters satisfying no erosion criterion	Erosion will cease if all seepage is intercepted		
	Filtered exit, satisfying excessive erosion, or incomplete seepage interception	Rapid to very rapid	Moderate to difficult	Seepage, pore pressure
	Filtered exit, not satisfying continuing erosion criterion, or unfiltered exit	Rapid to very rapid	Moderate to difficult	Seepage, pore pressure
Progression to form a pipe, and a breach		Assess time using method in Section 11.2.2.		
	Gross enlargement in the foundation or in the embankment; and slope instability linked to development of a pipe.	Very rapid or rapid Medium or slow	Moderate to readily Readily	Seepage, visual Visual, survey, seepage
	Crest settlement or/and sinkhole in embankment	Usually slow to medium.	Readily	Visual, survey, seepage

Phase of Development	Mechanism	Usual Rate of Development	Ease of Detection	Method of Detection
	Slope instability, and unraveling or sloughing, for internal erosion embankment to foundation	Usually slow to medium unless linked to rapid development of pipe.	Moderate to readily	Visual, survey, seepage

Table 11.3 Rate of development, ease of detection and methods of detection of internal erosion into and along conduits or adjacent to walls (adapted from Fell et al, 2005)

Phase of Development	Mechanism	Usual Rate of Development	Ease of Detection	Method of Detection
Initiation	Concentrated leak in crack or hydraulic fracture	Rapid or very rapid	Difficult	Visual, seepage
	Erosion into open crack or joint in the conduit or wall	Slow	Moderate	Visual, seepage
Continuation: Concentrated leak in crack or hydraulic fracture	Filters (A) satisfying no erosion criterion	Erosion will cease		
	Filtered exit, satisfying excessive erosion criterion	Rapid to very rapid	Moderate to difficult	Seepage, pore pressure
	Filtered exit, not satisfying continuing erosion criterion, or unfiltered exit	Rapid to very rapid	Moderate to difficult	Seepage, pore pressure
Continuation: Erosion into open crack or joint in the conduit or wall	Crack or joint width satisfies excessive erosion criterion	Slow (C)	Moderate to difficult (B)	Seepage, pore pressure
	Crack or joint width does not satisfy continuing erosion criterion	Slow to medium (C)	Moderate to difficult (B)	Seepage, pore pressure
Progression to form a pipe and a breach		Assess time using method in Section 11.2.2.		
	Gross enlargement; and slope instability linked to development of a pipe	Very rapid or rapid	Moderate to readily Readily	Seepage, visual

Phase of Development	Mechanism	Usual Rate of Development	Ease of Detection	Method of Detection
	Crest settlement or/and sinkhole in the embankment	Medium or slow Usually slow to medium.	Readily	Visual, survey, seepage
	Slope instability, unraveling or sloughing	Usually slow to medium unless linked to rapid development of pipe.	Moderate to readily	Visual, survey, seepage
Conduits and walls	Erosion into a conduit or crack in a wall, usually only progresses towards breach by initiation of erosion along the conduit or wall, in which case factors are as for gross enlargement and instability. For cases which continue towards development of a sinkhole in a crest mode of breach, ability to hold a roof/or sinkhole is critical and the rate of development is usually slow, readily detectable by visual, survey or seepage.			

Notes:

(A) Filters are those controlling internal erosion around the conduit, or adjacent walls.

(B) Often the crack and seepage is visible on inspection, but erosion may be intermittent, or only when conduit is flowing with water, so not readily observed.

(C) The evidence seems to be that even for conduits surrounded by erodible soils, e.g. fine sand, the rate of erosion into the conduit is slow.

12. METHODS FOR ASSESSMENT OF THE LIKELIHOOD OF FAILURE OF A DAM BY INTERNAL EROSION

12.1 INTRODUCTION

As discussed in Section 9.2 there are a number of approaches for assessing a dam for internal erosion. These are:

(1) **Deterministic:** This can be regarded as a load versus resistance approach using the Spillway Design Flood and Maximum Design Earthquake.

(2) **Engineering Judgment:** This can also be regarded as a load versus resistance approach, but applied not just to the Spillway Design Flood and Maximum Design Earthquake, so engineering judgment or qualitative risk assessments can be made.

(3) **Quantitative Risk Assessment**, which requires more formalized estimates of probabilities of failure and the consequences.

This section provides information on how the likelihood of failure of an existing dam by internal erosion can be determined. This Bulletin does not set out to be a guideline on risk assessment. It provides information which may be used in such assessments. For detailed information on risk management see ICOLD Bulletin 130, Risk Assessment in Dam Safety Management (ICOLD, 2005); ANCOLD Guidelines on Risk Assessment (ANCOLD, 2003), Hartford and Baecher (2004), Risk and Uncertainty in Dam Safety, and UK Risk Assessment and Reservoir Safety (Environment Agency, 2013). These publications also give guidance on methods for estimating loss of life and financial and economic losses.

12.2 GENERAL PROCEDURE

The general procedure for estimation of the likelihood of failure a dam by internal erosion in the embankment, the foundations, and from embankment to foundations is very similar to that for the engineering analysis described in Sections 9.10 and 9.11, but ultimately requires that the likelihood (probability) of failure be assessed:

1. Assemble all the relevant information about the dam and its foundations as described in Section 9.8.
2. Determine the loading conditions for the dam resulting from the reservoir and earthquakes as described in Sections 9.3, 9.4, 9.5 and 9.6 and Section 3.4 in relation to concentrated leaks.
3. Carry out a detailed Potential Failure Modes Analysis (PFMA) as described in Section 9.9.
4. Screen out PFM which are not applicable to the dam or which will contribute a negligible amount to the likelihood of failure. Some guidance on how this may be done is given in Section 9.10. Screening may be done on the zoning of the dam and the foundation conditions, or a more detailed understanding of the factors influencing the initiation and continuation of erosion.
5. For the remaining PFM use engineering judgment, qualitative or quantitative risk analysis methods to estimate the likelihood of failure taking account of the frequency of the applied reservoir and seismic loads. Some guidance on how this may be done is given in Section 12.4.

12.3 ESTIMATION OF LIKELIHOODS OF FAILURE FOR THE POTENTIAL FAILURE MODES APPLICABLE TO THE DAM

12.3.1 Some general principles

There are some general principles which should be applied when estimating the likelihood of a PFM leading to failure. They are similar to those in Section 9.11.1, except that a range of loads applied at different frequencies (e.g. varying water levels) must be considered.

12.3.2 Concentrated leak erosion-some aids to judgment

Fell et al (2008) use tables which for each PFM of concentrated leak erosion list the factors which most affect the likelihood of a crack and hydraulic fracture. These are assigned a relative importance factor (RF) and likelihood factor (LF) and the product is summed to give an overall $\sum$ (RF x LF), between 6 and 24. In Fell et al (2008) these are then linked to estimate conditional probabilities of cracking or hydraulic fracture. The tables are included in the "Piping Toolbox" described in Section 12.4.2, which also gives sources of copies of the document.

12.4 QUANTITATIVE AND SEMI QUANTITATIVE RISK ASSESSMENT METHODS

12.4.1 Methods based on historic databases of failures and incidents

12.4.1.1 Interim Guide to Quantitative Risk Assessment for UK Reservoirs

The Interim Guide to Quantitative Risk Assessment for Reservoirs in the United Kingdom (Brown & Gosden, 2004), is a quantitative method, quantified on the historic performance of the large population of similar dams in the United Kingdom, for which records of incidents of poor performance have been kept for many years (Brown & Tedd, 2003). The guide has a comprehensive text and includes probability assessments estimated on spreadsheets for the major threats, overtopping, stability and particularly, internal erosion; a simple dambreak assessment to estimate the consequences of failure, and guidance on unacceptable and acceptable risk, with emphasis on following the ALARP principle to maintain the probability of failure as low as practicable at proportionate cost.

The method was developed from an earlier scoping project by KBR (2002) to compare the probability of failure due to floods with internal erosion, the project being managed under a Steering Group appointed by Defra and including trials of the prototype risk assessment methodology on ten UK dams.

The methodology is shown on Figure 4.1 of the Guide and comprises devising Current and Intrinsic Condition scores in ranges of 0 to 10, and then mapping these onto a probability distribution based on historic performance of UK dams. Default values of annual probability of failure are given in Table 4.2 of the Guide (reproduced here in Table 12.1), with adjustments for dam type included in Sheet 4.3 of the workbook. Experience in application of the Guide (Brown & Gosden, 2006, Brown et al, 2008) included that some panel engineers found difficulty in use of Excel workbooks with multiple sheets, and that guidance on scoring, particularly where there was no data on dam construction, should be improved to obtain better consistency.

This method is only applicable to UK dams which are mostly puddle clay core dams, a design not used widely elsewhere. There are also many ‘homogeneous’ dams. Both types do not have filters. The probability elements of the method are derived from the performance of these dams in UK and cannot strictly be applied elsewhere, unless users can identify substantial similarities to their dams.

Table 12.1 Suggested default values of probability of failure for anchor points for internal erosion for UK dams

Current Condition score	Description	Dams regulated under Reservoirs Act 1975 (Table 4.2 of Brown & Gosden, 2004)		Small dams which do not come under legislation (Table 4.5 of Brown & Gosden, 2004)	
		Internal stability (embankment)	Internal stability (appurtenant works)	Internal stability (embankment)	Internal stability (appurtenant works)
10	Emergency drawdown considered necessary to avert failure	1.4×10^{-2}	1.0×10^{-2}	3.8×10^{-2}	2.6×10^{-2}
8	Concern over behavior leading to works outside periodic safety review	3.8×10^{-4}	2.7×10^{-4}	3.3×10^{-3}	2.3×10^{-3}
0	Best condition dam	4.7×10^{-8}	$3.3. \times 10^{-8}$	As dams under the Act	

An updated UK guide will soon be published (Environment Agency, 2013). It updates the 2004 approach but remains focussed on UK dams. The updated UK guide also recommends that this Bulletin be used to examine the mechanics of internal erosion in conjunction with the quantitative risk assessment.

12.4.1.2 UNSW Method

The UNSW method is described by Foster et al (2000(a), (b)). The database of dam incidents was compiled primarily from the three ICOLD studies (ICOLD 1974, 1983 and 1995) and supplemented with additional incident cases from the other existing compilations, from the literature and dam engineering organizations.

The method allows estimation of the annual probability of a dam failing within the embankment, in the foundation and from embankment to foundation based on the dam zoning, the material in the core, the filters, foundation geology and cut-off details.

The method allows a broad categorization of the dam types into those less and more likely to fail. However the authors of the method have found that it does not allow for important details of the dam to be allowed for, and it cannot model the frequency of reservoir loading which is an important factor in estimating the likelihood of failure. They prefer to use event tree methods such as the "piping toolbox" described below.

12.4.2 Event tree methods

Event tree methods have been used by a number of organizations including USBR, USACE, and Australian Dam organizations since the late 1990's. The methods have been refined with time and are still being refined as the understanding of the mechanics of internal erosion is improved.

These methods mostly rely on "expert opinion elicitation" (e.g. Brown & Aspinall, 2004) to estimate conditional probabilities within the event trees. They are successful provided those doing the analysis are familiar with the mechanics of internal erosion and quantitative risk analysis. Because of this their use has been largely restricted to USBR, USACE, and Australian, USA and UK Consultants.

The "Piping toolbox" or "Risk analysis for Dam Safety, A Unified Method for Estimating Probabilities of Failure of Embankment Dams by Internal Erosion and Piping" is an event tree system for estimating the annual probability of breach of embankment dams by internal erosion and piping in the embankment, the foundation and from the embankment into the foundation.

It was developed as a unified approach for carrying out risk analyses by the United States Bureau of Reclamation (USBR), United States Army Corps of Engineers (USACE), Consulting Engineers URS and the University of New South Wales.

The document (Fell, Foster, Davidson, Cyganiewicz, Sills and Vroman (2008), see references for details) is in two parts, a Guidance Document, and a Supporting Information Document. It is available as a University of New South Wales UNICIV Report in electronic form by emailing Emeritus Professor Robin Fell, UNSW, r.fell@unsw.edu.au, or Dr Mark Foster, URS Australia, mark.a.foster@urs.com. Note that it includes the tables referred to in 12.3.2 giving guidance on cracks and concentrated leak erosion.

The "Piping toolbox" is designed for use in quantitative risk analysis. If it is to be used for that purpose the Development Team and their organizations strongly recommend that all risk analysts that will use this toolbox be trained in its use. The complexity of the issues and importance of the end product demands that all analysts fully understand the methodology.

The key goals of the training would be to provide an understanding of all features and components of the methodology; to outline for the analyst the supporting information and background that was used in the development of the methodology; and to guide the analyst through a detailed, real life example use of the methodology.

The authors of the Toolbox have seen a number of examples where the lack of adequate training has resulted in incorrect application of the methods.

Some perspectives on the use of the Toolbox in the United Kingdom are given in Eddleston et al (2009, 2010)

It is recognized that not many will wish to use the document for quantitative risk analysis. It is however a valuable document for those managing the safety of dams for internal erosion. The Tables of Conditional Probabilities it contains are a useful guide to the factors which affect the likelihood of initiation, continuation, progression and breach, and of the likelihood of successful detection and intervention.

An electronic copy of the document can be obtained by emailing Emeritus Professor Robin Fell, UNSW, r.fell@unsw.edu.au, or Dr Mark Foster, URS Australia, mark.a.foster@urs.com. Note that it includes the tables referred to in 12.3.2 giving guidance on cracks and concentrated leak erosion.

13. TERMINOLOGY – INTERNAL EROSION PROCESSES – INTERNATIONAL GLOSSARY

13.1 TERMINOLOGY USED IN RELATION TO INTERNAL EROSION

The following terminology is used in relation to internal erosion and piping.

It is important that all those working in the area adopt these terms so as to use a common language to describe the processes and avoid confusion. The terminology is based on that adopted at the International Workshop on Internal Erosion of Dams and their Foundations, held in Aussois, France in April 2005, (Fell and Fry, 2007).

Backward erosion: Backward erosion involves the detachment of soils particles when the seepage exits to a free unfiltered surface, such as the ground surface downstream of a soil foundation or the downstream face of a homogeneous embankment or a coarse rock fill zone immediately downstream from the fine grained core. The detached particles are carried away by the seepage flow and the process gradually works its way towards the upstream side of the embankment or its foundation until a continuous pipe is formed.

There are two forms of backward erosion, “backward erosion piping” in which the roof of the pipe is formed by a cohesive soil layer of embankment and the pipe is essentially horizontal (Figure 2.5); and “global backward erosion” where near-vertical pipes form within broadly graded silt sand gravel (non-plastic) cores of embankments (Figure 2.7).

Breach initiation: The initiation of breach is the final phase of internal erosion. It may occur by one of the following five phenomena (listed below in order of their observed frequency of occurrence):

- **Gross enlargement** of the pipe.
- **Overtopping** (e.g. due to settlement of the crest from suffusion and/or due to the formation of a sinkhole from a pipe in the embankment).
- **Slope instability** of the downstream slope
- **Unraveling** of the downstream face.
- **Static liquefaction:** A form of slope instability in loose non-plastic soils which involves a large increase of pore pressure and loss of undrained strength.

Breach process: There are three phases in the breach process, initiation, formation and development. The breach initiation is where there is an uncontrolled release of water from the reservoir but the crest does not collapse, the breach formation occurs after the crest collapses up to the erosion of the whole section of the dam and the development where there is finally the enlargement of the breach width by side erosion.

Concentrated leak erosion: Where there is an opening, through which concentrated leakage occurs, the walls of the opening may be eroded by the leaking water. Such concentrated leaks may occur through a crack caused by settlement or hydraulic fracture in a cohesive clay core for example, or through desiccation and tension cracks at high levels in the fill, or through cracks resulting from settlement of fill. In some circumstances, these openings may be sustained by the presence of structural elements such as spillways and pipes, or by the presence of cohesive materials able to 'hold a roof', as it is described, below which an opening is sustained, the periphery of which is eroded. It may occur in a continuous zone containing coarse and/or poorly compacted materials which form an interconnecting voids system. The concentration of flow causes erosion (some organizations call this scour) of the walls of the crack or interconnected voids.

Contact erosion: (also known as **parallel contact erosion**): Soil contact erosion is a form of internal erosion which involves selective erosion of fine particles from the contact with a coarser layer, caused by the flow passing through the coarser layer. For instance along the contact between silt and gravel sized particles. It relates only to conditions where the flow in the coarser layer is parallel to the interface between the coarse and fine layer (Figure 13.1). Selective erosion of fine particles from the contact with a coarser layer caused by the flow through the fine layer with flow normal to the contact surface is considered by another phase of the erosion process: filtration or continuation.

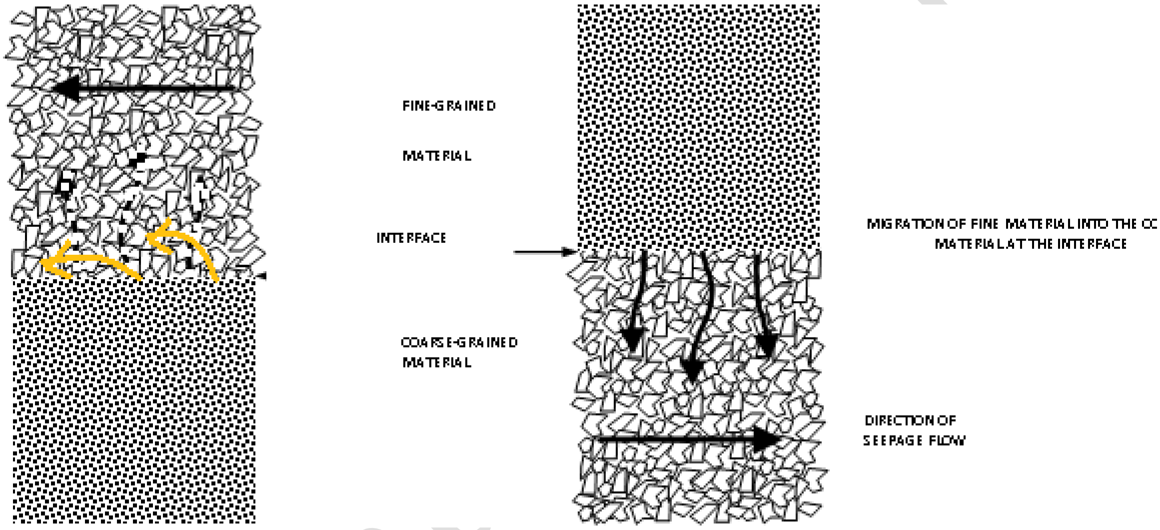


Figure 13.1 Diagram of soil contact erosion.

Continuation (filtration): Continuation is the phase where the relationship of the particle size distribution between the base (core) material and the filter controls whether or not erosion will continue. Foster and Fell (2001) and Foster (1999) define four levels of severity of continuation; no erosion, some erosion, excessive erosion and continuing erosion. These are shown conceptually in Figure 13.2 and defined individually below:

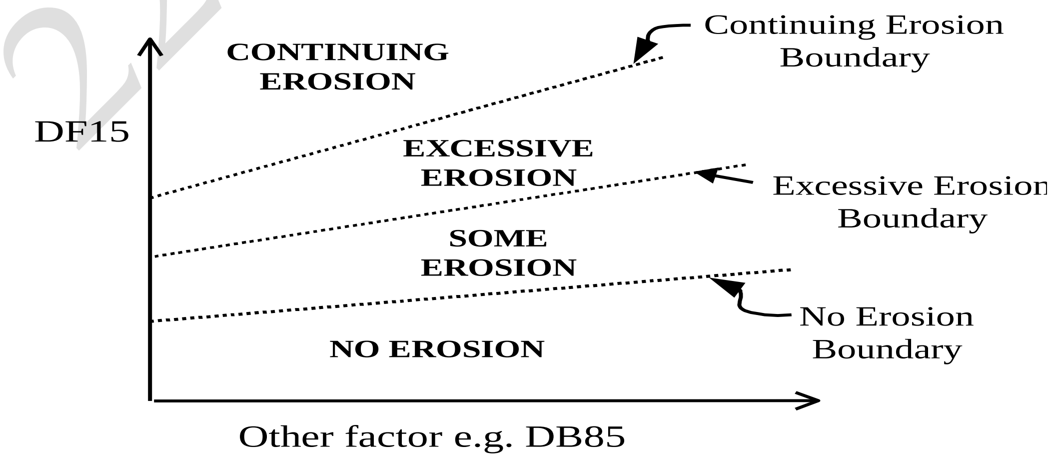


Figure 13.2 Conceptual filter erosion boundaries (Foster, 1999), Foster & Fell (2001)

- **Continuing erosion:** The filter is coarser than the continuing erosion criteria and is too coarse to allow the eroded base materials to seal the filter allowing unrestricted erosion of the base soil.
- **Excessive erosion** defines conditions where erosion of the base soil will be excessive before it seals. Filters between the excessive erosion boundary and the continuing erosion boundary will eventually seal but only after significant erosion of the base soil. In dams there may be large leakage flows before the filter seals by clogging of the surface of the filter by eroded base soil.
- **Some erosion:** The filter is between the no erosion and excessive erosion boundaries. The filter quickly seals after particles of the base material clog the surface of the filter.
- **No erosion:** The filter is finer than the no erosion criterion and seals with no or practically no erosion of the base material. Filters designed and constructed according to modern filter design criteria will satisfy no erosion criteria.

Detachment: Detachment is the first stage of the erosion process. Particle detachment occurs by the hydraulic shear forces developed by the seepage flow velocity. The mechanics are determined by whether the soil is cohesive or cohesionless.

Detection: Detection is the ability to detect the internal erosion process during one of the erosion phases. Currently, internal erosion is most easily detected in the progression or breach phases.

Dispersion (also known as **deflocculation**) is a physico-chemical phenomenon contributing to concentrated leak erosion.

Soils in which the clay particles will detach from each other and from the soil structure without a flow of water, and go into suspension are termed dispersive clays.

The dispersivity of a soil is directly related to its clay mineralogy. In particular soils with a high exchangeable sodium percentage such as montmorillonite present, tend to be dispersive, while kaolinite and related minerals (e.g. halloysite) are non dispersive. Soils with illite present tend to be moderately dispersive.

The dispersivity depends also on the pore water chemistry. Low pore water salt concentrations lead to greater dispersivity and high salt concentrations can suppress dispersion in susceptible soils.

Erosion of a soil particle is the detachment and transport of that particle by forces caused by seepage flow (drag and lift forces, which are related to viscosity and turbulent shear stresses). Detachment is a first condition expressing that drag and lift forces are larger than resisting forces. Transport is the second condition expressing first a hydro-mechanical criterion that the drag forces are larger than the rolling or sliding resistance of the buoyancy weight of the particle and secondly a geometric criterion: voids or cracks exist which are large enough for eroded particles to pass through. This last criterion makes the difference between internal and external erosions.

Failure process: Failure process describes how the dam could fail by internal erosion for the particular loading condition. It includes the definition of loading, the internal erosion path and internal erosion phases (initiation of erosion, continuation, progression, failure to intervene, and breach). In event tree analysis, each internal erosion failure process is represented by one complete event tree.

Failure mode: Failure mode describes how the dam could fail for the particular loading condition. The three principle failure modes for embankment dams are overtopping due to inadequate spillway capacity, internal erosion and slope instability. For internal erosion, it includes the definition of loading, the internal erosion path and internal erosion phases (initiation of erosion, continuation, progression and initiation of breach). In event tree analysis, each failure mode is represented by one complete event tree.

Flaw: A flaw is a physical condition that allows seepage concentration Examples include continuous crack; or poorly compacted layer in which a concentrated leak may form.

Heave (also known as **'blow out'**) occurs in cohesionless soils which are confined by an overlying lower permeability stratum when seepage pore pressures are such that the effective stress becomes zero (pore pressure equals total stress). 'Heave' may often be followed by backward erosion of the cohesionless soil if the seepage gradients remain high at the surface.

Hydraulic fracture occurs in the core or foundation of embankment dams when the pore pressure exceeds the minor principal stress and the effective stress in the core or foundation becomes zero (or even slightly negative if the soil can withstand tensile stresses). The pressure of the water seeping through the core from the reservoir exceeds the remaining compressive stress and forms a crack or further opens an existing crack in which internal erosion by concentrated leak may initiate.

Initiation: Initiation is the first phase of internal erosion, when one of the phenomena of detachment of particles occurs. Four initiation phenomena are defined: concentrated leak, backward erosion, suffusion and soil contact erosion (each of which are defined separately in this section).

Internal erosion occurs when soil particles within an embankment dam or its foundation, are carried downstream by seepage flow. Internal erosion can initiate by concentrated leak erosion, backward erosion, suffusion and soil contact erosion.

Internal erosion path is the path of moving eroded particles inside the dam and/or its foundation.

Internal erosion phases (or **internal erosion mechanisms)** are the mechanisms during an internal erosion process leading to failure. Internal erosion of embankment dams and their foundations can be represented by four phases (or mechanisms):

1. **Initiation** of erosion.
2. **Continuation** of erosion (i.e. whether there are filters capable of stopping the erosion process).
3. **Progression** to form and sustain a pipe and/or to increase seepage and pore pressures in the downstream part of the embankment or its foundation.
4. **Breach** initiation resulting in uncontrolled release of the water from the reservoir.

Internal Instability: In soils subject to internal instability finer particles in the soil are able to move within the soil mass under the forces imposed on the particles by seepage flow. The phenomenon does not require a crack within the soil in which erosion may occur as is required in concentrated leak erosion, or a free surface from which particles detach and a roof of cohesive soil as is required for backward erosion.

Coarse graded and gap graded soils, such as those shown schematically in Figure 13.3 are internally unstable. In these circumstances, the internal instability is known as suffusion. The term internal instability has sometimes been used in the literature to describe internal erosion of soils which do not self filter, and internal erosion of broadly graded soils which do not satisfy the rules for suffusive soils but which erode under very high gradients.

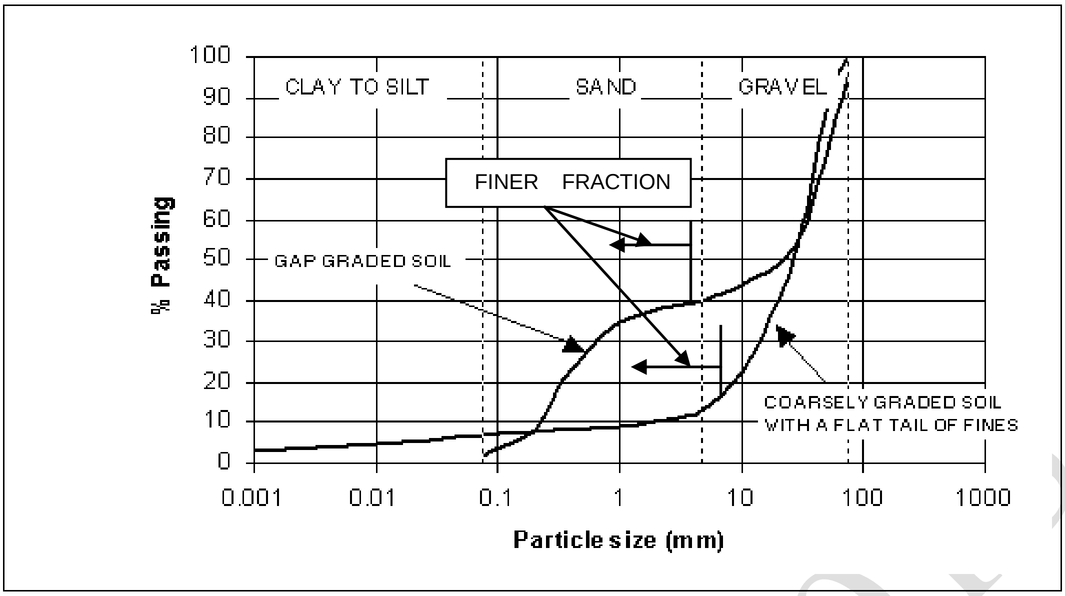


Figure 13.3 Soil gradation types which are potentially internally unstable and susceptible to suffusion

Intervention: Intervention is the ability to stop the internal erosion process during one of the first three phases and prior to the initiation of a breach.

Load: The main loads exerted on a dam are hydrostatic loads (which are related to the reservoir level) and seismic loads (which may be expressed in terms of earthquake magnitude and peak ground acceleration [PGA]). Other potential loads include acts of terrorism or an aero plane crashing into the dam. The likelihood of a given reservoir level load is expressed as the annual probability or frequency that the maximum reservoir level is within a certain range. The likelihood of a given seismic load is expressed as the annual probability or frequency of a loading parameter (e.g. peak ground acceleration) being in a certain range.

Load Condition: A load condition is a theoretical combination of different loads exerted on a dam at a given time for analysis purposes. The loading condition is the first node of the internal erosion assessment. The loading condition controls whether certain failure processes are possible, and affects the likelihood of each of the internal erosion phases.

Location of initiation of erosion: This is the physical initiating point of internal erosion in the structure such as in the embankment, foundation, along a conduit or along a spillway interface.

Piping: Piping a potential progression phase of internal erosion which initiates by backward erosion, or erosion in a crack or high permeability zone, and results in the formation of a continuous tunnel called a ‘pipe’ between the upstream and the downstream side of the embankment or its foundation. Internal erosion is commonly described as ‘internal erosion and piping’ but piping is actually the culmination of a process of erosion in which a number of phases must occur and be sustained in order that a ‘pipe’ develops through the dam or its foundation and allows the passage of considerable quantities of water which may lead to a breach.

Progression: Progression is the phase of internal erosion where hydraulic shear stresses within the eroding soil may or may not lead to the erosion process being on-going, and in the case of backward and concentrated leak erosion to formation of a pipe. The main issues are whether the pipe will collapse, or whether upstream zones may control the erosion process by flow limitation.

Self-filtering: In soils which self-filter, the coarse particles prevent the internal erosion of the medium particles, which in turn prevent erosion of the fine particles. Soils which potentially will not self-filter include clayey sandy gravels, soils which are susceptible to suffusion, and very broadly graded soils such as those which fall into the grading envelope shown in Figure 13.4.

Movement of the particles in such broadly graded soils may be in concentrated leaks in cracks or openings caused by hydraulic fracture as proposed by Sherard (1979).

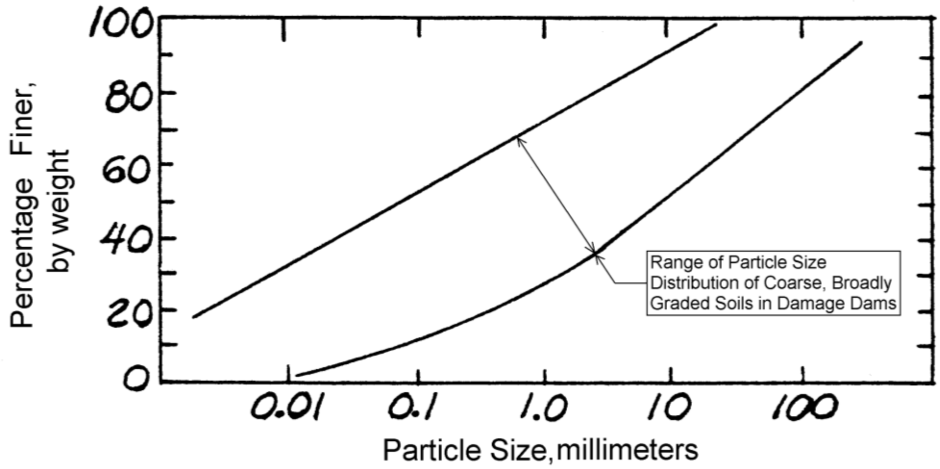


Figure 13.4 Examples of grading envelopes of some broadly graded soils which did not self filter (Sherard, 1979)

Sinkhole: A sinkhole is a cavity formed in the core of a dam or in a dike which may form as a result of erosion initiated by a concentrated leak, contact erosion or suffusion, or erosion into open joints in a foundation or conduit. The sinkhole cavity is usually near vertical and forms above the zone or point where erosion initiates. The progression of development of sinkholes is often a slow process with soil falling from the top of the sinkhole being carried away by the initiating erosion mechanism. Sinkholes may take years to manifest themselves at the crest of the dam or dike

Static Liquefaction: Static liquefaction occurs in loose saturated contractive cohesionless soils. As shear strains develop under the static loads in the slope, the pore pressures in the soil increase significantly and effective stresses approach zero, resulting in a large reduction in shear strength in undrained loading. The soil may exist in a contractive state in its natural condition or may become contractive as a result of suffusion.

Suffusion is a form of internal erosion of internally unstable soils which involves selective erosion of finer particles from the matrix of coarser particles, in such a manner that the finer particles are removed through the voids between the larger particles by seepage flow, leaving behind a soil skeleton formed by the coarser particles. The volume of finer particles is such that they fit within the voids formed by the coarser particles, that is the voids are under-filled. Suffusion usually involves little or no change in volume of the soil mass. Suffusion occurs at vertically upward seepage gradients less than the Terzaghi critical gradient and the effective stresses are carried largely by the coarser particles. *This phenomenon is sometimes referred to as suffosion in the literature.*

13.2 DEFINITIONS RELATED TO THE ANALYSIS OF THE PROBABILITY OF FAILURE

The following definitions relate to the analysis of the probability of failure, more details are given in ICOLD Bulletin 130:

Annual Exceedance Probability (AEP): The estimated probability that an event of specified magnitude will be exceeded in any year.

Frequency: A measure of likelihood expressed as the number of occurrences of an event in a given time or in a given number of trials (see also likelihood and probability).

Likelihood: Conditional probability of an outcome given a set of data, assumptions and information. Also used as a qualitative description of probability and frequency.

Probability: A measure of the degree of certainty. This measure has a value between zero (impossibility) and 1.0 (certainty). It is an estimate of the likelihood of the magnitude of the uncertain quantity, or the likelihood of the occurrence of the uncertain future event. There are two main interpretations from ICOLD:

Statistical – ***frequency*** or fraction – The outcome of a repetitive experiment of some kind like flipping coins. It includes also the idea of population variability. Such a number is called an "objective" probability because it exists in the real world and is in principle measurable by doing the experiment.

Subjective probability (degree of belief) – Quantified measure of belief, judgment, or confidence in the likelihood of an outcome, obtained by considering all available information honestly, fairly, and with a minimum of bias. Subjective probability is affected by the state of understanding of a process, judgment regarding an evaluation, or the quality and quantity of information. It may change over time as the state of knowledge changes.

Risk assessment: The process of reaching a decision recommendation on whether existing risks are tolerable and present risk control measures are adequate, and if not, whether alternative risk control measures are justified or will be implemented.

Screening: An initial assessment of the risk of internal erosion at a dam, used to inform the need for further investigations or studies, or whether the risk is tolerable

13.3 ICOLD DEFINITIONS OF LARGE DAM, FAILURE, ACCIDENT AND INCIDENT

Large dam: ICOLD define a large dam as a dam which is more than 15 meters in height (measured from the lowest point in the general foundations to the crest of the dam), or any dam between 10 and 15 meters in height which meets one of the following conditions:

- the crest length is not less than 500 meters
- the capacity of the reservoir formed by the dam is not less than one million cubic meters
- the maximum flood discharge dealt with by the dam is not less than 2000 cubic meters per second
- the dam is of unusual design.

Accident: Three categories of accidents are listed by ICOLD (1974):

- Accident Type 1 (A1): An accident to a dam which has been in use for some time but which has been prevented from becoming a failure by immediate remedial measures, including possibly drawing down the water.
- Accident Type 2 (A2): An accident to a dam which has been observed during the initial filling of the reservoir and which has been prevented from becoming a failure by immediate remedial measures, including possibly drawing down the water.
- Accident Type 3 (A3): An accident to a dam during construction, i.e. settlement of foundations, slumping of side slopes etc., which have been noted before any water was impounded and where the essential remedial measures have been carried out, and the reservoir safely filled thereafter.

Failure: Collapse or movement of part of a dam or its foundation, so that the dam cannot retain water. In general, a failure results in the release of large quantities of water, imposing risks on the people or property downstream (ICOLD, 1995). Incidents to dams during construction are only considered as failures when large amounts of water were involuntarily released downstream. Two categories of failures are listed by ICOLD (1974):

- Failure Type 1 (F1): A major failure involving the complete abandonment of the dam.
- Failure Type 2 (F2): A failure which at the time may have been severe, but yet has permitted the extent of damage to be successfully repaired, and the dam brought into use again.

Incident: Either a failure or accident, requiring major repair.

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