Chapter 2: Review of Previous Work

This chapter summarizes previous research work conducted in areas related to the long-term performance of seepage barriers in dams. Relatively little has been published regarding the long-term performance of seepage barriers, however, the few references that are directly associated with this topic are discussed below. Summaries of previous research on several additional topics are presented herein because of their relevance to this study. The following research topics are discussed in this chapter:

- The long-term performance of seepage barriers,
- Theoretical seepage barrier performance,
- Construction techniques and material properties, and
- Piping development in dams.

Much of the research presented in this study is based on the review of unpublished data and reports regarding the design, construction, and performance of the dams and seepage barriers that are a part of this study. In addition to the unpublished data, many papers have been published on design and construction aspects of the seepage barriers included in this study. In order to provide a comprehensive presentation of the available data for each dam, summaries of the published and unpublished reports, papers and data that have been collected for each dam are presented in the following chapter "Review of Case Histories".

The Long-Term Performance of Seepage Barriers

Marsal and Resendiz (1971)

In this paper presented at the Fourth Pan-American Conference on Soil Mechanics and Foundation Engineering, Marsal and Resendiz (1971) discuss short-term and long-term mechanisms that may be in effect in dams with seepage barriers and discuss design considerations for seepage barriers in dams. For the purposes of their paper, cutoffs are described as any structure whose main purpose is to reduce the flow of water through the foundation. Therefore this paper includes the performance of upstream seepage blankets and compacted earth cutoffs as well as the seepage barrier types that are the subject of this study.

Marsal and Resendiz (1971) discuss simplified models for evaluating the effectiveness of seepage barriers in reducing flow, uplift pressures, or exit gradients. Two ways of quantifying barrier effectiveness are presented: E_H, which describes the effectiveness in terms of the head loss across the barrier as a percentage of the total head difference across the dam, and E_Q, which describes the reduction in seepage rate due to the barrier as a percentage of the seepage rate without the barrier in place. The effects on barrier efficiency caused by seepage through intact barriers and barriers with defects are presented as well as a model for seepage around a barrier through fractured bedrock.

The risk of rigid seepage barriers developing stress concentrations in soil embankments is discussed. Finite element analyses using linear elastic soil models are used to show that tension is likely to develop near the crest of dams with rigid seepage barriers in the foundation. Devices for monitoring hydraulic pressure, seepage flow rates and deformation of dams with seepage barriers are discussed.

Several case histories are presented, two of which have relevance to the topic of this study: Los Tortolas Dam and Jose M. Morales Dam. The remainder of the case histories are for seepage barrier types not included in this study (upstream blanket and grout curtain). Summaries of the relevant case histories are presented below.

<u>Las Tortolas Dam.</u> This dam was constructed from 1965 through 1968 in the State of Durango, Mexico. The dam is a zoned earth fill dam 108 feet high and 1570 feet long. A partially penetrating seepage barrier consisting of a slurry trench backfilled with "well-graded soil" was constructed along the centerline of the low permeability dam core. The barrier is 10 feet thick and extends to a depth of 82 feet and was designed to cut off seepage flows in the most permeable part (the upper 65 meters) of the alluvial foundation.

The anticipated effectiveness of the seepage barrier (based on piezometric heads) was calculated during the design of the dam using finite element seepage analyses. Piezometric levels observed following completion of the dam and filling of the reservoir did not coincide with the calculated values. Several unsuccessful attempts were made to calculate piezometric heads that match the observed values by altering the hydraulic properties of the alluvial materials and embankment fill. In the end, the discrepancies between the calculated values and the measured values were attributed to "stratigraphic complexities" that were undetected in the site exploration.

In addition to the piezometers, deformation monuments and pressure cells were installed in the dam core at an elevation 5 feet above the top of the seepage barrier. This instrumentation showed about 1 foot of settlement above the centerline of the barrier. The amount of settlement decreased to less than half of a foot at the next monument that was not located above the trench. The pressure cells located above the barrier measured total stresses that were 65 percent of the calculated total vertical stress (the total unit weight multiplied by the height of the fill). These measurements illustrate the mechanism of differential settlement and stress redistribution in the core that could lead to hydraulic fracturing.

<u>Jose M. Morales Dam.</u> This dam was constructed in 1967 and 1968. It consists of a zoned earth fill embankment 170 feet high and about 1400 feet long. To reduce seepage through the highly permeable foundation alluvium (hydraulic conductivity of about 1.5 cm/sec) a seepage barrier consisting of concrete piles and concrete panels was constructed to depths of up to 180 feet. A 45-foot wide zone was grouted adjacent to the upper 90 meters of the seepage barrier for the purpose of reducing the compressibility of the foundation soils.

Assessments of readings of the 45 piezometers installed in the dam indicate that the hydraulic effectiveness of the seepage barrier is 96 percent or better. This high effectiveness is largely a result of the extremely high permeability of the foundation soils rather than the low effective permeability of the barrier. Finite element analyses performed on the dam and foundation calculated stresses in the concrete seepage barrier that should have caused shearing failure of the concrete. Details of these analyses are not provided in the paper.

<u>Foster, Fell and Spanagel (1998, 2000a and 2000b).</u>

In this research report (Foster et al. 1998) and two papers (Foster et al. 2000a, 2000b) the authors have reviewed performance data on a large number of dams and have statistically analyzed the characteristics of the dams versus the incidence of dam accidents and failure. In the research report (Foster et al. 1998) and first paper (Foster et al. 2000a) statistical analyses are presented comparing the characteristics of dams (zoning, filters, core soil type, compaction, foundation conditions, and foundation cutoff) with the incidence of failures and accidents. In the second paper (Foster et al. 2000b) a method is presented (termed the University of New South Wales or UNSW method) for assessing the relative likelihood of dam failure by piping based on the results of the statistical analyses presented in the first paper.

The UNSW method starts by assessing the average annual probability of failure by piping, P_p, based solely on construction method, embankment materials and zoning of the dam and whether the dam has been in operation for more than or less than 5 years. This probability is taken as the sum of the average annual probabilities of three modes of piping failure: piping through the embankment, P_e, piping through the foundation, P_f, and piping from the embankment into the foundation, P_{ef}. Weighting factors are then calculated for each of the three modes. The weighting factors take into account design, construction, monitoring and performance factors to modify the modal probabilities. Thus, the probability of piping failure for a dam is expressed by the equation:

$$P_f = w_E P_e + w_F P_f + w_{EF} P_{ef}, \qquad \text{2-1}$$

Where w_E, w_F, and w_{EF} are the weighting factors for the three modes of piping failure.

The results of the statistical analyses indicate that dams with seepage barriers have a higher probability of failure or accident than dams that don't. For dams on soil foundations, Foster et al. (2000b) recommend weighting factors of 1.0 for dams with sheetpile wall cutoffs or poorly constructed slurry trench walls, 0.8 for dams with well constructed slurry trench walls and 0.7 for dams with deep cutoff trenches. For dams with rock foundations the recommended weighting factors are 3.0 for sheetpile walls or poorly constructed diaphragm walls, 1.5 for well constructed diaphragm walls and 1.0 for cutoff trenches. For both foundation types the weighting factors are higher for dams with the types of seepage barriers that are the topic of this study (deep, thin barriers). Thus, statistically the presence of a seepage barrier in a dam increases the probability of failure or accidents. It should be noted that this statistic may be misleading due to the fact that seepage barriers are generally constructed in dams with less than ideal foundation conditions or preexisting seepage problems.

It was also reported that dams with high depth to width ratios in a cutoff trench have a higher incidence of failure than do those with wider seepage barriers (Foster et al. 1998). This statistic is attributed to the potential for developing low stress conditions in the trench backfill that could result in the initiation of hydraulic fracturing. While these thinner cutoff trenches do not classify as seepage barriers as defined in this study, this statistic is relevant to this study because it illustrates how the seepage-related failure mechanisms in dams change as the seepage retarding structures get thinner and become more like seepage barriers.

Ahmed-Zeki, Logan, McQuarrie, and Jaduram (2000).

Ahmed-Zeki et al. (2000) present an assessment of the long-term performance of a seepage barrier installed during the construction of Upper Huia Dam in New Zealand in 1929. Upper Huia Dam is a 120-foot high concrete gravity structure constructed on a foundation of breccia/conglomerate interbedded with tuff. A 3-foot-thick concrete cutoff wall was constructed to a maximum depth of 20 feet beneath the upstream face of the dam. The paper presents an assessment of the effectiveness of the seepage barrier in maintaining sufficiently low uplift pressures beneath the dam to maintain sliding and overturning stability. Several piezometers were installed below the dam in 1998 to monitor hydraulic heads during various operational stages of the dam.

The results of the study indicate that the seepage barrier is performing well and continues to maintain sufficiently low hydraulic pressures for dam stability. While this assessment is of the long-term performance of the barrier, monitoring of the seepage conditions were not performed prior to this assessment and thus the change in performance over time cannot be assessed.

Theoretical Seepage Barrier Performance

Telling, Menzies and Simons (1978a, 1978b, 1978c)

These papers present methods for assessing the effectiveness of seepage barriers that contain joints or that partially penetrate a permeable layer. The first paper (Telling et al 1978a) presents a summary of methods for assessing and predicting seepage barrier efficiency. Efficiency is defined in two ways, the head efficiency and the flow efficiency. The head efficiency, E_H, is defined as the ratio of the head drop across the barrier, h, to the total head drop across the dam, H, or:

$$E_H = h/H \qquad 2\text{-}1$$

The flow efficiency, E_Q, relates the flow with the barrier in place, Q, with the flow without the barrier, Q_0, and is defined as the ratio of the change in flow with the barrier in place to the flow without the barrier or:

$$E_Q = (Q_0 - Q)/Q_0 \qquad 2\text{-}2$$

Several theoretical expressions are presented for estimating head efficiency in terms of the ratio of the dam foundation permeability to the barrier permeability, the length of the dam, the thickness of the barrier and the thickness of the permeable layer. These equations assume that the dam itself is impermeable.

In the first paper, Telling et al. (1978a) also present case histories of dams with seepage barriers in place and assesses their efficiency. Fort Peck and Garrison Dams have steel sheet pile seepage barriers installed across sands and gravels. Both Fort Peck and

Garrison Dams showed increased measured head efficiencies over time (12 percent to 30 percent over 17 years for Fort Peck and 18 percent to 38 percent over 5 years for Garrison). The increase in efficiency is attributed to filling of the sheet pile interlocks with fines and the effects of corrosion along the interlocks. A concrete diaphragm wall at Allegheny Reservoir Dam was reported to have a very high efficiency (reported as 100 percent with no data provided). However, concrete seepage barriers at Selset, Balderhead, Limoeiro, and Selevir Dams all were reported to have very low head efficiencies. All four of the poorly performing barriers were partially embedded into permeable bedrock (shale, fractured gneiss, or schist) and extended with single-line grout curtains. The ineffectiveness of the barriers is attributed to ineffectiveness of the grout curtains.

In the second paper (Telling et al. 1978b) theoretical expressions are derived to calculate the head efficiency of seepage barriers containing uniformly spaced joints. Expressions are derived for barriers with open joints and joints with entrapped soil infill. The expressions are in terms of the wall geometry (barrier thickness, joint spacing, and flow distance along the joint), the permeability of the surrounding soil, and the permeability of the entrapped soil infill.

Small scale tests were performed to assess the affect of perforations in a barrier by using a perforated metal barrier in a 100 mm by 250 mm by 1 meter sand-filled box. The results of these tests indicated that once the perforation area to total area ratio reached about 1 percent, the barrier had the same effectiveness as a partially penetrating barrier with all of the penetrations concentrated at the base of the barrier. Thus, based on the theoretical expressions and the results of the small scale tests, walls with closely spaced joints, such as sheet pile walls, can be as ineffective as a partially penetrating wall with no imperfections.

The paper concludes that, due to the close spacing of joints (interlocks) in sheet pile cut-off walls, the head efficiency of these walls is expected to vary from 30 to 90 percent with the higher efficiencies coming in walls where there is fine-grained soil entrapped within the joints. On the other hand, concrete diaphragm walls are expected to have efficiencies of 90 percent or better. Case histories are presented supporting these conclusions. In cases where the efficiency of concrete diaphragm walls was observed to be lower than 90 percent, the inefficiencies were attributed to underseepage beneath the barrier.

The third paper (Telling et al 1978c) presents the results of a study on the efficiency of partially penetrating cutoff walls. A series of graphs are presented for calculating the efficiency of partially penetrating seepage barriers from three dimensionless ratios: the ratio of the thickness of the barrier to the depth of the permeable layer, the ratio of the depth of the seepage barrier to the depth of the permeable layer, and the ratio of the permeabilities of the permeable layer to the seepage barrier. Two main conclusions are drawn:

1. For set value of the percentage of the permeable layer thickness that the barrier penetrates and the thickness of barrier there is a limiting permeability ratio (permeability of the surrounding soil to the permeability of the barrier) beyond which there is negligible increase in effectiveness of the barrier. This value varies between 10^2 and 10^4 depending on the percentage of penetration and barrier thickness.
2. The depth of penetration has a marked influence on the efficiency of the seepage barrier.

Lee and Benson (2000)

This paper presents the results of an experimental study conducted to evaluate the effectiveness of three types of seepage barriers: soil-bentonite (SB), geomembrane (GM), and composite geomembrane-soil (CGS). The study included intact barriers and barriers with defects and investigated the effects of keying the barrier into an aquitard. The study was performed by conducting a series of bench-scale tests on a variety of wall conditions including sealed and unsealed GM joints and three scenarios of keying the base of the barrier: 1) with a gap between the barrier and aquitard, 2) with the barrier in contact with the aquitard, and 3) with the barrier keyed into the aquitard. The tests were performed at hydraulic gradients on the order of 0.04 to 0.12.

The study results indicate that CGS barriers had the lowest flow rates followed by SB and GM barriers. CGS barriers had flow rates 100 times less than GM barriers. Unsealed joints in GM barriers resulted in flows of up to 160 times larger than those measured with sealed joints. In order to achieve low flow rates, GM walls were required to be keyed into the aquitard. Conversely, low flow rates were achieved with SB and CGS walls by simply having the base of the wall in contact with the aquitard. This final result is though to be the result of the SB backfill conforming to the surface of the aquitard.

Lefebvre, Lupien, Pare, and Tourier (1981**)**

This paper presents the results of finite element analyses performed on two hypothetical dams to investigate the effectiveness of using a downstream relief trench, drains, impermeable upstream blankets, or partial cutoffs in controlling downstream exit gradients. The analyses varied parameters such as anisotropy of the permeability of the foundation, the width of seepage berms, the permeability of the drainage blanket, and the depth of seepage barriers and relief drains. The results of the analyses were presented graphically showing the effect varying the parameters had on the factor of safety against piping failure at the toe of the dam. The results of the analyses indicate that all of the mitigation measures increased the factor of safety to varying degrees. The analysis results indicated that the downstream relief trench was more effective than the upstream blanket and a partially penetrating seepage barrier. The analyses also showed that variation of the hydraulic anisotropy of the foundation soils plays a significant role in the initial factor of safety of a dam and the effectiveness of the seepage mitigation measures.

Construction Techniques and Material Properties

Ressi di Cervia (1992)

This paper focuses on the historical development of slurry wall construction. The focus is on three different types of slurry walls: structural walls, deep cutoff walls, and slurry trenches. The paper outlines the historical development of slurry walls starting with the first patents acquired in Italy in the 1940s. The evolution of the excavation techniques from clamshells to draglines, excavators, and hydrofraises is covered. Other developments discussed include: material backfill, panel joints, reinforcement, and wall continuity.

The section on deep cutoff walls most closely relates to the topic of this study. The focus here is on the use of the hydrofraise as the state of practice in construction of deep cutoff walls. It is noted that the hydrofraise is severely limited to the soil conditions. It has been experimented with in hard rock, but (at the time of the paper's publication) was not an economically viable solution to wall excavation in hard rock or soils that were difficult to excavate. This paper does not give guidelines or recommendations for the design or construction of deep cutoff walls.

Fenoux (1985)

This report discusses the engineering properties of backfill materials for seepage barriers including conventional concrete and plastic conglomerates (plastic concrete and plastic mortar). The properties discussed include: deformability, shrinkage, strength, permeability, durability and erodibility. There is a brief discussion of the techniques used for constructing the barriers and for mixing and placing the infill materials.

The report discusses the basic materials used in the construction of the backfill and the effects the various cementing agents, aggregates, and additives have on the resulting engineering properties. Construction issues that can affect the properties of the backfill such as desiccation due to filtering, water content/saturation of soil, and temperature effects are also discussed. Methods for testing and monitoring the performance of the barriers and the infill materials are briefly discussed.

Kaul, Kauschinger, and Perry, 1991

This report discusses the properties of plastic concrete as they apply to seepage barriers. The concept behind the use of plastic concrete in seepage barriers is to match the deformation characteristics of the barrier to those of the surrounding soil, thus decreasing the potential for cracking during deformation of the dam and foundation. The report presents the results of research conducted to quantify the stress-strain behavior and permeability of plastic concrete.

The report outlines the testing procedures and presents the results of Q test and CIUC triaxial tests performed on plastic concrete samples prepared using various mix designs. The specimens varied in cement content, bentonite content, water-cement ratio, and curing time. The report presented the effects that varying the mix designs had on the following:

- Unconfined compressive strength,
- Splitting tensile strength,
- Elastic modulus,
- Compressibility
- Pore pressure generation and stress path, and
- Permeability.

In general, it was found that the addition of bentonite clay to concrete increases the ductility and decreases its shear strength. The permeability of concrete with bentonite is expected to be the same or less than concrete without bentonite. The report provides a design technique for designing plastic concrete for use in seepage barriers that has a modulus similar to the surrounding soil. The design technique is based on the unconfined compressive strength and uses an empirical correlation between unconfined compressive strength and modulus for plastic concrete.

Ryan and Day (2002)

This paper describes the methods of design and construction of soil-cement-bentonite (SCB) slurry walls. The construction methods are described from excavation of the slurry trench to mixing of the SCB backfill for the wall. SCB backfill mix design is discussed and the effects of the various components are discussed. Sampling and testing techniques and programs are covered. Three case studies are presented and the strengths and permeabilities that were achieved in these projects are discussed.

Britton, Filz and Little (2005)

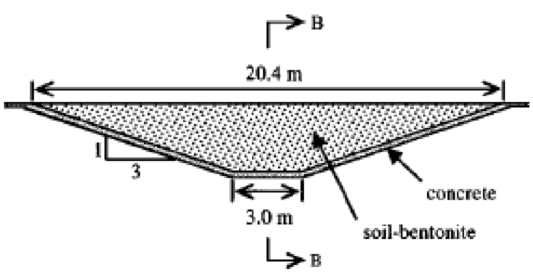
This reference covers the effects of variability in hydraulic conductivity in contaminant transport through soil-bentonite cutoff walls. While the mechanics of contaminant transport is of little relevance to this study, the paper does discuss the occurrence of hydraulic conductivity variability and the sources of the variability, both of which are relevant to this study.

The authors group the sources of variability in hydraulic conductivity into three categories: 1) variability in the backfill mixture, 2) defects that occur during trench excavation and backfill placement, and 3) post-construction changes in the wall. Data from five case studies are presented to statistically evaluate the variability in soil-bentonite walls due to source category 1. The evaluation indicated the distribution of hydraulic conductivity in soil-bentonite barriers could be defined better with a log-normal distribution than with a normal distribution. The standard deviation of the negative natural log of the hydraulic conductivities in the cutoff walls studied ranged from 0.13 to 0.32.

Britton, Filz and Herring (2004)

Britton et al. (2004) compare the results of several methods of measuring the hydraulic conductivity of soil-bentonite barrier backfill in a pilot-scale cutoff wall constructed in a testing facility at Virginia Tech. The study measured the hydraulic conductivity using laboratory tests on disturbed samples, laboratory tests on undisturbed samples, piezocone dissipation tests, piezometer tests (slug tests), and a global measurement of the average hydraulic conductivity of the backfill in the pilot-scale cutoff wall.

Three pilot-scale walls were constructed in a barrier pit testing facility consisting of a 5.9-foot wide concrete lined trench with a maximum depth of 9.2 feet. Compacted clay liners were constructed at the base of the pit and the remainder of the pit was backfilled with compacted sand. Soil-bentonite barriers were then constructed though the sand and embedding into the compacted clay liner. Schematic longitudinal and transverse profiles of the barrier pit are soil-bentonite barrier are presented in Figure 2-1. During construction of the barriers grab samples of the soil-bentonite backfill were taken and tested for hydraulic conductivity using the American Petroleum Institute (API) cell device and following the procedures and interpretation method outlined in Filz et al. (2001).



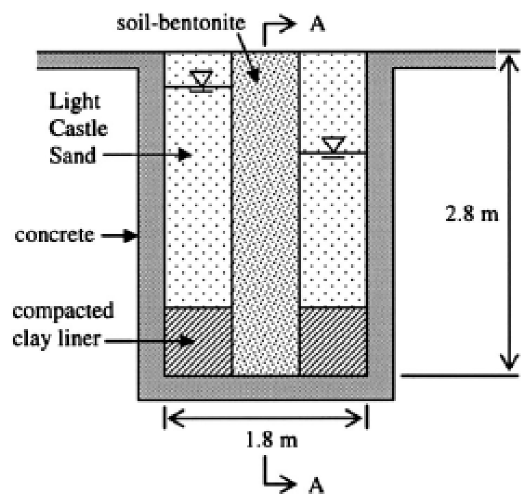


Figure 2-1 Longitudinal (left) and transverse (right) profiles of barrier pit and soil-bentonite barrier (after Britton et al. 2004, with permission from ASCE)

After completion of the barrier construction global measurements of the barrier were performed by imposing a known hydraulic gradient across the barrier by maintaining water levels on either side of the barrier. The water levels were maintained by adding and removing water from monitoring wells. The average hydraulic conductivity was calculated theoretically from the amount of water that flowed through the barrier.

Piezometer tests and piezocone dissipation tests were also performed within the soil-bentonite backfill material. After the testing described above was complete the barrier was disassembled and undisturbed samples were taken of the soil-bentonite backfill. The undisturbed samples were tested for hydraulic conductivity using the falling headwater/constant tailwater procedure.

The results of the testing indicated that the trend in hydraulic conductivity from the various tests methods is as follows:

$$k_{\text{API}} < k_{\text{undisturbed}} < k_{\text{piezocone}} < k_{\text{piezometer}} < k_{\text{global}} \qquad \text{2-4}$$

Where k_{API}, $k_{\text{undisturbed}}$, $k_{\text{piezocone}}$, $k_{\text{piezometer}}$, and k_{global} are the hydraulic conductivity values for the soil-bentonite backfill resulting from the disturbed samples tested on the API cell, the falling headwater/constant tailwater tests on undisturbed samples, the piezocone dissipation tests, the piezometer tests, and the global measurements, respectively. This trend indicates a dependence of the hydraulic conductivity test results on the volume of material tested by each of the techniques where the methods testing the larger volume of material resulted in higher hydraulic conductivity values. A plot of the various hydraulic conductivity results versus sample volume is presented in Figure 2-2. Filz et al. (2004) argue that this trend suggests that the effective permeability of a soil-bentonite seepage barrier is strongly affected by variable hydraulic conductivity within the barrier. As larger sample volumes are tested, there is a greater chance of encountering defects in the barrier or zones of higher hydraulic conductivity due to incomplete mixing of the soil-bentonite material.

Filz, Adams, and Davidson (2004)

Filz et al. (2004) present closed-form expressions for the stability of bentonite-water slurry supported trenches. Filter cake formation on the walls of the trench was shown to significantly increase the overall stability of the trench. The expressions presented consider two cases, first, the case of global trench stability when a filter cake forms on the trench wall and, second, the case of local trench wall stability when no filter cake forms. For the case where a filter cake forms, the following expression is derived for the factor of safety, F:

$$F = \frac{2\sqrt{B}}{B-1}\tan\phi \qquad \text{2-5}$$

where B is a dimensionless stability index defined as:

$$B = \frac{2q + H[(1-m^2)\gamma_{\mathrm{m}} + m^2\gamma_{bw}}{H(n^2\gamma_s - m^2\gamma_w)} \qquad \text{2-6}$$

and q is the surcharge pressure, m = H_w/H, H is the depth of the trench, H_w is the height of groundwater above the bottom of the trench. These variables are defined schematically on Figure 2-3.

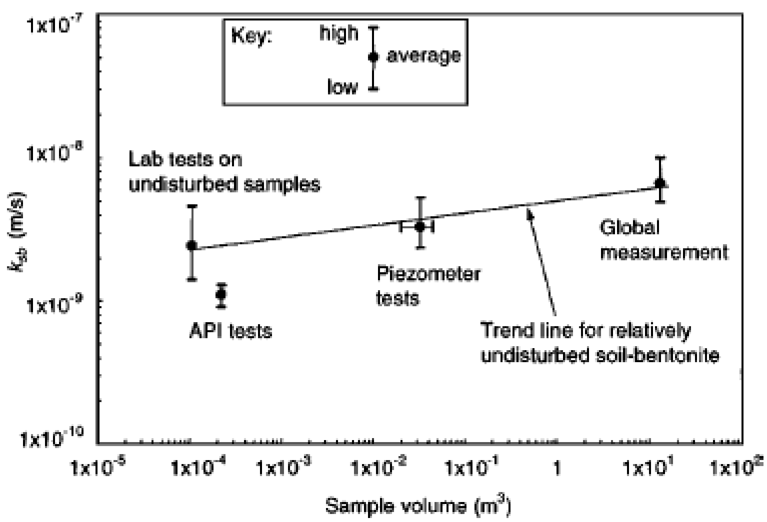


Figure 2-2 The influence of sample size on the resulting value of hydraulic conductivity (after Britton et al. 2004, with permission from ASCE)

For the case of local stability where no filter cake is formed the factor of safety is presented in terms of the stagnation gradient, i_0, where:

$$i_0 = \frac{h_s(\gamma_s/\gamma_w) - h_w}{L} \qquad \text{2-7}$$

where h_s = head of slurry, γ_s = unit weight of slurry in the trench, γ_w = unit weight of the groundwater, h_w = head of groundwater, and L is the distance of penetration of the slurry into the trench wall. Because the value distance L is unknown the value of i_0 is obtained empirically from laboratory test results using Figure 2-4. The local factor of safety, F_{local}, is then presented as:

$$F_{local} = \frac{i_0 \gamma_w \tan \phi_s}{\gamma_{bs}} \qquad \text{2-8}$$

where ϕ_s = the angle of internal friction for the saturated soil, and γ_{bs} = the buoyant unit weight of the slurry saturated soil.

The authors also discuss factors that affect whether the filter cake will form and how far the slurry will penetrate the trench walls when no filter cake forms. Based on laboratory test results, a filter cake will form if the D_{15} size of the soil is less than 0.4mm for pure water-bentonite slurry or if the D_{15} size of the soil is less than 9 times the D_{85} size of the slurry with suspended particles. Factors that affect the penetration depth of the slurry include the head difference between the trench and the surrounding groundwater, and the bentonite concentration in the slurry.

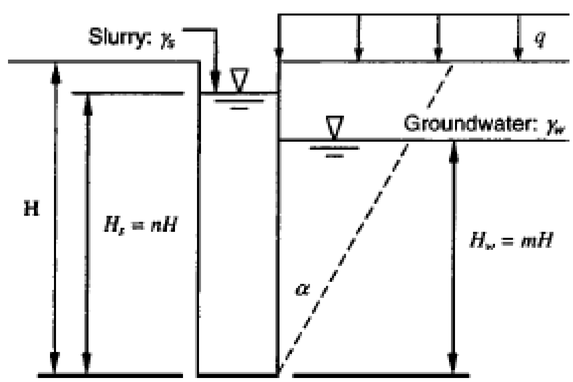


Figure 2-3 Definitions of variables for global stability of a trench where a filter cake has formed (after Filz et al. 2004, with permission from ASCE)

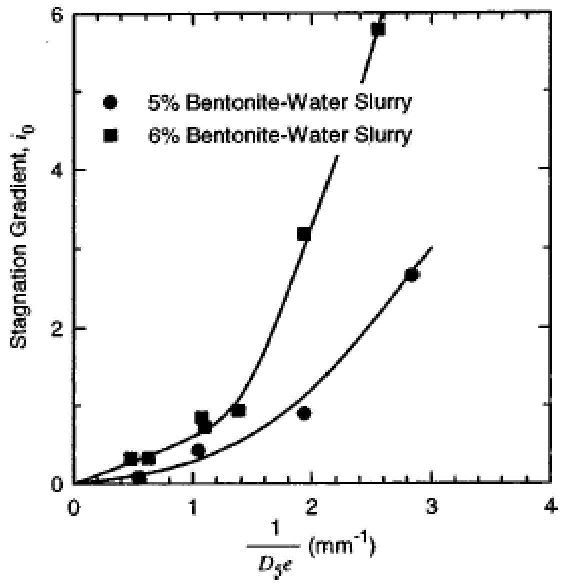


Figure 2-4 Chart for estimating stagnation gradient, i_0, of slurry (after Filz et al. 2004, with permission from ASCE)

Britton (2001)

This PhD dissertation presents the results of hydraulic conductivity and contaminant transport studies performed on pilot scale soil-bentonite cutoff walls constructed at a testing facility at Virginia Tech. The aspects of the study that are relevant to this dissertation where published in two journal articles Britton et al. (2005) and Britton et al. (2004), which are summarized above. The dissertation does present test data and results and greater details of the study than is presented in the journal articles.

Fox (2004)

Fox (2004) presents an analysis procedure for two- and three-dimensional stability of a trench supported by water-bentonite slurry. The analysis is based on Coulomb-type force equilibrium and considers two cases of drained effective stress conditions and one case of undrained total stress conditions. The first effective stress case considers the formation of a tension crack above the groundwater table. The geometry and forces considered for this analysis are presented in Figure 2-5. The second effective stress case is for cases where the base of the tension crack is below the groundwater table and the total stress case uses undrained strength and considers no pore water pressure (surface fluid pressures in the trench and tension crack are considered). Both of these cases and use similar geometry and forces to those shown in Figure 2-5. Using force equilibrium, equations are derived for three- and two-dimensional stability for all three cases. Due to the complexity of the equations they are not presented in this study. However, the equations could easily be programmed into a spreadsheet.

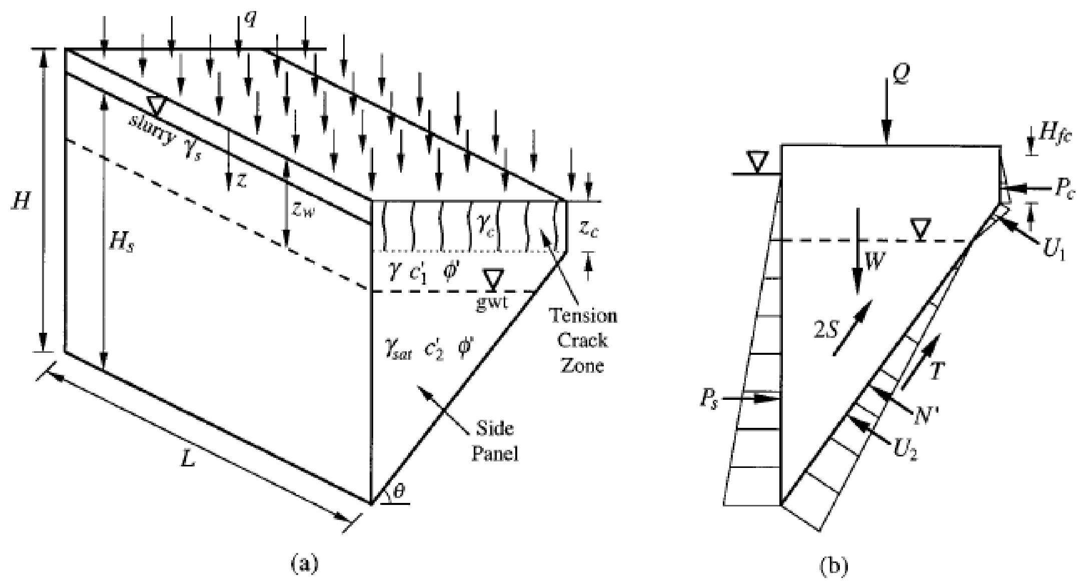


Figure 2-5 Geometry (a) and forces (b) on a failure wedge into a slurry-supported trench with a tension crack above the groundwater table (after Fox 2004, with permission from ASCE)

An example calculation was performed to illustrate the effects of trench length on the three-dimensional factor of safety. For a 65-foot deep trench in a fine grained soil deposit the factor of safety was found to decrease rapidly with increasing trench length up to a trench length of about 65 feet. Beyond 65 feet the decrease in stability continued at a slower rate asymptotically approaching the factor of safety for the two-dimensional case. This example illustrates the benefits of excavating a trench in short sections and provides a means for assessing a maximum allowable length of trench during construction.

Xanthakos (1979)

This textbook presents the 1979 state-of-the-art of slurry wall construction. The book covers the construction of slurry trenches including: stability, soil-slurry interaction, slurry design, and excavation equipment. Types of slurry walls covered include ductile cutoff walls, semirigid and rigid cutoff walls, and diaphragm walls. Various wall types and applications are also covered.

Millet, Perez, and Davidson (1992)

This paper presents the 1992 state-of-the-practice in the USA for the construction of structural diaphragm walls and cutoff barriers. Three types of cutoff barriers are presented: soil-bentonite, cement-bentonite, and plastic concrete. The authors discuss three main design criteria that should be considered for cutoff walls: permeability, deformability, and permanence.

The permeability is considered to be a function of the continuity and integrity of the wall, the wall thickness, the backfill properties, the backfill placement techniques, and the type of connection with surface structures. The continuity and integrity of the wall are largely functions of the excavation and trench cleaning techniques, the stability of the excavation, backfill properties during placement, and backfill placement techniques. Obviously, wall thickness has a large effect on the hydraulic transmissivity through the wall. In addition, the wall thickness of soil-bentonite walls is an important factor contributing to the potential for hydraulic fracturing of the wall. In narrow spoil-bentonite walls settlement of the backfill and drag along the walls may lead to low effective stress within the wall and the potential for hydraulic fracturing through the wall. The effects that the proportions of the various constituents have on the final properties of the backfill are discussed. Connections with surface structures can be a source of concentrated leaks in barriers if the settlement potential of the barrier infill is not taken into consideration and a gap is allowed to form below the surface structure.

Deformability of the barrier is important within and under dam embankments due to the strain and stress redistribution that may occur in embankments and foundations after the construction of the cutoff. Deformability of a barrier is largely a function of the backfill type and mix design. Soil-bentonite barriers are usually considered deformable. However, the coarser the gradation of the base soil used in making the soil-bentonite backfill the stiffer the barrier will be. The deformability of a cement-bentonite wall is generally a function of the cement-water ratio and the bentonite-water ratio. Plastic-concrete barriers are expected to increase in strength and stiffness (decrease in ductility) with the addition of aggregate and increase in cement-water ratio and increase in ductility with increasing bentonite content.

The permanence of cutoffs is generally considered to be a function of the chemical properties of the environment. Under normal environmental conditions the cutoffs are usually considered to be permanent structures. However, in corrosive or contaminated environments the permeability of backfill materials containing bentonite has been shown to increase over time.

Mitchell and Rumer (1997)

This reference provides an evaluation of the existing technology for subsurface waste containment barriers. While geared toward the containment of subsurface contamination, it does present an overview of many of the construction technologies used in the construction of seepage barriers in dams. The relevant construction techniques include: soil-bentonite cutoff walls, plastic concrete cutoff walls, deep soil mixed cutoff walls, jet-grouted cutoff walls.

Vanel (1992)

This paper presents a system, called the CWS system, for making the joints between panels of a diaphragm wall watertight. The CWS system uses a form to cast one to three plastic or rubber waterstop blades along the leading edge of a primary panel of the wall. The form stays in place after the concrete of the primary panel is placed and acts as a guide for the excavating tool during the excavation of the secondary panel. After excavation of the secondary panel the guide is removed allowing concrete to be placed around the remainder of the waterstop blades. Several examples where the CWS system has been successfully used are presented.

Tamaro and Poletto (1992)

This reference discusses the process of constructing structural slurry walls and identifies areas where special scrutiny is required to provide adequate quality control. The importance of using guide walls to control the line and grade of the slurry wall is presented along with recommendations for guide wall design and specification. The configuration of the panels and the construction details of the panel joints are discussed. Wall reinforcement techniques are reviewed. Recommendations are also provided for the design, specification, and inspection of the following design components:

- Concrete mix,
- Bentonite slurry mixing and placement,
- Excavation tools and techniques,
- Placement of reinforcement,
- Concrete placement,
- Tolerances and finishes,
- Recordkeeping.

A sample inspector's checklist is provided at the end of the paper.

Breek (1992)

This paper presents a method for constructing thin slurry walls using a driven steel beam. The procedure uses a specialized steel beam driven into place using a pneumatic hammer. While the beam is being withdrawn by vibrating movement, concrete is tremmied into place through the beam. The benefits of this walls system are the ability to construct walls on the order of 10 to 20 centimeters thick and not removing polluted soil from the ground.

Piping Development in Dams

Foster and Fell (1999a)

This University of New South Wales research report presents criteria for assessing the performance of filters in dams that do not satisfy modern design criteria. The criteria established divide the substandard filters into four categories based on filter test behavior. These categories, *no erosion*, *some erosion*, *excessive erosion*, and *continuing erosion*, are separated by the *continuing erosion boundary*, the *excessive erosion boundary*, and the *no erosion boundary* as illustrated in Figure 2-6. The boundaries are defined in terms of ratios of the DF15 (the D_{15} of the filter material) to the DB85 (the D_{85} of the base soil). The numerical values of the DF15/DB85 ratios differ for different types of base soil, therefore, the boundaries presented on Figure 2-6 should be considered schematic rather than quantitative.

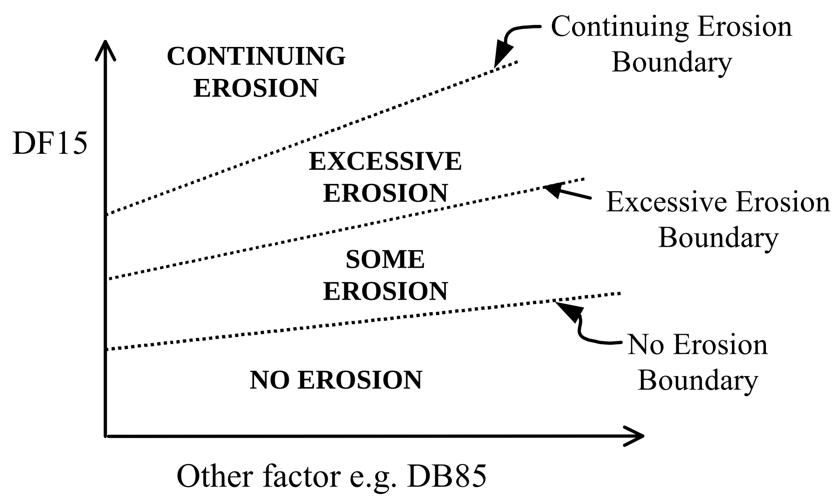


Figure 2-6 Filter behavior categories and boundaries (after Foster and Fell 1999, with permission from the University of New South Wales)

The remainder of the report discusses statistical analyses of laboratory test results and case histories that are performed to define the *continuing erosion* and *no erosion* boundaries (grouping the *some erosion* and *excessive erosion* categories into one). These analyses were performed for four base soil groups:

- Base Soil Group 1: Fine silts and clays (≥85 percent finer than the No. 200 sieve),

- Base Soil Group 2: Sandy clays and silts (35-85 percent finer than the No. 200 sieve),

- Base Soil Group 3: Clean sands and silty and clayey sands with low fines content (0-15 percent finer than the No. 200 sieve)

- Base Soil Group 4: Clayey and silty sands (15-35 percent finer than the No. 200 sieve).

The results of the analyses were distilled into proposed criteria for the two aforementioned boundaries. The analysis results and proposed criteria are presented in Table 2-1 for the *no erosion* boundary and in Table 2-2 for the *excessive* and *continuing erosion* boundaries.

Table 2-1 Summary of statistical analyses results and proposed criteria of the *no erosion* boundary (after Foster and Fell 1999, with permission from the University of New South Wales)

Base Soil Group	**Fines content *(1)***	**Design Criteria of Sherard and Dunnigan (1989)**	**Range of DF15 for No Erosion Boundary**	**Proposed Criteria for No Erosion Boundary**
1	≥ 85%	DF15 ≤ 9 DB85	6.4 - 13.5 DB85	DF15 ≤ 9 DB85 **(2)**
2	35 - 85%	DF15 ≤ 0.7mm	0.7 - 1.7mm	DF15 ≤ 0.7mm **(2)**
3	< 15%	DF15 ≤ 4 DB85	6.8 - 10 DB85	DF15 ≤ 7 DB85
4	15 - 35%	DF15 ≤ (40-pp%75µm) x (4DB85-0.7)/25 + 0.7	1.6 - 2.5 DF15 of Sherard and Dunnigan design criteria	DF15 ≤ 1.6 DF15d, where DF15d = (35-pp%75µm)(4DB85-0.7)/20+0.7

Notes: (1) The subdivision for soil group 2 and 4 was modified from 40% passing 75µm, as recommended by Sherard and Dunnigan (1989), to 35% based on the analysis of the filter test data. The modified soil groups are termed group 2A and 4A.
The fines content is the % finer than 75µm after the base soil is adjusted to a maximum particle size of 4.75mm.
(2) For highly dispersive soils (Pinhole classification D1 or D2 or Emerson Class 1 or 2), it is recommended to use a lower DF15 for the no erosion boundary.

- For soil group 1 soils, suggest use the lower limit of the experimental boundary, i.e. DF15 ≤ 6.4 DB85.
- For soil group 2A soils, suggest use DF15 ≤ 0.5mm.

Published papers summarizing the information in this report were prepared by the authors for the 8th Australia New Zealand Conference on Geomechanics (Foster and Fell 1999c) and the American Society of Civil Engineers Journal of Geotechnical and Geoenvironmental Engineering (Foster and Fell 2001).

Table2-2 Summary of statistical analyses results of statistical analysis and proposed criteria of the *excessive* and *continuing erosion* boundaries (after Foster and Fell 1999, with permission from the University of New South Wales)

<table>
<tr><th>Base Soil</th><th>Proposed Criteria for Excessive Erosion Boundary</th><th>Proposed Criteria for Continuing Erosion Boundary</th></tr>
<tr><td>Soils with DB95<0.3mm</td><td>DF15 > 9 DB95</td><td rowspan="5">For all soils:
DF15 > 9DB95</td></tr>
<tr><td>Soils with 0.3<DB95<2mm</td><td>DF15 > 9 DB90</td></tr>
<tr><td>Soils with DB95>2mm and fines content >35%</td><td>• average DF15 should be greater than the DF15 which results in an erosion loss of 0.25g/cm^2 in the continuing erosion filter test
• coarse limit DF15 should be greater than the DF15 which gives an erosion loss of 1.0g/cm^2 in the continuing erosion filter test</td></tr>
<tr><td>Soils with DB95>2mm and fines content <15%</td><td>DF15 > 9 DB85</td></tr>
<tr><td>Soils with DB95>2mm and fines content 15-35%</td><td>DF15 > 2.5 DF15design,
where DF15design is given by:
DF15design=(35-pp%75µm)(4DB85-0.7)/20+0.7</td></tr>
</table>

Fell, Wan, Cyganiewicz, and Foster (2001)

This University of New South Wales research report presents a method for estimating the time for the progression of internal erosion and piping and the development of a breach of the dam leading to failure. These processes, *progression* and *breach,* account for the later two phases of the four phased model for seepage failures in dams developed by Foster and Fell (1999b, 2000). The authors discuss three main issues that affect the progression phase and the potential breach mechanisms. The factors affecting the progression phase are the ability of the soil to support a roof, potential for enlargement of pipe, and potential for limiting flows by crack filling. Breach mechanisms include: crest settlement, gross enlargement of the pipe, sinkhole on the crest, unraveling of the downstream slope, or instability of the downstream slope.

To assist in the assessment of the time for progression the authors present three tables (presented as Tables 2-3 to 2-5) that provide guidance for assessing the influence that the characteristics of the dam and the soils making up the dam have on the likelihood of the following processes occurring: a pipe roof being supported, enlargement of the pipe, and upstream flow limitation within the pipe. Table 2-3 is for assessing the likelihood that a pipe will be supported and the fines content, degree of saturation, and plasticity of the embankment and foundation soils. Table 2-4 is for assessing the likelihood of pipe enlargement and takes into account soil properties such as: plasticity index, clay fraction, dispersivity, level of compaction, and compaction water content of the embankment and foundation soils. Finally, the likelihood of flow limitation occurring is assessed by Table 2-5 that correlates the likelihood with the presence of a crack filling (cohesionless) material in the dam zoning and the permeability of the exterior zones (upstream shell).

Table2-3 Table for assessing the likelihood to support a pipe (after Fell et al. 2001, with permission from the University of New South Wales)

Factor	Influence on Likelihood of Fill or Foundation Materials Supporting the Roof of a Pipe		
	More likely	Neutral	Less likely
(a) Embankment materials			
Fines content (% finer than 0.075mm)	Fines content > 15%	Fines content <15% and >5%	No fines or fines content < 5%
Degree of saturation	Partially saturated (first filling)		Saturated
(b)Foundation materials			
	Piping through soils with cohesive fines Cohesive layer overlying piped material Piping through solution features in rock Piping below rigid structures (e.g. spillway)	Well graded sand and gravel	Homogeneous, cohesionless sands

Table2-4 Table for assessing the likelihood of pipe enlargement (after Fell et al. 2001, with permission from the University of New South Wales)

Factor	Influence on Likelihood of Pipe Enlargement		
	More likely	Neutral	Less likely
Embankment and foundation			
Hydraulic gradient across the core	High	Average	Low
Soil Type	Very Uniform fine cohesionless sand (PI<6) or well graded cohesionless soil (PI<6)	Well graded material with clay binder (6<PI<15)	Plastic clay (PI>15)
Clay fraction (percent passing 0.002 mm)	Low clay percentage (e.g.<5%)		High clay percentage (e.g. > 50%)
Pinhole dispersion test	Dispersive soils, pinhole D1, D2	Potentially dispersive soils, pinhole PD1, PD2	Nondispersive soils, pinhole ND1, ND2
Embankment			
Compaction density ratio	Poorly compacted, <95% standard compaction density ratio	95 - 98% standard compaction density ratio	Well compacted >98% standard compaction density ratio
Compaction water content	Dry of standard optimum water content (approximately 3% or less)	Approximately 1-2% drier than standard optimum water content	Standard optimum or wet of standard optimum water content
Saturation	As compacted, partially saturated		Saturated after compaction
Foundation			
Relative density or Consistency	Loose Soft	Medium dense Stiff	Dense Very stiff

Table2-5 Table for assessing the likelihood of upstream flow limitation (after Fell et al. 2001, with permission from the University of New South Wales)

Factor	Influence on Likelihood of Upstream Flow Limitation		
	Unlikely	Neutral	Likely
Filling of cracks by washing in of material from upstream	Homogeneous zoning. Upstream zone of cohesive material.		Zone upstream of core capable of crack filling (cohesionless soil)
Restriction of flow by upstream zones or concrete element in dam	Homogenous zoning. Very high permeability zone upstream of core (e.g. coarse grained rockfill)	Medium to high permeability zone upstream of core	In zoned dam, medium to low permeability granular zone upstream of core (e.g. fine grained or dirty rockfill). Central concrete corewall and concrete face rockfill dams

The authors discuss the fact that it is difficult to distinguish between the completion of the progression stage and the beginning of the breach phase. Therefore, the assumption is made that dams in which the progression phase develops rapidly are also likely to breach rapidly. For this reason, only one additional table (Table 2-6) is presented to assist in the assessment of time to failure. Table 2-6 considers the gradation and plasticity of the soils in the downstream zone of the embankment and relates these properties to the rapidity of breach.

Table2-6 The influence of material in downstream zone of embankment or in foundation on likely time for development of breach (after Fell et al. 2001, with permission from the University of New South Wales)

Material description	Likely breach time
Coarse grained rockfill	Slow-medium
Soil of high plasticity (liquid limit>50%) and high clay sized content including clayey gravels	Medium-rapid
Soil of low plasticity (liquid limit<35%) and low clay size content, all poorly compacted soils, silty sandy gravels	Rapid-very rapid
Sand, silty sand, silt	Very rapid

The likely time for progression and breach is then assessed by considering the combined effects of the four tables discussed above (three tables for assessing progression and one for assessing breach). Table 2-7 presents the approximate likely times in intervals that range from less than 3 hours to weeks, months or years.

Table 2-7 Table for estimation of time for progression of breach linked to development of piping (after Fell et al. 2001, with permission from the University of New South Wales)

Factors Influencing the Time for Progression and Breach				Approximate likely time - qualitative	Approximate likely time - quantitative
Ability to support a roof[a]	Rate of erosion[b]	Upstream flow limiter[c]	Breach time[d]		
Yes	R or VR	No	VR or R-VR	Very rapid	<3 hours
Yes	R	No	R	Very rapid to rapid	3-12 hours
Yes	R-M	No	VR		
Yes	R	No	R-M	Rapid	12 -24 hours
Yes	R-M, or M	No	R		
Yes	R	Yes	R or VR		
Yes	R	No	M or S	Rapid to medium	1-2 days
Yes	R-M, or M	No	M or M-S		
Yes	R or R-M	Yes	R or R-M		
Yes	M or R-M	No	S	Medium to slow	2-7 days
Yes	R-M or M	Yes	S		
Yes	M	Yes or No	S	Slow	Weeks –even months or years

Notes: VR = Very Rapid, R = Rapid, M = Medium, S = Slow.
[a]Estimated using Table 2-3
[b]Estimated using Table 2-4
[c]Estimated using Table 2-5
[d]Estimated using Table 2-6

The authors discuss detection of internal erosion and piping and make the following conclusions:

- The most common means of detection of piping and internal erosion is monitoring of seepage visually or with instrumentation measurements,

- It is not common to find conclusive evidence of piping and internal erosion based on the monitoring devices,

- The inability to detect internal erosion often has to do with the location where the mechanism begins (i.e. within the dam core) and the rapidity with which the mechanism initiates (i.e. cracking of the embankment or core, hydraulic fracture).

- Failures by piping in the foundation or by piping beginning in the embankment and through the foundation are mostly from backward erosion following heave or hydraulic fracture, and would not often be expected to be preceded by large increases in seepage flow.

- In a vast majority of cases of failure by piping, the reservoir was within 1 meter of the historic high reservoir level when progression of erosion to form a pipe occurred in the embankment. There is less correlation with reservoir levels and piping in the foundation and from the foundation to the embankment.

A summary of the findings presented in this report are published in a paper in the American Society of Civil Engineers Journal of Geotechnical and Geoenvironmental Engineering (Fell et al. 2003).

<u>Foster and Fell, 1999b</u>

This University of New South Wales research report presents some of the early work that eventually led to the publication of UNSW reports R-428 (Fell et al. 2004) and R-436 (Fell and Wan 2005) which are summarized below. The methodologies and guidance presented in this report have either been represented in the R-428 and R-436 reports or have been superseded by more up to date methodologies and guidance.

Fell, Wan, and Foster (2004)

Fell et al. (2004) is a University of New South Wales research report that presents methods for estimating the probability of failure of embankment dams by internal erosion and piping within the embankment. The authors present a framework model within which they divide the process of dam failure by internal erosion and piping into four sequential phases: *initiation, continuation, progression and breach*. The *initiation* phase consists of the development of a concentrated leak and the initiation of erosion along this leak. The *continuation* phase will develop if eroded soil particles are able to be removed from the area of the concentrated leak. The *progression* phase consists of the enlargement of the concentrated leak to form a pipe. Finally, the *breach* phase occurs when a mechanism has developed to the point where failure of the dam occurs. A schematic illustration of the four phases of failure for backward erosion in the embankment is presented in Figure 2-7. Four-phased models are presented for the development of piping for failure mechanisms involving backward erosion in the embankment, erosion in a concentrated leak in the embankment, backward erosion in the foundation, and piping from the embankment to foundation.

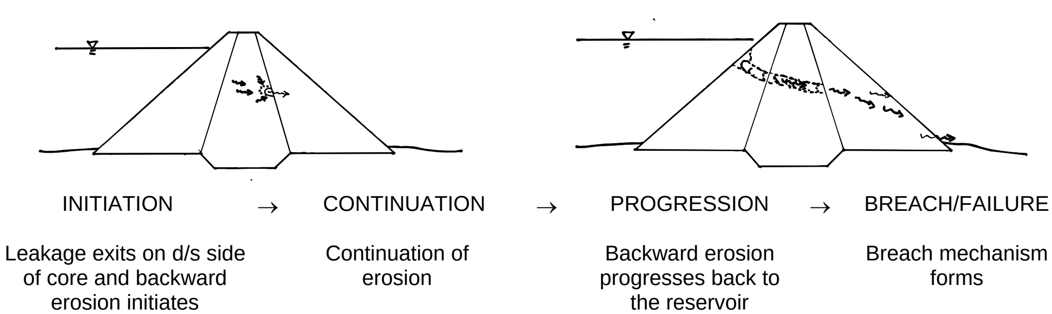


Figure 2-7 A four-phased model for the development of piping failure initiated by backward erosion (after Fell et al 2004, with permission from the University of New South Wales).

The report presents an event tree-based method for assessing the risk of failure of embankment dams due to seepage or internal erosion through the embankment. Guidance for developing failure modes and event trees for individual dams is provided. The subsequent chapters of the report present recommended methods for assigning probabilities to the nodes of the event trees. Specifically, the chapters deal with the four phases of seepage failure development: *initiation, continuation, progression* and *breach*.

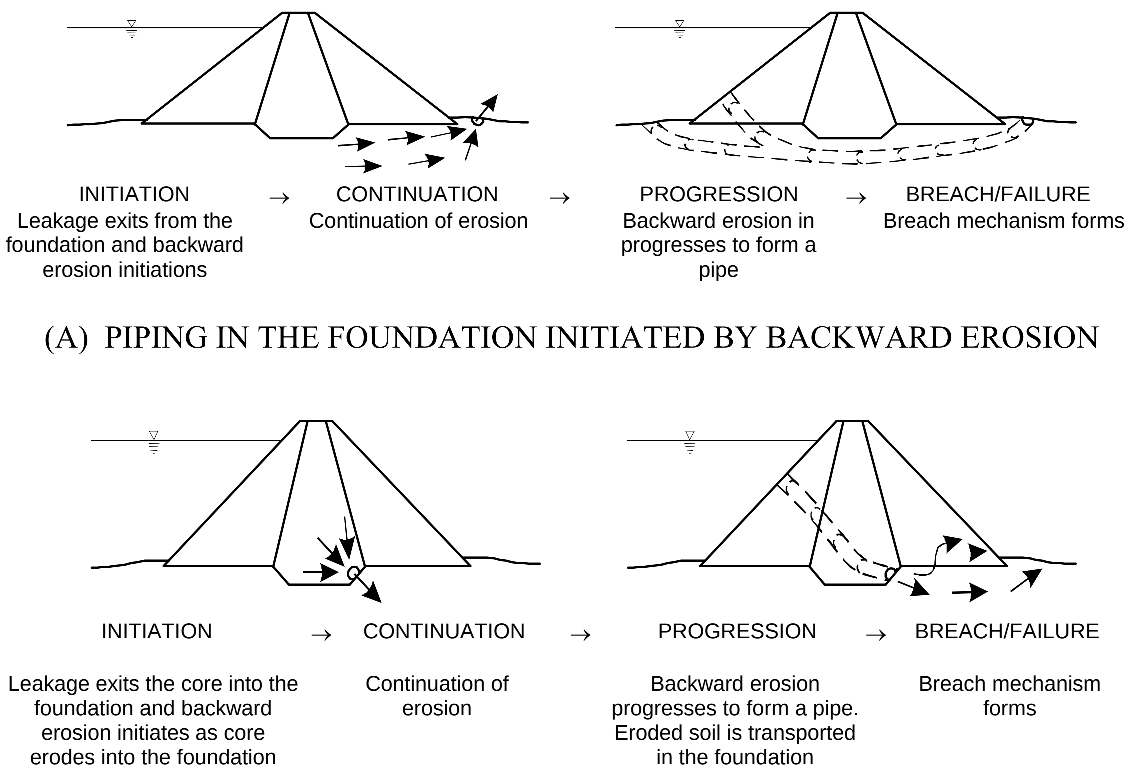
The first of these chapters provides aids for assessing the probability of the *initiation* phase, development of a crack in an embankment and the initiation of erosion. A series of tables are presented to aid in assigning probabilities to the various branches of the event trees based on an historic data approach. The historic data approach looks at the past performance of dams to predict the future performance of dams with similar characteristics. The first table presents values of the average probability of initiation of several piping modes based on a statistical evaluation of a large number of dams. Four subsequent tables present factors based on dam characteristics (height, zoning, construction, materials, observed and measured performance, etc.) that are used to modify the probabilities obtained in the first table to the specific characteristics of the dam being assessed. Other aids, mostly in the form of graphs and tables, are presented to assist engineering judgment with regard to the likelihood of crack development and erosion initiation.

The chapter addressing the continuation phase deals predominantly with filtering behavior. Guidance is also provided to assist in the assessment of the likelihood of erosion occurring into cracks and bedrock joints. The chapter regarding the progression phase provides guidance for assessing the likelihood of eroding soil maintaining a pipe without collapse, the likelihood of crack filling action occurring and the likelihood of upstream flow limitation preventing progression from occurring. The chapter on the breach phase provides guidance and judgment aids for assessing the probability of various beach modes.

The final chapter of the report aids in the assessment of the probability that a failure mode will be detected and intervention and repair will be possible before dam breach occurs. The guidance involves assessing the rate at which the internal erosion and piping will develop and progress, and the possible means for intervention.

Fell and Wan (2005)

This report parallels the report presented above (Fell et al. 2004) and presents methods for estimating the probability of failure of embankment dams by internal erosion, piping within the foundation, and piping from the embankment to the foundation. A similar four-phased model of the process is presented. Schematic illustrations of these models are presented in Figure 2-8.



(B) PIPING FROM THE EMBANKMENT TO FOUNDATION INITIATED BY BACKWARD EROSION

(C)

Figure 2-8 Four-phased models for the development of piping failure in the foundation and from the embankment to the foundation (after Fell and Wan 2005, with permission from the University of New South Wales).

The authors discuss the ways that internal erosion is initiated in the foundation or from the embankment to the foundation. For internal erosion in the foundation three main categories of initiating mechanism are presented: backward erosion, suffusion, and erosion in a concentrated leak. The triggering conditions for these mechanisms are discussed. For piping from the embankment to the foundation to initiate there must be defects or concentrated seepage paths into which the embankment soil can erode. These seepage paths may be bedrock joints, open graded soils, or other defects.

The report discusses an event tree-based method for assessing the risk of failure of embankment dams due to seepage or internal erosion through the foundation or from the embankment into the foundation, and provides guidance for developing failure modes and event trees for individual dams. The subsequent chapters present discussion and guidance to assist in assigning probabilities to the nodes of the event trees. Specifically, the chapters deal with the four phases of seepage failure development: initiation, continuation, progression and breach.

The first of these chapters provides discussion on the initiation of the various modes of internal erosion and presents a recommended methodology for assessing the probability of these modes initiating. The methods of Sellmeijer and his co-workers (de Wit et al. 1981, Sellmeijer and Koenders 1991) and those of Schmertmann (2000) for assessing the potential for initiation of backward erosion are discussed and a proposed method for assessing the probability of initiation based on exit gradients is proposed. A method for assessing the probability for initiating internal erosion by suffusion is proposed that considers the gradation of the soil and the hydraulic gradient imposed on the soil. A method for assessing the probability for initiating internal erosion along a concentrated leak is proposed that takes into account the *erosion rate index* developed by Wan and Fell (2002, 2004a, 2004b), the width of the concentrated leak, and the hydraulic gradient in the leak. Finally, the historic data approach to assessing the likelihood of initiation of internal erosion is proposed as a starting point from which the quantifying procedures discussed above can be used to further refine estimates of initiation probability.

The chapter that addresses the *continuation* phase deals predominantly with filtering behavior. Guidance is provided to assist in the assessment of the filtering effectiveness of soils based on gradation, the ability of bedrock joints to filter soil particles, and the fraction of the soils gradation that may be eroded by suffusion.

The chapter that addresses the *progression* provides guidance for assessing the likelihood of eroding soil maintaining a pipe without collapse, the likelihood of crack filling action and the likelihood of upstream flow limitation preventing progression. In assessing the *progression* phase each of the modes previously considered for the *initiation* and *continuation* phases; backward erosion, suffusion, concentrated leak in foundation, and internal erosion from embankment to foundation; are treated separately due to the unique nature of the mechanisms associated with each mode. Aids to judgment are provided to assist in assessing the likelihood of eroding soil holding a roof, crack filling action, and upstream flow limitation.

The chapter addressing the *breach* phase provides guidance and judgment aids for assessing the probability of various beach modes. The final chapter of the report aids in the assessment of the probability that a failure mode will be detected and intervention and repair will be possible before dam breach occurs. The guidance involves assessing the rate at which the internal erosion and piping will develop and progress, and the possible means for intervention.

McCook (2004)

McCook (2004) presents a comprehensive discussion on the mechanisms of piping and internal erosion in dams. The paper reviews the various terminologies used to describe the processes by which water flow can cause erosion within an embankment and proposes lumping the processes into the two categories of 1) piping and 2) internal erosion. Definitions for *piping* and *internal erosion* are then presented and the various mechanisms that contribute to seepage erosion are discussed.

The term *piping* is used by McCook to describe the erosion of soil particles due to percolation of water through a soil body with no preferential seepage path. The water flow in this scenario is thus assumed to be concentrated along soil particles or adjacent structures. The key parameter in assessing whether piping will occur is the hydraulic gradient at locations where soil particles can be eroded. The term *internal erosion* is used to describe the erosion of soil particles that occurs due to flow of water in concentrated pathways such as defects or cracks in the soil mass. The key parameter in assessing whether internal erosion will occur is the velocity of the water flowing through the concentrated pathway. In dams internal erosion failures and incidents are much more common than piping incidents.

McCook (2004) discusses the susceptibility of soil types to piping and internal erosion. Fine-grained, poorly sorted sands are the most susceptible to piping erosion. Courser sand particles are more difficult to dislodge and thus have a higher resistance to erosion. Soils with plastic clay fines have a high resistance to piping due to the electro-chemical bonding forces associated with these soils that resist detachment by seepage forces. In contrast, soils having the ability to support an open crack without collapse are generally considered susceptible to internal erosion. Dispersive clays are especially susceptible, followed closely by low plasticity silts (ML and CL-ML). Sands and gravels with sufficient fines to support a crack are also susceptible to internal erosion. Erodible bedrock such as poorly cemented sandstone, gypsum, and limestone may also be susceptible to internal erosion.

McCook (2004) presents seven scenarios where piping failures can develop in dams and six scenarios where internal erosion piping failures can develop. For each of the scenarios, the sequence of events needed for the failure to progress is presented and examples of failures and incidents that occurred by the process described are referenced. Finally, design measures to prevent failures and incidents are presented.

Tomlinson and Vaid (2000)

Tomlinson and Vaid (2000) performed an experimental study on the effects of several parameters on the initiation of piping erosion in relatively uniform soil samples. Using a laboratory apparatus they investigated the effects of grain size ratio, gradient, rapidity of gradient increase, confining pressure, and thickness of filtering layer. They found that the grain size ratio had the largest effect on the susceptibility of piping of soil through a filter system. In general, when the ratio of the D_{15} of the filter (D_{15f}) to the D_{85} of the soil (D_{85s}) was less than 8, piping erosion did not occur in the tested soils under the test conditions. When the D_{15f}/D_{85s} was greater than 12, piping erosion occurred under moderate gradients. When the D_{15f}/D_{85s} was between 8 and 12, piping could be induced by increasing the gradient or changing the other parameters listed above.

The results of the study indicated that when gradients are applied rapidly to the test specimens that the potential for erosion increases. This is thought to be due to the ability of the soils to form bridges in the filter materials at lower gradients that inhibit erosion at higher gradients. Increasing confining pressure was found to slightly increase the erosion potential. There was found to be a minimum filter thickness below which the filters effectiveness was significantly reduced. This effect was attributed to the inability of the larger soil particles in the filter to form bridges that restrain the finer particles.

Gould and Lacy (1993)

This paper discusses control of developing seepage problems in dams. The first part of the paper discusses evidence of developing seepage problems. The most common evidence of developing seepage problems is an increase in seepage outflow. Other evidence such as changes in piezometer levels, embankment settlement, sinkholes, seeps at embankment toe, and turbidity in seepage outflow is discussed.

Conditions that lead to seepage problems are discussed including: design defects, construction defects, and unfavorable geology not taken into account in design. Older dams are more susceptible to design defects since the design of newer dams have benefited from the lessons learned in the past.

Remedial measures for dealing with seepage problems are discussed in two categories: drainage and barriers. Of interest to this study is the discussion of seepage barriers. The authors present a brief summary of the state-of-the-practice for slurry trench cutoffs. They discuss the benefits of each of the three types of trench backfills: soil-bentonite, cement-bentonite, and concrete. A brief discussion of hydraulic fracturing of embankments is presented.

Summary

There is limited published literature that is directly related to the topic of long-term performance of seepage barriers in dams. Marsal and Resendiz (1971) discuss mechanisms that can affect the long-term performance of seepage barriers and present two case studies that are relevant to this study. Ahmed-Zeki et al. (2000) and Telling et al. (1978a) present case histories on aspects of the performance of seepage barriers over time. Foster et al. (1998, 2000a, 2000b) present statistical analyses on how the presence of various components of dams affect the long-term performance the dam. The above references represent the published literature identified that directly relate to this study.

The remainder of this literature review concentrates on subjects that are related to the topic of long-term seepage barrier performance and provide background information that is useful in making the assessments of dam performance, performing the analyses, and developing the conclusions that are presented in this study. The literature on seepage barrier construction methods provides background on the construction techniques and barrier material properties that is useful in modeling the behavior of the barriers and identifying mechanisms that may be affecting the long-term performance of the barriers. Similarly, the literature on theoretical seepage barrier performance provides useful insights into the effects cracks, joints, and partial penetration have on the effectiveness of the barrier. Finally, the literature on the development of seepage failures in dams provides both a framework for modeling the mechanisms that may be affecting the long-term performance of seepage barriers in dams as well as providing aids for qualitatively and quantitatively assessing the factors that may influence the likelihood of these mechanisms affecting the barrier performance.