Granular Filter Design and Performance Considerations for Aging Embankment Dams

by

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A project report submitted in partial fulfillment of the requirements for the degree of

Master of Engineering

in

Geotechnical Engineering

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Abstract

Embankment dams have been constructed in Canada and the United States for over one hundred years. Embankment dams are owned by government bodies, mining and resource companies, hydroelectric producers, and other entities. Dams that have been in service for many years may have a high hazard rating due to the consequences of potential failure, as development tends to expand towards these structures over time. Older embankment dams precede the development of modern filter design criteria, which were first developed in the 1920s and have undergone considerable progress and refinement since. The lack of appropriate filter design in aging dams may increase risk of internal erosion and eventual failure. Risk assessment procedures and monitoring programs are the primary tools used by dam owners and operators to evaluate the risk of and detect potential failure events.

A brief summary of modern filter design has been presented. Traditional particle-sized based criteria were reviewed and recent developments in constriction-based criteria were discussed. Considerations for challenging base soils, such as dispersive and broadly graded soils, were provided.

The effects of aging on dam filter properties and performance were discussed. Evidence of filter degradation over time exists in the literature and has been attributed to several causes. These factors include filter clogging, changes in water quality, mechanical degradation, and the development of internal instability and each has been examined.

A discussion of risk assessment and monitoring for aging dams has been provided. Three of the most widely-used methods were summarized. Most risk assessment methods use a potential failure mode analysis framework. The numerous uncertainties associated with an aging structure presents challenges when attempting to apply potential failure mode analysis. The results of a risk assessment investigation are typically used to identify areas that may require additional investigation, potential remediation actions, or increased monitoring. They are also used as a basis for emergency planning. Monitoring techniques provide information about the location and rate of seepage in and through the embankment. Monitoring is typically the first indication of an internal erosion event that may lead to failure. Improvements in monitoring techniques have decreased detection time and increase likelihood of successful interventions that prevent failure.

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1.0 Introduction

Canada is rich in water resources, and dams have been an important engineering tool for managing them. According to the Canadian Dam Association (2019), owners of dams include both corporate entities such as resource extraction companies, electricity producers, and agricultural groups and public organizations from local municipalities to provincial and federal governments (Canadian Dam Association, 2019). There are approximately 14,000 dams in Canada, and over 1,100 of these meet the International Commission on Large Dams (ICOLD) classification for large dams (Canadian Dam Association, 2019). ICOLD defines a large dam as: a dam over 15 metres in height from the lowest point in the foundation to the crest; or a dam between 5 and 15 metres in height impounding over 3 million cubic metres (Canadian Dam Association, 2019). Around the middle of the 20th century, dam builders increasingly capitalized on the widespread availability and good engineering properties of glacially-derived soils, and earth and rockfill embankment dams became more common than concrete gravity dams (Canadian Dam Association, 2019). However, there are many examples of embankment dams that precede this increase in popularity. According to the 2019 Inventory of Large Dams in Canada, there are 223 earthfill embankment and 24 rockfill dams which were completed before 1969 (Canadian Dam Association, 2019). In the United States, approximately 15,600 of the 91,000 dams in the country are classified as high-hazard, meaning loss of life and significant economic damage would occur should the structure fail (ASCE, 2021). With an average age of 57 years, many of the dams in the US have been in service for decades (ASCE, 2021). While the age of a dam does not mean it is inherently hazardous, Vaughan and Bridle (2004) state "although the general experience is that dams grow safer with time, there is no justification for assuming that because they have not leaked or failed after a given time, they will never leak or fail."

Figure 1.1 illustrates some of the many configurations embankment dams can take. The varying zones are made up of different materials that provide structure, stability, seepage control, and drainage (Fell, et al., 2015). Coarse rockfill and riprap provide structure and erosion control to the outer face of the dam, while the core is protected from internal erosion by filters of sand and gravel. In the labelling nomenclature seen in Figure 1.1 by Fell et al., the zones labeled with 3 and 4 indicate riprap or rockfill, while Zone 1 indicates the impervious material that provides seepage control (2015). Figure 1.1 illustrates examples of dams with protective filters, which are denoted in Figure 1.1 by 2A and 2B. Fine filter next to the core (2A) prevents erosion of the core by seepage water, while coarser filters (2B) in turn protect the fine filter (Fell, et al., 2015).

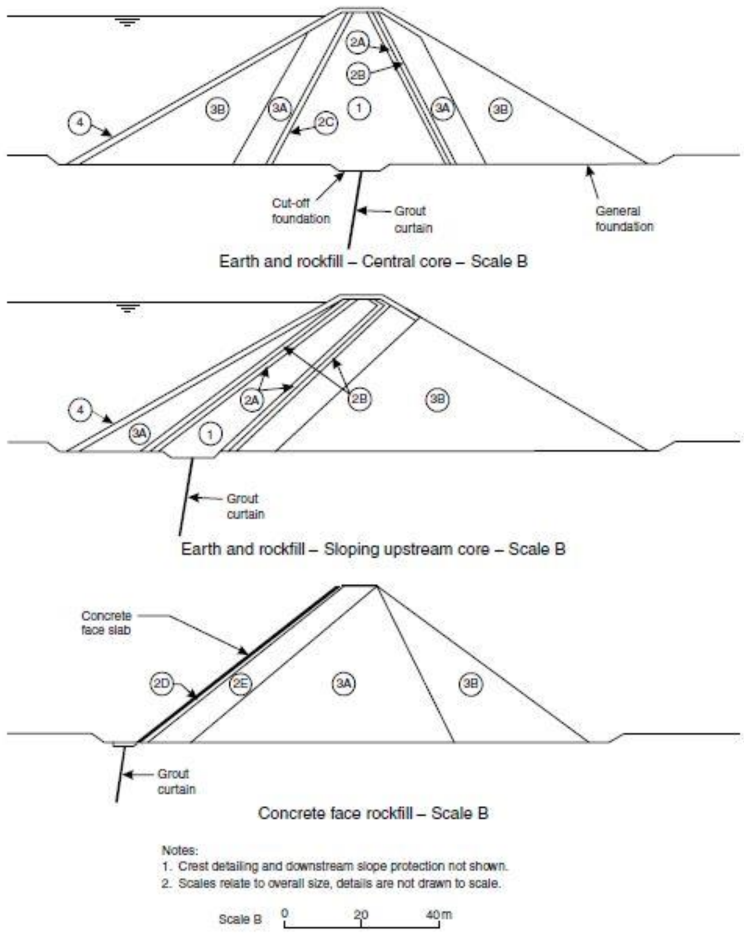

Figure 1.1: Diagrams of types of earth and rockfill dams (Foster, et al., 2005)

Aging embankment dams may lack protective filters entirely or have filters that are inadequate by modern standards of practice (Foster, et al., 2000). Past studies (ICOLD, 1995 cited in Mousavin, 2017; Foster, et al., 2000) have determined that internal erosion is a major cause of dam failure, with 20% to 30% of failures attributed to it. One study concluded that 25% of the internal erosion failure cases included in their investigation were due to inadequate filtration within the embankment (Bonala and Reddi, 1998, cited in Mousavin, 2017).

Modern filter design has progressed and improved the safety and performance of newly constructed dams, but the ability to recognize and assess risks associated with aging dams is vital as population growth “steadily encroaches on once-rural dams and reservoirs” (ASCE, 2021). A brief summary of filter design follows in the next section, along with a discussion of risks that have been identified in aging dams and measures developed to assess these risks.

2.0 Filter Design Criteria

Many researchers have contributed to the progress and development of filter design criteria since it was first introduced by Terzaghi in the early 1920s (USACE, 1953). Filter design criteria can be separated into particle size bases and constriction size based methods. A good filter must be “durable under the wetting and drying processes, chemical action of water, and against mechanical actions occurring during placement and compaction.” (Fell, et al., 2015) A good filter must also be internally stable and self-healing. It must able to “prevent loss of its own small particles due to disturbing agents such as seepage and vibration.” (Kenney & Lau, 1985)

2.1 Particle Size Criteria

Methods based on the particle size of the base and soil material have largely been developed experimentally. The original criterion described by Terzhagi was valid for uniform, cohesionless base and filter materials and is a ratio of the base and filter gradations, given by $D_{15}/d_{85} < 8$ (USACE, 1953) where D_{15} is the particle size that 15% of the filter material by weight is smaller than. The notation d_{85} indicates the particle size of the base material of which 85% by weight is finer. Sherard and Dunnigan (1984a, 1984b) devised the slot and slurry tests to determine the appropriate filter criteria for a variety of base soils. A series of experiments on fairly uniformly graded sands and gravels with a D_{15} of 1.0 to 10 mm were carried out and results supported a ratio of $D_{15}/d_{85} \leq 5$ but found that suggested alternate ratios of D_{50}/d_{50} and D_{15}/d_{15} were not sufficient (Sherard, et al., 1984a). Experiments were also carried out on silts and clays to determine appropriate filter criteria for cohesive soils. The authors were concerned with identifying criteria for what was referred to as a ‘critical’ filter- a downstream filter that must be “capable of controlling and sealing a concentrated leak through the core, and should also be stable in conventional laboratory filter test under a relatively high gradient, such as 1000” (Sherard, et al., 1984b). The investigation once again supported use of the ratio $D_{15}/d_{85} \leq 5$ for sandy clays and silts, and criteria of $D_{15} = 0.5$ mm for sand filters protecting clays (Sherard, et al., 1984b). It was also concluded that the Atterbeg limits had no significant bearing on the filter size, and that the void spaces between the finer particles in a sandy gravel filter control the filtration performance (Sherard, et al., 1984b). Certain problematic soils including highly plastic silts, residual soils, and gap-graded soils were noted to be out of the scope of these criteria (Sherard, et al., 1984b). These results were refined shortly thereafter by the introduction of the "no erosion filter" (NEF) test, which was superior to the previous slot and slurry tests due to its ability to simulate the extremely detrimental conditions a critical filter may be subjected to by a concentrated leak from the core (Sherard & Dunnigan, 1989). Figure 2.1 illustrates how in the event of a concentrated leak, high gradients would form at the face of the filter after the leak is sealed by the capture of fine particles eroded from the base soil.

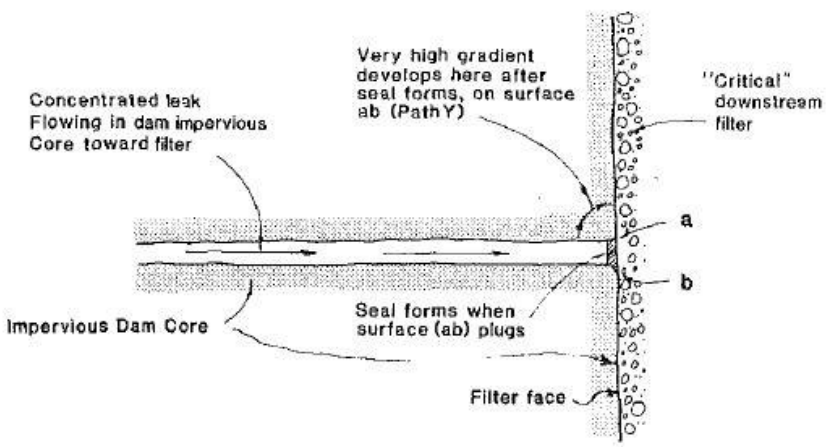

Figure 2.1: Illustration of a concentrated leak flowing from core into filter (Sherard and Dunnigan, 1984b)

The NEF test also enabled the identification of a filter boundary size, the D_{15b}, at which no visible erosion occurred to a preformed hole in the base specimen over the course of the test (Sherard & Dunnigan, 1989). The tests were carried out on a number of base soils that were sorted into four general groups:

Group 1: fine silts and clays with over 85% passing the No. 200 sieve

Group 2: silty and clayey sands and sandy silts and clays with 40-85% pasing the No. 200 sieve

Group 3: silty and clayey sands and gravelly sands with 15% or less passing the No. 200 sieve

Group 4: soils with distributions between Group 2 and Group 3

When categorizing the base soil, the gravel fraction retained on the No. 4 sieve (particles with a diameter greater than 4.75 mm) was first removed and the gradation curve was recalculated using the remaining soil (Sherard & Dunnigan, 1989). General filter boundary criteria were then determined for each of the four soil groups, shown in Table 2.1 below.

Table 2.1: Filter boundary criteria for soils (Sherard and Dunnigan, 1989)

Soil Group Number	Fines content by % passing No. 200 sieve	Filter boundary (D_{15b})
1	85-100	D_{15b} = 7 d_{85} -12 d_{85}
2	40-80	D_{15b} = 0.7 - 1.5 mm
3	0-15	D_{15b} = 7 d_{85} - 10d_{85}
4	15-40	D_{15b} = between Groups 2 and 3

The work by Sherard and Dunnigan was used as the basis for the method developed by the National Resources Conservation Service (NRCS) (Indraratna, et al., 2008). The NRCS method sought to improve filter performance for broadly-graded base soils and not only called for discarding the gravel fraction prior to categorizing the base soils, but also attempted to limit the risk of segregation of the filter material during placement by limiting the maximum size of the filter particles and the uniformity coefficient C_u (the ratio between the D_{60} and D_{10} sizes of the filter material) (Indraratna, et al., 2008). Later investigation by Foster and Fell (2001) supported the previous studies but proposed slight adjustments to criteria for Soil Group 2, namely reducing the lower bound from 40% to 35% and reducing the maximum D_{15} to 0.5 mm for dispersive soils of Group 2.

While particle-based methods are simple to use and applicable to a wide range of soils, they do have several limitations. General particle size-based criteria based on D_{15} size are overly conservative if used for broadly-graded filter soils, which may increase susceptibility to clogging over the life of the filter (Indraratna & Raut, 2006). Similarly, the d_{85} of the base soil gives no indication of the width of gradation and its use can lead to inadequate filter design for well-graded base soils (Indraratna & Raut, 2006), although this is mitigated slightly by regrading the base soil on the No. 4 sieve as indicated in NRCS (1994). Finally, no consideration is given to the density of the filter and its effect on constriction size (Indraratna & Raut, 2006).

Later studies sought to alleviate these limitations. A Reduced PSD approach was introduced for use with broadly graded base soils which enabled an estimation of the self-filtering ability of a potential base soil (Indraratna, et al., 2008). This information could then be used to determine whether a coarse filter would be sufficient due to the base soil's internal stability, or whether a gradation containing finer particles would be needed (Indraratna, et al., 2008).

2.2 Permeability Based Criteria

Filter criteria based on permeability were also developed. In their case study of the failure of the Balderhead dam, caused by inadequate filter design, Vaughan and Soares (1982) introduced the idea of a 'perfect' filter to be used for cores of flocculating clay. The dam was built in 1959, prior to the development of a standard of practice regarding protective filters for silts and clays and failed due to cracking and internal erosion the filter was unable to seal (Vaughan & Soares, 1982). The authors described a 'perfect' filter as one that is fine enough to capture and retain even the smallest particles of the base soil, which then negates the necessity of applying a safety factor to the filter gradation, although it does require that the fine filter protected in turn by coarser filters (Vaughan & Soares, 1982). A relationship was developed between the permeability of the filter and the floc size it was able to retain:

$$k = 6.7x10^{-6} \cdot \delta^{1.52}$$

where k is permeability in m/s and δ is the size in microns (10^{-6} mm) of the particle just passing the filter, which can be determined using Stokes' Law or experimentally (Vaughan & Soares, 1982). This relationship was found to be successful in the design of non-cohesive sand filters for cohesive clay cores and was not originally indicated for use in filters for dispersive soils. The 'sand castle' test, a simple method for testing the cohesion of a filter material, was described for use in the laboratory or field. A sample of moistened filter material is compacted into a cylindrical or conical mould such as a beach bucket or compaction mould, then turned out into a shallow tray to which water is added. The sample is confirmed to be non-cohesive if it collapses to its angle of repose due to the loss of capillary suction (Vaughan & Soares, 1982). Due to the difficulty involved in the measurement of the permeability of a filter in comparison to its gradation, this method was later updated by the development of a relationship between permeability and D_{15} size of a uniform filter:

$$k = 3 * 10^{-8}(D_{15})^{1.767}$$

where D_{15} is in microns and k is permeability in m/s (Vaughan, 2000 cited in Vaughan & Bridle, 2004). This method is applicable to non-cohesive filters with permeability greater than 1 x 10^{-5} m/s (Vaughan & Bridle, 2004). A comparison of ‘critical’ filters as defined by Sherard and Dunnigan (1989) and ‘perfect’ filters was carried out in terms of the minimum size of particle retained, using the relationship between D_{15} and permeability (Vaughan & Bridle, 2004). For base soils of Group 1, critical filters were more conservative than perfect filters but significantly less conservative for Group 2 base soils (Vaughan & Bridle, 2004). Criticism of this method argued that the ‘perfect’ filter approach is overly conservative (Sherard, 1983) which could increase project costs by prescribing an extremely fine filter.

2.3 Constriction Based Criteria

Other design criteria sought to define an appropriate filter by controlling the constriction size- the size of the void spaces between particles in the filter material. Kenney et al. (1985) defined the controlling constriction size D_c* as “the diameter of the largest particle that can possibly pass through a filter material of specific thickness”. An effective filter functions by capturing eroded base soil particles larger than the controlling constriction size. This decreases the constriction size, and so finer particles are captured, until eventually the leak is sealed. This process is referred to as self-filtration and the layer of base particles retained in the filter is the self-filtration layer (Kenney, et al., 1985). Different examples of constriction are illustrated in Figure 2.2. Constriction size is a function of the size distribution and packing of particles in the soil skeleton.

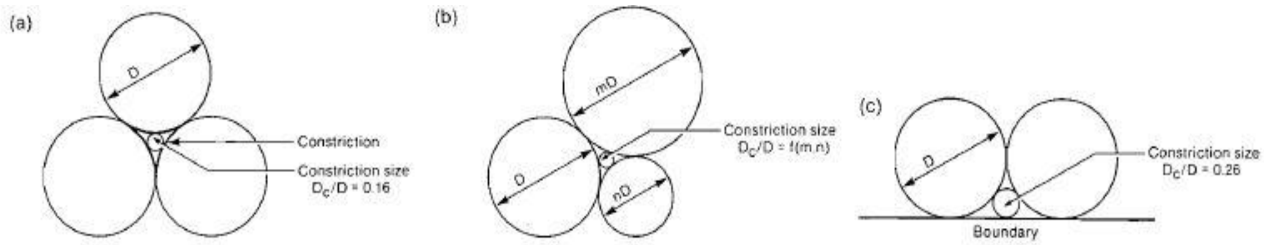

Figure 2.1: Constriction sizes in a dense matrix of spherical particles (Kenney et al., 1985)

Experimentation demonstrated that D_c* is mainly influenced by the smallest particles in the filter matrix and only slightly affected by the width of gradation and the filter thickness (Kenney, et al., 1985). However, it was determined that the D_c* is strongly affected by hydrodynamic conditions within the filter, and therefore is not an adequate design criterion if used alone (Kenney, et al., 1985). Further research developed the idea of a constriction size distribution (CSD) of percent passing by particle surface area, which was then used to determine the constriction size at which self-filtration becomes possible (Indraratna, et al., 2008). The dominant filter constriction size D_{c95} was found to represent the cut-off value at which larger base particles were unable to enter constrictions within the filter and therefore unable to contribute to self-filtration, and the base soil PSD could be modified to exclude particles above this size for design purposes (Indraratna, et al., 2008; Indraratna & Raut, 2006).

The D_{c95} criteria was conceived using a one-dimensional constriction model under the assumption of independent flow channels (Indraratna, et al., 2008). Later studies applying the principle of self-filtration to a 3D pore model revealed that the filter constriction size corresponding to 35% finer is the minimum size of base particle that could be retained in the filter (Locke, et al., 2001; Indraratna, et al., 2008). This constriction size was referred to as the controlling constriction size, D_{c35} (Indraratna, et al., 2007 cited in Indraratna, et al., 2008). The base soil representative parameter d_{85SA}, determined from the PSD by surface area, was combined with the controlling constriction size to yield the criterion:

$$\frac{D_{c35}}{d_{85SA}} < 1$$

This criterion, while succinct, is extremely thorough as "it takes into consideration an array of fundamental filter parameters, including PSD, CSD, C_u and R_d…" (Indraratna, et al., 2008).

The various capabilities of some of the particle based models discussed previously are compared to those of the D_{c95} and D_{c35} models in Table 2.2.

Table 1.2: Parameters considered in various particle and constriction based models, reproduced from Indraratna, et al., 2008

Criteria properties	Terzaghi (USACE, 1953)	NRCS (1994)	D_{c95} model (Indraratna & Raut, 2006)	D_{c35} model (Indraratna et al., 2007)
Regrading required	Yes	Yes	No	No
Inherent analysis of internal stability	No	No	Yes	No
Considers self-filtration PSD	No	No	Yes	No
Clear distinction between effective and ineffective filter	No	No	Yes	Yes
Considers porosity, R_d, and C_u	No	No	Yes	Yes

While the particle-size based models can potentially oversimplify filter design by failing to consider certain factors, they "offer the advantage of being simple to use and the implicit consideration of all major factors affecting filtration..." while the constriction-based models are more "comprehensive, quantifiable, and realistic" and are preferred for use with broadly graded base soils (Indraratna, et al., 2008).

2.4 Filters for Dispersive Base Soils

Filter design for dispersive base soil has been an important topic of study as dispersive soils carry an increased risk of internal erosion. Dispersive soils are clayey soils that rapidly deflocculate when exposed to water with low salt concentration, and they are commonly used for core material in areas where there is a lack of better-quality material (Vakili, et al., 2018).

Further challenges are introduced when the base soil is both dispersive and broadly graded. The Sherard and Dunnigan (1989) filter design criteria do not ensure adequate filtration for broadly graded dispersive base soils (Vakili, et al., 2015). The Vaughan and Soares (1982) model of a 'perfect' filter, despite its conservatism when used with less problematic soils, was found to be applicable for dispersive base soils (Vakili, et al., 2018). A criterion of $D_{15}/d_{85} \leq 5.5$ was found appropriate for broadly graded dispersive base soils of Group 1, with the criterion from Foster and Fell (2001) $D_{15}/d_{85} \leq 6.4$ proving adequate for uniformly graded soils (Vakili, et al., 2015). For dispersive soils of Group 2, $D_{15} \leq 0.28$ is sufficient (Vakili, et al., 2015). A 2018 review of empirical and theoretical design criteria for dispersive base soils concluded that while most of the empirical design criteria could be used if care was taken to ensure validity for the specific base soil and field characteristics, the constriction-based criteria were widely applicable as "limitations caused by filter gradation, filter compaction, and filter PSD do not affect these criteria." (Vakili, et al., 2018).

3.0 Effects of Aging on Filter Performance

It has been established that many existing dams were built without the benefit of modern filter design criteria and may contain inadequate filters. However, even filters that were designed appropriately can underperform if issues are introduced as the dam ages. Aging in this context is defined as "time-related changes in the properties of the materials of which the structure and its foundation are composed." (Oladejo, 2014) It is directly evaluated by measuring the material properties within the structure over time, while indirect evaluation is possible through "monitoring the effects and consequences of changes and the actions causing them." (USSD Committee on Materials for Embankment Dams (USSD), 2010) In designs where the filter must protect the base material while also allowing seepage to drain, issues can arise if self-filtration reduces permeability to the extent that drainage is inhibited. Consolidation of the materials within the dam can occur at different rates, leading to differential deformation and the formation of fissures which could lead to internal erosion (USSD, 2010). The erosion could occur within the core material (USSD, 2010) and form a concentrated leak as studied by Sherard and Dunnigan (1989); or it is frequently encountered at the embankment-foundation contact. Other potential sources of internal erosion include the erosion of embankment material into fissures in foundation rock, and erosion of embankment filter material due to dispersion, internal instability, or leaching of soluble minerals (USSD, 2010).

3.1 Physical Clogging of Filters

There are various geotechnical and geoenvironmental applications where filter materials may be required to perform as drainage layers in addition to providing protection to the base soil. These may include wastewater filtration seen in landfills, oil recovery processes (Reddi, et al., 2000), and seepage collection from consolidating tailings in a tailings storage pond. Figure 3.1 shows an example schematic of a permeable embankment dam used for tailings storage where the filters must permit drainage of water expressed from the tailings during consolidation.

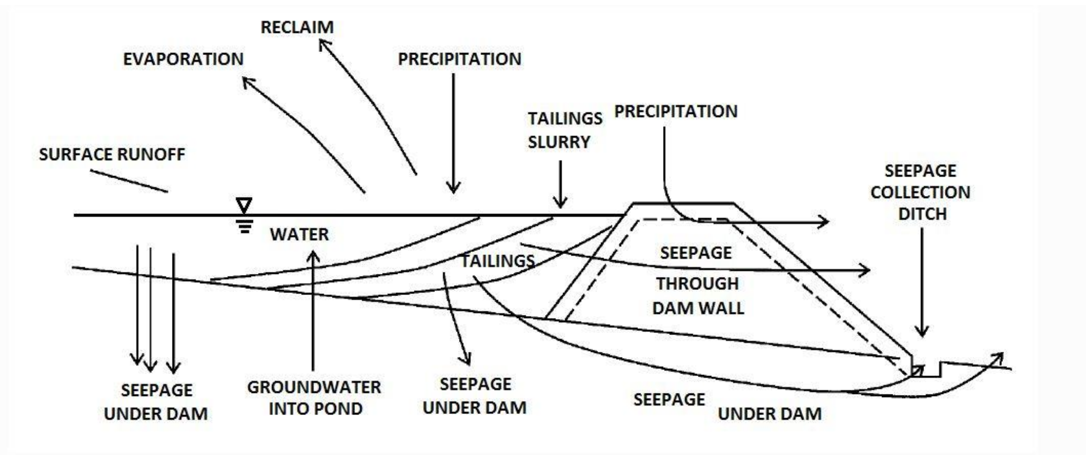

Figure 3.1: Dam water balance showing seepage through dam (European Commission 2009 cited in Niemeläinen et al.)

The process of physical clogging by micron-sized particulate matter was studied by Reddi et al. (2000) using a sandy soil typical of filter materials permeated at various flow rates by a kaolinite particle suspension and a polystyrene microsphere suspension. The results indicated a reduction in permeability an order of magnitude or greater after the filter was permeated by 300-600 pore volumes of the suspension, with particulate concentration showing greater influence than flow rate in observed clogging (Reddi, et al., 2000). The study was expanded in 2005 with an investigation of the influence of flow regime on physical clogging via a comparison of clogging observed under constant flow rate and constant head (Reddi, et al., 2005). Geotechnical engineers may encounter constant head conditions in dewatering wells (Reddi, et al., 2005). Constant head conditions are more commonly found in geotechnical applications, with constant head flow regime encountered in “protective soil filters and drainage layers in earth-retaining structures, tunnels, and earth dams.” (Reddi, et al., 2005) The results indicated similar reduction in permeability with respect to time, but the reduction occurred in fewer pore volumes in the constant head regime which was attributed to a greater degree of self-filtration (Reddi, et al., 2005).

3.2 Biochemical Clogging of Filters

In certain applications, the compounds within the seepage passing through filters may provide favorable conditions for the formation of biofilm, which can clog filters. Rowe et al. (2000) conducted a study on the effects of particle size on clogging rate in columns of uniform granular material permeated with leachate collected from the leachate collection system of a municipal landfill. The results indicated that smaller particles with diameters of 4 or 6 mm provided more favourable conditions for the formation of biofilm, which in turn encouraged increased deposition of inorganic material by altering the chemical oxygen demand (Rowe, et al., 2000). In finer filters, clogging was observed to concentrate near the intake area of the filter column, whereas more even distribution was observed in columns with 15 mm diameter particles (Rowe, et al., 2000). Reductions in hydraulic conductivity of 7 to 8 orders of magnitude were observed in both fine and large particles, but the reduction occurred quickly in the column of fine material and more slowly in the larger material, suggesting that selection of appropriate particle size strongly influences the life cycle of filters exposed to seepage which is favourable to the formation of biofilm (Rowe, et al., 2000). Ochre is a particular biofilm, formed by microbial colonization in seepage water containing high concentrations of iron compounds, that frequently accumulates within the void spaces of drainage material (de Mendonca, et al., 2003). As in the similar observations by Rowe, et al., accumulated ochre can form clogs that reduce the hydraulic conductivity of the filter (Rowe, et al., 2000; de Mendonca, et al., 2003). The impaired drainage performance increases saturation levels and pore pressures within the structure, which may destabilize the structure or contribute to piping (de Mendonca, et al., 2003). Numerous examples of performance issues within the filters of earth dams have been attributed to ochre clogging (Xu et al., 1976; Infanti and Kanji, 1974; cited in de Mendonca, et al., 2003). In a report on the design and construction of the Vermilion Dam completed in California in 1953, it was noted "During first filling of the reservoir, the toe drain became obstructed…probably by an accumulation of iron oxide which is apparently leached from the natural soils in the vicinity. This rust-colored, slimy sediment collects on the weir plates and in the stilling basins of the outlet weir and therefore is believed to be the cause of the obstruction in the toe drain." (Terzhagi & Leps, 1958) Ochre formation has also been observed on geotextiles used in the drainage system of tailings dams (Scheurenberg, 1982, cited in de Mendonca, et al., 2003).

3.3 Mechanically Induced Filter Degradation

The gradation of a filter can also be altered by mechanical forces. In some mining operations, the tailings storage facility is composed of an embankment dam encircling a 'pond' where tailings are deposited and allowed to consolidate, like the diagram shown in Figure 3.1. The dam generally begins as a small 'starter' dam and undergoes periodic raises to accommodate the tailings production from the processing facility. In some cases coarse, dewatered tailings are used as construction material for the dam (Singh, et al., 2021). As the height of the dam increases, material at the base can be subjected to stresses which "may be sufficient to initiate grain crushing…which can effectively increase the fines content, leading to pore clogging and reduced hydraulic conductivity, and eventually jeopardize the stability of the dam" (Singh, et al., 2021). Failure of a tailings dam can be an extremely catastrophic event with significant environmental and economic impacts, as seen in the 2014 Mount Polley mine disaster in British Columbia and the 2019 Brumadinho disaster in Brazil. Considering the high potential for harm, it is crucial that tailings dams function as designed. Singh et al. (2021) conducted a study of underflow tailings and filter material from active mine sites and measured the change in hydraulic conductivity after the material was subjected to stresses ranging from 3.5 to 120 megapascals (MPa). Underflow tailings are the coarse fraction of tailings separated from raw tailings by hydrocycloning (Singh, et al., 2021). The results suggested that significant loss of hydraulic conductivity could be observed if the magnitude of the compressive stress induced grain breakage, with experimental results indicating "a stress increase from 5 to 40 MPa can cause a reduction in hydraulic conductivity by one order of magnitude for both materials." (Singh, et al., 2021) The filter material was observed to experience both a reduction in overall void ratio from 0.48 to 0.3, and an increase in fines content from 11% to 17% (Singh, et al., 2021). The reduction in hydraulic conductivity in the filter material could impair its ability to drain seepage water, causing the phreatic surface within the dam to rise and weakening the stability of the structure (Singh, et al., 2021). While the authors indicate that stresses in tailings dams rarely exceed 5 MPa (Obermeyer and Alexieva, 2011 cited in Singh, et al., 2021) previous work (Reimer, et al., 2008 cited in Singh, et al., 2021) had shown that loading from 0 to 5.5 MPa caused the fines increase in underflow to increase by approximately 5% to 6%. While the large stresses studied by Singh et al. (2021) may not develop within a dam, it is nonetheless prudent to consider all factors that may influence the performance of a structure.

3.4 Water Quality Effects on Filters for Dispersive Soils

Changes in the influent water quality can affect the performance of filters, especially in cases where the dam includes dispersive materials. An investigation into the effects of water quality of the performance of filters for dispersive cores indicated that the use of river water improved filter performance in comparison to distilled water (Vakili, et al., 2021). The NEF test boundary size achieved in tests using river water increased by 1.75 in comparison to the size determined with distilled water, meaning less rigorous filter requirements were needed (Vakili, et al., 2021). The dispersivity of the base soil is influenced by the mineral and total dissolved salt content within the reservoir water and pore water, and changes to the water quality could influence the long-term performance of the structure (Vakili, et al., 2021). A case study was cited where a dam in Australia, constructed with dispersive material, failed within days after the water it was impounding underwent a decrease in concentration of dissolved salts from 26 to 1.2 milliequivalents per litre (Knodel, 1991 cited in Vakili, et al., 2021). In dams built using dispersive materials, manipulating the dissolved salt content of the water could be used to improve the performance of the filter, as the authors state “the results proved that a once dispersive soil could have a reduced dispersivity when exposed to water with high dissolved salt concentration and would only require a coarser, thus cheaper, sandy filter to control the colloidal erosion.” (Vakili, et al., 2021) The results also indicate the importance of using the anticipated reservoir fluid for NEF tests during the design phase of future dams (Vakili, et al., 2021).

3.5 Development of Internal Erosion or Piping

Filter degradation by any of the previously described methods may initiate internal erosion and piping within aging dams. The risk of internal erosion may also have been present since construction due to the use of incompatible filter and base materials due to unconservative filter design criteria, or the use of material prone to internal instability.

An interesting case study by Oskoorouchi and Lane (2004) of the Lake Storey Embankment Dam, completed in 1929, noted the presence of a moderately sized slope failure on the downstream shoulder of the dam and seepage from the downstream toe. The reservoir created by the dam is 133 acres (5.38E5 m^2) and residential development has occurred in near proximity to the downstream, as shown in Figure 3.2 (IDOT, 1993 cited in Oskoorouchi and Lane, 2004).

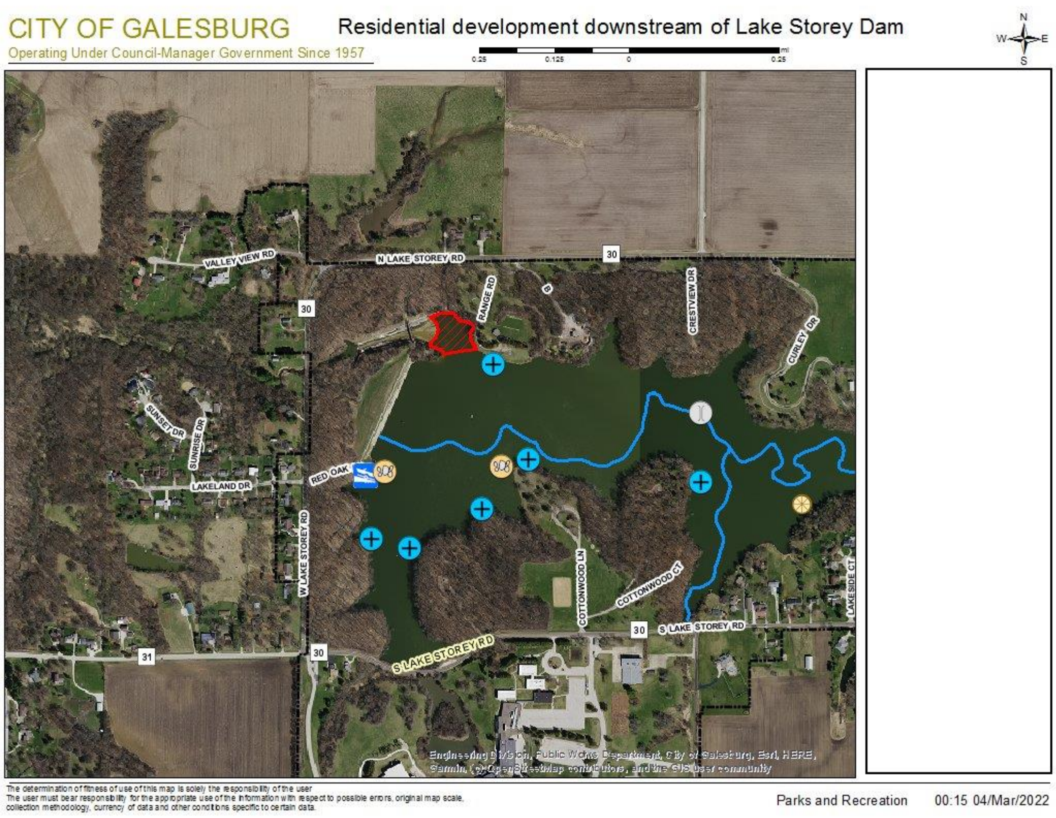

Figure 3.2: Residential development downstream of the Lake Storey Dam, Galesburg IL (Engineering Division, Public Works Department, n.d.*)*

The slope failures on the downstream slope of the dam were relatively recent developments, with one occurring about eight years prior to the authors' April 2000 inspection, with the other estimated to have occurred only weeks prior to the inspection (Oskoorouchi & Lane, 2004). While it was noted that the construction of the dam was completed over a decade "before engineers came to know the importance of transition filter and drainage system [sic] in embankment dams" (Oskoorouchi & Lane, 2004) the lengthy interval between construction and onset of failure was puzzling. It was revealed that trees had been permitted to grow on the slopes of the dam to a mature size before being removed at the ground surface (Oskoorouchi & Lane, 2004). The authors suggested that decomposition of large root systems left behind by grown trees caused the observed failures and could develop into piping and eventual failure of the dam if left uncorrected (Oskoorouchi & Lane, 2004). Extensive remediation efforts were made to the downstream slope, including construction of a drainage system and shear key with protective filters at the downstream toe, flattening the slope, and covering the surface with a fine transition filter to impede soil migration (Oskoorouchi & Lane, 2004).

4.0 Assessing Risks of Aging Dams

The best time to minimize future aging-related performance risks is during the design and construction phases. The owners and operators of aging dams must depend on risk assessment techniques to gauge the likelihood of potential issues, and robust monitoring systems to provide early detection of developing issues (Oladejo, 2014). Several risk assessment techniques and monitoring procedures have been developed.

4.1 Risk Assessment and Prediction

Risk analysis is concerned with determining the most probable answers to the "risk triplet" posed by Kaplan and Garrick: what possible negative event could happen? how likely is this to happen? if it does happen, what are the possible outcomes? (1981) Most risk assessment methods are based on a potential failure mode analysis (PFMA), which is defined by the U.S. Federal Energy Regulatory Commission (FERC) as "an exercise to identify all potential failure modes …and to assess those potential failure modes of enough significance to warrant continued awareness and attention to visual observation, monitoring and remediation as appropriate." (FERC, 2017)

4.1.1 Assessing Internal Erosion by NEF Tests

To assess the performance of filters in dams that predate modern filter criteria, Foster and Fell (2001) proposed definitions for no erosion (NE), excessive erosion (EE), and continuing erosion (CE) particle size boundaries for filter and base soil particles based on the results of No Erosion Filter (NEF) test laboratory analysis. No erosion was defined as losses of 10 g or less in cohesionless base soils or no visible erosion in cohesive soils; base soils that experienced over 100 g of loss were classified as excessive erosion; and continuing erosion occurred when the filter was incapable of sealing and the base soil eroded without restriction (Foster & Fell, 2001). The boundaries and relationship between D_{15} and d_{85} are shown in Figure 4.1. As the D_{15} increases relative to the d85, the risk of inadequate filter performance increases.

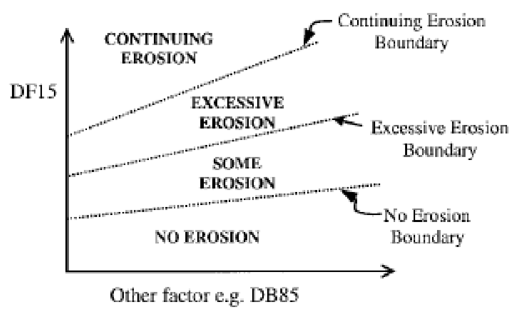

Figure 4.1: Erosion performance boundaries from filter tests (Foster & Fell, 2001)

Size boundaries for excessive, continuing, and no erosion boundaries were proposed for the four based soil groups defined by Sherard and Dunnigan (1989) and may be used to assess performance of existing filters. The results of the NEF tests were compared to case studies of existing dams and used to assign relative likelihood ratings to potential erosion outcomes based on the D_{15} size within the dam, shown in Table 4.1. More favorable filter performance outcomes were determined to be more likely in dams having an average D_{15} finer than the NE boundary and trended towards unfavorable performance as the D_{15} increased.

Table 4.1: Assessment of potential filter performance in event of concentrated leak (Reproduced from Foster and Fell, 2001)

Comparison of D15 in Dam to Filter Test Erosion Boundaries		Likelihood of Filter Performance in Event of Concentrated Leak		
Average D_{15} in Dam	Coarsest D_{15} in Dam	Seals with no erosion P_{NE}	Seals with some erosion P_{SE}	Partial or no seal with large erosion P_{LE}
< NE	< NE	Highly likely	Unlikely	Highly unlikely
< NE	> NE and < EE	Equally likely	Equally likely	Unlikely
> NE	< EE	Unlikely	Equally likely	Equally likely
> NE and <EE	> EE	Unlikely	Unlikely	Likely
> EE	> EE	Highly unlikely	Unlikely	Highly likely

The predictions were assigned based on the assumption that the filter was not susceptible to internal erosion or segregation (Foster & Fell, 2001). The probability terms should be adjusted towards increased likelihood of erosion if used for soils that may have internal erosion or segregation issues (Foster & Fell, 2001). In a related study, information from 28 dams that had experienced piping or internal erosion events was analysed. All dams included in the study included downstream filters or transitions zones coarser than specified by modern standards of practice (Foster, et al., 2018). The actual gradations of core and filter material were analysed to determine NE, EE, and CE boundary sizes. Poor compaction and thick fill placement layers (500-800 mm) were identified as factors that strongly increased risk of erosion (Foster, et al., 2018).

A study of 19 samples of widely-graded soils from existing dams in New Zealand investigated the applicability of several methods that have been developed to assess internal erosion. The soils available for use in embankment dams in New Zealand are generally “widely-graded alluvium or glacial tills, volcanically-derived soils, soft alluvial deposits, and loess.” (Clawford-Flett & Haskell, 2016) The wide variations possible in the engineering parameters of these soils introduces challenges and uncertainties surrounding their performance (Clawford-Flett & Haskell, 2016). The study applied the Kenney and Lau (1985), Li and Fannin (2008), Burkenova (1993) and Wan and Fell (2008) methods for assessing internal stability to the soil samples and determined that one method was invalid for all soil samples, one was unreliable, and two were well-verified for use with widely-graded glacial tills (Clawford-Flett & Haskell, 2016). The results could be used to identify appropriate methods to assess internal instability in widely-graded soils found in other parts of the world, such as the glacial tills common to Scandinavia (Rönnqvist & Viklander, 2015) and Canada. The filter compatibility of the soils was assessed using the Foster and Fell (2001) method described previously. The results showed considerable overlap between the D_{15} size for NE and EE for several samples which did not permit clear analysis of filter performance (Clawford-Flett & Haskell, 2016).

4.1.2 University of New South Wales (UNSW) Method

A quantitative risk assessment method was developed by Foster, Fell, and Spannagle (2000) based on case studies of dams which had experienced partial or complete failure by piping. Previous investigation had indicated that up to half of all dam failures were caused by piping- in a statistical analysis of large dam failures, 31% had been attributed to piping through the embankment, 15% due to piping through the foundation, and an additional 2% caused by piping from embankment to the foundation (Foster, 1999; Foster, et al., 2000). While most of these failures occurred at first filling (42%) or within the first five years of service (66%) (Foster, et al., 2000), some failures occurred later, indicating the risk of piping remains throughout the life cycle of the dam. Quantitative risk assessment is a common method used by dam safety authorities, but it can be very difficult to assess relative likelihood of piping events without the benefit of knowledge of similar dams to draw comparisons from (Foster, et al., 2000). The method, termed the University of New South Wales (UNSW) method, considers factors including "dam zoning, filters, age of the dam, core soil types, compaction, foundation geology, dam performance, and monitoring and surveillance." (Foster, et al., 2000) The method first estimates the average annual failure rate for the three methods of piping failure, then estimates the weighting factors for each failure mode by multiplying the weighting factors for the individual factors contributing to each piping mode, then combines these into a representative probability using the equation below. The tables for each contributing factor are included in Appendix A.

$$P_p = w_E P_e + w_F P_f + w_{EF} P_{ef}$$

Where P_p is annual likelihood of failure by piping; P_e, P_f, and P_{ef} are average annual frequencies of failure for piping through embankment, foundation, and embankment into foundation, respectively; and w_E, w_F, and w_{EF} are the weighting factors assigned to each failure mode (Foster, et al., 2000). The USNW method is recommended for preliminary assessments and to help prioritize dams for further study (Foster, et al., 2000).

4.1.3 Federal Energy Regulatory Commission (FERC) method

The Federal Energy Regulatory Commission (FERC) method is based on use of a risk informed decision making process to evaluate potential failure modes (PFMs) in the dam (Federal Energy Regulatory Commission (FERC), 2014). The first step is a thorough desk study to gain understanding of the embankment dam. All available documentation and information, including construction reports, design reports, construction material and techniques should be reviewed (Federal Energy Regulatory Commission (FERC), 2014). Next, a series of PFMs are developed based on the initial assumption that "each PFM step will occur, and the dam will fail, regardless of the likelihood of that ultimate outcome." (FERC, 2014) After a full PFM has been developed, an event tree with decision nodes at each step is used to assign a likelihood rating to the PFM (FERC, 2014). Probabilities are expressed in fractions as shown in Table 5.

Table 4.2: Risk estimation descriptors and probability values (FERC, 2014)

Descriptor	Probability
Virtually Certain	0.999
Very Likely	0.99
Likely	0.9
Neutral	0.5
Unlikely	0.1
Very Unlikely	0.01
Virtually Impossible	0.001

The team performing the risk assessment must carefully select the risk descriptor and probability value for each ‘yes’ branch of the event tree. The ‘no’ branch is then assigned a probability value of 1.0 less the probability selected for the ‘yes’ branch. The probabilities of the ‘yes’ branches are multiplied to yield a comprehensive annual probability of occurrence for the PFM (FERC, 2014).

The FERC method requires careful consideration of all potential failure modes that could occur within the structure and its procedure and aims to yield a stringent, comprehensive analysis of potential risks. However, there may be a high degree of uncertainty regarding the relative likelihood of events in the PFM due to a lack of information available to assess. The method is heavily contingent both on the quality and quantity of information available and the collective experience and expertise of those analysing it.

The FERC method is similar but less exhaustive than the method outlaid in the joint federal publication Best Practices in Dam and Levee Safety Risk Analysis (USBR and USACE, 2019). The joint federal guidelines were in turn “based heavily upon the work of Fell, et al.” (FERC, 2014)

The results of any risk assessment should include estimates of confidence and uncertainty (USACE, 2015). The USACE describes uncertainty as relating “to the variability of the natural processes and parameters, and our ability to model and understand them” while confidence describes “the completeness of the information and its interpretation, and where data gaps or limitations in interpretations may lie.” (USACE, 2015). Results of risk assessment can be used to inform further site investigations, monitoring programs, and emergency planning.

4.2 Monitoring and Detection

A thorough monitoring program is the first warning system for internal erosion. Early detection increases the likelihood that the most unfavorable outcomes can be avoided (FERC, 2014). It has been suggested that even an hour’s notice of impending failure can save lives. (USBR 1999 cited in Fell et al. 2003). The USSD states “embankment dams are most susceptible to failure within the first 5 years of construction, or after operation for more than 50 years… this is strong justification to be vigilant in maintaining a good monitoring program of any existing seepage." (USSD, 2010) Internal erosion is detected by visual inspection and ongoing monitoring of pore pressures within the dam in addition to turbidity and flowrate of seepage water (USSD, 2010). Borehole testing can be used to determine the permeability of fill material, which can then be used to assess risk of piping (Vaughan & Bridle, 2004). If internal erosion is detected, possible remediation actions include grouting the piping path, installation of a diaphragm wall, replacement of the drain system, or complete replacement of the problematic section (USSD, 2010).

Geophysical methods have been successfully used in monitoring programs in recent years. Self-potential measures naturally occurring electrical potentials created by seepage and its use in monitoring for embankment dams and other geotechnical structures has been well documented (Brosten, et al., 2005). Electrical resistivity measurement is a progressive technique that has been adopted for seepage monitoring and combined with probability techniques to predict locations in an embankment where seepage is likely to occur (Mousavian, 2017).

5.0 Conclusion

Embankment dams have been an important tool which has allowed many industries and entities to develop and manage water resources. Embankment dam design, especially the design of protective filters in earth and rockfill embankment dams, is a relatively young practice that continues to progress. Many existing embankment dams contain filters that were not constructed to the specifications required by current state of practice. While most dams become safer over time due to the development of the self-filtration layer, there are several mechanisms by which aging filters can degrade and become susceptible to failure. These mechanisms include mechanical deformation, change in water quality, filter clogging by physical or chemical means, and development of internal erosion.

The failure of an embankment dam can have devasting effects. Dam owners, operators, and stakeholders are concerned with potential failures caused by aging. Methods have been developed to assess the risks of degradation, but challenges include potential lack of information regarding materials used and construction records. The accuracy of risk assessment procedures is completely dependent on the depth of understanding of the structure possessed by the parties conducting the assessment. It is crucial that all available information regarding the design, construction, and performance of the structure be carefully considered and augmented with instrumentation, visual inspection, and material sampling if needed. The results of the risk assessment procedure are then used to inform monitoring schedules, potential remediation work, and emergency management and response planning. Refinements to risk assessment techniques allows more accurate understanding of potential risks.

Detection of potential failure events has been generally conducted by monitoring conducted periodically, such as surveying and visual inspection and data collection from instrumentation. Recent developments in detection and monitoring have included the use of remote monitoring technology and geophysical investigation methods such as self-potential, resistivity, and seismic. Continuous monitoring permits earlier detection and response to internal erosion and increases the chances of prevention of complete structural failure.

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Appendix A

Tables for use of UNSW method

Table A.1: Average historic failure frequencies by piping path and dam zoning (Foster, et al., 2000)

	Embankment			Foundation			Embankment into foundation		
		Average annual P_e ($\times 10^{-6}$)			Average annual P_f ($\times 10^{-6}$)			Average annual P_{ef} ($\times 10^{-6}$)	
Zoning category	Average P_{Te} ($\times 10^{-3}$)	First 5 years operation	After 5 years operation	Average P_{Tf} ($\times 10^{-3}$)	First 5 years operation	After 5 years operation	Average P_{Tef} ($\times 10^{-3}$)	First 5 years operation	After 5 years operation
Homogeneous earthfill	16	2080	190	1.7	255	19	0.18	19	4
Earthfill with filter	1.5	190	37	1.7	255	19	0.18	19	4
Earthfill with rock toe	8.9	1160	160	1.7	255	19	0.18	19	4
Zoned earthfill	1.2	160	25	1.7	255	19	0.18	19	4
Zoned earth and rockfill	1.2	150	24	1.7	255	19	0.18	19	4
Central core earth and rockfill	(<1)	(<140)	(<34)	1.7	255	19	0.18	19	4
Concrete face earthfill	5.3	690	75	1.7	255	19	0.18	19	4
Concrete face rockfill	(<1)	(<130)	(<17)	1.7	255	19	0.18	19	4
Puddle core earthfill	9.3	1200	38	1.7	255	19	0.18	19	4
Earthfill with core wall	(<1)	(<130)	(<8)	1.7	255	19	0.18	19	4
Rockfill with core wall	(<1)	(<130)	(<13)	1.7	255	19	0.18	19	4
Hydraulic fill	(<1)	(<130)	(<5)	1.7	255	19	0.18	19	4
All dams	3.5	450	56	1.7	255	19	0.18	19	4

Note: P_{Te}, P_{Tf}, and P_{Tef} are the average frequencies of failure over the life of the dam; P_e, P_f, and P_{ef} are the average annual frequencies of failure. Values in parentheses are based on an assumption of <1 failure.

Weighting factors contributing to failure by piping through embankment (Foster, et al., 2000)

Table A.2: Weighting factors for piping through embankment (w_E values in parentheses) (Foster, et al., 2000)

	General factors influencing likelihood of failure				
Factor*	Much more likely	More likely	Neutral	Less likely	Much less likely
Embankment filters $w_{E(filt)}$		No embankment filter (for dams that usually have filters; refer to text) (2)	Other dam types (1)	Embankment filter present, poor quality (0.2)	Embankment filter present, well designed, and well constructed (0.02)
Core geological origin $w_{E(cgo)}$	Alluvial (1.5)	Aeolian, colluvial (1.25)	Residual, lacustrine, marine, volcanic (1.0)		Glacial (0.5)
Core soil $w_{E(cst)}$	Dispersive clays (5); low-plasticity silts (ML) (2.5); poorly graded and well-graded sands (SP, SW) (2)	Clayey and silty sands (SC, SM) (1.2)	Well-graded and poorly graded gravels (GW, GP) (1.0); high-plasticity silts (MH) (1.0)	Clayey and silty gravels (GC, GM) (0.8); low-plasticity clays (0.8)	High-plasticity clays (CH) (0.3)
Compaction $w_{E(cc)}$	No formal compaction (5)	Rolled, modest control (1.2)	Puddle, hydraulic fill (1.0)		Rolled, good control (0.5)
Conduits $w_{E(con)}$	Conduit through the embankment, many poor details (5)	Conduit through the embankment, some poor details (2)	Conduit through embankment, typical USBR practice (1.0)	Conduit through embankment, including downstream filters (0.8)	No conduit through the embankment (0.5)
Foundation treatment $w_{E(ft)}$	Untreated vertical faces or overhangs in core foundation (2)	Irregularities in foundation or abutment, steep abutments (1.2)		Careful slope modification by cutting, filling with concrete (0.9)	Careful slope modification by cutting, filling with concrete (0.9)
Observations of seepage $w_{E(obs)}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes, seepage emerging on downstream slope (2)	Leakage steady, clear, or not observed (1.0)	Minor leakage (0.7)	Leakage measured none or very small (0.5)
Monitoring and surveillance $w_{E(mon)}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequencies of failure by piping through the embankment depending on zoning type.

Table A.3: Weighting factors for piping through foundation (wF values in parentheses) (Foster, et al., 2000)

	General factors influencing likelihood of failure				
Factor*	Much more likely	More likely	Neutral	Less likely	Much less likely
Filters $w_{F(filt)}$		No foundation filter present when required (1.2)	No foundation filter (1.0)	Foundation filter(s) present (0.8)	
Foundation (below cutoff) $w_{F(fnd)}$	Soil foundation (5)		Rock, clay-infilled or open fractures and (or) erodible rock substance (1.0)	Better rock quality ⟶	Rock, closed fractures and non-erodible substance (0.05)
Cutoff (soil foundation) $w_{F(cts)}$		Shallow or no cutoff trench (1.2)	Partially penetrating sheetpile wall or poorly constructed slurry trench wall (1.0)	Upstream blanket, partially penetrating, well-constructed slurry trench wall (0.8)	Partially penetrating deep cutoff trench (0.7)
Cutoff (rock foundation) $w_{F(ctr)}$	Sheetpile wall, poorly constructed diaphragm wall (3)	Well-constructed diaphragm wall (1.5)	Average cutoff trench (1.0)	Well-constructed cutoff trench (0.9)	
Soil geology (below cutoff) $w_{F(sg)}$	Dispersive soils (5); volcanic ash (5)	Residual (1.2)	Aeolian, colluvial, lacustrine, marine (1.0)	Alluvial (0.9)	Glacial (0.5)
Rock geology (below cutoff) $w_{F(rg)}$	Limestone (5); dolomite (3); saline (gypsum) (5); basalt (3)	Tuff (1.5); rhyolite (2); marble (2); quartzite (2)		Sandstone, shale, siltstone, claystone, mudstone, hornfels (0.7); agglomerate, volcanic breccia (0.8)	Conglomerate (0.5); andesite, gabbro (0.5); granite, gneiss (0.2); schist, phyllite, slate (0.5)
Observations of seepage $w_{F(obs)}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes, sand boils (2)	Leakage steady, clear, or not observed (1.0)	Minor leakage (0.7)	Leakage measured none or very small (0.5)
Observations of pore pressures $w_{F(obp)}$	Sudden increases in pressures (up to 10)	Gradually increasing pressures in foundation (2)	High pressures measured in foundation (1.0)		Low pore pressures in foundation (0.8)
Monitoring and surveillance $w_{F(mon)}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequency of failure by piping through the foundation depending on zoning type.

Table A.4: Weighting factors for accidents and failures caused by piping from embankment into foundation (w_{EF} values in parentheses) (Foster, et al., 2000)

	General factors influencing likelihood of initiation of piping				
Factor*	Much more likely	More likely	Neutral	Less likely	Much less likely
Filters $w_{EF(filt)}$	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)	Appears to be independent of presence–absence of embankment or foundation filters (1.0)
Foundation cutoff trench $w_{EF(cot)}$	Deep and narrow cutoff trench (1.5)		Average cutoff trench width and depth (1.0)	Shallow or no cutoff trench (0.8)	
Foundation $w_{EF(fnd)}$		Founding on or partly on rock foundations (1.5)			Founding on or partly on soil foundations (0.5)
Erosion-control measures of core foundation $w_{EF(ecm)}$	No erosion-control measures, open-jointed bedrock, or open-work gravels (up to 5)	No erosion-control measures, average foundation conditions (1.2)	No erosion-control measures, good foundation conditions (1.0)	Erosion-control measures present, poor foundations (0.5)	Good to very good erosion-control measures present and good foundation (0.3–0.1)
Grouting of foundations $w_{EF(gr)}$		No grouting on rock foundations (1.3)	Soil foundation only, not applicable (1.0)	Rock foundations grouted (0.8)	
Soil geology types $w_{EF(sg)}$	Colluvial (5)	Glacial (2)		Residual (0.8)	Alluvial, aeolian, lacustrine, marine, volcanic (0.5)
Rock geology types $w_{EF(rg)}$	Sandstone interbedded with shale or limestone (3); limestone, gypsum (2.5)	Dolomite, tuff, quartzite (1.5); rhyolite, basalt, marble (1.2)	Agglomerate, volcanic breccia (1.0); granite, andesite, gabbro, gneiss (1.0)	Sandstone, conglomerate (0.8); schist, phyllite, slate, hornfels (0.6)	Shale, siltstone, mudstone, claystone, (0.2)
Core geological origin $w_{EF(cgo)}$	Alluvial (1.5)	Aeolian, colluvial (1.25)	Residual, lacustrine, marine, volcanic (1.0)		Glacial (0.5)
Core soil type $w_{EF(cst)}$	Dispersive clays (5); low-plasticity silts (ML) (2.5); poorly graded and well-graded sands (SP, SW) (2)	Clayey and silty sands (SC, SM) (1.2)	Well-graded and poorly graded gravels (GW, GP) (1.0); high-plasticity silts (MH) (1.0)	Clayey and silty gravels (GC, GM) (0.8); low-plasticity clays (CL) (0.8)	High-plasticity clays (CH) (0.3)
Core compaction $w_{EF(cc)}$	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)	Appears to be independent of compaction, all compaction types (1.0)
Foundation treatment $w_{EF(ft)}$	Untreated vertical faces or overhangs in core foundation (1.5)	Irregularities in foundation or abutment, steep abutments (1.1)		Careful slope modification by cutting, filling with concrete (0.9)	Careful slope modification by cutting, filling with concrete (0.9)
Observations of seepage $w_{EF(obs)}$	Muddy leakage, sudden increases in leakage (up to 10)	Leakage gradually increasing, clear, sinkholes (2)	Leakage steady, clear, or not monitored (1.0)	Minor leakage (0.7)	No or very small leakage measured (0.5)
Monitoring and surveillance $w_{EF(mon)}$	Inspections annually (2)	Inspections monthly (1.2)	Irregular seepage observations, inspections weekly (1.0)	Weekly–monthly seepage monitoring, weekly inspections (0.8)	Daily monitoring of seepage, daily inspections (0.5)

* Refer to Table 1 for the average annual frequency of failure by piping from the embankment into the foundation depending on zoning type.

Appendix B

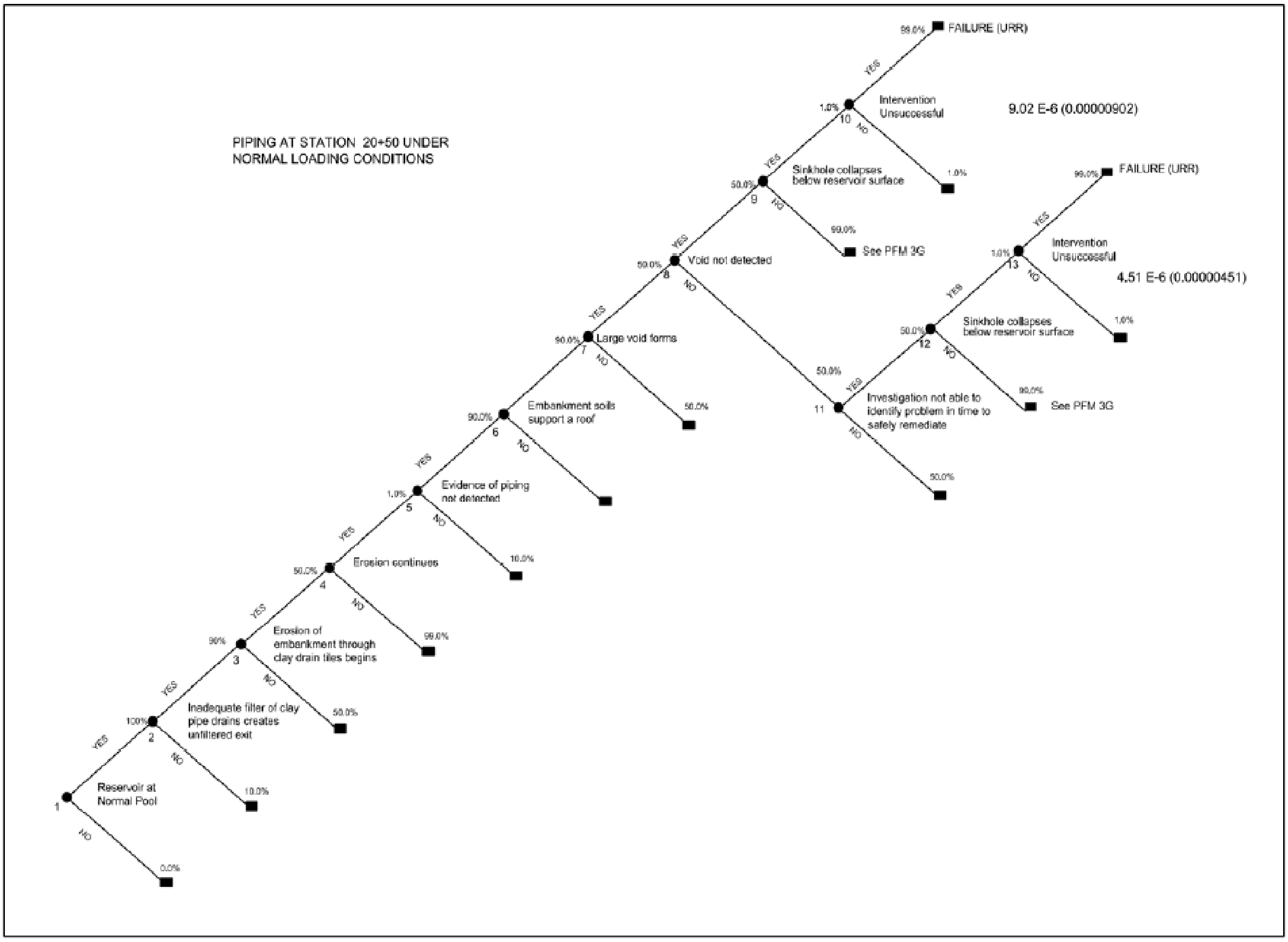

Figure B.1: Example event tree for a PFM (FERC, 2014)