IV-4. Internal Erosion Risks for Embankments and Foundations

Key Concepts

One of the leading causes of dam and levee failures in the United States is from internal erosion of embankments (or their foundations). Unfortunately, this is a potential failure mode that cannot be completely analyzed using numerical formulae or models. However, valuable information on embankment and foundation behavior is available to help in assessing internal erosion risks. The term "internal erosion" is used by Reclamation and USACE as a generic term to describe erosion of soil particles by water passing through a body of soil. "Piping" is often used generically in the literature but actually refers to a specific internal erosion mechanism (described below).

It is recognized that risk estimating procedures, although quantitative, do not provide precise or accurate numerical results. The nature of the risk evaluation should be advisory and not prescriptive, such that site specific considerations, good logic, and all relevant external factors can be applied in decision making, rather than reliance on a "cookbook" numerical approach (Von Thun 1999). Thus, although the numbers are important, the more important aspects of a risk analysis are to: 1) develop an improved understanding of the embankment's strengths, weaknesses, and vulnerability to potential failure modes; and 2) to "build the case" for the estimates that are presented and the resulting recommended action (or inaction). As such, one of the primary objectives of the risk analysis/assessment[1] is to understand and "build the case" for the risk estimates that are developed and the resulting recommended actions. Prior to the risk assessment, the risk team should review and discuss available information, and some analyses may be necessary (e.g., filter compatibility, internal instability, vertical exit gradient, etc.). The risk team should also review pertinent case histories. A few are summarized at the end of this chapter as a starting point.

Although this chapter represents a substantial revision to the previous published version, periodic revisions will continue to be needed.

General Categories of Internal Erosion

Internal erosion failure modes can develop in response to a loading applied to the embankment or its foundation. The loading is generally characterized as either:

- Static/normal operation (i.e., reservoir level at or above a threshold elevation that would cause initiation of internal erosion) – only Reclamation explicitly considers
- Hydrologic (i.e., related to a flood or reservoir level higher than the normal operating reservoir level)
- Seismic (i.e., earthquake causes deformation and/or cracking that would cause initiation of internal erosion)

[1] Within this chapter, the terms "analysis" and "assessment" are considered synonymous.

Internal erosion potential failure modes can be grouped into general categories related to the physical location of the internal erosion pathway. Case histories of embankment failures can be related to the following general categories of internal erosion:

- Internal erosion through the embankment (Figure IV-4-1)
- Internal erosion through the foundation (Figure IV-4-2)
- Internal erosion of the embankment into the foundation (Figure IV-4-3a), including along the embankment-foundation contact (Figure IV-4-3b)
- Internal erosion along or into embedded structures such as conduits or spillway walls (Figure IV-4-4)
- Internal erosion into drains such as toe drains, stilling basin underdrains, etc.

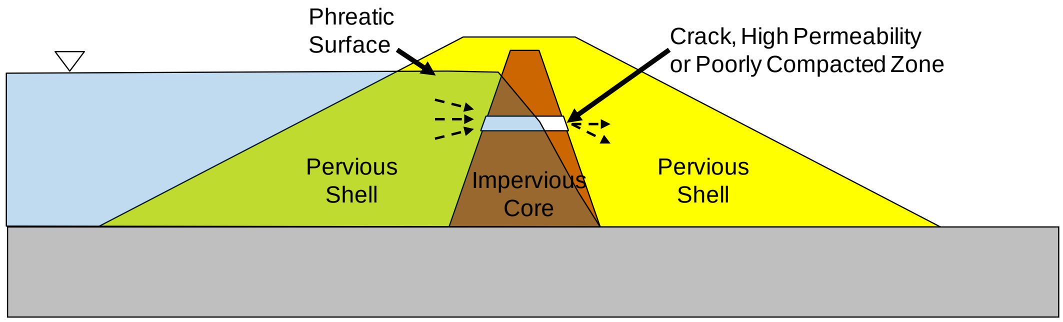

Figure IV-4-1. Internal Erosion through the Embankment

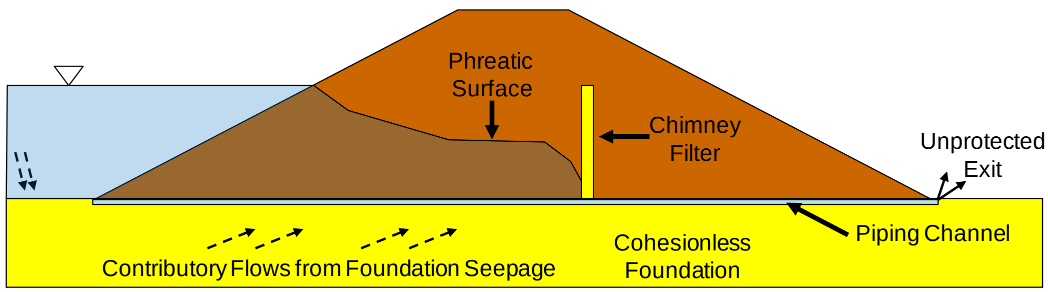

Figure IV-4-2. Internal Erosion through the Foundation

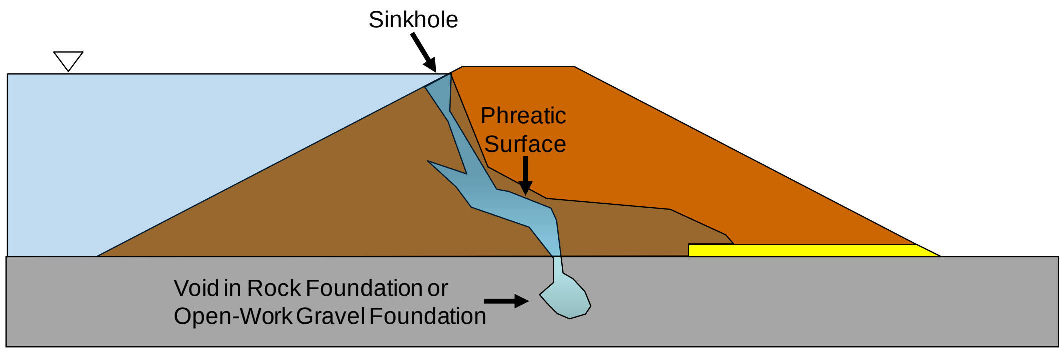

Figure IV-4-3a. Internal Erosion of the Embankment into the Foundation

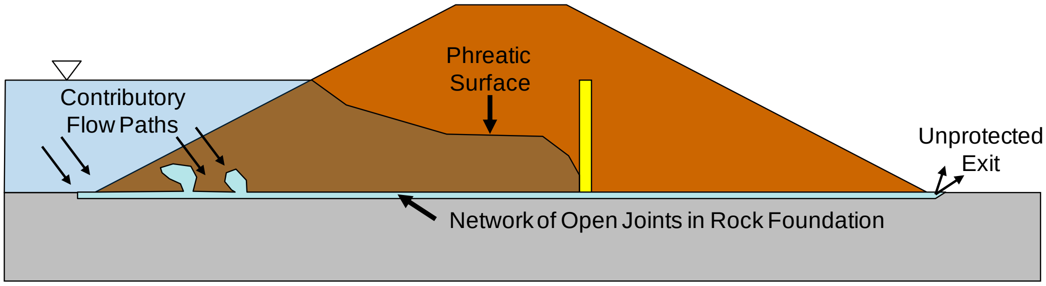

Figure IV-4-3b. Internal Erosion along the Embankment-Foundation Contact

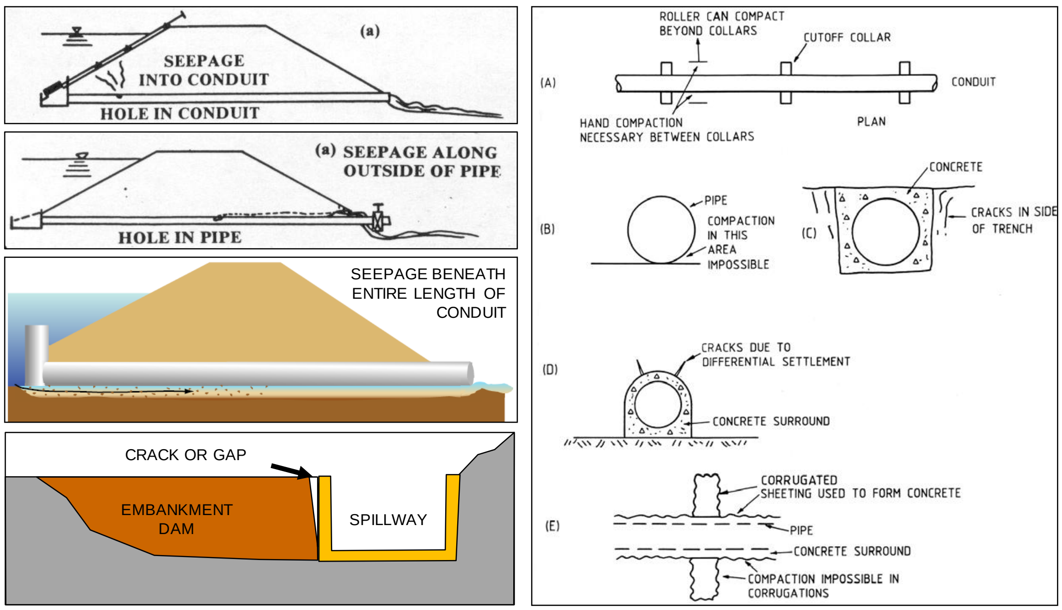

Figure IV-4-4. Internal Erosion along or into Embedded Structures (adapted from FEMA 2005, 2008 and Fell et al. 2008)

The stilling basin case history described at the end of this chapter is an example of internal erosion into drains. It is important to note that no dam failures have occurred as a result of internal erosion into drains. This is most likely because this potential failure mode would take a long time to develop, and case histories indicate intervention through early detection has been successful in stopping the internal erosion process.

The categories of internal erosion identified here are <u>not</u> potential failure mode descriptions. The potential failure mode should be identified in detail based on site-specific information and clearly described from initiation to breach. It is important to identify where the concentrated seepage path will likely form, where erosion first initiates, where the soil particles will be carried, how the erosion will progress, opportunities for detection and intervention, and how the embankment will breach.

Mechanisms of Internal Erosion

Whereas the previous discussion centered on the "categories" (or locations) of internal erosion, the following discussion describes the specific internal erosion "mechanisms."

Organizations that have studied the mechanics of internal erosion incidents have observed several different mechanical processes and have classified those incidents by mechanism to establish some degree of common terminology along with an understanding the physical factors that can lead to different internal erosion mechanisms. Both USACE's and Reclamation's mechanisms are described below. Although the terminology is different, each organization includes the same mechanical processes, as summarized in the table below. Those evaluating internal erosion should consider the specific mechanics of the potential failure modes envisioned at a specific site and provide a full description of the process, regardless of the mechanism name.

Table IV-4-1. Mechanisms of Internal Erosion

USACE (adapted from ICOLD)	Reclamation
(Note: Reclamation's description of the BEP mechanism is applicable to USACE.)	**Backward erosion piping (BEP)**: Occurs when soil erosion (particle detachment) begins at a seepage exit point and erodes backwards (upstream), supporting a "pipe" or "roof" along the way. As the erosion continues, the seepage path gets shorter, and flow concentrates in plan view, leading to higher gradients, more flow, and the potential for erosion continues to increase. Four conditions must exist for BEP to occur: 1) flow path or source of water; 2) unprotected or unfiltered exit; 3) erodible material within the flow path; and 4) continuous stable roof forms allowing pipe to form. BEP is particularly dangerous because it involves progression of a subsurface pipe towards the reservoir.
(Note: Reclamation's description of the internal migration mechanism (stoping) is applicable to USACE.)	**Internal migration (stoping)**: Occurs when the soil is not capable of sustaining a roof or pipe. Soil particles migrate downward primarily due to gravity, but may be aggravated by seepage or precipitation, and a temporary void grows in the vicinity of the initiation location until a roof can no longer be supported, at which time the void collapses. This mechanism may be repeated progressively until the core is breached or the downstream slope is over-steepened to the point of instability. Since by definition roof support is lacking, this mechanism typically leads to a void that may stope to the surface as a sinkhole. Stoping can occur in narrow central core dams constructed with broadly graded cohesionless soils (e.g., glacial till) due to

USACE (adapted from ICOLD)	Reclamation
	internal instability/suffusion, or due to open defects in rock foundations or structures embedded in the embankment. (Note: ICOLD includes stoping as global backward erosion.)
Concentrated leak erosion: involves erosion of the walls of an opening (crack) through which concentrated leakage occurs. (Note: Reclamation uses the term scour for concentrated leak erosion.)	**Scour**: Occurs when tractive seepage forces along a surface (i.e., a crack within the soil, adjacent to a wall or conduit, along the embankment-foundation contact) are sufficient to move soil particles into an unprotected area, or at the interface of a coarse and fine layer in the embankment or foundation. Once this begins, a process similar to backward erosion piping or internal migration could result. Scour does not necessarily imply a backward (upstream) development of an erosion pathway. Enlargement of an existing defect may occur anywhere along the seepage pathway.
Contact erosion: The selective erosion of fine particles from the contact with a coarser layer caused by the passing of flow through the coarser layer parallel to the contact. Lab testing suggests a difference may exist if the hydraulic attack is from leakage in a crack versus the seepage (more tortuous) in a coarse layer. (Note: Reclamation also uses the term scour for contact erosion.)	
Internal instability (Note: Reclamation's description of the mechanisms for internally unstable soils are applicable to USACE.)	**Internal Instability - Suffusion, and Suffosion**: Both are internal erosion mechanisms that can occur with internally unstable soils. It is possible that these mechanisms as well as internal migration (stoping) can occur in complex glacial environments where tills, glacio-lacustrine and outwash deposit co-exist. **Suffusion** involves selective erosion of finer particles from the matrix of coarser particles (that are in point-to-point contact) in such a manner that the finer particles are removed through the voids between the larger particles by seepage flow, leaving behind a soil skeleton formed by the coarser particles. With suffusion there is typically little or no volume change. **Suffosion** is a similar process, but results in volume change (voids leading to sinkholes) because the coarser particles are not in point-to-point contact. Suffosion is less likely under the stress conditions and gradients typically found in embankment dams. Note: This

USACE (adapted from ICOLD)	Reclamation
	condition might require consideration of BEP, cracking and concentrated leak erosion, or contact erosion.

Flaws and other physical factors that can lean to each internal erosion mechanism, and guidance for evaluating the probability of initiation of internal erosion are discussed in this chapter.

Conceptual Framework for Internal Erosion Process

Internal Erosion Process

The process of internal erosion is generally broken into four phases: 1) initiation of erosion; 2) continuation of erosion; 3) progression of erosion; and 4) initiation of a breach. The first 3 phases are illustrated in Figures IV-4-5 and IV-4-6 for internal erosion in the embankment. Similar processes apply for internal erosion in the foundation (Figure IV-4-7) and internal erosion of the embankment into or at the foundation (Figure IV-4-8).

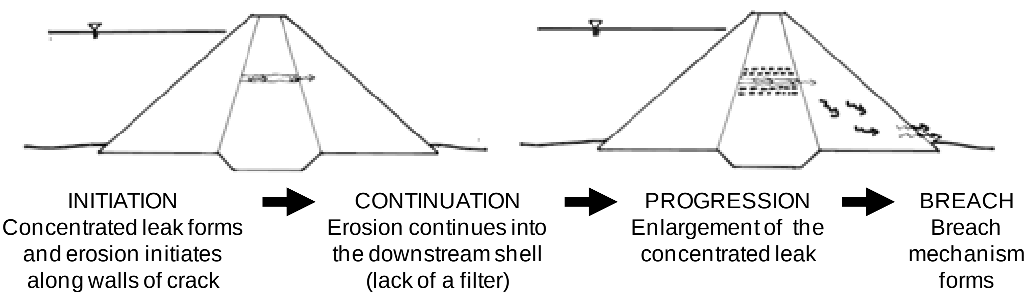

Figure IV-4-5. Internal Erosion through the Embankment Initiated by a Concentrated Leak (adapted from Fell et al. 2008)

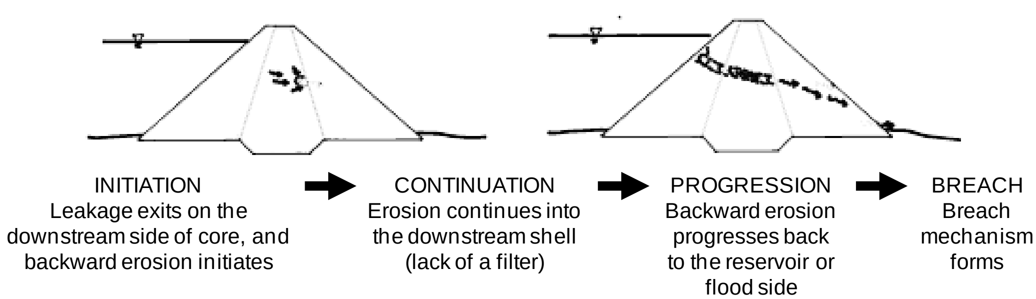

Figure IV-4-6. Internal Erosion through the Embankment Initiated by Backward Erosion (adapted from Fell et al. 2008)

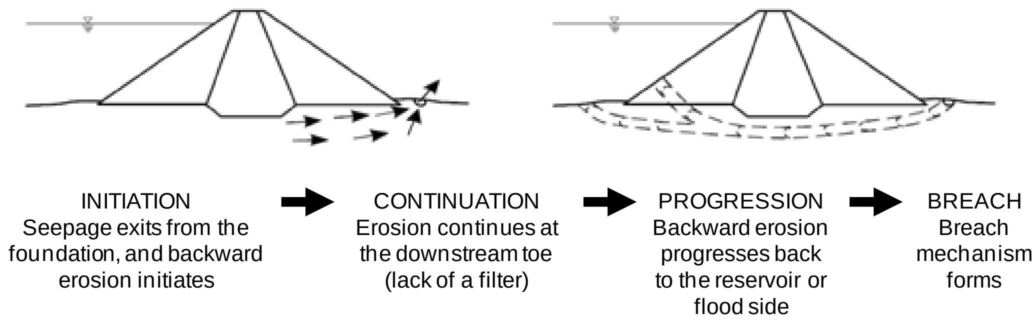

Figure IV-4-7. Internal Erosion through the Foundation Initiated by Backward Erosion (adapted from Fell et al. 2008)

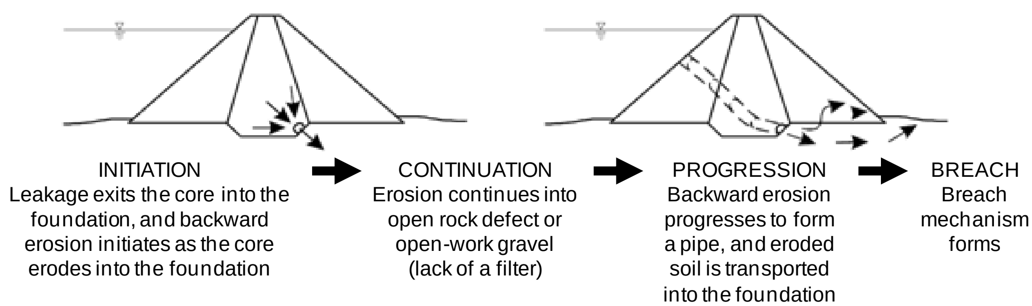

Figure IV-4-8. Internal Erosion of the Embankment into the Foundation (adapted from Fell et al. 2008)

Generic Event Tree

A generic sequence of events has been developed for internal erosion failure modes that is based on the four phases of internal erosion shown above. In addition, a threshold reservoir elevation (or several different ranges of elevations) and the likelihood of unsuccessful detection and/or intervention are assessed. Consequences are also evaluated for each event tree as discussed in Chapter III-1.

- ↳Water level at or above threshold level
  - ↳Initiation – Erosion starts
    - ↳Continuation – Unfiltered or inadequately filtered exit exists
      - ↳Progression – Continuous stable roof and/or sidewalls
        - ↳Progression – Constriction or upstream zone fails to limit flows
          - ↳Progression – No self-healing by upstream zone
            - ↳Unsuccessful detection and intervention
              - ↳ Breach (uncontrolled release of impounded water)

Depending on how the potential failure mode is envisioned, it might be appropriate to decompose the initiation event into two events: 1) flaw exists; and 2) erosion initiates given the flaw exists. Reclamation generally (but not exclusively) considers one event because historical rates of initiation within Reclamation's inventory (discussed later) can be estimated, whereas historical rates of a flaw existing where erosion has not initiated are unknown.

For projects designed to include flood risk management that have not been fully loaded and lack historical rates of initiation one could choose to decompose initiation into two events in the event tree. This will allow the identification of scenarios where the likelihood of a flaw may be a primary factor in the risk estimate. The quantification of this event tree node can provide a better understanding of how a flaw impacts both the estimate and the uncertainty in the risk estimate.

↳Water level loading (at or above threshold level)
  ↳Flaw exists – Continuous crack, high permeability zone,
                  zones subject to hydraulic fracture, etc.
    ↳Initiation – Particle detachment (erosion starts)
      ↳See above event tree for other nodes that apply.

These sequences of events can be illustrated as an event tree. The typical events are summarized in Table IV-4-2, along with brief descriptions of factors to consider for each event. More detailed tables listing numerous factors to consider for each event are included at the end of this chapter. ***The risk team should develop specific event trees for their identified potential failure modes.***

Table IV-4-2. Typical Internal Erosion Event Tree with Summary of Factors to Consider (adapted from ICOLD 2013 Draft)

Event	Basic Considerations
Water level loading	There are multiple ways to address water level loading. For dams that have been nearly fully loaded, such as many in Reclamation, one can separate potential failure modes under normal operating (static) conditions from hydrologic and seismic related potential failure modes. For dams that have significant flood storage, such as many in USACE, and in any case have not been significantly loaded or are not significantly loaded very often one can consider the full range of reservoir loading and not evaluate static loading separately. See the section later in this chapter for more details.
Flaw (if included as a separate event)	A "flaw" is a crack or gap that is or may become continuous, poorly compacted zone, or high permeability zone in which a concentrated leak may form. For Backward Erosion Piping (BEP), no flaw is required, but a continuous zone of cohesionless soil in the embankment or foundation is required.
Initiation	Does erosion initiate under the seepage gradients or potential leakage conditions? Consider the erodibility of the soil, internal erosion mechanism and estimated seepage gradient.
Continuation	Does an unfiltered or inadequately filtered exit exist to allow erosion to continue?
Progression (3 nodes included here)	Does a continuous material exist that can act as a roof for a developing pipe, or will continuous stable sidewalls be maintained along a crack? Does the upstream zoning fail to constrict or limit flows? Does the upstream zoning fail to self-heal an erosion pathway? Consider the global or horizontal gradient required for progression of a pipe to the reservoir.
Unsuccessful Detection and Intervention	Is the location of the seepage or signs of erosion (i.e., particle transport) observable? Are the type and frequency of surveillance, monitoring, and inspection sufficient to detect the seepage? Are adequate personnel, equipment, and materials available to respond? Is the project site or affected area on the embankment accessible? Is there sufficient time to intervene based on the rate of erosion? Can the reservoir be drawn down in time to prevent initiation of a breach?
Breach	Gross enlargement of a pipe or concentrated leak Sloughing or unraveling of the downstream face Sinkhole development Slope instability

Suggested Method for Evaluating Internal Erosion Risks

Table IV-4-3 below outlines the general recommended approach, with some agency specific guidance, for evaluation of internal erosion potential failure modes:

Table IV-4-3. Suggested Method for Evaluating Internal Erosion Risks

General Task	Approach and Guidance
Develop potential failure modes	Based on all available information, develop the site-specific internal erosion potential failure modes in accordance with Chapter I-3 on Potential Failure Mode Analysis. Fully describe each potential failure mode from initiation through embankment breach.
Develop event tree	Start with the generic event tree described above for each identified potential failure mode that is thought to contribute significantly to the risk, Adapt the tree where needed due to site-specific conditions of the embankment and its foundation.
Assemble background information	Collect, review and assemble available information related to geology, material properties, gradations, instrumentation data, design and construction details, construction photographs, and any other information that would help with estimating risk. The level of effort of this data collection, review and assembly varies depending on the agency and the level of the risk analysis. For many dams and levees, much of this information is readily available in previous dam/levee safety reports (e.g., Reclamation's Comprehensive Facility Reviews (CFRs/CR), FERC's Part 12 Inspection Reports, USACE's Periodic Inspection (PI) and Periodic Assessment (PA) reports, or other similar reports). For most Reclamation CR-level risk analyses, little effort is necessary beyond collecting and reviewing the available information. For higher-level Reclamation risk analyses, greater effort may be justified for additional researching construction photos and for preparing geologic drawings, additional instrumentation plots, etc., as needed. For USACE quantitative risk assessments (i.e., issue evaluation studies and dam safety modification studies), large format drawings are also typically prepared that characterize the subsurface conditions and synthesize all of the available information listed above (if not available).
Select reservoir load partitions	The number of reservoir load partitions will depend on a number of operational, physical, and performance related factors. Factors such as changes in embankment zoning, geology, or performance thresholds may provide justification for a greater number of reservoir load partitions to account for non-linearity in the loading.
Perform supporting evaluations	Perform necessary supporting analyses and estimates that are appropriate for the level of risk analysis. Typical analyses might include: filter compatibility evaluation, seepage analyses, uplift stability, critical gradient for particle transport and/or progression of a pipe, instrumentation trends analysis, etc. Review pertinent case histories.
Develop "more likely" and "less likely" factors for each event tree node	Develop a list of the adverse factors that make each event tree node more likely and the favorable factors that make each event tree node less likely based on the key evidence and degree of belief that the likelihood is low or high. Detailed tables at the end of this chapter provide a comprehensive list of more likely, neutral, and less likely

General Task	Approach and Guidance
	factors that can influence the likelihood of each event. ***Complete lists of more likely and less likely factors should be developed for each event of each potential failure mode on a site-specific basis and should not be limited to only those listed on the tables.***
Estimate the probability (and range of uncertainty) for each event	The approach for estimating the probability for each internal erosion event depends on the specific event, the level of risk analysis and agency best practice. For Reclamation CR-level risk analyses, probabilities are typically estimated by a small team that might use previous risk estimates as a starting point. They might elect to use the same risk estimates because nothing has changed, or they might consider new information and develop new estimates. For higher-level Reclamation risk analyses, risk estimates are developed in a team setting during a facilitated, approximately week-long meeting. For estimating the probability of initiation event, Reclamation's best practice is to start with the best estimate range based on historical base-rates for the potential failure mode category being considered (included later in this chapter), and increase or decrease that estimate range based on factors listed in the tables. Any variation from the recommended best estimate ranges should be supported by solid evidence and/or a strong case. USACE's best practice for estimating probabilities of each node of the event tree is to utilize the best available and multiple methods in support of the risk estimate, including analytical methods as described in this chapter, case histories, etc. Although multiple approaches are encouraged to obtain supporting data to build the case, all final probabilities are estimated using team elicitation procedures based upon the totality and strength of the evidence. For both agencies, documentation of the basis for the estimate is critical to support the case for the risk estimate. This is particularly true for the initiation estimate, which can vary significantly and can strongly influence the overall probability of failure estimate. Chapter I-6 provides guidance on subjective probability and elicitation.
Perform supplemental supporting evaluations	Perform supplemental evaluations that might help with estimating initiating probabilities, including but not limited to: • If the potential failure mode is driven by a flaw through the embankment core or the foundation, consider the guidance on flaws provided in this chapter. • If the potential failure mode is driven by backward erosion piping in fine-grained cohesionless soils and appropriate grain-size information is available, consider the guidance on critical gradient for particle transport and/or progression provided in this chapter. • If the potential failure mode is driven by uplift and blowout at the toe of the embankment, calculate the factor of safety against uplift and consider guidance provided in this chapter. • If the potential failure mode is driven by internal instability and grain-size information is available, evaluate the potential that the soils are internally unstable and the potential that suffusion could occur and possibly lead to stoping in adjacent soils.
Evaluate sensitivity	Perform sensitivity analyses, if necessary, if key pieces of information

General Task	Approach and Guidance
	(e.g., plasticity, gradation, erodibility, etc.) are unavailable, limited, or vary greatly. Use caution in trying to estimate parameters that require this information. One technique to consider when key data are unavailable is to base estimates on two different potential conditions (one being the most reasonably favorable and the other the most reasonably adverse) to provide an indication whether the results are sensitive to this information, and whether further investigations or analyses are justified.

Performance

The statistics of historical failures and incidents embankment dam incidents can provide some insight when estimating the likelihood of a flaw or initiation of internal erosion. However, such rates should be used with caution based on the general method in which they were developed. Whenever historical rates are provided, they may be more representative of the product of several probabilities on the internal erosion event tree. The rates must be carefully considered based on site-specific information. Seepage from the downstream slope (from visual observation, measurement, or other non-invasive methods), measured high pore pressures, settlement, deformation, and cracking are possible indicators of a flaw or defect in the embankment. Similarly, seepage at the downstream toe is a possible indicator of a flaw or defect in the foundation. The influence of observations on the probability of a flaw should take into account the mechanism causing the flaw, the available data, and the relative importance of the observations.

Historical Background

Based on the records of dam incidents and the dam register in ICOLD (1974, 1983, 1995), Foster et al. (1998, 2000) evaluated the statistics of failure of large dams constructed between 1800 and 1986, excluding dams constructed in Japan before 1930 and in China. The total number of failures is 136, of which the total number of failures while in operation is 124. Where the mode of failure is known, the total number of failures was 128, of which the total number of failures in operation is 117. The results are summarized in Table IV-4-A-1 in Appendix IV-4-A for internal erosion through the embankment, internal erosion through the foundation, and internal erosion of the embankment into the foundation. The largest number of failures occurred in the embankment, and nearly one-half of these were associated with conduits which penetrate the embankment or walls which support the embankment. For all internal erosion failure modes, approximately two-thirds of all failures and one-half of all accidents occur on first-filling or in the first 5 years of reservoir operation. Therefore, approximately one-half of all incidents have occurred after 5 years of reservoir operation. The historical frequencies of failures and accidents are summarized in Table IV-4-A-2 in Appendix IV-4-A, and the timing of the incidents is summarized in Table IV-4-A-3 in Appendix IV-4-A for internal erosion through the embankment and Table IV-4-A-4 in Appendix IV-4-A for internal erosion through the foundation. Foster et al. (1998, 2000) also found that nearly all internal erosion failures in the embankment occurred when the reservoir level was at or near (within one meter) the pool of record. Excluding conduits and spillways, 63 percent of the incidents are associated with cracking, and 37 percent of the incidents are associated with poorly compacted and high permeability zones (Foster et al. 1998, 2000). A further assessment of the case study information is summarized in Table IV-4-A-5 in Appendix IV-4-A for incidents of cracking and hydraulic fracturing in the embankment and Table IV-4-A-6 in Appendix IV-4-A for incidents of poorly compacted and high permeability zones.

Reservoir Loading Considerations

Historically, most internal erosion failures have occurred when the reservoir was within about 3 feet of the historical maximum level or greater (Fell et al. 2003). The annual likelihood of achieving this sort of level can be estimated using methods described in Chapter II-1 on Reservoir (and River Stage) Exceedance Probabilities. If seepage flows or boils, etc., emerge at lower reservoir elevations, these lower reservoir elevations should also be considered and included in the reservoir load ranges used in the event tree (i.e., typically more than one reservoir load branch).

For dams that have been nearly fully loaded, such as many of Reclamation's, one can separate potential failure modes under normal operating (static) conditions from hydrologic and seismic-related potential failure modes. For reservoirs that serve primarily as water storage, it is not unusual that they fill nearly every year, and in such cases a value of 1.0 for this event is frequently assigned. In cases where the reservoir does not typically fill, the likelihood of achieving a high reservoir can be input based on reservoir exceedance curves. When the next event (initiation) is based on an annualized evaluation of Reclamation's internal erosion incidents, care must be exercised when assigning values to this reservoir probability. Note: It is assumed that all of Reclamation's incidents occurred during high reservoir levels (which is not an unreasonable assumption).

For dams operated primarily for flood risk management, such as most USACE, or for dams that have significant flood storage and in any case have not been significantly loaded or are not significantly loaded very often the annual loading can vary significantly from year to year. Therefore, one can consider the full range of reservoir loading and not evaluate static loading separately. The "static" loading in this case can essentially included in the hydrologic loading evaluation. Cumulative plots of annual probability of failure, annualized incremental life loss, and annualized incremental economic consequences associated with "normal" operating ranges or floods of interest can be used to evaluate and portray risks for various levels of loading (e.g., for reservoir levels up to conservation pool) and help identify critical load ranges that may be contributing the most risk. If the static evaluation of the risk at a dam included the use of an annualized evaluation of internal erosion incidents, such as Reclamation's, care must be taken in the selection of the loading interval to start the evaluation of the flood loading to avoid double-counting of the risk. For flood loadings (which are considered to be hydrologic failure modes), an estimate of the likelihood of reaching the historical high and higher elevations must be determined from flood frequency analysis and possibly flood routings (see Chapter II-2 on Probabilistic Hydrologic Hazard Analysis).

For each potential failure mode, the risk team can establish load increments for evaluation. These can be used in developing a system response curve that relates the conditional probability of failure to the reservoir level, for the full range of loading. Non-linear portions of the loading or system response can unknowingly lead to results that are controlled by less well-defined portions of curves. Therefore, the reservoir levels must be carefully selected to define the shape of the system response curve, especially at elevations where significant changes in the probabilities may occur. In general, partitioning of reservoir levels should consider the following elevations:

- Elevation of the maximum annual pool
- Elevation where the probability of initiation of erosion becomes non-zero (e.g., bottom of a crack, elevation of rock defect, etc.)
- Geological features which occur above a particular level in the foundation (e.g., highly permeable gravel layer)
- Elevations where there is a documented change in performance (e.g., boils, high piezometric levels, etc.)
- Topographic features (e.g., major changes in foundation profile)
- Elevations corresponding to changes in design (e.g., top of filter, top of impervious core, or top of downstream berms)
- Elevation of pool of record. This is an important elevation because the embankment and its foundation have been tested up to this level.
- Uncontrolled spillway crest or key elevations associated with controlled spillway operations.
- Probable Maximum Flood (PMF) elevation
- Elevation of the embankment crest

The water levels do not have to be consistent between failure modes or with the stage-frequency curve as long as the full range of loading is covered. Typically, 3 or 4 water levels are selected, but the actual number should be adequate to define the shape of the system response curve. After estimating the risks associated with the full range of reservoir loading, a team can compare to those obtained from the annualized evaluation of Reclamation internal erosion incidents.

Initiation – Erosion Starts

"Initiation" is the first part of the conceptual model of an internal erosion failure mechanism. Arguably, this is the most difficult node to evaluate and estimate, and also the most important (i.e., tends to have the most potential impact on the estimated annual probability of failure). Therefore, in-depth discussion is warranted.

Garner and Fannin (2010) developed a Venn diagram, as shown in Figure IV-4-9, to illustrate that erosion initiates when an unfavorable coincidence of 1) material susceptibility; 2) stress conditions; and 3) hydraulic load occur. In their work, material susceptibility is related to the potential for soil to experience loss of a portion of its finer fraction, as a consequence primarily of grain size and also shape of the grain size distribution curve. For the purposes of this manual, another critical component of "material susceptibility" is the relative erosion resistance (plasticity) and dispersivity of a soil. The critical hydraulic load is related to the hydraulic energy required to invoke a mechanism of internal erosion, by means of seepage flow through the embankment. In other words, this factor relates to the seepage gradients and velocities present in the embankment or foundation and whether they are sufficient to induce particle movement. The critical stress condition is related to the inability to resist internal erosion due to the magnitude of effective stress, with recognition that stress varies spatially and/or temporally within the body of the dam or levee. The stress condition plays a role in internal instability, but can also be viewed to reflect the presence of "defects" in an embankment or foundation, whether due to cracking, hydraulic fracturing, arching, or similar phenomena. The central subset describes a zone within the embankment that is susceptible to all three factors. The combination of material susceptibility, hydraulic loading and critical stress gives rise to the release or detachment, and transport of soil grains.

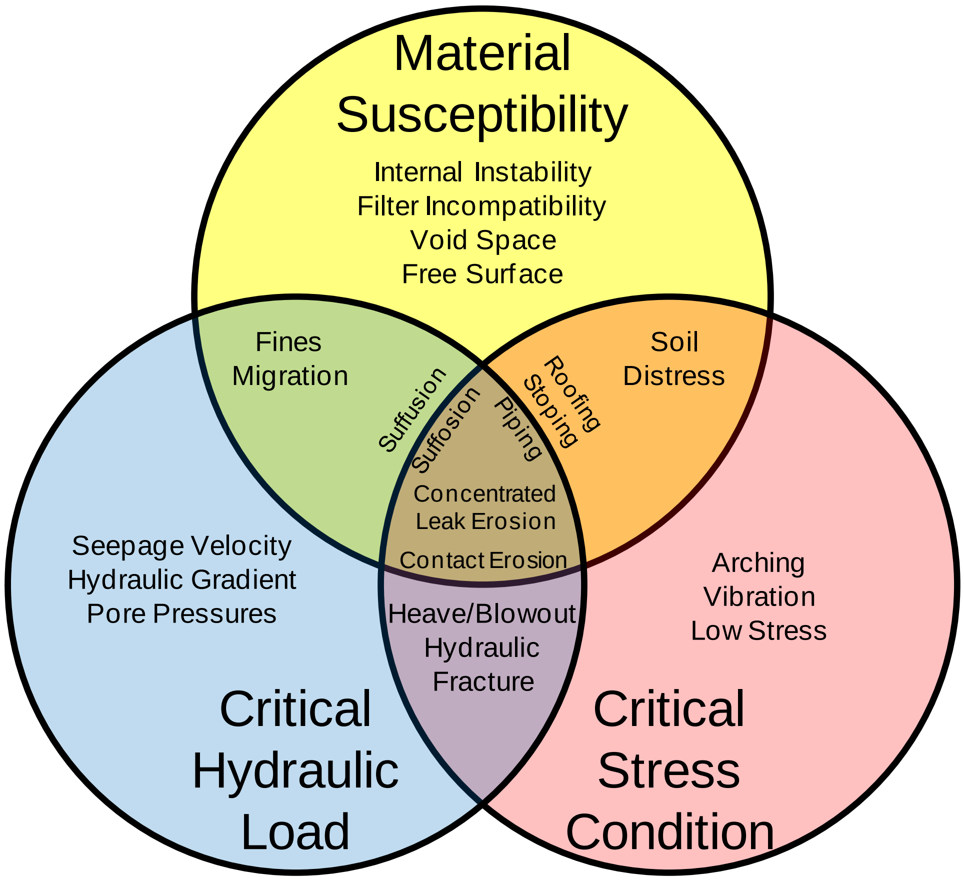

Figure IV-4-9. Factors Affecting the Initiation of Internal Erosion (adapted from Garner and Fannin 2010)

This concept that the initiation of internal erosion is in part dependent on material properties, hydraulic conditions, and in situ stress conditions provides a good starting point to discuss each of these factors in relation to “initiation.”

Effect of Material Properties on Initiation

Plasticity

Plasticity appears to be the most important factor affecting the potential for backward erosion piping to initiate. Backward erosion piping is simply far more likely to occur in cohesionless (or low plasticity) soils than in cohesive or plastic soils. However, plastic soils will also be less likely to experience other internal erosion mechanisms as well. This is apparent from case histories. The likely reason is that inter-particle bonding present in non-dispersive clayey soils provides additional resistance to seepage than in silts and coarse-grained soils. The effect of plasticity varies with water content, and this can be complex. Low plasticity soils can be brittle and can sustain a roof or crack.

Gradation and Particle-Size

Gradation and particle-size are also important. As particle size increases (as in coarser sands and gravels, cobbles, and boulders), it takes a higher seepage velocity (more energy) to move soil particles. However, the laboratory gradations may not be representative of soils with larger particle sizes or soils susceptible to segregation or washout. Another gradation factor is the potential for internal instability, which is key to the development of suffusion or suffosion. Internal instability of soils is a concern for broadly-graded soils (i.e., soils with wide range of particle sizes – cobbles and gravels with sands, clays, and silts) with a flat tail of fines, particularly if the soil is gap-graded (missing mid-sized particles). Examples of these types of soils are shown on Figure IV-4-10. Glacial soils can frequently fall into either of these categories.

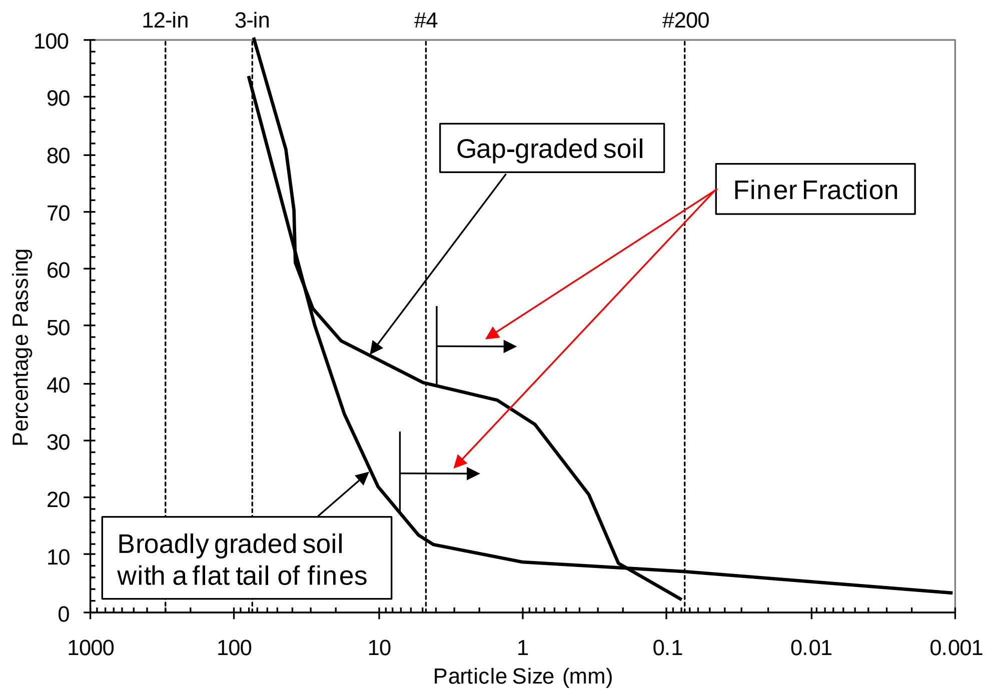

Figure IV-4-10. Potentially Internally Unstable Soils (adapted from Wan and Fell 2004)

According to Sherard (1979), soils are generally considered “internally unstable” if the coarser fraction of the material does not filter the finer fraction. He obtained data from a number of embankment dams, where sinkholes appeared on the crest and slopes of widely graded embankment embankments of glacial origin, and plotted a band around these gradations, as shown in Figure IV-4-11. The internally unstable soil gradations usually plotted as nearly straight lines or as curves with only slight curvature within the range shown. Reclamation’s filter design standard also considers the slope of the gradation curve. This slope is illustrated in Figure IV-4-11 and is noted as “4x.” The slope of this line is approximately equal to the boundary slopes of Sherard’s band. The location of the “4x” line on the plot is unimportant. Any portion of a gradation curve that has a flatter slope than this line indicates a potentially unstable soil, whereas portions of the gradation curve steeper than the line indicate a stable soil. This technique can also be used to evaluate gap-graded soils.

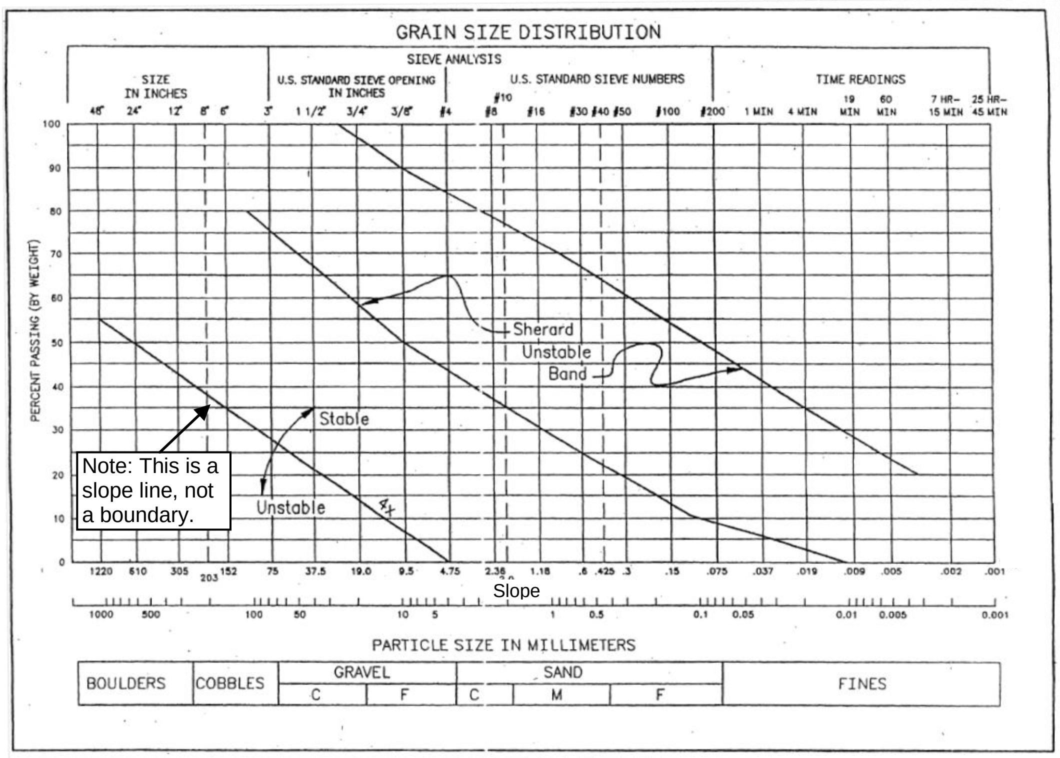

Figure IV-4-11. Potentially Internally Unstable Soils (Sherard 1979 and Reclamation 2011)

Density

Density plays an important role as well. The denser the soil, the harder it becomes to dislodge the soil particles and initiate erosion.

Erodibility

A key consideration in the likelihood of initiation of erosion in any case is the erodibility of the embankment core and/or foundation materials. The likelihood of erosion initiating is much higher in highly erodible soils. Sherard (1953) published an early erosion resistance classification which is still useful in evaluating the likelihood of erosion, shown in Table IV-4-4; the lower the number, the greater the erosion resistance. Note that plasticity and compaction moisture content plays a key role in erodibility. Table 8 of Sherard (1953) provides a more detailed examination of the soil characteristics from the case histories examined than Table IV-4-4for both backward erosion piping and cracking. Due to its size, that table is not reproduced in this document. ICOLD (2013 Draft) has prepared a similar classification for resistance to concentrated leak erosion based on Fell et al. (2008), as shown in Table IV-4-5. Chapter IV-1 describes the erodibility parameters including critical shear stress and erosion coefficient that are used in the prediction of erosion of soils subject to concentrated leak erosion.

Table IV-4-4. Piping Resistance of Soils (adapted from Sherard 1953)

Greatest Piping Resistance Category (1)	1. Plastic clay, PI > 15, well compacted.
	2. Plastic clay, PI > 15, poorly compacted.
Intermediate Piping Resistance Category (2)	3. Well-graded material with clay binder, 6 < PI < 15, well compacted.
	4. Well-graded material with clay binder, 6 < PI < 15, poorly compacted.
	5. Well-graded, cohesionless material, PI < 6, Well compacted.
Least Piping Resistance Category (3)	6. Well-graded, cohesionless material, PI < 6, poorly compacted.
	7. Very uniform, fine cohesionless sand, PI < 6, well compacted.
	8. Very uniform, fine, cohesionless sand, PI < 6, poorly compacted.

Table IV-4-5. Erosion Resistance of Soils from Concentrated Leaks (Scour) (adapted from ICOLD 2013 Draft)

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

Based on an examination of Reclamation internal erosion incidents (Engemoen 2011), it is estimated that 87 percent of cases of internal erosion at Reclamation embankments has been associated with soils of no to low plasticity, and only 13 percent associated with soils having a plasticity index greater than 6 or 7. Dispersive soils are not addressed in Table IV-4-4 but can be even more erodible. Dispersive soils are typically clays in which the clay particles can disperse or deflocculate (go into suspension) under still conditions, quite the opposite of most clays that require considerable seepage velocities to begin the erosion process. Dispersivity is related to clay mineralogy and particularly the electrochemical forces between soil particles as well as the pore water; soils having a high exchangeable sodium percentage are more susceptible. Dispersive clays are not limited to specific types, colors, geomorphology, or climatic conditions. Marine clays located in southern states where inland seas were present are often susceptible (e.g., Mississippi, Alabama, Arkansas, Louisiana, Texas, Oklahoma, New Mexico, and Arizona). It is difficult to tell whether a clay is dispersive without specific tests. However, it is common that some erosion features are observed in natural deposits of dispersive soils. Applicable laboratory tests that provide a measure of soil dispersivity include the (Emerson) Crumb test, the (SCS) double hydrometer test, the (Sherard) pinhole tests, and chemical tests that evaluate ESP (exchangeable sodium percentage) or SAR (sodium absorption ratio). It is frequently suggested that at least two different tests be run to check for dispersivity. Experience suggests initiation of backward erosion piping in dispersive clay has generally only occurred either on first reservoir filling or upon raising the reservoir to new levels for the first time (Sherard 1979).

Effect of Hydraulic Conditions on Initiation

Role of Concentrated Seepage

Embankments and foundations are not completely impervious, and thus, virtually all facilities have some degree of seepage. It is not necessarily the amount of seepage that leads to internal erosion incidents; rather it tends to be whether concentrated seepage is occurring in soils that are susceptible to erosion. In other words, the initiation of erosion typically requires a particular seepage pathway that allows a concentrated flow within a generally limited or localized area or feature within an embankment or its foundation (e.g., cracks, joints, etc.). General seepage models that portray seepage through porous media represented by large zones or layers feature an idealized situation that is generally unlikely to accurately portray the potential for internal erosion in most cases. Instead, it is the "weak link" or anomaly in an embankment or foundation where a concentrated flow is likely to occur and result in an incident. Such weak links or reasons for concentrated flows typically include the types of defects previously discussed, as well as naturally occurring pervious layers that are susceptible to erosion.

Gradients

It is important to recognize that there are two types of gradients associated with seepage through porous soils and internal erosion: vertical and horizontal gradients. Vertical (upward) gradients are considerations in the potential for heave, uplift or blowout, and sand boils and can lead to unfiltered exits or potential initiating conditions for an internal erosion mechanism. Horizontal (internal) gradients through an embankment or its foundation play a key role in the probability that internal erosion will initiate and progress.

Critical Gradient, Heave, Uplift, and Blowout: Traditional soil mechanics or seepage discussions on critical vertical exit gradients (e.g., by Terzaghi and Peck, and Cedergren) have typically only presented examples using sand foundations. The term "heave" was used to describe the condition when the saturated sand specimen, subjected to upward seepage flow in the laboratory, suddenly decreases in density and increases in permeability. This limit-state condition occurs when the seepage pressure on a plane in the specimen equals the weight of the specimen, and the effective pressure becomes zero. The traditional equation can be rearranged to solve for the upward hydraulic gradient (or critical vertical gradient) which is then further reduced to the more recognizable form in practice as the ratio of the buoyant unit weight of the soil (γ_b) to the unit weight of water (γ_w):

$$i_{cr} = \gamma_b / \gamma_w$$

This simplified relationship for the critical vertical exit gradient can also be expressed as the condition when the pore water pressure equals the submerged unit weight of the soil, and thus the effective stress is zero. At the critical vertical gradient in cohesionless foundations, a "quick" condition exists in the sand, and the foundation materials may "heave" or "boil" as shown in Figure IV-4-12. Sand boils are an indicator of locations where the critical vertical exit gradient is close to or may have been reached.

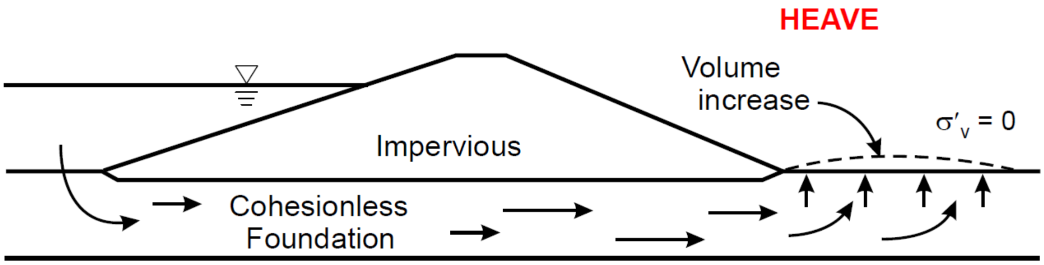

Figure IV-4-12. Heave at the Toe of an Embankment (Pabst et al. 2012)

In the case of cohesionless foundations with no confining layer and vertical (upward) flow at the toe, vertical exit gradients (i_e) can be estimated using seepage analyses or piezometric data and then compared to the critical vertical gradient. Estimated vertical exit gradients less than the critical vertical gradient provide an indication that they may not be sufficient to create heave/boiling conditions at a seepage exit. Depending on the state of knowledge about given site conditions, there can be significant uncertainty with the estimated values of gradients. It should be noted that Darcy's flow equation is only valid until the critical gradient is reached. At the critical gradient, the sudden rearrangement of particles results in a sudden increase in discharge at the same gradient indicating the flow is no longer proportional to the gradient and permeability is no longer a constant. It is possible that sand boils may form, but significant particle transport may not occur due to other conditions, such as inability to hold a roof, heterogeneity of actual soil deposits, or insufficient horizontal gradients over a long enough time to fully develop an internal erosion mechanism.

A "blanket-aquifer" foundation consists of a low permeability, confining layer (such as clay) overlying a pervious layer (such as sand). If the pervious layer is not cut off upstream, seepage pressures in the pervious layer at the base of the confining layer may exceed the overburden pressure of the confining layer (i.e., soil blanket) at the downstream toe of an embankment, and uplift (or "blowout") of the confining layer may occur as shown in Figure IV-4-13. This is a primary concern for levees, and the term "heave" has also been used to describe uplift/blowout of the soil blanket by USACE and others in the literature. When soil blankets are ruptured, sand from an underlying aquifer will often be forced up through the confining layer, producing sand boils. A quick/boiling condition that often forms in cohesionless material may not exist around the sand boil, but "spongy" ground conditions are often noted and can be seen and felt when walking on a ruptured or an uplifted soil blanket.

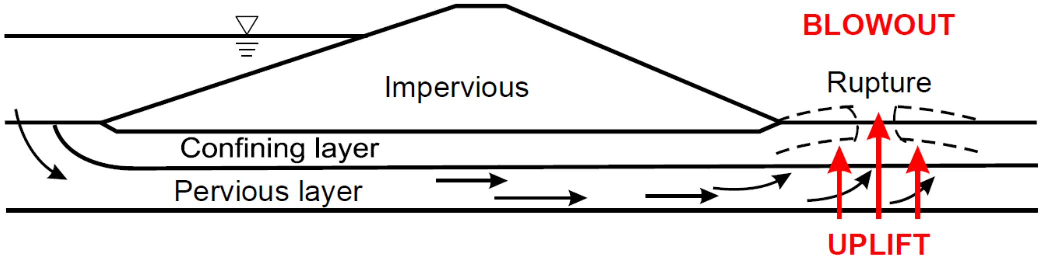

Figure IV-4-13. Uplift and/or Blowout at the Toe of an Embankment (Pabst et al. 2012)

The limit-state condition for uplift of the confining layer is reached when the uplift pressure at the base of the confining layer equals the weight of the confining layer (at the time the corresponding uplift pressure is applied). If the uplift pressure in the field exceeds the weight of the confining layer at any time, uplift is likely to initiate and result in significant changes in the seepage regime. If uplift or blowout does occur, a potential unfiltered exit location is provided where internal erosion may initiate. As uplift occurs, a new seepage exit can form beneath the confining layer where hidden deterioration can occur from concentrated seepage if the horizontal gradients are high enough. With blowout, the confining layer is ruptured proving an unfiltered exit to the ground surface. The specific location of a rupture may be the result of a defect and/or the location of the maximum uplift pressure. If hidden deterioration was occurring before the blanket ruptured, these locations may be coincident. As previously mentioned, significant particle transport may not occur due to other conditions related to material erodibility, roof formation, insufficient horizontal gradients to develop an internal erosion mechanism.

The assumptions for seepage conditions, tailwater conditions, degree of saturation, and density of the confining layer must be carefully considered in the evaluation of the limit state for uplift of the confining layer. Uplift can also initiate in partially saturated confining layers, especially for light weight soils (peat or OH soils) or cohesive soils in cases of drought.

For blanket-aquifer foundations, two methods have been used to evaluate uplift and/or blowout of the confining layer. One method involves simply comparing the uplift pressure acting at the base with the weight of the fine-grained soil blanket at the time the corresponding uplift pressure is applied. The critical gradient approach is commonly used for cohesionless foundations with no confining layer and vertical (upward) flow at the toe. Both approaches are applicable to some dams, for example dams which have blankets that vary significantly in key properties such as erosion resistance (ML versus CH) or blankets that are discontinuous due to an old ox bow. When in doubt, both can be used to inform the risk assessment team. The critical gradient method is used primarily for levees and involves comparing the actual gradient across the landside soil blanket (confining layer) to the maximum allowable gradient. An “underseepage factor of safety” is calculated assuming steady state-seepage conditions (USACE 2012), which reduces to the same form as the critical vertical gradient approach mentioned above. Text books and literature are not always clear in defining what approach is preferred, or even in distinguishing between the two approaches. USACE is currently working to resolve the differences in methods used between dams and levees. When conducting a risk assessment, the team should consider the most appropriate methodology for their site-specific conditions to help them better understand the potential failure mode and its likelihood to develop. The potential for heave, uplift, and/or blowout in the field can and has been greatly influenced by geologic details, the details of man-made features, climatic conditions, as well as biological and chemical processes such as excavation by rodents, plugging of seepage exits by bio-fouling, or mineral deposition. Risk assessors should be aware and consider these key factors and whether or not they are included in analyses.

Horizontal Gradients: Horizontal (or nearly so) gradients are internal gradients along a seepage path through an embankment and/or foundation. They affect the likelihood that internal erosion can occur by such means as concentrated leak erosion, backwards erosion piping, or suffusion/suffosion. There is a fundamental difference between upward gradients and horizontal gradients. Upward gradients are resisted by gravity and relate to the potential for heave or uplift and the possible initiation of internal erosion. However, gravity is not a resisting force for a horizontal seepage exit such as in a ditch at the toe of the embankment, and little to no horizontal gradient is required for initiation of internal erosion.

A typical “critical” vertical (upward) exit gradient in cohesionless soils is often thought to be around 1.0 for a specific gravity of 2.7 (where heave is concerned) and higher for cohesive soils not subject to uplift. However, the magnitude of horizontal gradient that has led to internal erosion is much lower. For example, the horizontal gradient at Reclamation’s A.V. Watkins Dam incident was calculated to be 0.08, and the horizontal gradient at USACE’s Wister Dam, which suffered concentrated leak erosion, was reported to be 0.02 (but contained some dispersive clays). Horizontal gradients as low as 0.02 were estimated for levees along the Mississippi River in 1937, 1947, and 1950, as shown in Figure IV-4-14. Evaluation methods for horizontal gradients are discussed in Appendix IV-4-C.

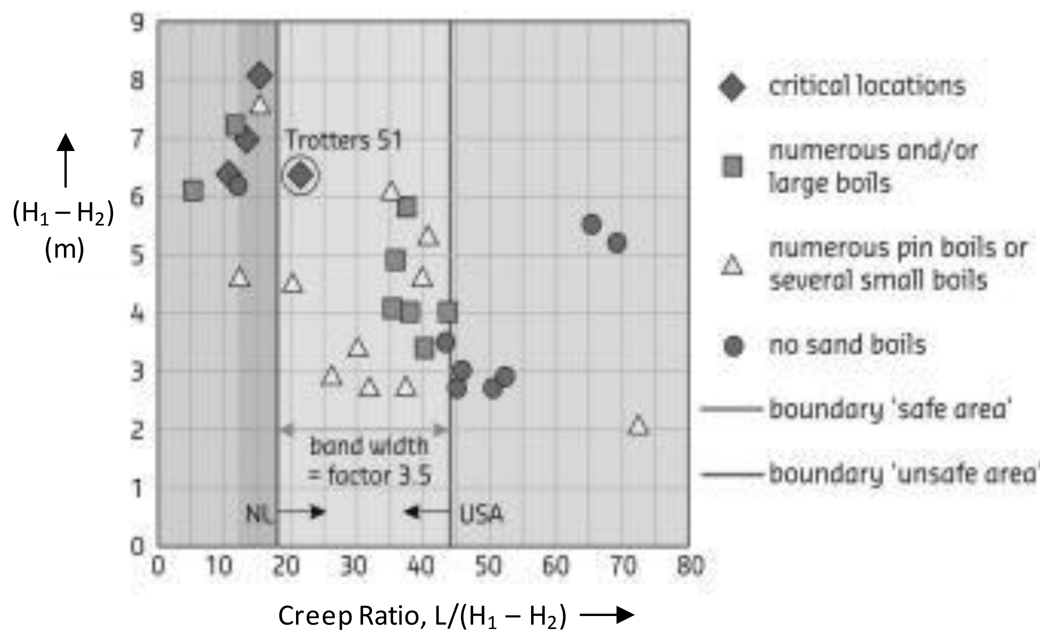

Figure IV-4-14. Critical San Boil Locations along Mississippi River Levees (adapted from Ammerlaan 2007)

Soil plasticity and moisture content is of extreme importance:

- Fell et al. (2008) indicate that the likelihood of backward erosion piping and suffusion in cohesive soils (PI > 7) is essentially zero under the seepage gradients which typically occur in embankments and their foundations.
- As mentioned previously, Reclamation cataloguing of internal erosion incidents shows that only 13 percent have involved cohesive soils (PI > 7).
- As demonstrated by this experience, as well as laboratory tests such as the Hole Erosion Test and Jet Erosion Test, cohesive soils are able to withstand much higher gradients than cohesionless soils before erosion initiates.

- However, this is not the case if the cohesive soils are dispersive. Case histories demonstrate embankments comprised of dispersive soils can erode quickly by internal erosion.

Unlike vertical exit gradients in sands, there is no widely accepted formula with which to evaluate when horizontal gradients might lead to internal erosion. In large part, this is due to no simple test to capture the physics that control whether or not it will occur. Unlike critical gradient piping by heave in cohesionless soils, no single laboratory test can be performed to determine the "gradient" at which erosion initiates and/or progresses. There is a great deal of uncertainty and variability inherent in lengthy seepage flow paths through embankment or foundation soils. These uncertainties include:

- Internal gradients are likely quite different at various places along the seepage pathway since natural, or even engineered, soils can be highly variable. The seepage path is undoubtedly not a straight line and likely meanders considerably, with seepage flows experiencing different amounts of head loss along the way.

- It is extremely unlikely that sufficient piezometers would be located in a number of critical locations along a seepage pathway in or beneath an embankment to accurately measure the piezometric pressures at key points in the critical (weak link) flow path.

- Furthermore, it is exceedingly difficult to accurately assess how the soils along an entire seepage pathway will respond to seepage gradients. Laboratory tests can provide insights into how a relatively small segment of representative soil will behave under various hydraulic gradients, and these studies suggest that key factors like soil plasticity and grain-size are important parameters in determining the potential for internal erosion. In actual field conditions, both soils and gradients are expected to vary in most instances.

These complex variables, as well as many other physical or chemical factors which play a role in an internal erosion process, help explain why there is no widely accepted means to determine the factor of safety against internal erosion or backward erosion piping. Rather than using deterministic safety factors, Reclamation and USACE practitioners typically use available laboratory testing, research, and empirical evidence to probabilistically estimate internal erosion potential in risk analyses. References to consider in aiding these determinations include research from the University of New South Wales, the work by Schmertmann and Townsend, papers by researchers from Delft/Deltares in the Netherlands, and Kovács. Some of these studies indicate seepage forces on sand grains due to upward flow into the head of a developing pipe is a key reason that the critical horizontal gradient can be significantly lower than the critical vertical gradient.

Effect of Stress Conditions and Presence of "Flaws or Defects" on Initiation

Influence of Stress Conditions on Internal Instability

As described earlier, suffusion and suffosion deal with finer particles being washed out by seepage flows through a broadly graded or gap-graded, internally unstable soil. Stress conditions play a role in this process, specifically relating to whether the space between coarse particles in the soil is “over-filled” or “under-filled.” When the coarser-grained portion of an internally unstable soil are essentially in point-to-point contact, the space between coarse particles can be thought of being under-filled, and the stresses are being carried by those point-to-point contacts of the coarse particles. Thus, the finer matrix material feels little to no stress, and can consequently be washed out by seepage flows. However, when the larger particle sizes are essentially floating in the finer matrix material, there is no load carried by a coarser skeleton and thus all particles generally experience the same stress. In this scenario, the stress conditions upon all soil grains would require a much higher seepage velocity to move the materials. This is why suffusion (or the erosion of finer soils within an internally unstable soil with point-to-point contacts of the coarser grains) is much more likely to occur than suffosion (which features the coarser grains floating in finer matrix material) under the gradients typically present in embankment dams and foundations.

Low Stress Zones and “Arching”

The formation of low stress zones, or even tension zones, in an embankment is known to have led to many failures and incidents involving internal erosion. The zones can occur in areas of severe differential settlement. Foundation anomalies and conduits in narrow trenches have led to numerous instances of cracking and potentially hydraulic fracturing. In many cases, these low stress zones essentially lead to flaws or defects that are described below.

Flaws in the Embankment and Foundation

Fell et al (2008) suggested that a primary mechanism for internal erosion initiation is through flaws in the embankment core or foundation, which frequently result from unfavorable stress conditions. Based on that document, the following conditions may lead to an increased likelihood of a flaw existing <u>through the embankment</u> (including considerations for <u>conduits</u> through the embankment):

- Wide benches or “stair steps” in the upper to middle portion of the abutment profile can lead to transverse cracking from differential settlement.
- Steep abutments near the top of the embankment, can also lead to transverse cracking from differential settlement.
- Very steep abutments and a narrow valley can lead to “arching” of the soil across the valley leading to a reduction in vertical confining stress within the embankment and increased potential for cracking due to hydraulic fracturing (i.e., pore pressures exceed confining stress).
- Fell et al. (2008) suggest that differential settlement between the shell and the core (if deformability of the materials differ) can lead to “dragging and transverse shearing” of the core. However, more typically, this type of differential settlement leads to longitudinal cracks at the interface between the two materials.
- Different foundation conditions (deformability) across the profile can lead to differential settlement and cracking of the embankment core.
- Low-density fine-grained loess soils or weakly cemented “desert” soils present within the foundation may collapse upon wetting, leading to differential settlement or hydraulic fracturing through the low density material and transverse cracking through the embankment.
- Different deformability conditions between fill and foundation soils (such as at diversion channels) may lead to transverse cracking through the embankment.

- Desiccation of the embankment material can lead to transverse cracking through the upper part of the core.
- Excessive settlements as a percentage of the embankment height (i.e., more than about 3 to 5 percent during construction or about 1 percent at 10 years post-construction) increases the chances of transverse cracking – even lesser settlements may lead to cracking in particularly brittle soils. Note that cracking is often masked; case histories suggest that such cracking can go unnoticed for years and even decades.
- An irregular foundation contact surface, possibly with overhanging rock features, or sloppy or loose foundation soil conditions upon embankment placement can lead to inadequate compaction and a pervious channel along the embankment-foundation contact.
- Irregular rock surfaces and overhangs beneath foundation soils that are no cutoff can cause differential settlement and/or defects beneath overhangs.
- Poor core density due to lack of formal compaction, lack of compaction control, or excessively thick compacted layers can result in pervious layers through the core.
- Seasonal shut-downs or placement in freezing weather can lead to a pervious layer through the core if not properly treated (i.e., frozen material and desiccation cracking was not removed and the surface thoroughly scarified with good moisture control upon re-compaction). In the unlikely event that post-shutdown construction results in lower modulus material in comparison to the underlying embankment, differential settlement of the overlying embankment can lead to transverse cracking in that portion.
- The presence of a conduit through the embankment core creates a potential high permeability pathway due to the potential for inadequate density or compaction, especially if one or more of the following conditions are also present:
    - A round conduit with no concrete encasement where it is difficult to get good compaction on the under-side.
    - The presence of seepage cutoff collars which are difficult to get good compaction around and against.
    - Cracks or open joints in the conduit, or corrugated metal pipe which is subject to corrosion deterioration and through-going holes, into which embankment core material can be washed.
    - Steep and narrow trench into which the conduit was placed, which makes compaction difficult and creates the potential for arching of soil across the trench, leaving a low density zone susceptible to hydraulic fracturing.
- A stiff conduit projecting up into a brittle embankment also creates the potential for differential settlement above and adjacent to the conduit and the potential for cracking.
- Presence of frost-susceptible soils in which ice lenses can form, particularly when these materials are adjacent to conduits or other structures that could increase the possibility of freezing conditions.
- If a spillway passes through the embankment such that the core is compacted against the spillway wall, difficulties in compacting against the wall (especially if vertical or counterforted), and settlement away from the wall parallel to the abutment, can potentially lead to a high permeability zone or small gap adjacent to the wall.
- For composite concrete/embankment dams, vertical faces, overhangs, and changes in slopes of the concrete section (against which the embankment core is compacted) can lead to higher permeability seepage paths, especially if post-construction embankment settlements are large.

- Direct observations such as observed transverse cracks in the crest of the embankment, or concentrated seepage or wet areas on the downstream face of the embankment, adjacent to an outlet works conduit, or adjacent to a spillway wall could be indications that flaw may extend through the embankment.
- Evidence of sinkholes or depressions (especially along the alignment of a penetrating outlet works conduit), could be indications that material has moved by means of seepage flows.
- Rodent holes and root balls, if not properly treated, can be locations for piping to initiate. Rodents may burrow into dry areas of an embankment when the reservoir is low, but these areas may be exposed to the reservoir as it rises. Similarly, decaying root systems can form pathways for piping initiation.
- Combinations of some of these conditions are common in case histories of incidents involving internal erosion.

The following conditions may indicate an increased likelihood of internal erosion <u>through the foundation</u> or <u>from the embankment into the foundation</u>:

- A low permeability confining layer at the toe of the embankment beneath which high artesian pressures exist, which increases the chance of blowout.
- Sand boils observed in the channel downstream of the embankment which could be indications of material movement associated with a foundation seepage path, especially if material is moving out away from the boils.
- Open joints, seams, faults, shears, bedding planes, solution features, or other discontinuities in the rock foundation at the contact with the embankment core into which core material can erode, especially if the following also apply:
    - The discontinuities trend upstream to downstream across the foundation, providing a pathway for reservoir seepage.
    - There was no or questionable foundation surface treatment performed during construction in the way of dental concrete or slush grout, especially if the treatment area was narrow with respect to the height of the embankment.
    - The effectiveness of foundation grouting is questionable due to grout holes being parallel to open discontinuities, poor grout mixes, widely-spaced holes with uncertain closure, uncaulked surface leaks during grouting, and/or little pore-pressure drop across the grout curtain as measured by piezometers.
    - The discontinuities are open, or perhaps filled with erodible silty or sandy material. Wider discontinuities are more problematic than narrow ones.
- Poor clean-up at the core-foundation rock surface can lead to a low density or erodible pathway at the contact.
- Ridges and valleys formed by excavation along geologic features (e.g., tilted bedding planes forming an irregular surface) that trend upstream to downstream, into which compaction is difficult, can lead to low density pathways near the embankment-rock contact and hydraulic fracture (Quail Creek Dike).
- Embankment core material placed against the downstream slope of a cutoff trench cut into pervious gravels with no intervening filter leaves an interface through which core material can be eroded.
- A narrow steep-walled cutoff or outlet conduit trench forms a location where arching of core material placed into the trench can lead to a low density zone in the core susceptible to transverse hydraulic fracturing. This can be problematic if there is a pathway downstream through which the core material can erode.

- Highly permeable foundation materials which can transmit significant flow capable of eroding material at the base of the embankment and carrying it downstream.
- Combinations of some of these conditions (such as a narrow trench and poor treatment with open untreated joints) are common in case histories.

Reclamation and USACE Research

USACE is currently researching internal erosion incidents for their portfolio of over 600 embankment dams and over 2,500 registered levee systems. Reclamation, which has a similar inventory of over 200 embankment dams, has conducted a similar effort and developed a list of internal erosion incidents at their projects. A summary of Reclamation's research and experience is discussed in the following section on initiation.

Approach to Estimating the Probability that Internal Erosion will Initiate

Given an open or unfiltered exit exists, in most cases the key event in the event tree generally used to estimate the probability of an internal erosion failure is the probability that internal erosion will initiate. Initiation is typically judged to have a relatively low probability of occurring; is based on a number of variables including presence of a concentrated seepage path (flaw), length of the seepage path, hydraulic "gradients," and soil erodibility; and is thus difficult to estimate. Both Reclamation and USACE use a process of evaluating all information and existing conditions at a site to flesh out potential failure modes and utilize expert elicitation to assess the likelihood that the internal erosion process will initiate. However, the two agencies follow a somewhat different philosophy in how to develop the probabilities of initiation. USACE utilizes a broad amount of information to support probability estimates made using team elicitation, including analytical methods and application of researchers' findings on the potential for various soils to erode under various conditions. Reclamation tends toward an approach based on the empirical observations (or base rate frequencies) gained from a study of internal erosion incidents within their inventory. The fundamental reason for Reclamation's simpler approach is the difficulties and numerous uncertainties associated with applying laboratory findings and gradient assumptions to the spatially vast and variable embankment/foundation system typically being evaluated. Neither approach should be considered more "right" as both approaches have merits. Furthermore, there may be situations when either (or both) approach might be more appropriate, and risk teams should not feel constrained to a particular approach. However, given the fundamental differences in these two approaches, each agency approach is separately described below.

Probability of Initiation of Internal Erosion: Reclamation Approach

Use of Historical Frequencies

Estimating the probability of an internal erosion failure is very difficult and lacks deterministic approaches. Thus, thus the use of similar case histories provides some degree of "ground truth" or empiricism/precedence to the evaluation. Laboratory testing of small specimens to develop erosion properties and similar data may not be representative of the weak link or true condition in the spatially large embankment-foundation system. Similarly, seepage models may not be representative of the key

Probability of Initiation of Internal Erosion: Reclamation Approach

hydraulic conditions that would drive the development of an internal erosion PFM along a long seepage path through variable materials. It may not be a wise use of limited funds to spend significant monies in an effort to estimate a probability that is arguably no more than an index value.

Estimated historic rates of internal erosion initiation can provide risk teams with a relative range or average value for various types of internal erosion (i.e., a "starting" or "anchoring" point). Reclamation in the past has typically based the likelihood of this node on the documented historical rate of internal erosion failures and incidents (specifically work by the UNSW), and adjusted upward or downward based on site specific factors. Most recently, reviews of Reclamation internal erosion incidents have been made (Engemoen and Redlinger 2009; Engemoen 2011). The following is a discussion from those studies:

- Reviews of Reclamation internal erosion incidents indicate there have been a total of 98 known incidents and one failure. Internal erosion incidents have occurred throughout the history of Reclamation embankments, and sometimes multiple instances at the same dam. The total number of dams that have experienced incidents is 54, or about 1 in every 4 Reclamation embankments.

- ***These incidents are not limited to first filling but can occur at any time in a dam's life.*** About 36 percent of Reclamation incidents have occurred during the first five years of reservoir operation, and 64 percent of all incidents have occurred after more than five years of successful operation.

- The incidents have also not been limited to older or deteriorated dams; newer dams have also had incidents. Approximately half of the incidents occurred in dams that were more than 47 years old, and the other half in dams that were less than 47 years old.

- Each incident was classified into one of five categories: 1) internal erosion through the embankment; 2) internal erosion through the foundation; 3) internal erosion of embankment into foundation; 4) internal erosion into or along a conduit; and 5) internal erosion into a drain.

- In addition, each incident was also classified into one of four internal erosion mechanisms: 1) backward erosion piping; 2) internal migration (formerly called progressive erosion); 3) scour; and 4) suffusion/suffosion (related to internal instability). Admittedly, the assignment of an internal erosion mechanism to a past incident requires a lot of judgment – in many cases a definitive understanding of just what type of process or mechanism is not clear. Furthermore, some incidents may well involve a combination of mechanisms.

- For both classification exercises, the evidence for developing internal erosion is also shown, as either "excessive seepage" or "particle transport." The use of particle transport was limited to those cases where clear evidence of internal erosion was noted, such as the presence of sinkholes, voids, sand boils that were moving soils, or turbid seepage water. Of the total 99 incidents/failures at Reclamation embankments, there have been a total of 53 cases where particle transport was observed.

- The following two tables portray the incidents in two different ways; first by category (location), and secondly by type of mechanism.

Table IV-4-6. Category of Internal Erosion Incidents at Reclamation Embankments

Category of Internal Erosion	Incidents/Failures with Definitive Particle Transport	Incidents with Excessive Seepage and Perhaps Sand Boils	All Incidents and Failures
Embankment only	3	4	7
Foundation only	31	39	70
Embankment into foundation	3	3	6
Into/along conduit	5	0	5
Into drain	11	0	11
Total	53	46	99

- The following observations from Table IV-4-6 are of note.
  - Approximately two-thirds of the tabulated internal erosion incidents have involved internal erosion through the foundation, perhaps due to the significant number of Reclamation dams without a fully penetrating cutoff over their entire length, the pervious nature of the foundation materials leading to significant seepage, the original deposition being the same general direction as the foundation seepage, and the presence of erodible soils in the foundation.
  - Of the 70 foundation incidents, 24 involved glacial soils, and 24 were attributed to bedrock seepage.
  - The relatively low rate of initiation of internal erosion through the embankment might be explained by Reclamation's use of wide cores (long seepage path) often flanked by shells of sands/gravels/cobbles (providing some filtering capability) and good compaction.
  - The relatively high rate of initiation of internal erosion into drains (includes through the foundation) may be due to decades of relatively poor design details for drains (open jointed pipe, brittle pipe materials, coarse gravel envelopes, and thin filters).

Table IV-4-7. Postulated Internal Erosion Failure Mechanisms Involved in Incident

Category of Internal Erosion	Incidents/Failures with Definitive Particle Transport	Incidents with Excessive Seepage and Perhaps Sand Boils	All Incidents and Failures
Backward Erosion Piping	7	9	16

Internal Migration	20	13	33
Scour	18	15	33
Suffusion/Suffosion	8	9	17
Total	53	46	99

- The following observations from Table IV-4-7 are of note.
  - Two-thirds of all incidents are suspected to have involved internal migration or scour, with each mechanism accounting for a third of the total.
  - Piping and suffusion/suffosion are believed to have each accounted for about 1/6 of the total incidents.
    16 of the 17 suspected suffusion/suffosion incidents involved glacial soils.
  - 21 of the 33 suspected scour incidents involved bedrock seepage.
  - ***The vast majority (87%) of incidents involved cohesionless or low plasticity soils (PI < 7).***

- The following table portrays the age of the dam (or modifications to a dam) at the time of each incidents.

Table IV-4-8. Age of Dam at Incident and Mechanism Type

Dam Age at Incident	No. of Piping Incidents	No. of Internal Migration Incidents	No. of Scour Incidents	No. of Suffusion-Suffosion Incidents	Total No. of Incidents
≤ 5 years	3	12	13	8	36
6-15 years	1	7	8	1	17
16–25 years	2	3	4	1	10
26-35 years	1	5	1	0	7
36-45 years	4	1	0	3	8
46-55 years	2	0	2	1	5
56-65 years	1	1	2	0	4
66-75 years	0	1	2	1	4
76-85 years	0	2	1	2	5
> 85 years	2	0	0	0	2
Totals	16	32	33	17	98

- The following observations from Table IV-4-8 are of note.
  - After about 20 years of reservoir operation, incidents become less common.
  - However, incidents continue to occur beyond 20 years, with no dramatic decline in rate of incidents after 20 years.
  - Most (60 to 75%) of the incidents involving internal migration, scour, and suffusion/suffosion occur in the first 25 years of operational history.
  - However, piping incidents tend to occur throughout the operational history; i.e., they are as likely to occur late as early.

An estimate of the historical rate of initiation of internal erosion in a Reclamation embankment dam can be obtained by dividing the number of incidents and failures by the number of dam-years of operation. The total number of dam-years was obtained by

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identifying each of Reclamation's approximately 220 major embankments considered in this study, determining their present age (to year 2010), and summing all ages. In this manner, the number of dam-years of operation at Reclamation facilities is estimated at approximately 13,000. Using this value, the following tables present the estimated historical rate at which erosion has initiated (and continued to progress in most cases to at least some degree).

Table IV-4-9. Historical Rate of Initiation* of Internal Erosion at Reclamation Embankments Based on Category

Type of Internal Erosion	Estimated Historical Rate of Erosion Initiation	
	Incidents/Failures with Definitive Particle Transport	All Incidents/Failures
Embankment only	$2x10^{-4}$	$5x10^{-4}$
Foundation only	$2x10^{-3}$	$5x10^{-3}$
Embankment into foundation	$2x10^{-4}$	$5x10^{-4}$
Into/along conduit	$4x10^{-4}$	$4x10^{-4}$
Into drain	$8x10^{-4}$	$8x10^{-4}$
Total	$4x10^{-3}$	$8x10^{-3}$

*Note: See later discussion of whether these data include more than just "initiation"

Table IV-4-10. Historical Rate of Initiation* of Internal Erosion at Reclamation Embankments Based on Mechanism

Type of internal erosion	Estimated Historical Rate of Erosion Initiation	
	Incidents/Failures with Definitive Particle Transport	All Incidents/Failures
Backward Erosion Piping	$5x10^{-4}$	$1.2x10^{-3}$
Internal Migration	$1.5x10^{-3}$	$2.5x10^{-3}$
Scour	$1.3x10^{-3}$	$2.5x10^{-3}$
Suffusion/Suffosion	$6x10^{-4}$	$1.3x10^{-3}$
Total	$4x10^{-3}$	$8x10^{-3}$

*Note: See later discussion of whether these data include more than just "initiation"

It is easy to question whether this review of past internal erosion incidents, admittedly not an in-depth research effort, is a reasonable portrayal of performance at Reclamation embankments. Another way to consider this frequency question might be to note that it is not unusual, on average, to see 1 or maybe even 2 new "incidents" of unusual seepage or piezometric behavior, new sand boils, or new sinkholes each year within the Reclamation inventory of embankment dams. Assuming 2 incidents per year with 250 embankments equates to an annual frequency of $8x10^{-3}$. This alternate approach to estimating a base rate frequency of the initiation of internal erosion happens to match the values obtained from the incident study – although far from definitive, this does support a measure of confidence in the reasonableness of these frequency data.

Rather than directly use the values reflected in these tables, it is recognized that additional adjustments may better reflect the ranges of potential "best estimate" values given the potential variables and uncertainties involved with categorizing internal erosion events. One key uncertainty deals with whether the historical base rate of incidents portrayed above reflects more than just the "initiation" phase of the internal erosion process. In other words, initiation may have occurred in more Reclamation embankments than these catalogued because the process never "progressed" far enough to manifest symptoms like detected/observed in the 99 cases.

It is difficult to estimate an additional number of dams where internal erosion may have initiated but did not continue or progress, and thus remained undetected. The original UNSW study of world-wide dams assumed the number of "unreported" incidents of initiation was probably in the range of 2 to 10 times the number of reported incidents. Given Reclamation's reporting and documentation capabilities, it would seem more likely that the lower portion of this range would be more applicable. Thus, if we were to apply a factor of 4 to the number of incidents involving definitive particle transport, the total base frequency rate could be considered to be $1.6x10^{-2}$.

Put another way, this is effectively a doubling of the historical rate gained by looking at all Reclamation incidents (including those with no observed particle transport). This assumption that there are 2 to 4 times as many cases of initiation that are not detected as there are observed incidents seems like a reasonable assumption.

Furthermore, rather than specifying a single value, it appears to make more sense to suggest a range of best estimate values. The term "best estimate" is used, as the true range of initiation of internal erosion probably spans several orders of magnitude. The lower end of the best estimate range is based on a doubling of the observed 53 cases of definite particle transport. The upper end of the best estimate range is based on a doubling of all 99 reported incidents, including the 46 that did not manifest any particle transport. Thus, the upper range values assume that only about one quarter of all cases of definitive initiation of internal erosion have actually been documented within Reclamation.

Recommended Tables for Use in Estimating Probability of Initiation

These adjusted values shown in the following tables are proposed as "starting points" or an empirical reference point for considering the probability of the initiation of internal erosion for Reclamation dams (or in an inventory of dams similar to Reclamation's). It should be noted that this inventory includes a large number of dams constructed prior to the failure of Teton Dam without well designed filters.

Table IV-4-11. Proposed Best Estimate Values of Annual Probabilities of Initiation of Internal Erosion by Category

Type of Internal Erosion	Range of Initiation Probability
Embankment only	$3x10^{-4}$ to $1x10^{-3}$
Foundation only	$2x10^{-3}$ to $1x10^{-2}$
Embankment into foundation	$2x10^{-4}$ to $1x10^{-3}$
Into/along conduit	$4x10^{-4}$ to $1x10^{-3}$

Into drain*	5×10^{-4} to 2×10^{-3}

*Note: Into drain values were adjusted downward given the limited number of instances where vulnerable drains are part of the inventory

Table IV-4-12. Proposed Best Estimate Values of Annual Probabilities of Initiation of Internal Erosion by Mechanism

Type of Internal Erosion	Range of Initiation Probability
Backward Erosion Piping	5×10^{-4} to 2.5×10^{-3}
Internal Migration	1×10^{-3} to 5×10^{-3}
Scour	1×10^{-3} to 5×10^{-3}
Suffusion/Suffosion	6×10^{-4} to 2.5×10^{-3}

These two tables can in effect be combined to show the estimated initiation probabilities for a given mechanism within a given category (location). This was done by looking at the 99 incidents in detail and cataloguing each incident in terms of: 1) whether the incident featured definitive evidence of particle transport; 2) the category (location) of the incident; and 3) the mechanism believed to be involved in the incident. In this manner, the Table IV-4-13 was developed. If a risk team has a reasonable model of a specific failure mode being evaluated, this table can be used as an initial guideline from which to anchor a “best estimate” of the probability of initiation of internal erosion.

Table IV-4-13. Best Estimate Values for Initiation of Internal Erosion, based on Historical Incidents at Reclamation Embankment Dams

	BEP	Internal Migration	Scour	Suffusion/ Suffosion	Total
Embankment	3*	1	2	1	3×10^{-4} to 1×10^{-3}
Foundation	8	21	25	16	2×10^{-3} to 1×10^{-2}
Embankment into Foundation	0	0	6	0	2×10^{-4} to 1×10^{-3}
Into/along conduit	1	4	0	0	4×10^{-4} to 1.5×10^{-3}
Into drain	4	7	0	0	5×10^{-4} to 2×10^{-3}
Total	5×10^{-4} to 2.5×10^{-3}	1×10^{-3} to 5×10^{-3}	1×10^{-3} to 5×10^{-3}	6×10^{-4} to 2.5×10^{-3}	

*The number of total incidents are shown in the boxes

Considerations for Usage of Table IV-4-13

1. These ranges are considered to be “**best estimates**” – <u>not the reasonable low and reasonable high</u>. Higher or lower estimates of initiation probability may be appropriate if conditions at the dam being evaluated are better or worse than the “average” condition at a Reclamation dam. For example, dams with very low hydraulic gradients and minimal seepage may lead to lower estimates of initiation, while dams with appreciable seepage or a history of concerns may warrant higher

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estimates.

2. The incidents used to develop these values were limited to only Reclamation embankment dams, so use on embankments designed/constructed by others should consider how well those dams compare to Reclamation practices.
3. A total of 87 percent of the incidents featured soils with no or low plasticity (i.e., PI < 7). If postulated failure modes involve low plasticity soils, there is no need to consider higher estimates. However, lower initiation rates may be considered with more plastic soils.
4. Approximately 1/3 of the incidents considered occurred in the first 5 years of operation, while 2/3 occurred in dams with more than 5 years of operational history. Thus, when evaluating new dams, consideration should be given to using somewhat higher values of initiation probability. Conversely, for dams with a long operational history, somewhat lower values could be considered.
5. ***Simply referring to the tabulated best estimates from Reclamation's history of incidents is not sufficient in evaluating the probability for erosion to initiate.*** Instead, site conditions must be considered in order to determine whether there are features, conditions, or behaviors present at a given site that will influence the potential for erosion to initiate. Comprehensive tables have been developed that offer a number of considerations that would affect the likelihood of initiation at a given site. There are separate tables for each category of internal erosion. Any estimate of initiation should consider the factors in the 11x17 tables at the end of this chapter.

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Analytical Approaches

A number of methods, tests, and tools are available to assist in evaluating the probability of initiation of internal erosion. These in combination with observations and the experience of the risk team provide the evidence against which the probability estimates are made. The risk team discusses these and other factors that were identified and decides which should receive the most weight. This section discusses some of the analytical methods and tests considered by USACE which the team would consider as a "more likely" or "less likely" factor during an elicitation for the probability of a flaw existing or probability of initiation. A range of reasonable estimates would then be made, and the "case" or evidence for why the estimates make sense would be described. Although methodology has been developed to aid in making reasonable probability estimates, it is the learning that occurs during the risk assessment process (which must be documented) that is key to making appropriate "risk-informed" decisions. ***USACE's best practice for estimating the probability of initiation of internal erosion is to utilize the best available and multiple methods, but all final probabilities are estimated using elicitation procedures based upon the totality and strength of the evidence.*** The team should compare final estimates with the historical rates presented earlier in this section.

Concentrated Leak Erosion

Where there is an opening through which concentrated leakage occurs, the walls of the opening may be eroded by the leaking water as shown in Figure IV-4-15. Such concentrated leaks may occur through a crack caused by settlement or hydraulic fracture (Figure IV-4-16) in a cohesive clay core, desiccation and tension cracks at higher levels in the fill, or cracks resulting from differential settlement of the fill. Situations in which concentrated leaks may occur were described previously, and figures depicting many of these situations are provided in Appendix IV-4-B. In some circumstances, these openings may be sustained by the presence of structural elements (e.g., spillways or conduits) or by the presence of cohesive materials able to "hold a roof" below which an opening is sustained and the periphery of which is eroded. It may also occur in a continuous zone containing coarse and/or poorly compacted materials which form a system of interconnected voids. The concentration of flow causes erosion (i.e., scour) of the walls of the crack or interconnected voids.

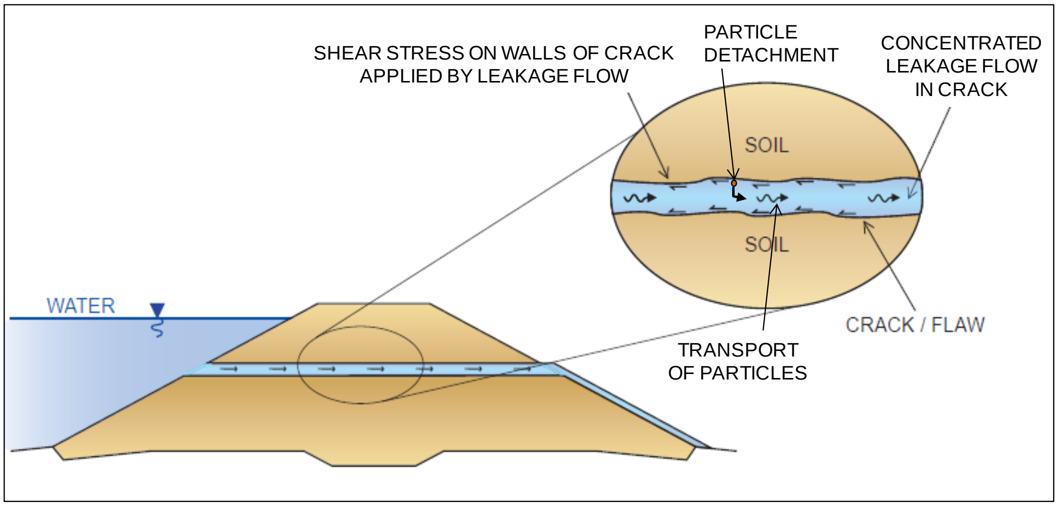

Figure IV-4-15. Concentrated Leak Erosion
(Courtesy of Mark Foster)

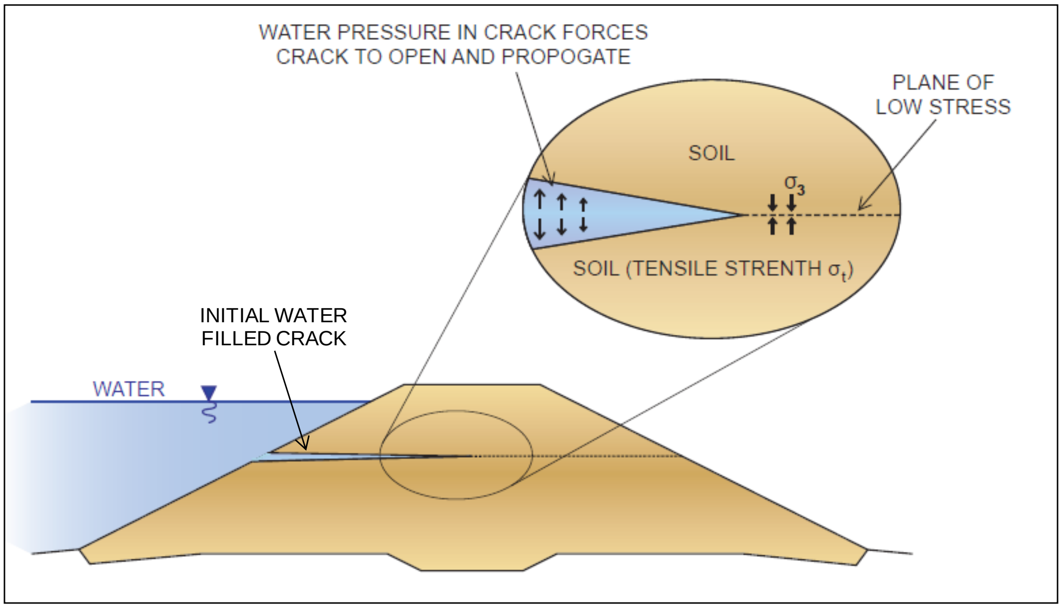

Figure IV-4-16. Hydraulic Fracture
(Courtesy of Mark Foster)

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Given a crack or gap (i.e., flaw) exists, the initiation of concentrated leak erosion depends on the depth or location of cracking relative to the reservoir level and the forces imposed on the sides of the crack by water flowing through it. The resistance to initiation of concentrated leak erosion is characterized by the critical shear stress (see Chapter IV-1 Erosion of Rock and Soil). To help assess the likelihood of initiation of concentrated leak erosion in a crack or gap, the hydraulic shear stress in the crack for the reservoir level under consideration (τ) can be compared to the critical shear stress which will initiate erosion for the soil in the core of the embankment (τ_c) at the degree of saturation of the soil on the sides of the crack. Further details are provided in Appendix IV-4-B.

Estimation of crack widths and depths involves a lot of uncertainty. ICOLD (2013) provides some examples of likely crack depths and widths due to cross-valley differential settlement or differential settlement in the foundation. These estimates are based primarily on the methods described in Fell et al. (2008), which includes methods for transverse cracking due to differential settlement, frost action, and desiccation, as well as hydraulic fracture. If a crack forms during construction, it may be masked by lifts placed near the crest after most of the deformation is complete, which may not propagate a crack upward, at least not to the same openness.

As reported in Fell et al. (2008), highly erodible soils such as silts, silty sands, or dispersive clays may be likely to erode at a crack width of 0.25 to 0.5-inch under a hydraulic gradient as low as 0.1, and at widths as small as 1 or 2 mm under hydraulic gradients of 0.5 or more. Clays may not be likely to erode until cracks reach 1 or 2 inches in width and hydraulic gradients approach 0.5 or more. However, cracks in clays may swell shut upon wetting.

Backward Erosion

Backward erosion involves the detachment of soils particles when the seepage exits to a free unfiltered surface, such as the ground surface downstream of a soil foundation, the downstream face of a homogeneous embankment, and a coarse rockfill zone immediately downstream from the fine-grained core. The detached particles are carried away by the seepage flow, and the process gradually works its way towards the upstream side of the embankment or its foundation until a continuous pipe is formed, as shown in Figure IV-4-17. Backward erosion can also occur vertically, such as in narrow central core embankment constructed with broadly graded cohesionless soils (e.g., glacial till) due to suffusion, or due to open defects in rock foundations or structures embedded in the embankment. For stoping, there is no need for a roof for the pipe since the particle movement is assisted by gravity, and the stoping process progresses to a sinkhole.

Backward Erosion Piping

Backward erosion piping occurs in cohesionless soils. It mainly occurs in foundations but may occur within embankments. The erosion process begins at a free surface on the downstream side of the embankment. For backward erosion in the foundation, the free surface may be in a ditch at the downstream embankment toe, the stream bed further downstream of the embankment, or may form due to a defect in a confining layer (e.g., due to desiccation cracking, uplift or blowout, animal burrows, excavation, or other penetrations). Backward erosion piping is often manifested by the presence of sand boils. Seepage and sand boils can represent a wide spectrum of potential conditions and risks

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(Von Thun 1996). Piping will develop when there is enough pressure and the supply of water from the pervious layer is sufficient. However, erosion will be slow when the pressure head which has caused the sand boil is sufficiently dissipated by the increased flow through the boil, similar to the effect of a relief well. For backward erosion in the embankment, the free surface may be an unfiltered or inadequately filtered zone downstream of the core. On sloping surfaces, the slow downward creep of soil particles is a sign of the development of the critical condition.

Terzaghi et al. (1996) showed that backward erosion piping will initiate when a "heave" or zero effective stress condition occurs in sands subject to upward through-seepage. The basis for design guidance is to prevent the uplift or blowout condition, and thus initiation and progression of backward erosion piping. Based on experience with Mississippi River flooding, USACE developed an analytical procedure for assessing levee underseepage and vertical exit gradients commonly known as "blanket theory" for seven scenarios (with and without confining layers) which are described in EM 1110-2-1913 (USACE 2000). Flow nets and two-dimensional finite element modeling (e.g., SEEP/W) are two commonly used techniques to estimate gradients.

To sustain piping, the seepage flow must be maintained at or above the critical gradient, and a mechanical condition is necessary to sustain a continuous roof for the developing pipe either by the embankment or a confining layer. Test results from studies by Weijers and Sellmeijer (1993), Schmertmann (2000), Sellmeijer et al. (2011) have shown that backward erosion can progress at global gradients of 40 to 60 percent of the critical gradients to cause backward erosion to initiate, especially for fairly uniform, fine to medium sands where critical global gradients can be as low as 0.02.

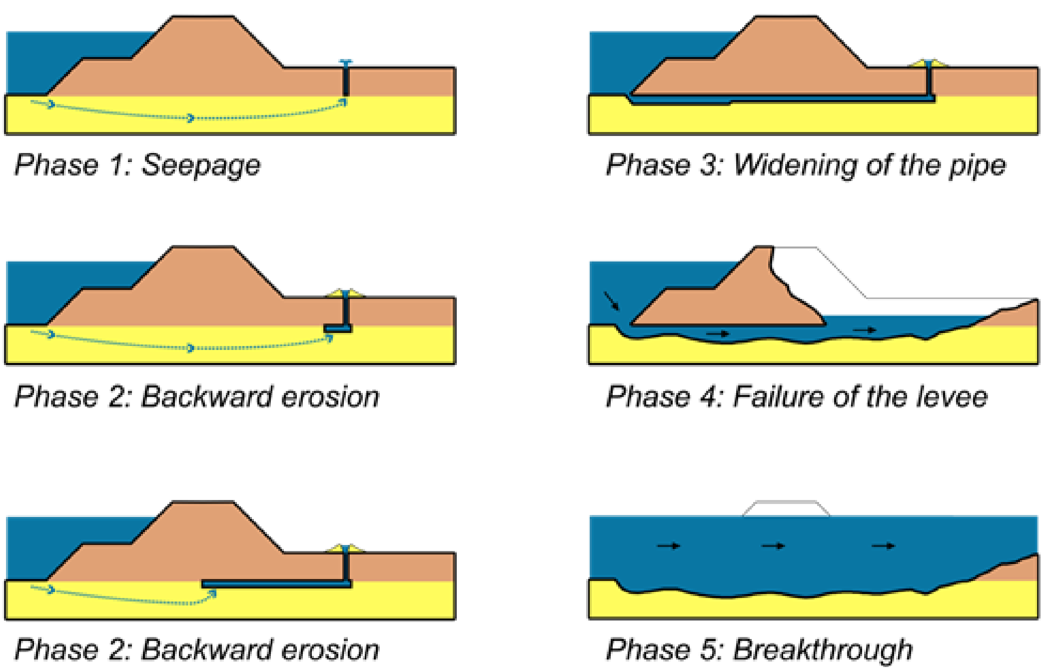

Figure IV-4-17. Backward Erosion Piping (adapted from van Beek et al. 2011)

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To help assess the likelihood of the hydraulic condition for progression of backward erosion piping, the global or horizontal gradient in the foundation can be compared to the critical gradient for progression of a pipe. Methods to evaluate the critical gradient for progression of a pipe include line-of-creep methods (Bligh 1910 and Lane 1935), Sellmeijer's piping rule (1993, 2011), Schmertmann (2000), and Hoffmans (2014). Further details on the critical gradients for initiation and progression of a pipe are provided in Appendix IV-4-C. ***Multiple methods are suggested to help inform judgment. The correct application of these methods requires an understanding of the context from which each method was developed.*** Robbins and van Beek (to be published in 2015) provide a more detailed review of the background, advantages, and disadvantages of each method and the various laboratory test conditions (e.g., density, exit configuration, soil characteristics, and scale effects) that significantly impact the findings. For example, the Sellmeijer and Schmertmann "average gradient" methods can only be used for situations that have a purely two-dimensional seepage regime (i.e., only applicable to situations that have uniform boundary conditions parallel to the embankment centerline such as an exposed ditch or no confining layer). Some methods may not apply to the materials under consideration. For example, Schmertmann's method is only recommended for $c_u < 3$, and Sellmeijer's piping rule is only applicable within the range of soils tested. ***For soils beyond the suggested ranges and differing exit configurations, the methods are not necessarily applicable, and the actual critical gradients may be quite different than what is estimated.***

All other parameters remaining the same, the likelihood of backward erosion piping is:

- Decreased by increasing particle size
- Decreased by increased coefficient of uniformity
- Decreased by increasing relative density
- Decreased by decreasing permeability
- Increased by the thickness of the piping layer
- Increased by presence of an underlying layer of higher permeability
- Increased by increased horizontal to vertical permeability ratio
- Slightly decreased by angularity of the particles
- Not changed by confining stress
- Increased for turbulent flow (Annandale 2007)

Terzaghi et al. (1996) indicate that the mechanics of piping "defy theoretical approach," and the "results of theoretical investigations into the mechanical effects of the flow of seepage serve merely as a guide for judgment." The analytical methods described in this chapter merely provide a starting point to help develop a list of more likely and less likely factors during an elicitation of probability estimates.

Stoping

Stoping can occur when the soil is not capable of sustaining a stable roof. Soil particles are eroded at an unfiltered exit and a void grows until the temporary roof can no longer be supported, at which time the roof collapses. This mechanism is repeated progressively causing the void to enlarge and migrate vertically upward. These voids can develop in both the saturated and unsaturated environments and typically result in formation of a sinkhole on the surface of the embankment.

Internal Instability (Suffusion)

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Suffusion (or internal instability) is a form of internal erosion which involves selective erosion of finer particles from the matrix of coarser particles of an internally unstable soil, in such a manner that the finer particles are removed through the voids between the larger particles by seepage flow, leaving behind a soil skeleton formed by the coarser particles, as shown in Figure IV-4-18. Suffusion results in an increase in permeability (greater seepage velocities and potentially higher hydraulic gradients) and possibly initiation of other internal erosion mechanisms into/along remnant coarser soil skeleton.

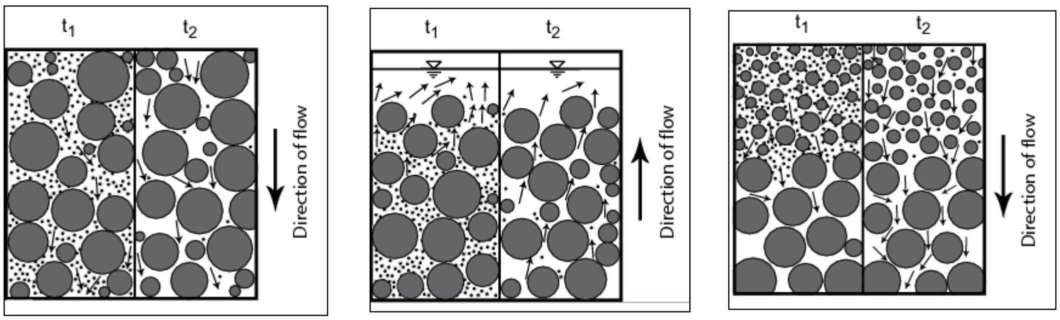

Figure IV-4-18. Internal Instability (Suffusion) (adapted from Ziems 1969)

According to Garner and Fannin (2010), the combination of three adverse conditions are needed for initiation of suffusion:

- Geometric condition: Size of finer soil particles must be smaller than size of the constrictions between coarser particles, which form the basic skeleton of the soil.
- Stress condition: Amount of finer soil particles must be less than enough to fill the voids of the basic skeleton formed by the coarser particles. Effective stresses are transferred by the coarser particles only, and some fines particles are not confined and free to move (i.e., "free fines").
- Hydraulic condition: Velocity of flow through the soil matrix must impose a high enough stress to overcome the particle weight of the finer soil particles and to move them through the constrictions between the larger soil particles.

Geometric Condition (Screening-Level Assessment of Susceptibility)

Assessing the susceptibility to internal instability for any risk assessment starts with a review of the particle-size distribution (i.e., geometric condition). Soils susceptible to internal instability include gap-graded soils and broadly graded soils with a flat tail of fines as shown in Figure IV-4-19.

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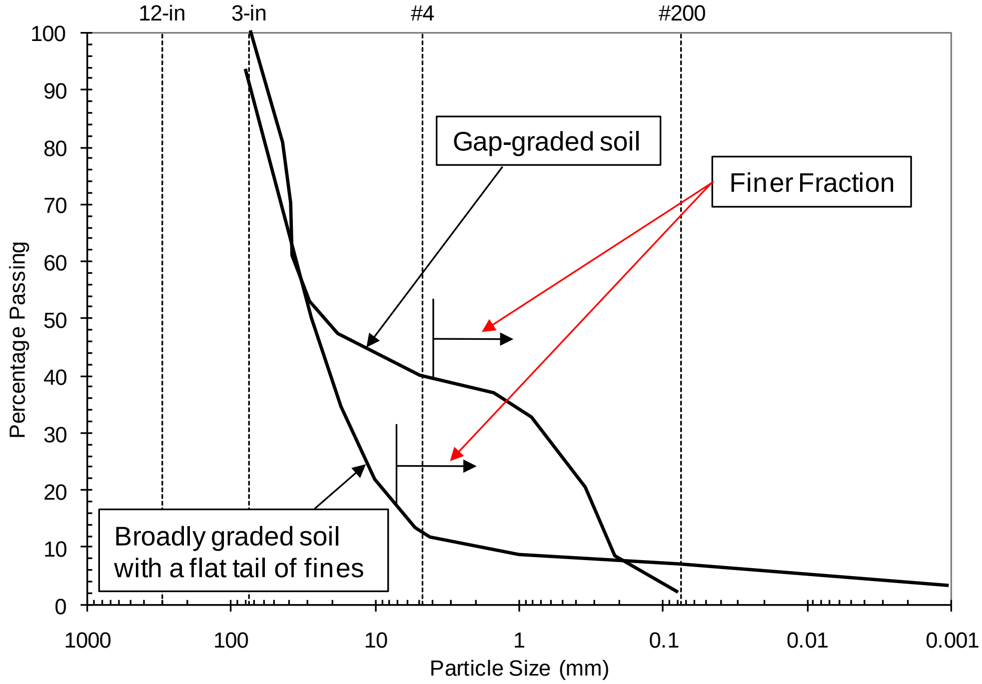

Figure IV-4-19. Potentially Internally Unstable Soils (adapted from Wan and Fell 2004a)

According to Sherard (1979), soils are generally considered “internally unstable” if the coarser fraction of the material does not filter the finer fraction. He obtained data on a variety of soils that were judged to be internally unstable and plotted a band around these gradations as shown in Figure IV-4-20. The internally unstable soil gradations usually plotted as nearly straight lines or as curves with only slight curvature within the range shown. Reclamation’s filter design standard also considers the slope of the gradation curve. This slope is illustrated in Figure IV-4-20 and is noted as “4x.” The slope of this line is approximately equal to the boundary slopes of Sherard’s band and is the same as Kenney and Lau (1985, 1986) criterion of H/F < 1 discussed in Appendix IV-4-D. The location of the “4x” line on the plot is unimportant. Any portion of a gradation curve that is flatter than this line indicates a potentially unstable soil, whereas portions of the gradation curve steeper than the line indicate a stable soil. This technique can also be used to evaluate gap-graded soils.

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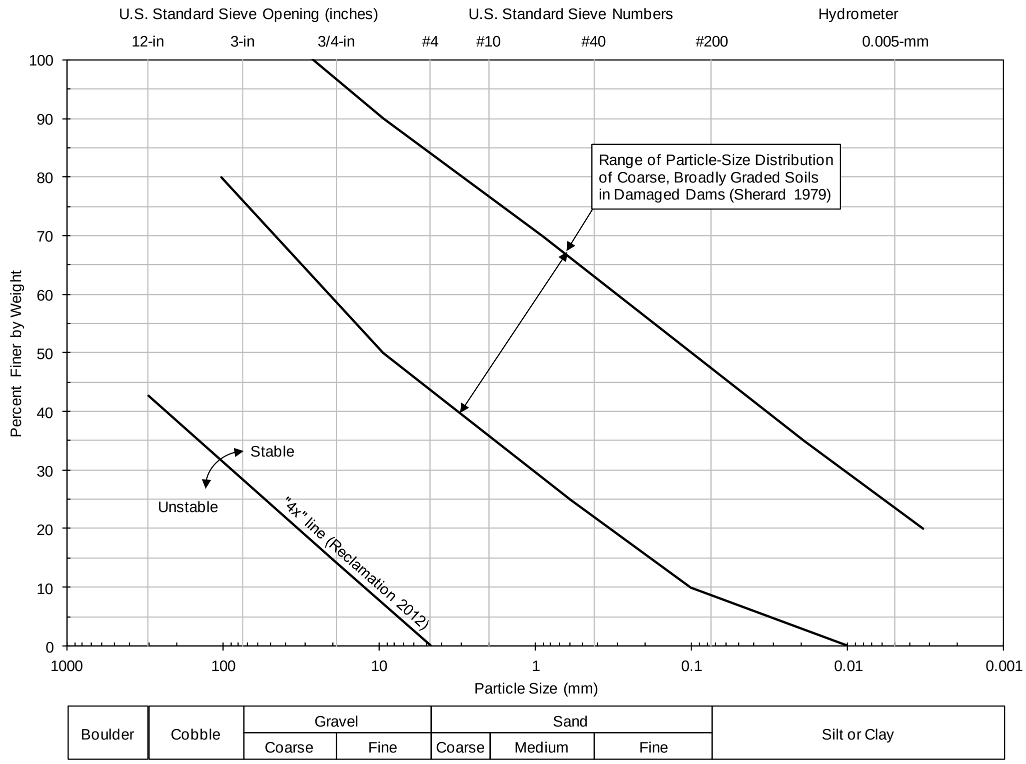


Figure IV-4-20. Potentially Internally Unstable Soils (adapted from Sherard 1979)

Geometric Condition (Detailed Evaluation of Susceptibility)

If the screening-level review of the gradation curves indicates the soil is potentially internally unstable, then the more robust methods in Appendix IV-4-D may be applied to further evaluate the susceptibility to internal instability. Several methods are described, and some methods may not apply to the materials under consideration. Multiple methods are suggested to help inform judgment. Marot et al. (2014) made the following suggestions for assessing the geometric criteria:

- Use Kézdi's criterion for gap-graded soils.
- Use Kenney and Lau's criterion for broadly graded soils with F < 15%.
- Use Kézdi's criterion for F > 15%, per Li and Fannin (2008).
- Use Wan and Fell's criteria (alternative method) for broadly graded silt-sand-gravel soils with F > 15%, per Marot et al. (2014).

Hydraulic Condition

There is little published literature on the seepage gradient required to initiate suffusion. Skempton and Brogan (1994) investigated the hydraulic criterion for the erosion of fine particles in well-graded and gap-graded sandy gravels and observed critical hydraulic gradients far less than the theoretical critical gradient for "heave."

Fell et al. (2004) summarized some general observations from laboratory testing:

- Soils with a higher porosity start to erode at lower hydraulic gradients.
- Soils with clayey fines erode at relatively higher hydraulic gradients than soils without clayey fines at similar fines contents.
- Soils with higher soil density erode at higher critical gradients, given the fines content of the soils are the same.
- Gap-graded soils erode at a relatively lower critical gradients than non-gap-graded soils with similar fines content.

According to Marot et al. (2014), the hydraulic loading on the particles is often described by three different approaches:

- Hydraulic gradient: Skempton and Brogan (1994) and Li (2008)
- Hydraulic shear stress: Reddi et al. (2000)
- Pore velocity: Marot et al. (2011, 2012)

However, more research is needed with a wider range of soils, hydraulic gradients, and flow orientation. In many of the internally stable soils tested in the laboratory, the gradients required to initiate suffusion were so high that they are unlikely to occur in dams, levees, or their foundations.

Contact Erosion

Contact erosion is a form of internal erosion which involves selective erosion of fine particles from the contact with a coarser layer caused by the passing of flow through the coarser layer (e.g., the contact between silt- and gravel-sized particles). It relates only to conditions where the flow in the coarser layer is parallel to the interface between the coarse and fine layer, as shown in Figures IV-4-21 and IV-4-22.

Probability of Initiation of Internal Erosion: USACE Approach

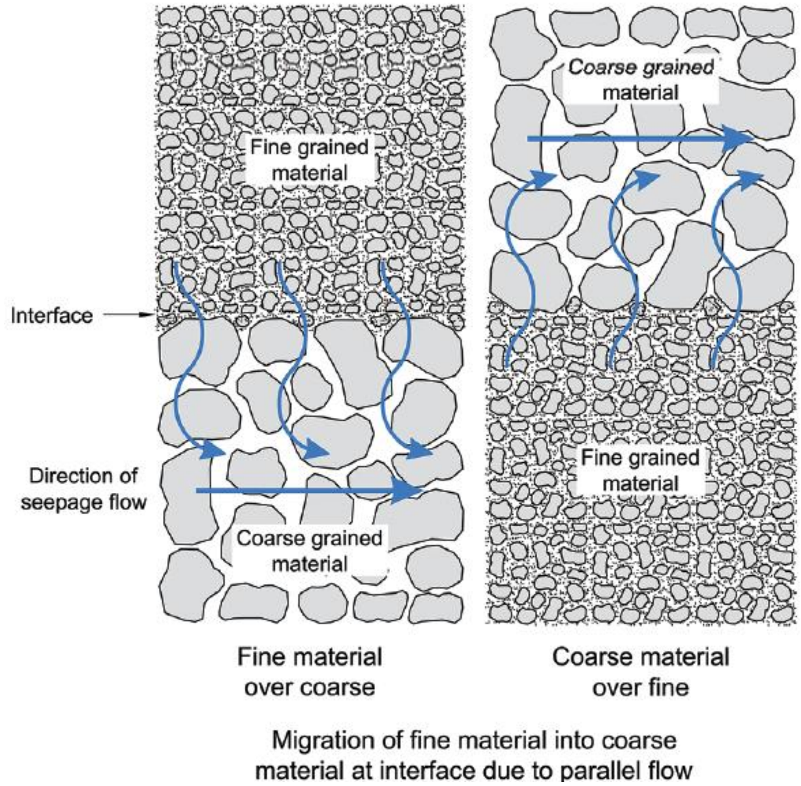


Figure IV-4-21. Contact Erosion Process (International Levee Handbook 2013)

Two conditions are needed for initiation of contact erosion:

- Geometric condition: Pores of the coarse layer have to be sufficiently large to allow fine particles to pass through (i.e., filtration criteria not satisfied).
- Hydraulic condition: Flow velocity has to be sufficient to detach the fine particles and transport them.

Probability of Initiation of Internal Erosion: USACE Approach

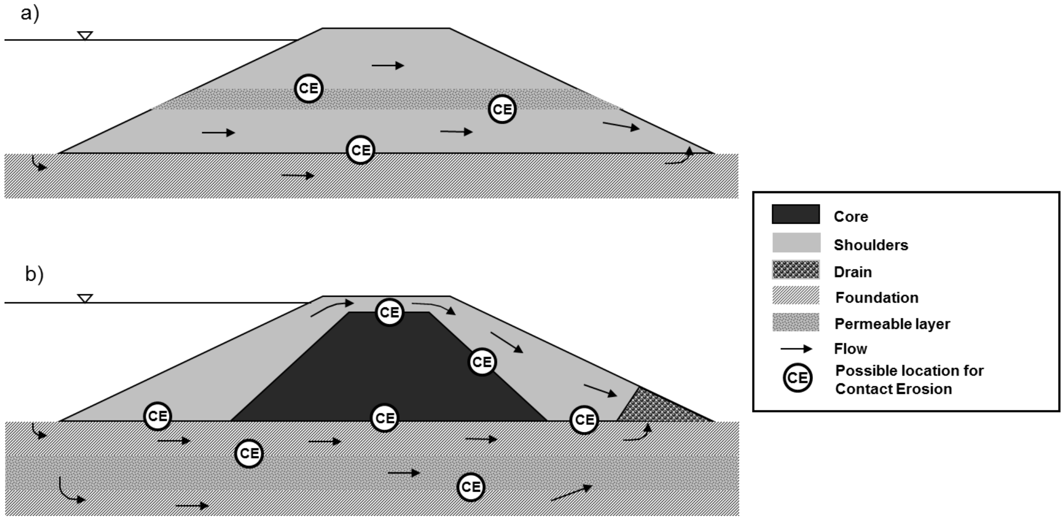


Figure IV-4-22. Possible Locations of Initiation of Contact Erosion (Béguin et al. 2009)

Contact erosion can lead to the formation of a roof at the interface, sinkhole development, creation of a weaker zone leading to slope instability, or clogging of permeable layers and increase in pore water pressure.

Geometric Condition (Screening-Level Assessment)

Fine soil layers that do not satisfy geometric criteria for filtration are not susceptible to contact erosion. Assessing the susceptibility to contact erosion for any risk assessment starts with a review of the particle-size distribution (i.e., geometric condition). Several researchers have proposed expressions for the geometric and hydraulic conditions, which are summarized in Appendix IV-4-E. Those geometric condition are very similar to the "no erosion" condition ($D_{15}/d_{85} < 9$) of Foster and Fell (1999, 2001). Modern filter design criteria can be used as an initial screening and must be used for the design of a new filter. For assessment of existing dams and levees, the Foster and Fell criteria (1999, 2001) can be used to assess filters (coarse layers) that do not satisfy modern filter design criteria (i.e., "no erosion" condition).

Hydraulic Condition

The critical gradient in the coarse layer can vary significantly depending on its permeability. However, the "critical" Darcy velocity for initiation of contact erosion does not significantly depend on its permeability and is only related to the fine soil's resistance to erosion. Therefore, the Darcy velocity is often a good indicator of the hydraulic loading and compared to the critical velocity for initiation of contact erosion. The critical velocity can be compared to the estimated Darcy velocity for the reservoir level under consideration to help assess the likelihood of initiation and progression of contact erosion.

Continuation

Erosion once initiated will continue unless the eroding forces are reduced or the passage of the eroded particles is impeded. Evaluation of this event in the event tree relies primarily on examining the filter compatibility of adjacent zones and layers in an embankment and foundation. In modern embankment dams, filters are used to prevent the migration of fines between various zones of the embankment or its foundation and to safely protect against leakage through cracks should they occur. Existing dams without modern filters depend on the filter compatibility of core and transition/shell materials (if present), as well as the compatibility at the core/foundation contact.

In essence, continuation is the phase of internal erosion where the relationship of the particle-size distribution between the base (core) material and the filters or adjacent materials controls if erosion will continue. The methodology to evaluate the probability of continuation of internal erosion will vary depending on the seepage exit. Generally, three exit conditions are considered: 1) open exit; 2) filtered/unfiltered exit; or 3) constricted (non-erodible) exit. Chapter 5 of Reclamation's Design Standards No. 13 (Reclamation 2011) entitled "Protective Filters" provides guidance for design and construction of soil filters, drains, and zoning of embankment dams that is useful for consideration in risk assessments of existing dams. Modern filters and drains defend against cracks and assure significant head loss occurs at the boundary if it already has not occurred because of cracks.

Open Exit

If there is a free or open face, then there is no potential for filtering action due to an unfiltered exit, and the probability of continuing erosion is virtually certain (i.e., $P_{CE} \approx 0.999$). Open exits can also be the result from common-cause cracking in the filter or transition materials.

Filtered/Unfiltered Exit

Some zones may be designated as "filters" but may not satisfy the current definition of a filter. Conversely, there may be material that does not meet filter criteria but can be considered an "opportunistic filter." The evaluation of filter compatibility between the base soil (core) and the filter or adjacent materials requires representative particle-size distribution data. When there are a greater number of gradations, the reliability of the filter compatibility assessment is improved. If the gradations are plotted on the same sheet, the normal range or gradation band can be observed for both the base soil and the filter, along with any outlier gradations. In soils containing coarse particles (gravel, cobbles, or boulders), it is important to realize they frequently do not show up in gradations. In zoned embankments, multiple filters or zones often provide transition from the finer to coarser materials. Each filter zone must be filter-compatible with the preceding zone if seepage across the boundary occurs. Therefore, the filter compatibility evaluation may be a multi-step process, depending on the embankment zoning. If a perforated pipe is installed in drain rock to transmit accumulated water, the compatibility with the perforations in the drain pipe must be evaluated. The following steps should be followed when assessing the likelihood of continuation of internal erosion for filtered exits:

- Gather the available information on particle-size distributions of the core or foundation and the filter or transition materials. This may include data from design-phase borrow area investigations, construction control testing, and post-construction testing on samples from the dam or its foundation. If only a few samples are available for each zone and only from borrow sources, care must be taken in drawing conclusions from the data to evaluate filter compatibility. Consider reviewing the borrow source and placement information. It may be that different portions of the embankment were placed using different borrow areas or zones within a borrow area. Therefore, some areas may have predominantly finer core material (or coarser adjacent material) and these areas should be evaluated using information specific to those areas and not the average conditions. A check if scalping was done in the field using a grizzly (i.e., screen) can be important.

- Plot the particle-size distributions for the base soil and filter materials. If the base soil contains gravel (i.e., materials larger than a No. 4 sieve), then re-grade the gradation curves for the base soil if any of the following conditions apply: FC ≥ 15 percent of the original base soil; the base soil is gap-graded; or the base soil is broadly graded ($C_u \geq 6$ and $1 \leq C_c \leq 3$). If the base soil's FC < 15 percent and it is neither gap-graded nor broadly graded, then re-grading is not required. The re-grading is typically performed on the No. 4 sieve so that the maximum size is 4.75 mm. Obtain a correction factor by dividing 100 by the percentage passing the No. 4 sieve. Multiply the percentage passing each sieve size of the base soil smaller than a No. 4 sieve by this correction factor. An example is shown in Figure IV-4-23. Plot the re-graded gradation curve using these adjusted percentages.

Sieve Size	Original Percent Passing	Adjustment	Final Percent Passing
3"	100.0		
1 1/2"	85.7		
3/4"	74.6		
3/8"	65.9		
#4	57.9	(57.9 / 57.9) x 100	100.0
#8	54.6	(54.6 / 57.9) x 100	94.3
#16	49.0	(49.0 / 57.9) x 100	84.6
#30	42.6	(42.6 / 57.9) x 100	73.6
#50	32.2	(32.2 / 57.9) x 100	55.6
#100	19.8	(19.8 / 57.9) x 100	34.2
#200	13.0	(13.0 / 57.9) x 100	22.5
1 min	9.9	(9.9 / 57.9) x 100	17.1
4 min	5.4	(5.4 / 57.9) x 100	9.3
19 min	2.9	(2.9 / 57.9) x 100	5.0
60 min	1.6	(1.6 / 57.9) x 100	2.8

Typ.

Figure IV-4-23. Example of Re-Grading Calculations (Reclamation 2011)

- Consider whether the filter materials are susceptible to cracking based on fines content, cementation, or the presence of plastic fines. Further details are provided in Appendix IV-4-F. If the filter materials are susceptible to cracking or subject to deformations that could cause cracking, then assume there is questionable potential for filtering action due to an unfiltered exit, and there is some probability (perhaps high) of continuing erosion (estimated by team judgment).

- Consider whether the filter materials are susceptible to segregation during storing, hauling, dumping, spreading, and compacting, and if the segregated layer is continuous. Further details are provided in Appendix IV-4-F. If a continuous segregated layer is likely, then the procedure of Fell et al. (2008) could be used to estimate the gradation after segregation. Further details are provided in Appendix IV-4-F.

- Consider whether the filter materials are susceptible to internal instability as previously described in this chapter. If internal instability is likely, then the procedure of Fell et al. (2008) could be used to estimate the gradation after washout of the erodible soil fraction. Further details are provided in Appendix IV-4-F.

- Assess if the filter materials will prevent continuation of internal erosion using modern filter design criteria. For filter materials which are coarser than required by modern filter design criteria, the Foster and Fell (1999, 2001) method may be used because it allows assessment of filters which are too coarse to satisfy modern no erosion design criteria. Further details are provided in Appendix IV-4-F. The “No Erosion” criteria must always be used for the design of a new filter. The other criteria are only used to evaluate existing dams.

- Check for blowout in cases where there is limited depth of cover over the filter material, comparing the seepage head at the downstream face of the core to the weight of soil cover (see section entitled “Critical Gradient, Heave, Uplift, and Blowout”). In addition, check for possible slope instability assuming appropriate pore pressures.

Continuity

An important point about continuation is whether the unfiltered exit is truly continuous. Zones in shell materials and layers of alluvial materials that act as unfiltered exits (i.e., don’t satisfy filter compatibility) need to be continuous to an open face or extensive void space need to exist in coarse soils or bedrock for eroded fines to be deposited into.

Constricted (Non-Erodible) Exit

For erosion to continue, the open joint, defect, or crack in conduits, walls, or rock foundations needs to be sufficiently open to allow the surrounding soil particles to pass through it. The effective opening size of such defects can be used to assess whether such features will allow internal erosion to continue. Poorly designed or inadequately filtered underdrains, toe drains, relief wells, or weep holes into which embankment or foundation materials can be eroded should be evaluated using similar “opening size” considerations, where applicable.

There are no commonly adopted criteria for assessing the likelihood of continuation for this scenario, although some have used design criteria for perforation size for drain pipes. For example, in order to prevent erosion into a drain opening, Reclamation traditionally recommended that $D_{85}E$ of the soil material closest to the crack or joint must be greater than or equal to 2 times the opening for uniformly graded materials or 4 times the opening for broadly graded materials. However, Reclamation and USCACE currently recommend that the maximum pipe perforation dimension when designing drain pipes should be no larger than the finer side of the $D_{50}E$ of the surrounding envelope material.

In other words, the maximum perforation dimension should be less than or equal to the $D_{50}E$ of the envelope material. Since both these criteria are used for design and could be assumed to represent no erosion limits, they are likely conservative unless the particles are flat.

Based on the results of filter tests on uniform base soils, Sherard et al. (1984) concluded that uniform filters act similar to laboratory sieves, with an opening sieve size approximately equal to $D_{15}F/9$. In a later series of tests (Sherard unpublished Memo 1G, 1985a), materials passing through the filters were caught and gradations of the material showed that approximately 97 to 99 percent of the particles were finer than $D_{15}F/9$. Foster and Fell (1999) obtained similar results. Based on these findings, Fell et al. (2008) suggested the following criterion for continuing erosion:

$$JOS_{CE} \geq D_{95}E$$

where JOS_{CE} = the opening size of the defect that would allow continuing erosion of the surrounding soil; and $D_{95}E$ = particle size adjacent to the open defect (i.e., envelope material) for which 95 percent by weight is finer after re-grading. This criterion assumes that the Foster and Fell (1999, 2001) continuing erosion criteria apply to erosion into an open joint, defect, or crack in conduits, walls, toe drains, or rock foundations, and that the crack width is equivalent to the filter opening size of the voids between the particles in a filter.

Again, it’s important to remember that constrictions that are retaining soils and preventing erosion need to be continuous to some exit point. For example, bedrock joints/fractures need to be continuous to an open face and not covered by alluvium. In some rare cases where extensive void spaces may exist in coarse soils or bedrock, an open exit may not be needed, but sufficient “storage space” for eroded fines must be available. It is also important to consider the flow direction and likelihood of flow reversal.

Progression

Progression is the process of developing and enlarging an erosion pathway through the embankment core or foundation. The progression phase can be subdivided into three separate processes for concentrated leak erosion (scour) and backward erosion piping. These processes include: 1) formation of a continuous stable roof and/or sidewalls through the core; 2) the possibility that flows are limited by a constriction or an upstream zone or structure; and 3) the potential for an upstream zone to provide self-healing. These three considerations are commonly used, but other factors may also need to be considered for the progression phase in some cases. The progression phase includes all steps after continuation and prior to breach with the exception of intervention.

Enlargement of the erosion pathway may occur in either an upstream or downstream direction. For internal erosion mechanisms that do not necessarily require formation of a pipe that connects to the reservoir (i.e., stoping or internal migration), then the progression phase as defined here would likely be different. Currently there is no uniform practice for evaluation of progression for these other internal erosion processes, although they need to be included in specific events trees. For example, a standard progression event description could be modified to include “the probability that a large sinkhole forms in a critical area allowing progression to continue.”

Progression – Continuous Stable Roof and/or Sidewalls

Formation of a continuous roof through the core or foundation is dependent on the soil conditions or presence of structures above the potentially erodible soils. Therefore, conduits, spillways, walls, and other concrete structures can form a roof along an identified potential internal erosion pathway. Interbeds of “hardpan,” caliche, or other slightly cemented materials also constitute potential roofs for underlying soils that are not capable of supporting a roof by themselves. Absent these conditions, the capability of the soil to support a roof is dependent mainly on the properties of the soil above those being eroded.

Fell et al. (2008) summarized work by Foster (1999) and Foster and Fell (1999) that evaluated case histories and found that the two most important factors for roof formation are the fines content and whether or not the soil is saturated. Soils with fines contents greater than about 15 percent were found to be likely to hold a roof regardless of the plasticity (whether non-plastic or plastic). Other influential factors include the degree of compaction (loose soil less likely to support a roof) and reservoir operation (cyclic reservoir levels were more likely to cause collapse than constant levels). Research by Park (2003)[2] showed that sandy gravel with 5 to 15 percent non-plastic fines collapsed quickly when saturated. Park also found that sandy gravel with 5 percent cohesive fines collapsed after some time, but very slowly with 15 percent cohesive fines.

Based on these studies, Table IV-4-14 adapted from Fell et al. (2008), provides guidance on the likelihood a soil will be able to support a roof, absent overlying harder materials.

For concentrated leak erosion that occurs high in the embankment (e.g., cracks in the crest or a gap adjacent to a spillway wall), a roof is not necessarily a requirement for the process to progress. It is possible that the sidewalls could collapse and prevent further progression rather than collapse of a roof material. If the primary internal erosion mechanism is stoping (internal migration) without formation of a roof, then this node can be eliminated from the event tree.

The presence of a structure or hard layer and soil properties are primary factors to consider in roof formation. Some other factors include soil variability along the seepage path, the length of the seepage path, and stress arching.

[2] Park’s research was related to cracking in filters. Some of the test results were considered applicable to the potential for roof formation of soils.

Table IV-4-14. Probability of Holding a Roof
(adapted from Fell et al. 2008)

USCS Soil Classification	Fines Content, FC (percent)	Plasticity of Fines	Moisture Condition	Probability of Holding a Roof (P_{PR})
Clays, sandy clays (CL, CH, CL-CH)	FC ≥ 50	Plastic	Moist or Saturated	0.9+
Silts (ML, MH)	FC ≥ 50	Plastic or Non-Plastic	Moist or Saturated	0.9+
Clayey sands, gravelly clays (SC, GC)	15 ≤ FC < 50	Plastic	Moist or Saturated	0.9+
Silty sands, silty gravels, silty sandy gravel (SM, GM)	15 ≤ FC < 50	Non-Plastic	Moist Saturated	0.7 to 0.9+ 0.5 to 0.9+
Granular soils with some cohesive fines (SP-SC, SW-SC, GP-GC, GW-GC)	5 ≤ FC < 15	Plastic	Moist Saturated	0.5 to 0.9+ 0.2 to 0.5
Granular soils with some non-plastic fines (SP-SM, SW-SM, GP-GM, GW-GM)	5 ≤ FC < 15	Non-Plastic	Moist Saturated	0.05 to 0.1 0.02 to 0.05
Granular soils (SP, SW, GP, GW)	FC < 5	Plastic	Moist or Saturated	0.001 to 0.01
		Non-Plastic	Moist or Saturated	0.0001
Notes: (1) Lower range of probabilities is for poorly compacted materials (i.e., not rolled), and upper bound is for well compacted materials. (2) Cemented materials give higher probabilities than indicated in the table. If the soils are cemented, use the category that best describes the particular situation.				

The probabilities should not be used directly in a risk assessment, but rather used to help develop a list of more likely and less likely factors during an elicitation of probability estimates.

Progression – Constriction or Upstream Zone Fails to Limit Flows

There are some cases where internal erosion can progress to the point where the dam core or foundation is eroded through, but a flow constriction at some point along the path, an upstream zone, or facing element limits the flow from the reservoir to the point where erosion is arrested and a breach will not form. This is contingent upon the upstream zone being stable under the flows and having small enough openings to limit flows through the zone to levels that would prevent further erosion of the core. In essence, the flow is limited so that shear stresses are insufficient to detach soil particles.

Fell et al. (2008) suggest that the success of the upstream zone in limiting flows is highly dependent on whether the mechanism leading to a flaw in the core is also present in the upstream zone, with its ability to support a roof or crack of secondary importance. If the potential for the flaw to extend through the upstream zone is high and the potential for the upstream zone to support a roof or crack is high, then flow limitation is unlikely.

Examples of constrictions may include concrete or sheet pile walls within the embankment or that fully penetrate foundation soils greatly increase the likelihood of flows being limited. Modern concrete walls (crossing the internal erosion pathway, typically extending into rock) that are in good condition have the best chance for success. Steel sheet pile walls may be less effective under poor driving conditions or poor construction techniques. Concrete or steel membranes, soil-cement slope protection, geomembranes, or other linings on the upstream face of the dam can be effective in limiting flows, depending on their condition, but potential erosion of the underlying support for the facing may be an issue.

For failure modes that involve seepage paths through bedrock discontinuities, the flow could be limited by the aperture of those discontinuities. Similarly, failure modes in which the seepage flows into a crack or joint in concrete, such as an outlet works conduit, the flow may be limited. However, flow velocities could be quite high, which could lead to stoping (internal migration).

For potential failure modes through the foundation, upstream fine-grained blankets beneath and around the dam may not prevent initiation of erosion but may be effective in limiting progression. Flow limitation may occur due to an increase in head loss across the upstream blanket after uplift of the downstream blanket and initiation of erosion.

In unusual cases, progression could create a large enough void that results in failure of the structure or zone providing the constriction.

Progression – No Self-healing Provided by Upstream Zone

Crack-filling action requires a granular zone upstream of the core with particles of a size, which can be transported by water flowing into the crack or pipe, and a downstream filter/transition zone or rockfill, which is sufficiently fine to act as a filter to these particles and the core.

Upstream granular zones have been observed to help supply crack-filling materials and contribute to self-healing. Typically, sinkholes appeared above the upstream filter/transition zone which is considered to be evidence of material being washed into the crack or pipe. Crack-filling action is only possible for central and sloping core earth and rockfill (or gravel shoulders) dams. The effectiveness of the crack-filling action depends on the compatibility of particle sizes of the granular material upstream of the core and in the downstream filter/transition zone, and then the compatibility of the downstream filter/transition material (with the washed-in particles) and the core. The internal erosion process may be arrested and not lead to breach if the crack or pipe progresses through the core, but there is an upstream zone which can collapse into it (i.e., the upstream zone is not capable of supporting a crack or a roof) and a downstream filter/transition zone which then acts as a filter. The washed-in materials aid in the filtering action against the downstream filter/transition zone, especially in cases of poor filter compatibility between the core and downstream filter/transition zone due to a lack of sand-sized particles in the core. In these cases, the probability of continuation may be high, but the washed-in particles may be capable of filtering against the downstream filter/transition zone reducing the potential for the pipe enlarging. There is less benefit when the washed-in particles are of similar sizes to the core material. There is limited benefit when there is no downstream filter/transition zone. The likelihood of success is difficult to estimate, but probably increases with thicker upstream zones, the presence of truly cohesionless materials, a variety of particle sizes, and the presence of a downstream shell or zone that will provide a filter for these materials that wash into and through the core. Finally, the size and nature of the defect in the core is a consideration (i.e., self-healing may occur early when the defect is a crack or later when the defect is a pipe).

Consideration should be given to whether the self-healing will occur early when the defect is small. In general, it is more likely to self-heal earlier in the process when sand size particles could be carried to downstream zone by relatively low flows. Gravel and larger sizes need high flows to be transported, so by the time flows are large enough to transport these sizes, significant enlargement of the erosion pathway may have already occurred. A well-documented example of this type of self-healing is in a case history for Matahina Dam in New Zealand (Gillon). Self-healing has also been observed at Suorva Dam in Sweden (Nillson 2005, 2007) and at Uljua Dam in Finland (Kuusiniemi 1991).

Assessing the Rate of Enlargement of a Pipe

The time for erosion to progress is an important factor for assessing the likelihood of successful intervention and is dependent on the soil erosion properties. In addition, breach mechanisms vary in their time to fully develop and catastrophically release the reservoir. Therefore, the likelihood of successful intervention should also consider the potential time available based on the breach mechanism being considered.

However, the duration of the critical loading is also an important condition, and episodic cycling of the reservoir may result in the progression of erosion occurring only sporadically (at high pool) over the course of weeks, months, years or even decades. This can complicate the assessment of the rate of enlargement significantly. Although the following procedure does not include the effects of pool duration and reservoir cycling, it may provide useful insights into the development time for erosion progression.

While the resistance to initiation of concentrated leak erosion is characterized by the critical shear stress, the rate of pipe enlargement in the progression phase under a significantly long loading event is characterized by the erodibility coefficient (rate of change of erosion rate). There are several methods for estimating the erosion properties of soils for concentrated leak erosion. The Hole Erosion Test (HET), Jet Erosion Test (JET), and Erosion Function Apparatus (EFA) are the most widely used tests. Further details on methods to estimate the erodibility parameters are discussed in Chapter IV-1 Erosion of Rock and Soil.

The erosion law for these tests can be expressed in terms of volume erosion (Hanson 1990):

$$\dot{\varepsilon} = k_d(\tau - \tau_c)$$

where $\dot{\varepsilon}$ = rate of volume of material removed per unit surface area per unit time (typically reduced to mm/hr or in/hr), τ = hydraulic shear stress for the reservoir level under consideration (typically Pa or psf); τ_c = critical shear stress for initiation of erosion (typically Pa or psf); and k_d = erodibility coefficient (typically $cm^3/(Ns)$ or (ft/hr)/psf).

The erosion law can also be expressed in terms of mass erosion (Wan and Fell 2002):

$$\dot{m} = C_e(\tau - \tau_c)$$

where $\dot{m}$ = rate of mass removed per unit surface area per unit time (typically kg/s/m^2), τ = hydraulic shear stress for the reservoir level under consideration (typically Pa or psf); τ_c = critical shear stress for initiation of erosion (typically Pa or psf); and C_e = coefficient of soil erosion (s/m).

For $\tau < \tau_c$, the erosion rate is zero. Values of k_d and C_e are related by the following expression:

$$C_e = k_d(\rho_d)$$

where ρ_d = dry density of the soil. Typical values for the erodibility parameters are presented in Chapter IV-1 Erosion of Rock and Soil.

The rate of enlargement of a pipe can be estimated using the erodibility parameters of the eroding soil and average hydraulic gradient along the pipe, as described in Appendix IV-4-G. Figure IV-4-24 illustrates the importance of soil erodibility (characterized by the Hole Erosion Test index) on the time for erosion to progress, based on the following assumptions: unrestricted potential for erosion (i.e., no flow limitation, continuing erosion condition); initial pipe diameter of 25 mm (1 inch); zero critical shear stress (which is conservative, particularly for $I_{HET} > 3.5$); shape of pipe remains circular; pipe can sustain a roof while it enlarges; and reservoir level remains constant. The time to erode to 2 m in diameter is about 20 percent greater.

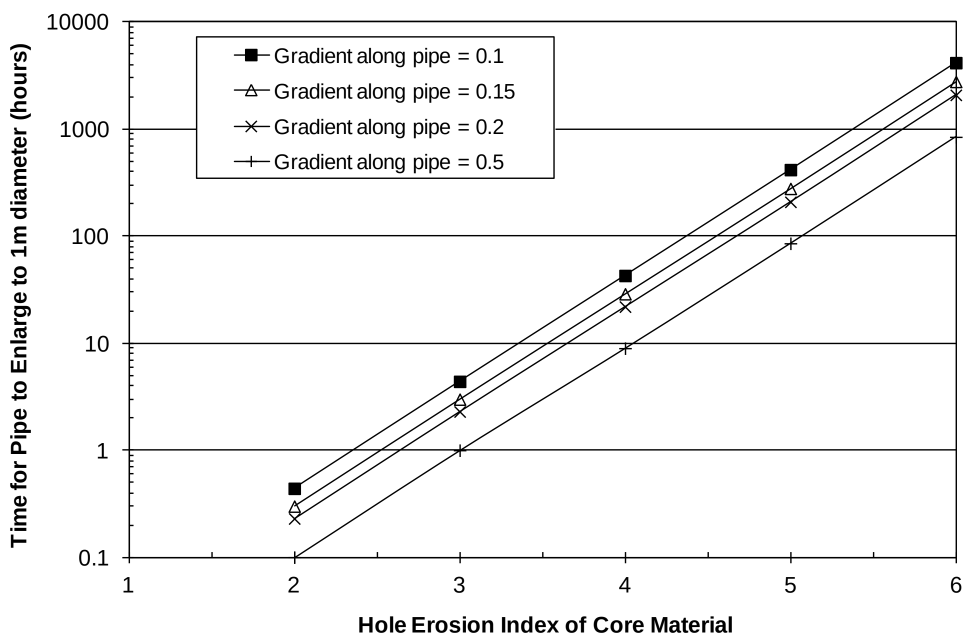


Figure IV-4-24. Approximate Time for a Pipe to Enlarge from 25 mm to 1m in Diameter (Fell et al. 2008)

Unsuccessful Intervention

This node considers the likelihood that human efforts to detect and stop (or slow) the internal erosion process from breaching the dam fail to work. This single node evaluates the potential that two components might occur: 1) detection (i.e., whether, or when, a developing failure mechanism would be observed and recognized as a problem); and 2) the ability to successfully intervene (i.e., can mitigating efforts be implemented in time to stop or slow the failure process to the point where dam breach does not occur?). The probability of unsuccessful intervention is captured in one node on the event tree, just before breach, although it is recognized that intervention efforts are likely to occur during all phases of an internal erosion process. When estimating the probability of unsuccessful intervention, it is acceptable to consider factors that would support earlier intervention, chronologically before the failure mode has completed the "progression" events.

Risk estimates must give due consideration for intervention actions. In order for intervention to be successful, the failure path must be detected, and repairs or lowering the reservoir must be performed prior to breach development. It is USACE's practice to prepare risk estimates for both with and without intervention because to understand the potential for detection and the benefits of intervention while at the same time not masking the seriousness of the issue by using intervention to reduce the estimated risk.

Reclamation's and USACE's experience includes internal erosion incidents that progressed for decades (although they were not recognized as such early on). Furthermore, both agencies have had a high rate of successful intervention. Only about 1 percent of Reclamation's incidents involving the initiation of internal erosion have led to dam breach (Teton Dam), and within USACE no incidents have resulted in dam breach. However, USACE has experienced several levee breaches. This success is due to a number of factors involving detection and successfully intervening at various points within the internal erosion process; however, two factors appear to particularly stand out in the tabulated cases.

- First, in most cases, signs of the potential initiation of internal erosion (e.g., sinkholes, sand boils, and excessive seepage) were observed and necessary remedial actions were quickly taken. Internal erosion incidents have typically been discovered by visual observation, sometimes by the public. For this reason, "eyes on the dam" is a key consideration. Is the dam in a remote location? Are likely downstream exit paths observable (consider rockfill, tailwater, marsh areas, beneath blankets, etc.)? How often is the dam visited/observed? How close does the public get? Are local officials (police, park rangers, and recreation staff) trained in dam safety? Few cases have been detected by routine instrumentation monitoring, although it has happened. Over the long-term, piezometer and seepage measurement trends can be indicative of slowly developing internal erosion failure modes.

- Second, there are a number of instances where it appears that self-healing or collapse of developing internal erosion took place and either stopped the process or provided warning such that intervention could take place. This episodic nature of internal erosion incidents, which can lead to these failure mechanisms taking decades to progress (or initiate in some cases), has been demonstrated in all categories of internal erosion, particularly in those involving foundation materials, conduits, or drains. The episodic nature has the benefit of increasing the likelihood of observation, but it can be dangerous because a connection to the reservoir can be sudden, after progressing undetected for a long time.

Evaluating factors related to detection and physical intervention actions is very site-specific and requires judgment and subjective probability estimates (see Chapter I-6 Subjective Probability and Expert Elicitation). For example, if there is coarse rockfill on the downstream slope or ponding at the toe of the dam, it may be very difficult to detect new muddy seepage. If the reservoir is large and the release capacity is small, attempting to draw the pool down may be of little help. If equipment and materials are not readily available from nearby sources, there may be little that can be done in the way of emergency repairs. These are important items to consider when evaluating detection and intervention. If this node is estimated to have a high likelihood for success, it should be highlighted in the documentation, as this is critical information for the operations of the facility. Even if the estimated likelihood of success is low, it should still be pre-planned and attempted should it occur.

Fell et al. (2001, 2003) studied case histories of failures and accidents for piping in the embankment, foundation, and embankment into the foundation. Based on the case histories and an understanding of the physical processes, they developed guidance on the time for progression beyond when a concentrated leak is first observed and development of a breach. Tables IV-4-24 to IV-24-26 are based on that study. In these tables, the qualitative terms for rates are defined in Table IV-4-27. Table IV-4-24 could be used to estimate the approximate time to dam failure after a concentrated leak is first observed. Most of the case studies were for breach by gross enlargement. Therefore, the method is only applicable to cases where the breach mechanism is gross enlargement. It is considered reasonable where the final breach is by slope instability, following development of a pipe. It will probably underestimate the time for breach by sloughing since breach by sloughing is a slowly developing mechanism which could take days to weeks to lead to breach. Breach by sinkhole development is potentially a rapid process in the final stages when the sinkhole emerges into the reservoir, but limited case history data exists.

Table IV-4-24. Rate of Erosion of the Embankment Core or Foundation Soil (adapted from Fell et al. 2001, 2003)

Factors Influencing the Time for Progression and Breach					
Ability to Support a Roof	Rate of Erosion (Table IV-4-25)	Upstream Flow Limiter	Breach Time (Table IV-4-26)	Approximate Likely Time (Qualitative)	Approximate Likely Time
Yes	R or VR	No	VR or R–VR	Very Rapid	< 3 hours
Yes	R	No	R	Very Rapid to Rapid	3 to 12 hours
Yes	R–M	No	VR	Rapid	12 to 24 hours
Yes	R	No	R–M		
Yes	R	No	M or S	Rapid to Medium	1 to 2 days
Yes	R or R–M	No	M or M–S		
Yes	M or R–M	Yes	R or R–M		
Yes	M or R–M	No	S	Medium	2 to 7 days
Yes	R–M or M	Yes	S		
Yes	M	Yes or No	S	Slow	Weeks, even months to years

Table IV-4-25. Rate of Erosion of the Embankment Core or Foundation Soil (used in Table IV-4-24) (adapted from Fell et al. 2008)

Soil Classification	(I_{HET})	Time for erosion in the core of the embankment or in the foundation	
		0.2-gradient along pipe	0.5-gradient along pipe
SM with < 30% fines	< 2	Very Rapid	Very Rapid
SM with > 30% fines	2 to 3	Very Rapid	Very Rapid
SC with < 30% fines	2 to 3	Very Rapid	Very Rapid
SC with > 40% fines	3	Rapid	Very Rapid
ML	2 to 3	Very Rapid to Rapid	Very Rapid
CL-ML	3	Rapid	Very Rapid
CL	3 to 4	Rapid	Very Rapid to Rapid
CL-CH	4	Rapid	Rapid
MH	3 to 4	Rapid	Very Rapid to Rapid
CH with LL < 65	4	Rapid to Medium	Rapid
CH with LL > 65	5	Medium to Slow	Medium

Note: I_{HET} is the index value from the Hole Erosion test (HET)

Table IV-4-26. Influence of the Material in the Downstream Zone of the Embankment on the Likely Time for Development of a Breach due to Gross enlargement of a Pipe (used in Table IV-4-24) (adapted from Fell et al. 2003)

Material Description	Likely Breach Time
Coarse-grained rockfill	Slow – Medium
Soil of high plasticity (PI > 50) and high clay-size content including clayey gravels	Medium – Rapid
Soil of low plasticity (PI < 35) and low clay-size content, all poorly compacted soils, silty sandy gravels	Rapid – Very Rapid
Sand, silty sand, silt	Very Rapid

Table IV-4-27. Qualitative Terms for Times of Development of Internal Erosion and Breach (adapted from Fell et al. 2003)

Qualitative Term	Equivalent Time
Slow (S)	Weeks or months, even years
Medium (M)	Days or weeks
Rapid (R)	Hours (> 12 hours) or days
Very Rapid (VR)	< 3 hours

Breach

Breach is the fourth and final phase of internal erosion in which materials in the dam are eroded, and the opening in the dam widens and deepens until an uncontrolled release of the reservoir occurs. The full contents of the reservoir may not be lost depending upon many factors. Breach occurs when either the failure mode is not detected or intervention is not attempted or is unsuccessful. The type of breach depends on the internal erosion mechanism being considered, embankment type, and the specific failure mode being considered. According to Fell et al. (2008), there are four breach mechanisms typically considered:

- **Gross enlargement of a pipe or concentrated leak:** If the erosion pathway or "pipe" connects to the reservoir, rapid erosion and enlargement of the pipe could develop until the crest collapses into the pipe. If the amount of crest drop is greater than the available freeboard, overtopping of the embankment could quickly lead to a breach. If overtopping does not occur, the embankment could be severely damaged, and breach could still occur by concentrated flow through cracks. If the likely breach mechanism for a potential failure mode is breach by gross enlargement, as opposed to sinkhole development or sloughing, a breach is generally more likely to occur. If the downstream shell is unable to support a roof, sloughing or unraveling would be the more likely breach mechanism.

- **Sloughing or unraveling of the downstream face**: In situations where the downstream zone is not capable of sustaining a roof, over-steepening of the downstream slope due to progressive slumping can eventually lead to complete loss of freeboard. Soil particles are eroded, and a temporary void grows near the downstream face until a roof can no longer be supported, at which time the void collapses. This mechanism is repeated progressively until the core is breached or the downstream slope is over-steepened to the point of instability. Unraveling refers to progressive removal of individual rocks by large seepage flows through a downstream rockfill zone. According to Leps (1973), the stability of rockfill against through-seepage depends on the following characteristics and conditions (listed in increasing importance): specific gravity of the rock particles, dominant particle size of the rock fill, gradation and shape of the rockfill particles, relative density of the rockfill, rate of discharge, maximum gradient, and inclination of the downstream slope of the rock fill. Methods to evaluate the stable rock size as a function of unit discharge and downstream slope include Olivier (1967), Solvik (1991), and EBL (2005).

Reclamation's Fontenelle Dam in Wyoming nearly breached in 1965 by sloughing, but the breach process occurred slowly enough so that the reservoir water surface was able to be lowered over the span of several days and arrest the breach. In contrast, Hell Hole Dam, a rockfill structure in California, failed from overtopping during construction in 1964, but it handled a leakage of about 13 cfs/ft before small slides and erosion began to progressively occur at the toe. Once this began, failure occurred within about 3.5 hours (Leps 1973). The core of Reclamation's Minidoka Dam overtopped during construction (1904 to 1906), and the downstream rockfill zone withstood flows estimated up to 1,000 cfs. The water surface elevation was 8 feet below the normal water surface when the core overtopped.

- **Sinkhole development**: This mechanism refers to stoping of material upward, creating a sinkhole or depression in the embankment that compromises the dam or lowers the crest below the reservoir level. For breach to occur, the sinkhole would need to be large enough to lead to overtopping. USACE's Wolf Creek Dam was constructed over karst features and has experienced numerous sinkholes. Due in part to concern that sinkholes may lead to potential breach, major mitigation measures have been completed.

- **Slope instability**: Internal erosion could cause high pore pressures in the foundation or embankment, resulting in reduced shear strength and slope failure. Breach could occur if the failure surface either intersects the reservoir, or the slope deformations are significant enough that the remnant can't resist the reservoir load. Although it is possible, this is generally not considered to be a very likely breach mechanism for most dams. No historical failures from slope instability due to increased pore pressures in the downstream slope are known to exist, and a unique set of circumstances would need to exist for it to be a major concern.

All four mechanisms lead to crest settlement and overtopping erosion. One or more of the mechanisms may occur during the breach process, and it is generally not necessary to know precisely which mechanism(s) would occur. However, risk estimates should typically be developed considering the most likely breach mechanism(s).

There are a few cases where once failure has initiated and progressed, and intervention has been unsuccessful, complete breach of the dam did not necessarily follow. Many Reclamation and USACE dams have large flood storage resulting in large normal freeboard. If the operative breach mechanism was stoping (forming a sinkhole near the crest) or progressive slumping and erosion at the toe of the dam during periods when the reservoir is low, the large freeboard may prevent failure by keeping the sinkhole above the reservoir surface, or by formation of a "berm" at the downstream slope from the slumped material that ultimately arrests breach development. In addition to large freeboard, other factors that have led to a reduced probability of complete breach include a concrete corewall to nearly full dam height (which is capable of retaining the reservoir even if a "pipe" or sinkhole develops). In the case of internal instability of core material, not only must the finer particles be washed through the coarser materials, but the remaining fraction must sustain enough flow such that it is also completely eroded. It is also possible that a small reservoir volume may empty through an opened seepage path before complete dam breach can occur. Breach mechanisms vary in their time to fully develop and catastrophically release the reservoir, and the intervention node should consider the potential time available based on the breach mechanism being considered.

Flood Considerations

Generally, internal erosion potential failure modes under normal operating conditions are of the greatest concern and pose the highest risks because the loading typically occurs every year. However, the same potential failure modes, and sometimes additional potential failure modes, need to be considered during flood loadings. During a large flood, the reservoir at a given dam may rise to unprecedented levels, increasing the driving head and seepage gradients. This would be analogous to a first-filling situation, and the conditional probability of internal erosion initiating (given the flood) could be significantly higher than under normal operating conditions, perhaps by an order of magnitude or more. On the other hand, the likelihood of a flood that would raise the reservoir to unprecedented levels could be quite remote. In general, for dams that are nearly full each year (i.e., many Reclamation projects), it has been found that the increase in initiation likelihood is less than the decrease in load probability for floods that would raise the reservoir to unprecedented levels, and internal erosion risks under flood loading are typically less than those under normal operating conditions. However, in some cases, where there is a significant flood control volume in the reservoir, this may not be the case, as the reservoir could rise significantly (perhaps up to double the normal pool depth) for relatively frequent floods (say less than a 100-year return period). In addition, when defects (such as cracks) are possible in the upper (and previously untested) portion of an embankment, there may be a much greater chance that internal erosion will initiate and progress to failure. For these conditions, typically the probability for nodes on the static internal erosion failure mode tree would be adjusted (which may include initiation, breach, and intervention) to account for a rise in the reservoir, and multiplied by the probability of the reservoir level for several flood loading increments.

At many embankment dams, the embankment core does not extend all the way to the crest of the dam, but stops a few feet short. During a flood, the reservoir may rise above the core and be retained by material not necessarily intended to hold back water. The question arises as to the likelihood of internal erosion through this material. If there is the potential for a crack to form in this upper part of the dam (which may be the case if the material contains significant fines, and particularly plastic fines), that must be taken into consideration. However, this material is generally coarser cohesionless soil, and it has been found that internal erosion through this type of material under a situation where water is stored against it during a flood is extremely unlikely. Typically, floods that would place the reservoir above the core but below the crest of the embankment would not wet the upstream face in this region long enough to develop flow all the way through the crest unless the material is permeable, in which case the soil particles would be larger and less likely to dislodge under the flow. In any case the water may not flow horizontally across the upper part of the dam to the downstream face, but rather flow downward into the shell on the downstream side of the core.

Accounting for Uncertainty

Given the difficulties in quantifying seepage and internal erosion behavior, there is a high degree of uncertainty in the estimates. Sensitivity analysis or other appropriate uncertainty analysis methods can be used to explicitly show how uncertainty influences the risk estimate. Reclamation and USACE utilize a suite of scalable assessment approaches that provide information to promote critical thinking and guide a risk analyst's judgment. For periodic assessment of risks (e.g., Comprehensive Review), simplified event trees are generally developed, and probabilities estimated directly for each branch of the event tree using judgment and subjective probability estimates (see Chapter I-6 Subjective Probability and Expert Elicitation) based on the available information for a particular dam. With a little more effort, uncertainty can be treated to a limited extent with sensitivity analysis by considering likely low, best, and high estimates for key variables. A range for annual probability of failure can be calculated using all of the likely low estimates and all of the likely high estimates. Appropriate weighting factors can also be assigned to the low, best, and high estimates to obtain the "best estimate" of annual probability of failure.

For Issue Evaluation (and other higher levels of risk assessment), the process is typically much more detailed and requires many steps of analysis. Uncertainty is accounted for in the calculations by assigning probability distribution functions for important variables in the risk analysis. Spreadsheet cells or event tree branches are described in terms of a probability distribution rather than a discrete value. Then, a Monte Carlo simulation is performed (typically with 10,000 iterations) to develop a probability distribution for the annual probability of failure and average annual life loss..

Relevant Case Histories

A summary overview of several key incidents is provided below, starting from early history to the present, illustrating that internal erosion can occur at virtually any time during the operational life of an embankment dam (Engemoen and Redlinger 2009).

Avalon Dam

Avalon Dam in New Mexico failed twice; once in 1893 from flood overtopping and later in 1904 from internal erosion. After the second failure, Avalon was taken over by Reclamation and reconstructed in 1907. Although this dam was not part of Reclamation's inventory when it failed, it was one of Reclamation's earliest dealings with an internal erosion incident. Avalon Dam was one of several dams built in the late 1800's or early 1900's that featured a rockfill downstream section which buttressed an upstream earthfill zone. It is notable that a number of failures or serious incidents occurred at other non-Reclamation dams having this similar configuration, including McMillan Dam, Black Rock Dam, and Fish Lake Dam. In all these cases, a seepage path existed through their earthfill zone that flowed down into underlying rockfill. The exact cause of failure of Avalon Dam is unclear, but explanations included piping of the embankment due to the severe incompatibility of the earthfill and rockfill from a filtering/retention perspective, or erosion at the base of the earthfill due to flows in the upper portion of the limestone foundation.

Fontenelle Dam

A very serious internal erosion incident occurred in 1965, when Fontenelle Dam nearly failed during first filling. Significant seepage traveled through the open jointed sandstone foundation rock, emanating 2,000 feet downstream in a low area as well as in the right abutment near the spillway. Seepage led to the erosion of more than 10,000 cubic yards of embankment materials before the intervention efforts of large outlet works releases and dumping of rockfill into the embankment erosion area eventually lessened the flows and the erosion. Fortunately, the large capacity outlet works was able to lower the reservoir by approximately 4 feet per day, quickly reducing the head at the abutment area where internal erosion was occurring. In less than 2 days of drawdown, the reservoir was lowered off of the spillway approach channel which undoubtedly was feeding seepage into the problem area. The primary cause of the near failure was thought to be inadequate grouting of the jointed sandstone and the lack of foundation treatment measures such as slush grouting and dental concrete, which led to seepage near the base of the dam that removed embankment material and led to the growth of voids and stoping. Contributing factors included the presence of infilling or soluble material in the jointed rock that may have inhibited grout travel; residual or redeposited soluble salts in the rock that may have reacted with the grout causing premature set or ultimate softening; the erodible nature of the embankment core material; and a steep right abutment that created difficulties in achieving good bond or contact between the embankment and abutment, encouraged differential settlement and cracking of the embankment, and made shallow grouting difficult because low pressures were required to prevent movement of the rock.

Another factor not mentioned in early reports was the unfavorable orientation of the abutment with respect to the potential for hydraulic fracturing. In hindsight, an obvious key factor in the near failure, in addition to the lack of sufficient foundation treatment, was the lack of an internal filter and drainage zone that would render seepage through both the foundation and embankment harmless with respect to the removal of soil particles and the buildup of pore pressures. A couple of key details are that the average zone 1 core material in the dam is reported as being a SC and CL with 13 percent plus No. 4 material and having a LL of 31 and a PI of 13. However, the core material remaining after the near breach in the abutment area was generally described as a well graded mixture of sandy gravel and silt. No crack in the core was noticed during close inspection of the piping channel through the zone 1. Zone 2 materials described as select sand, gravel and cobbles as well as the materials in the miscellaneous zone sloughed during this incident and an incident that occurred four months prior and were easily removed by the concentrated seepage.

Teton Dam

Teton Dam failed from internal erosion during first filling in 1976, marking the first and only failure of a Reclamation embankment dam. The failure was similar to the incident at Fontenelle Dam 11 years earlier, with excessive seepage through a highly jointed foundation rock leading to erosion of a highly erodible core material during initial reservoir filling. Contributing factors included a low permeability transition zone that contained too many fines to act as a drain for the core or serve as a filter, the lack of foundation filters on the downstream face of the cutoff/key trenches, insufficient treatment of the open joints in the rock foundation, the presence of a highly erodible core material, the rapid rate of initial reservoir filling, and an inoperable outlet works. Reports were prepared by an independent panel and a Government panel assembled to review the cause of failure.

There have been a number of reasons given as to how a defect in the core materials deep within the right abutment cutoff trench came about; some are as simple as resulting from the fill becoming frozen during winter shut down to more complex theories related to low stress zones and hydraulic fracturing caused by arching of the dam over the steep narrow cutoff trench. However, it is critical to recognize that the joints, fractures and openings in the downstream wall of the cutoff trench and the remaining foundation downstream of the trench were severely incompatible with respect to filtering and retention of the very fine grained, erodible core materials, as well as the silt infillings in some of the joints themselves. It would have been virtually impossible to construct a perfect core without defects to overcome these conditions, and the focus should have been on proper foundation treatment and filter protection for the core and the silt infillings.

Caldwell Outlet Works at Deer Flat Dams

The Caldwell Canal outlet works, with a capacity of 70 cfs, is a cut-and-cover conduit located through the left abutment section of the Upper Embankment at Deer Flat Dams in Idaho, and was completed in 1908. The foundation materials in the vicinity of the Caldwell conduit consist of mostly poorly graded sand and silty sand with some gravel. Caliche layers exist in some areas of the dam’s foundation as well. A dam safety inspection in 2001 (93 years after construction) noted some sediments in the seepage from a crack in the conduit located 65 feet upstream of the outlet portal. A large sand deposit approximately 6 feet wide by 15 feet long and 10 to 12 inches deep was observed downstream of the outlet structure. Although speculated to be windblown materials, it was also judged possible that the observed sediments could have been materials transported into the conduit by seepage flows. Then, in 2004 sediment was observed at the base of a crack in the conduit approximately 125 feet downstream of the regulating gate. Subsequently, ground penetrating radar was utilized in the conduit, and potential anomalies were detected between 100 and 150 feet downstream of the gate. Follow-up drilling through the conduit revealed voids beneath the conduit varying from ½-inch to 5 inches in depth, presumably caused by internal erosion of foundation soils into or along the conduit. Piezometers installed below the conduit revealed consistently low pressures similar to tailwater levels beneath the conduit from the downstream portal upstream to within 20 feet of the intake. It was concluded that backwards erosion piping had occurred along most of the conduit, with potentially high gradients existing at the upper end of the conduit. A large upstream berm was constructed to minimize the potential for upstream breakout of the piping pathway to the reservoir, until permanent corrective actions could be taken.

In the case of Upper Deer Flat Dam, even though in general there are gravels present in the embankment fill as well as the foundation, gravel sized particles were found to be absent over a large extent of the conduit foundation during the re-construction. Even if coarser particles were present in the soil mass, the mechanism of a soil filtering against a crack in the bottom of a conduit can be complicated by the fact that a flow path beneath the structure will not necessarily transport coarser particles up into or against the crack in the bottom of the conduit. Any particles transported to such a crack may drop away during times of lower gradients such as under lower reservoir operating conditions. Therefore, caution against the use of liberal filter/retention criteria in such a case is advised.

A.V. Watkins Dam

A.V. Watkins Dam (formerly known as Willard Dam) is a U-shaped (in plan view) zoned earthfill structure constructed within Willard Bay of the Great Salt Lake. Constructed from 1957 to 1964, the dam is 36 feet high at its maximum section and slightly more than 14.5 miles long. Upon first filling of the reservoir in 1965, as the reservoir reached within approximately 2 feet from full, numerous wet areas (with some areas displaying quick conditions) appeared at the downstream toe of the dam. After this discovery, filling of the reservoir was halted, the reservoir was lowered and a toe drain was constructed approximately 15 feet from the downstream toe from 4 to 5 feet deep in the foundation, consisting of 8-inch diameter bell and spigot concrete pipe with open joints and surrounded by gravel. Toe drain outfalls were constructed at approximate 1,000-foot intervals to discharge into the South Drain; a long open ditch excavated about 130 feet downstream of the dam toe to help drain farm land as well as seepage. The toe drain was apparently successful in drying up the downstream toe area and the reservoir was eventually filled to the top of active conservation water surface.

In November of 2006, A.V. Watkins Dam nearly failed at a location in the same general area that created problems during initial filling, as the result of piping and internal erosion of the foundation soils. Two days previously, a local cattle rancher working just downstream of the incident area noticed seepage and some silty material exiting from the cut slope of the South Drain. The rancher continued to observe the seepage and erosion into the South Drain until Monday, November 13, when he became concerned over the increase in seepage and the appearance of what he described as “dark clay” exiting into the South Drain. He called authorities and Reclamation began 24-hour monitoring and initiation of an emergency drawdown of the reservoir. Piping of the foundation soils was occurring from beneath the dam below a somewhat continuous downstream, but absent upstream, series of thin hardpan layers, and the fine-grained, silty sand soils were exiting from the dam’s downstream toe and from the base of the north slope of the South Drain. Approximately 140-190 gallons per minute of seepage was exiting from sand boils at the downstream toe of the embankment (but upstream of the toe drain), flowing across the ground surface and into sinkholes between the toe of the embankment and the South Drain. The seepage appeared to be re-emerging at the base of the bank of the South Drain and was depositing large amounts of sand into the South Drain. Figure IV-4-25 depicts the conditions.

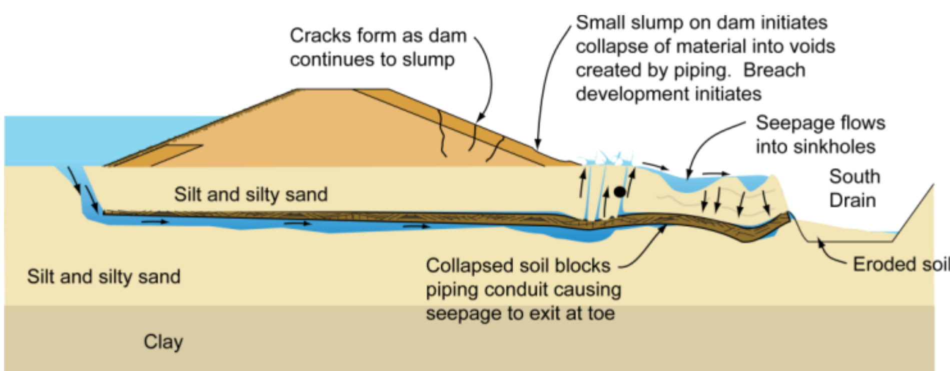


Figure IV-4-25. Failure Mode In-Progress (not to scale)

Efforts to save the dam focused immediately on transportation of filter sand and gravel materials to the site to begin placement of these soils directly over the sand boils in an attempt to stop the erosion of foundation soils. Initially filter sand was placed over the sand boils but that was quickly washed away due to the high exit velocities. Gravels were then placed over the sand boils until the exit velocities were reduced enough to allow placement of the filter sand. This reduced the flow and erosion of soil enough to allow the placement of a sufficiently large berm consisting of additional filter material, drainage material, and minus 5-inch pit-run material, to counter the uplift pressures in the emerging seepage at the toe.

On November 16, Reclamation technical staff determined the failure mode was still in progress and additional remedial action was required. It was noticed that seepage and erosion was still occurring into the South Drain. On November 17 and 18, a berm was added to the upstream slope of the embankment extending into the reservoir to stem the flow of the water into the foundation and any inlets to potential piping channels (located just beyond the upstream dam toe and within the riprap) that were postulated to be the sources for the concentrated seepage entering the foundation. These efforts were successful in stopping the foundation erosion and immediately reducing the overall seepage flows. Some key lessons learned at this dam to consider in future dam designs and risk analysis are:

- Internal erosion can initiate, progress and nearly fail a dam with an erodible foundation at very low head to seepage length ratios, in this case generally about 0.09 (locally may have been about 0.06 due to rodent holes), if the exit point for the unfiltered seepage is nearly horizontal, the soil is highly erodible, and a roof is present.

- Rodent activity can suddenly aggravate a meta-stable seepage situation, as rodents can fairly quickly excavate a burrow and shorten a potential seepage path, compared to the more gradual particle transport caused by seepage at these low gradients.

- Construction of open trenches downstream of the toe of a dam provides a location into which materials can be eroded.

- Toe drains installed as the primary defense against foundation internal erosion, especially when the drain is installed after an occurrence of piping was observed, can be critical to the performance of the structure. Plugging of the toe drains appeared to have been occurring at this site. It is not clear that the toe drain plugging was a significant contributor to the occurrence of the incident.

- Changes to seepage conditions that occur over a long period of time can be difficult to recognize and the knowledge about the presence of buried drains can be lost. Consideration should be given to estimating risks for (or at least considering as a potential failure mode) every location that seepage or wet spots are known to exist at a dam, as well as those areas typically analyzed (see also Bliss and Dinneen 2007).

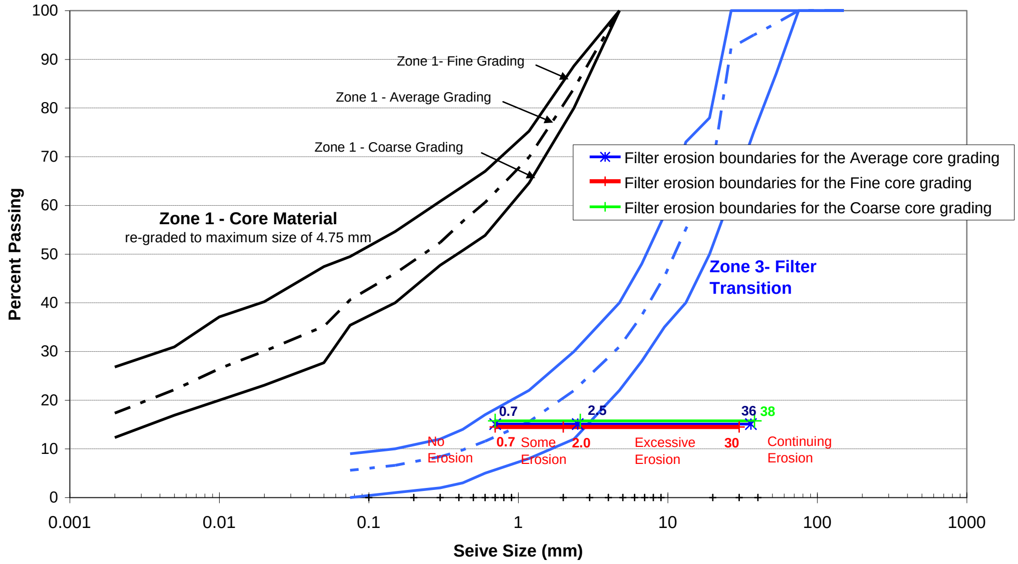
Stilling Basin at Davis Creek Dam

Davis Creek Dam is a modern embankment dam in Nebraska, completed by Reclamation in 1990. A sinkhole was reported adjacent to the outlet works on May 11, 2007. The sinkhole was located against the left side of the outlet works control house immediately upstream of the stilling basin, and measured approximately 5 feet along the wall, 2 feet wide away from the wall and about 6 feet deep. The sinkhole was located in the structural backfill composed of fine to medium sands. The perimeter of the sinkhole was probed with a steel rod, which could be inserted with ease vertically along the wall in the sand to a depth of about 10 feet below the bottom of the sinkhole or about 16 feet below the original ground surface. Subsequent video inspections of the spillway underdrain system found sand in the drain pipes. Due to a defect in the underdrain system, whether from broken pipe or inadequately constructed filters, structural backfill, filter sand, and possibly foundation sands were being internally eroded into the underdrain system and then removed downstream by the action of the drains during outlet works operation. A grouting operation was subsequently undertaken, and it took more than 20 cubic yards of grout to fill voids beneath the stilling basin and surrounding areas. The precise location and lateral extent of the void system could not be defined, and it is uncertain if the erosion had progressed upstream along the outlet works beyond the limit of the upstream edge of the sinkhole. A filtered drainage system was also constructed around the sides of the basin to encourage drainage and thus reduce uplift pressures.

The underdrains were installed to assist in preventing floatation of the stilling basin structure both during dewatering of the stilling basin and during operations should the hydraulic jump move downstream. They were constructed such that during certain operating conditions outlet works discharges running by the drain outlets created low pressures thus resulting in drain flow and lowered uplift pressures. Vents were installed to ensure negative pressures did not develop. This fairly sophisticated drain system, if damaged, can apparently be very efficient in causing particle transport from the foundation. Since the operations of the outlet are intermittent, removal of soil would be intermittent and could occur over a long period of time. The typical winter seepage regime could have primed the system with water and soil particles and then the underdrains could have nearly instantaneously removed the water and some soil from beneath the structure each year under certain operating conditions. Hydraulic connection of the stilling basin to the groundwater was potentially causing very severe transient seepage conditions and particle transport.

Exercise

Given the gradation curves shown in the following figure, estimate the probability of no erosion, some erosion, excessive erosion, and continuing erosion for the fine, average, and coarse Zone 1 base soil gradations. Assume the representative gradations of the re-graded base soil corresponds to 90 percent all gradation tests.



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

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

Figure IV-4-26. Example Exercise

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Appendix IV-4-A: Large Dam Failure Statistics

Table IV-4-A-1. Overall Statistics of Embankment Dam Failures (adapted from Fell et al. 1998, 2000)

No. of Cases		% of Failures (if Known)		Average Probability of Failure	
All Failures	Failures in Operation	All Failures	Failures in Operation	All Failures	Failures in Operation
Internal Erosion through the Embankment					
39	38	30	33	3.5E-03	3.5E-03
Internal Erosion through the Foundation					
19	18	15	15	1.5E-03	1.5E-03
Internal Erosion from the Embankment into the Foundation					
2	2	1.5	1.5	2.0E-04	2.0E-04

Table IV-4-A-2. Historical Frequencies of Failures and Accidents (adapted from Fell et al. 1998, 2000)

Case	Total	In Embankment	Around Conduits and adjacent to Walls
Internal erosion failures	36	19	17
Internal erosion accidents	75	52	23
Seepage accidents with no detected erosion	36	30	6
Total number of failures and accidents	146	101	46
Population of dams	11,192	11,192	5,596
Historical frequency for failures and accidents	0.013	0.009	0.0082
Proportion of failures and accidents on first-filling	36%		
Proportion of failures and accidents after first-filling	64%		
Historical frequency for first-filling		0.0032	0.0030
Historical frequency after first-filling		0.0058	0.0052
Historical annual frequency after first-filling		2.2E-04	2.0E-04

Table IV-4-A-3. Time of Incident for Internal Erosion through the Embankment (adapted from Fell et al. 1998, 2000)

Time of Incident	No. of Cases		% of Cases (if Known)	
	Failures	Accidents	Failures	Accidents
During construction	1	0	2	0
During first-filling	24	26	48	26
After first-filling and during first 5 years	7	13	14	13
After first 5 years	18	60	36	61
Unknown	1	3	–	–
Total	51	102	100	100

Table IV-4-A-4. Time of Incident for Internal Erosion through the Foundation (adapted from Fell et al. 1998, 2000)

Time of Incident	No. of Cases		% of Cases (if Known)	
	Failures	Accidents	Failures	Accidents
During construction	1	0	5	0
During first-filling	4	23	20	30
After first-filling and during first 5 years	10	19	50	24
After first 5 years	5	36	25	46
Unknown	1	7	–	–
Total	21	85	100	100

Table IV-4-A-5. Incidents of Cracking and Hydraulic Fracturing (adapted from Fell et al. 2008)

Cracking Mechanism	No. of Cases	Percentage
Differential settlement, cross-valley	20	35
Differential settlement, cross-section	6	11
Differential settlement, foundation	4	7
Differential settlement, embankment staging	0	0
Desiccation cracking	3	5
Closure section	3	5
Total	36	63

Table IV-4-A-6. Incidents of Poorly Compacted and High Permeability Zones (adapted from Fell et al. 2008)

Location	No. of Cases	Percentage
At the foundation-embankment interface	5	9
In the embankment	16	28
Total	21	37

Appendix IV-4-B: Concentrated Leak Erosion

Common Situations where Concentrated Leaks May Occur

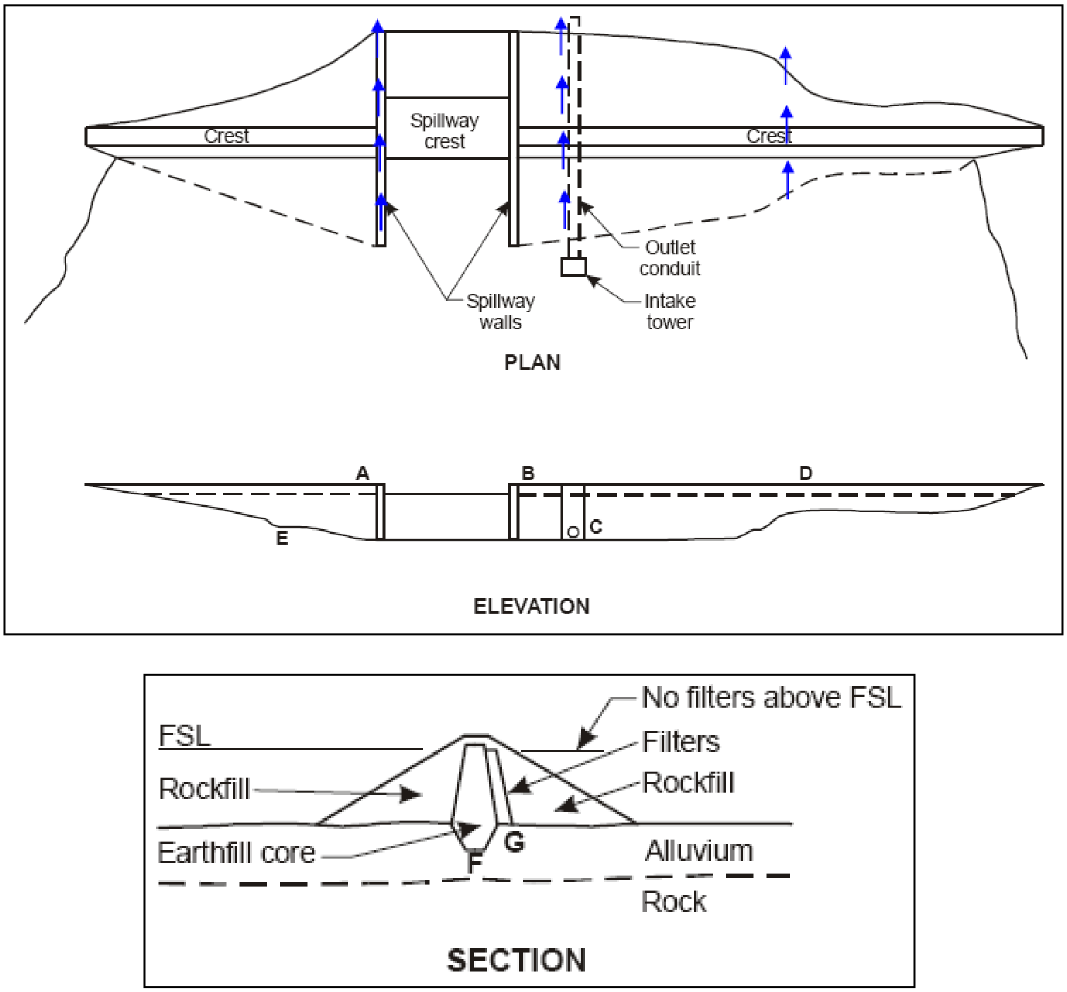

Figure IV-4-B-1. Potential Failure Paths
(Fell et al. 2008)

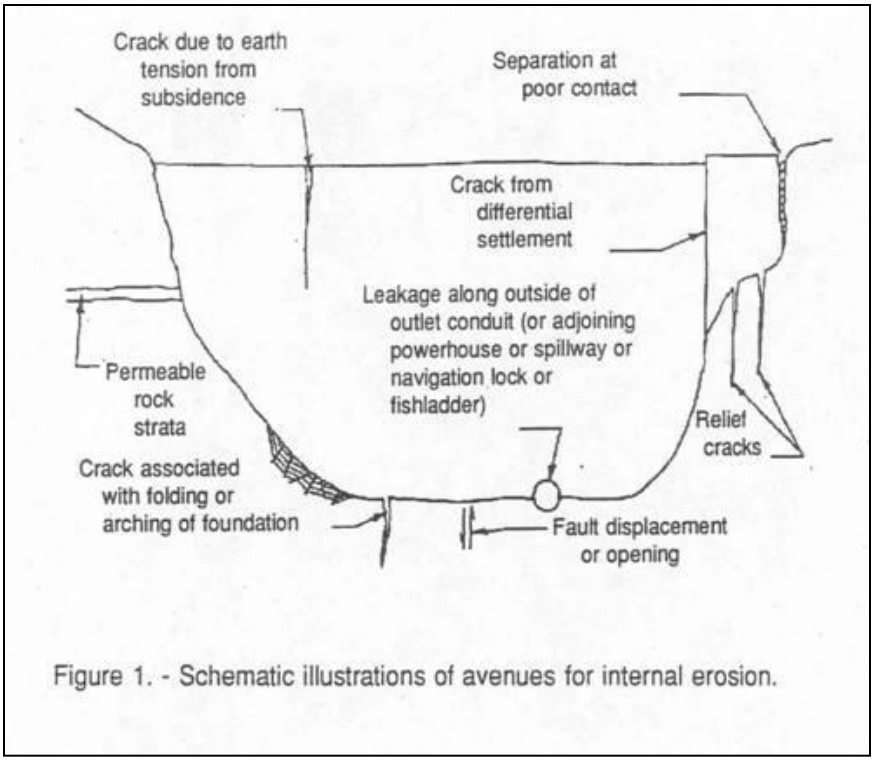

Figure IV-4-B-2. Potential Failure Paths
(Source?)

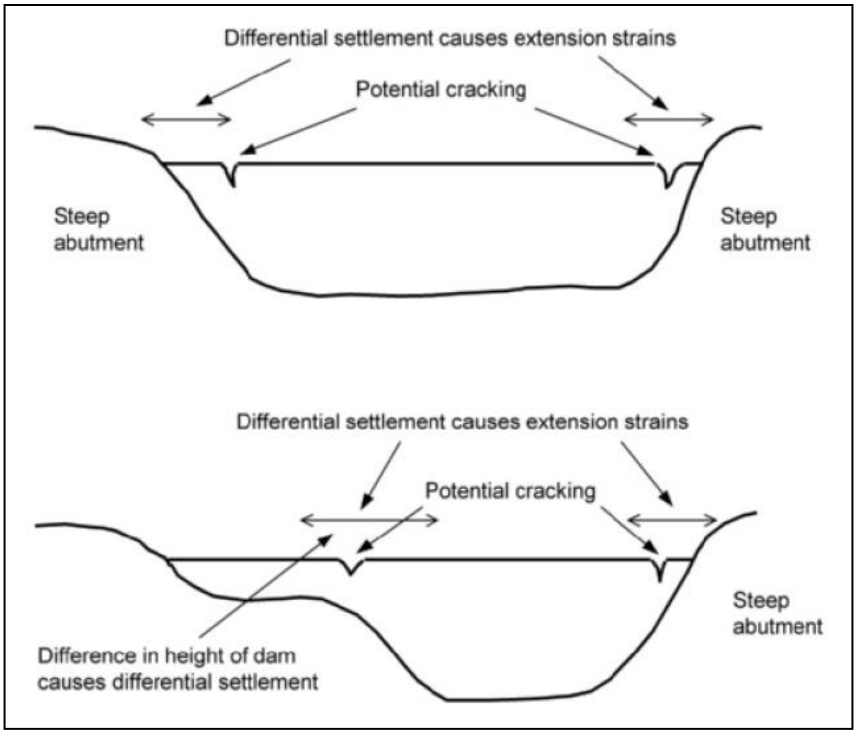

Figure IV-4-B-3. Cracking and Hydraulic Fracture due to Cross-Valley Differential Settlement of the Core (Fell et al. 2014)

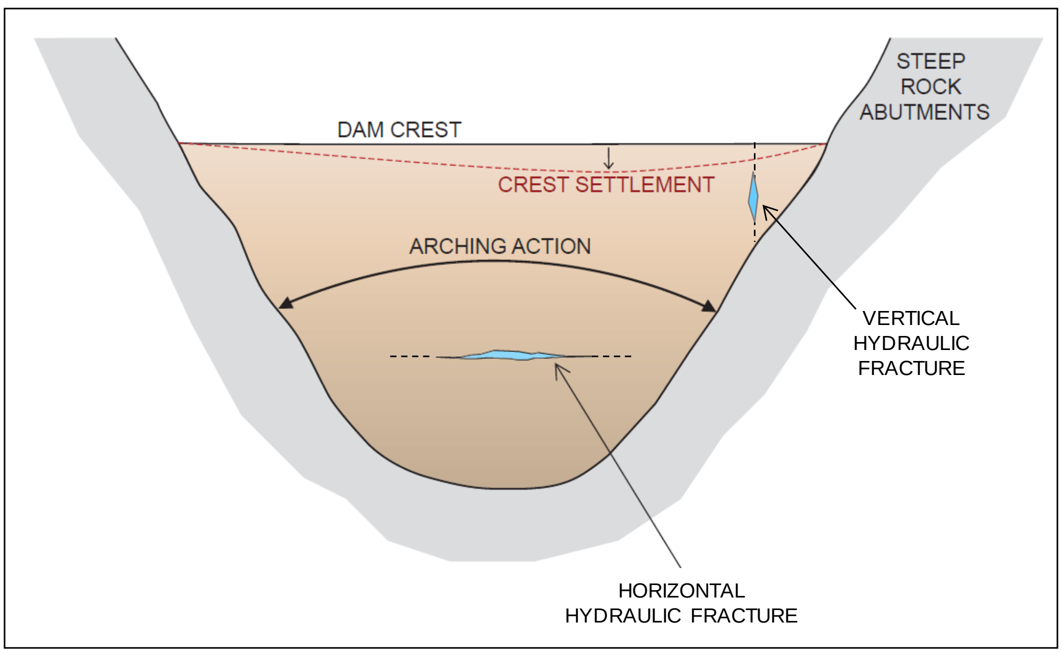

Figure IV-4-B-4. Cracking and Hydraulic Fracture due to Cross-Valley Arching and Steep Abutment Slopes (Courtesy of Mark Foster)

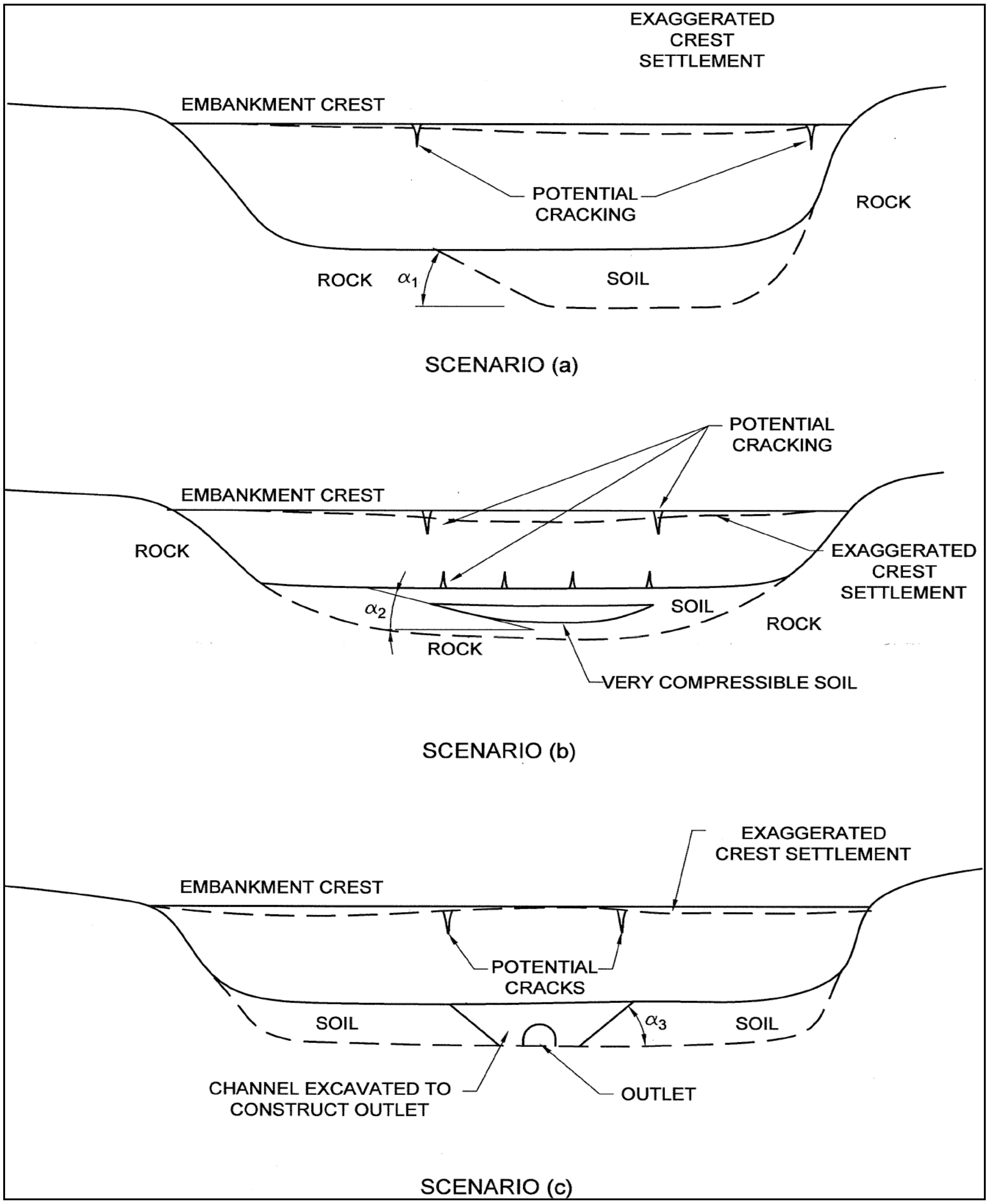

Figure IV-4-B-5. Cracking and Hydraulic Fracture due to Differential Settlement in the Foundation (Fell et al. 2008)

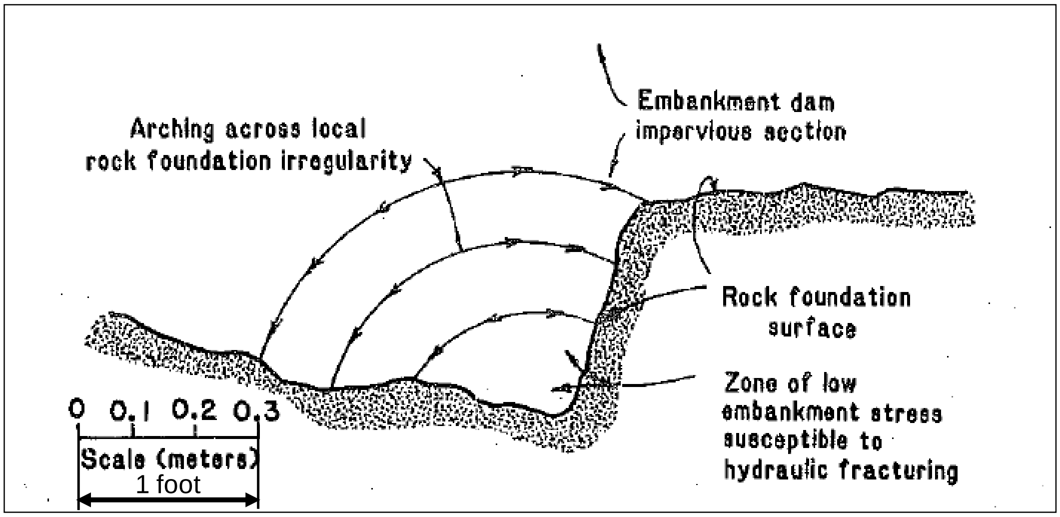

Figure IV-4-B-6. Cracking and Hydraulic Fracture due to Small-Scale Irregularities in the Foundation Profile (Sherard 1985b)

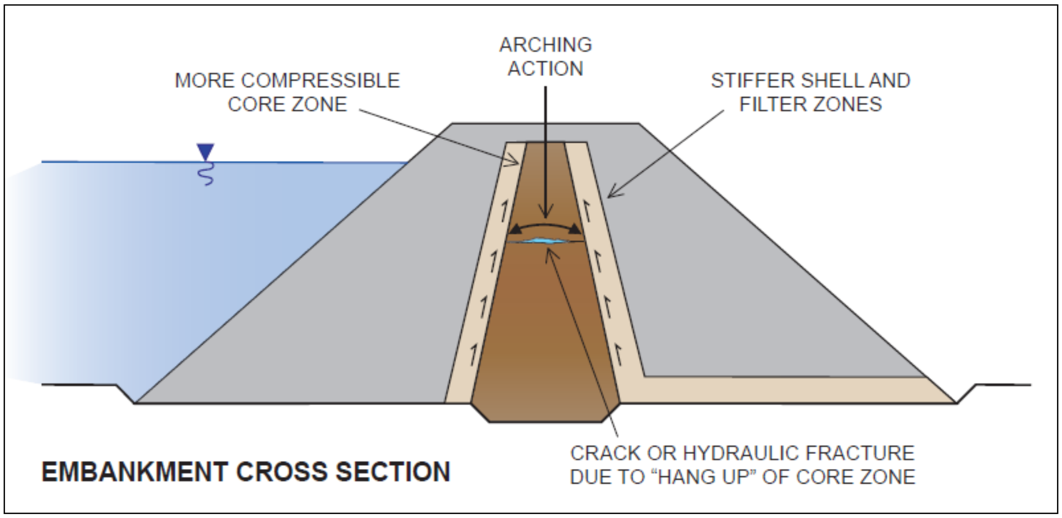

Figure IV-4-B-7. Cracking and Hydraulic Fracture due to Arching of Core onto Embankment Shells (Courtesy of Mark Foster)

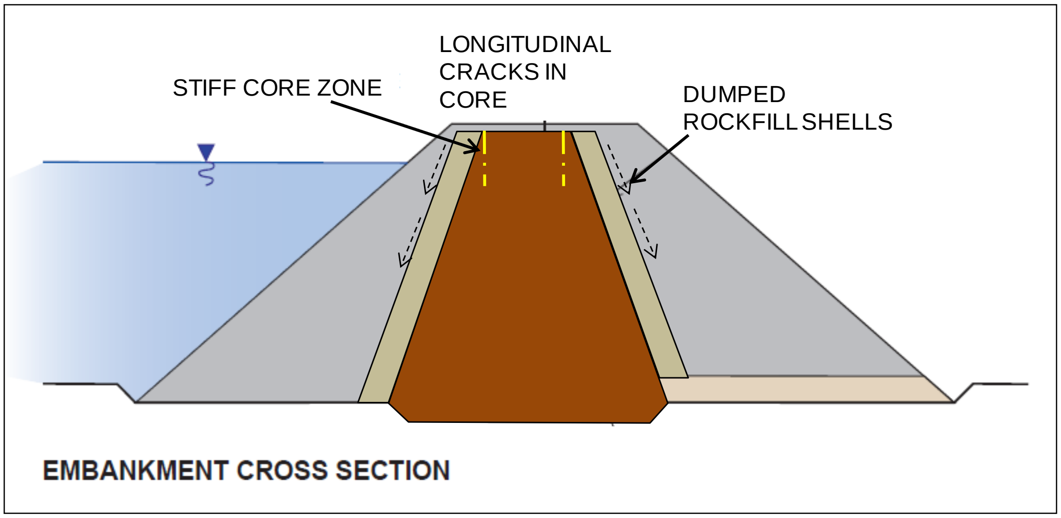

Figure IV-4-B-8. Cracking due to Cross-Sectional Settlement (Differential Settlement of Shell Zones) (Courtesy of Mark Foster)

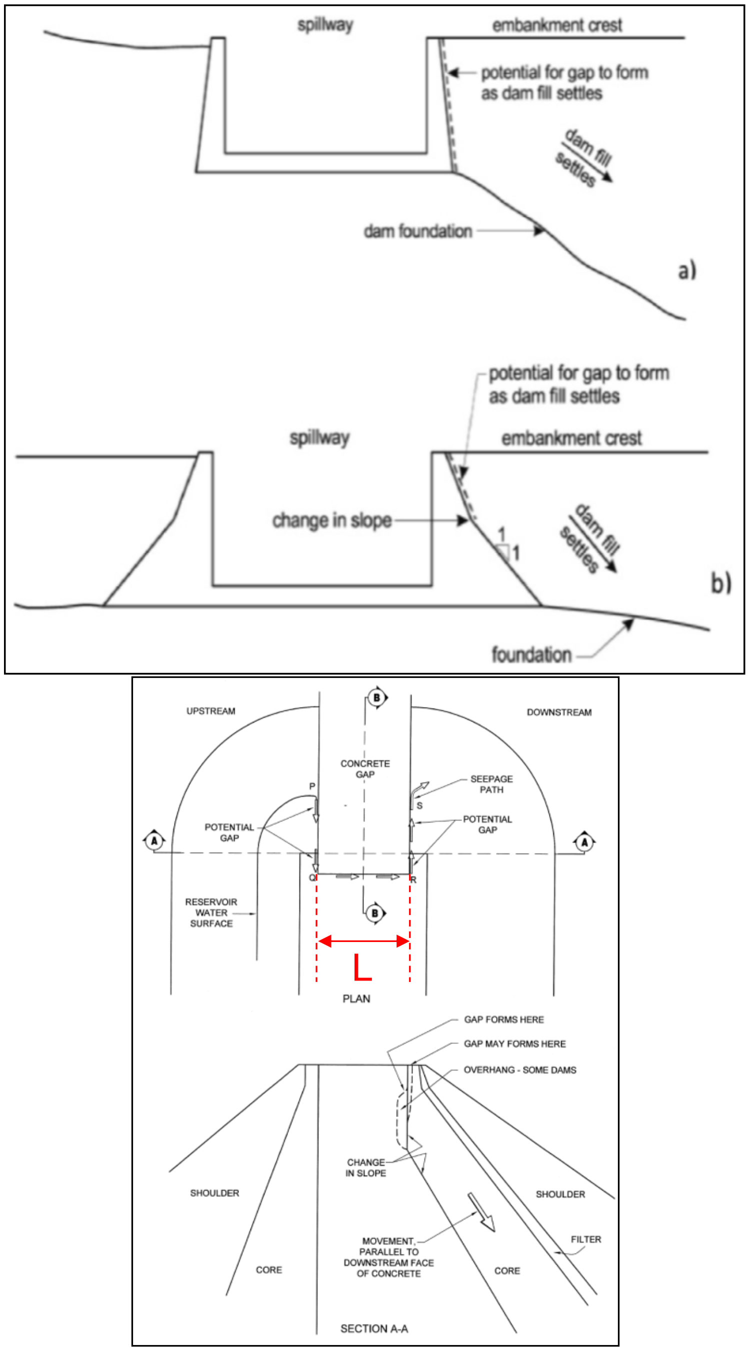

Figure IV-4-B-9. Crack or Gap adjacent to Spillway or Abutment Walls and Embankment-Concrete Interfaces (Fell et al. 2008)

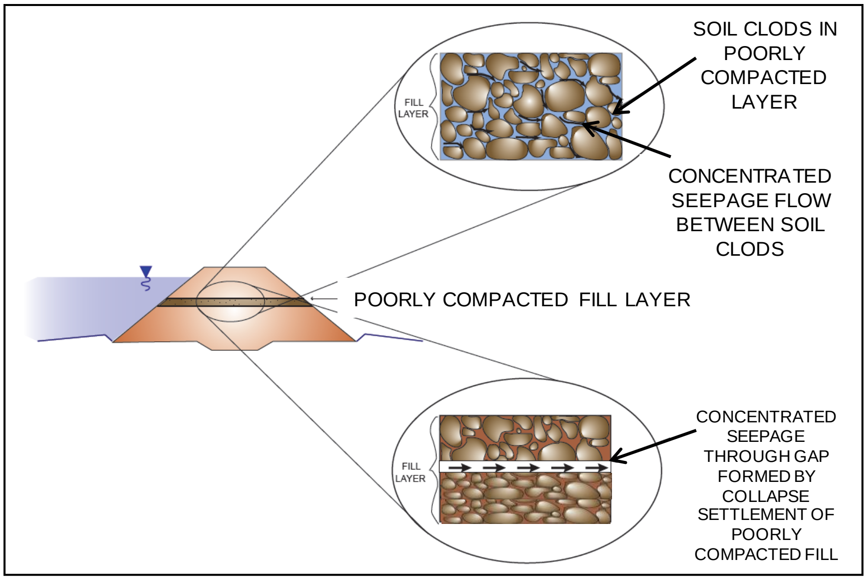

Figure IV-4-B-10. Crack, Hydraulic Fracture, or Openings in Poorly Compacted and/or Segregated Layers (Courtesy of Mark Foster)

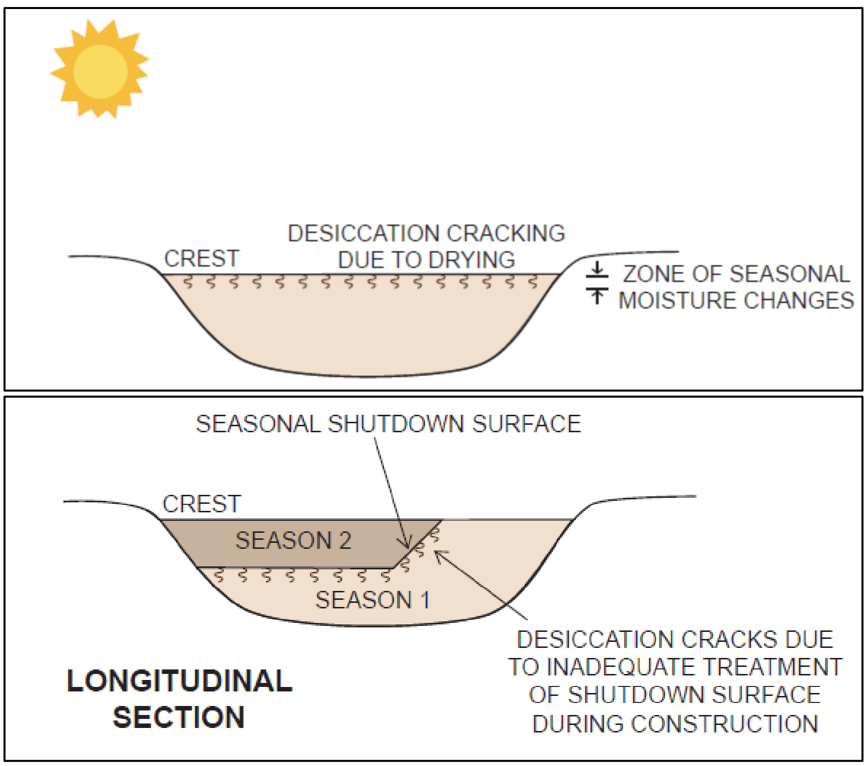

Figure IV-4-B-11. Cracking due to Desiccation (Courtesy of Mark Foster)

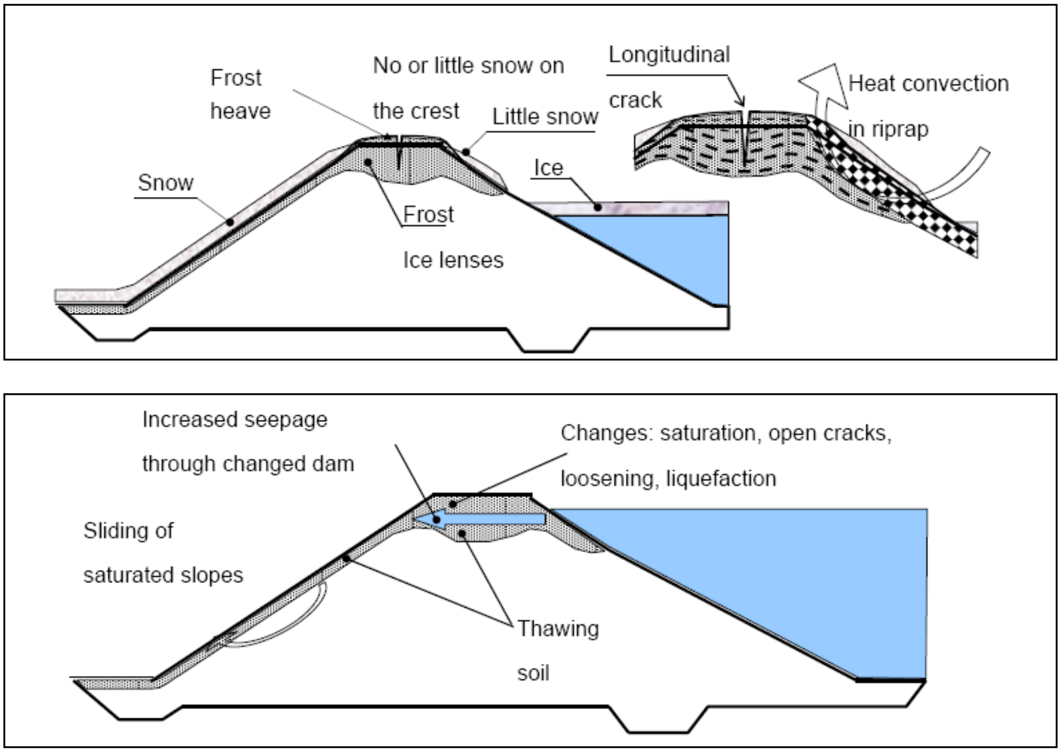

Figure IV-4-B-12. Cracking or High-Permeability Layers due to Freezing (Vuola et al. 2007)

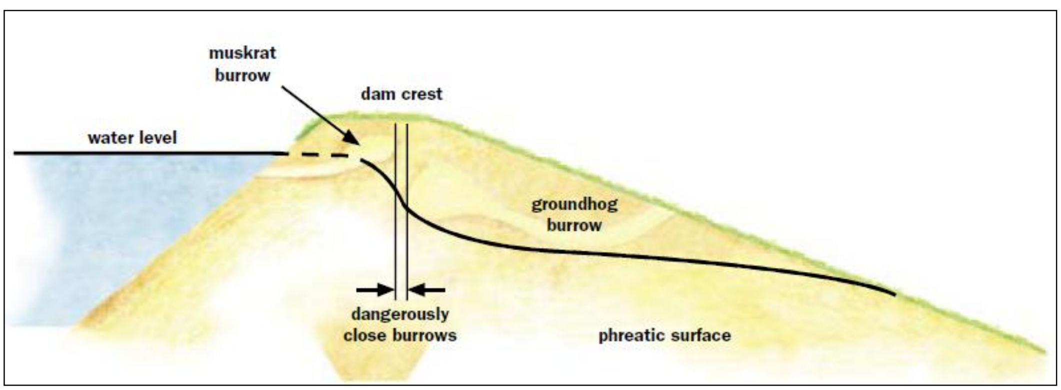

Figure IV-4-B-13. Effects of Animal Burrows (FEMA 2005)

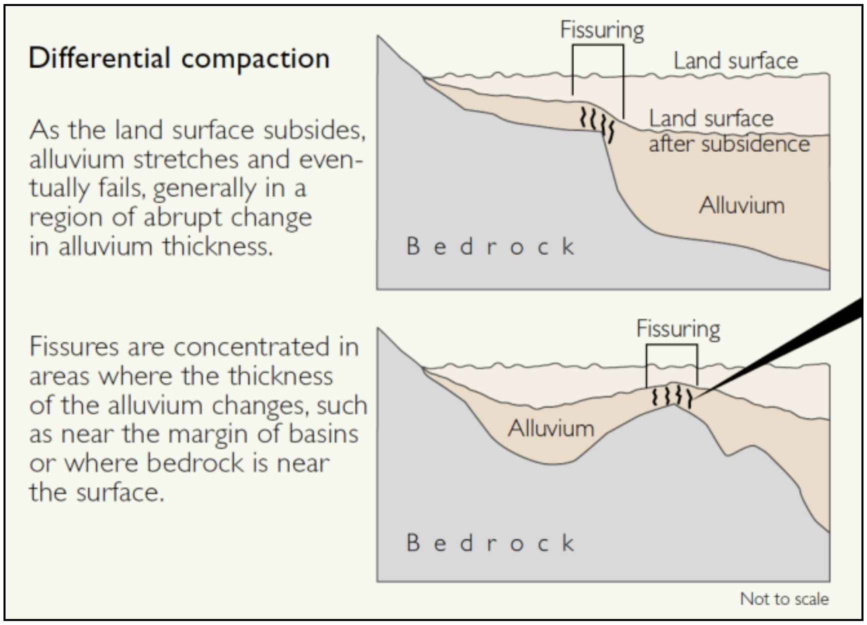

Figure IV-4-B-14. Cracking caused by Earth Fissures due to Subsidence (Galloway et al. 1999)

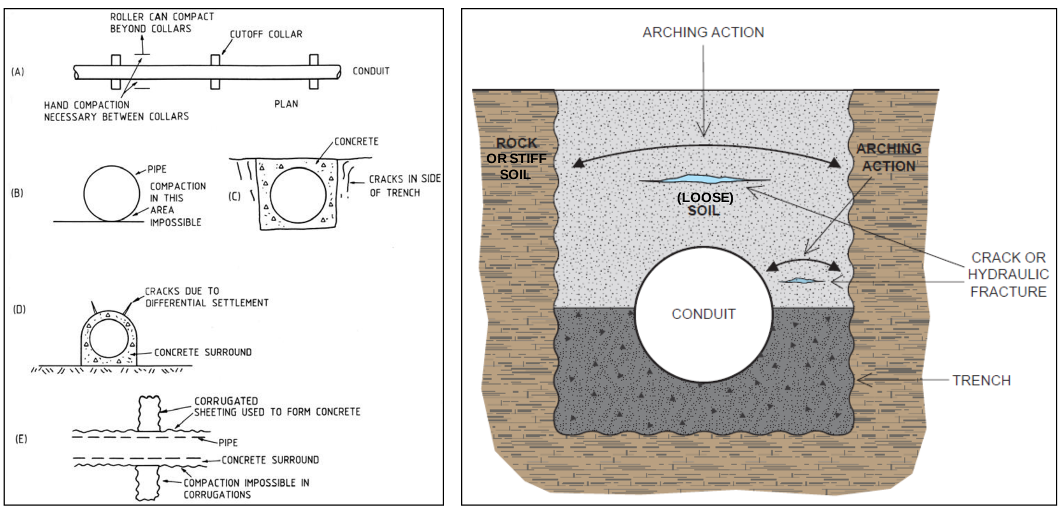

Figure IV-4-B-15. Some Causes of Piping Failures around Conduits (Fell et al. 2008)

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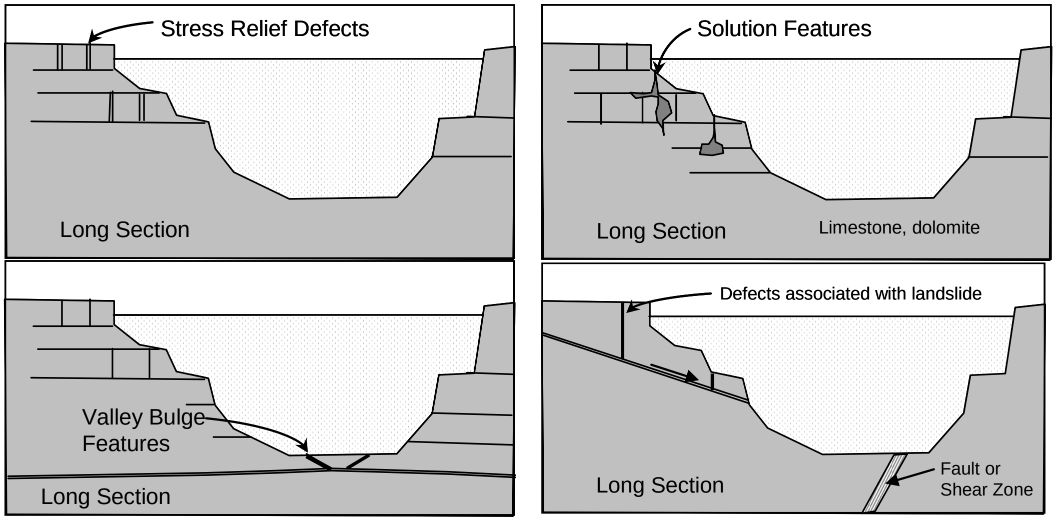


Figure IV-4-B-16. Defects in Rock Foundations due to Geologic Processes (Fell et al. 2008)

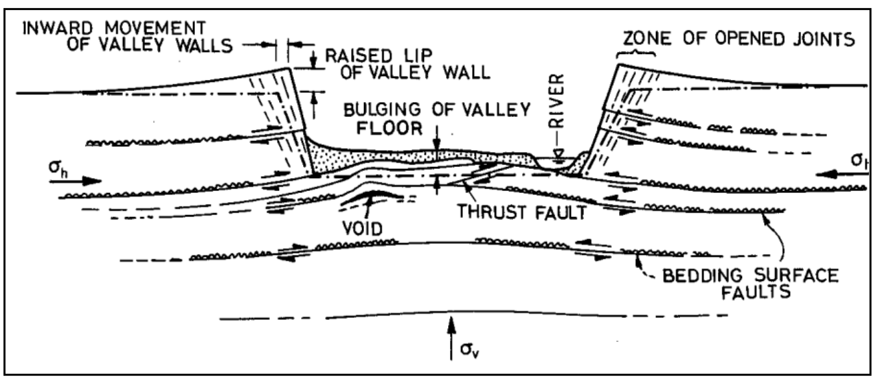


Figure IV-4-B-17. Valley Rebound and Stress Relief Effects in Valleys in Sedimentary and Other Horizontally Bedded Rocks (Fell et al. 2008)

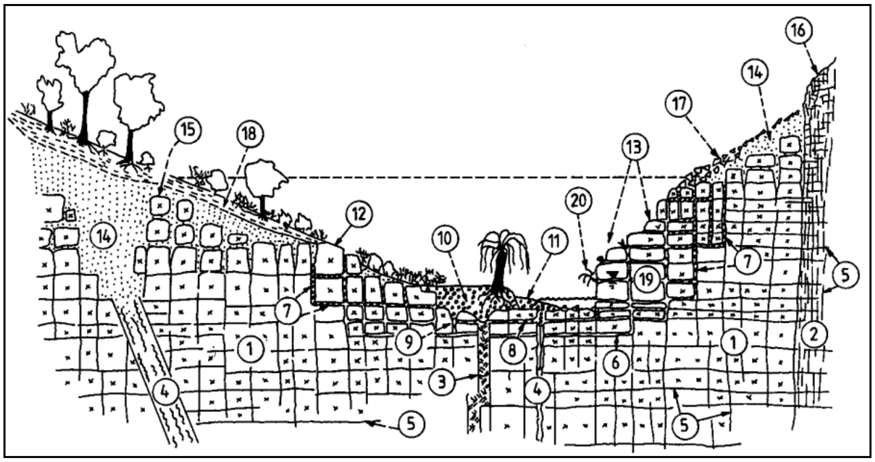


Figure IV-4-B-18. Features in Valleys Formed in Strong Jointed Rocks (Fell et al. 2008)

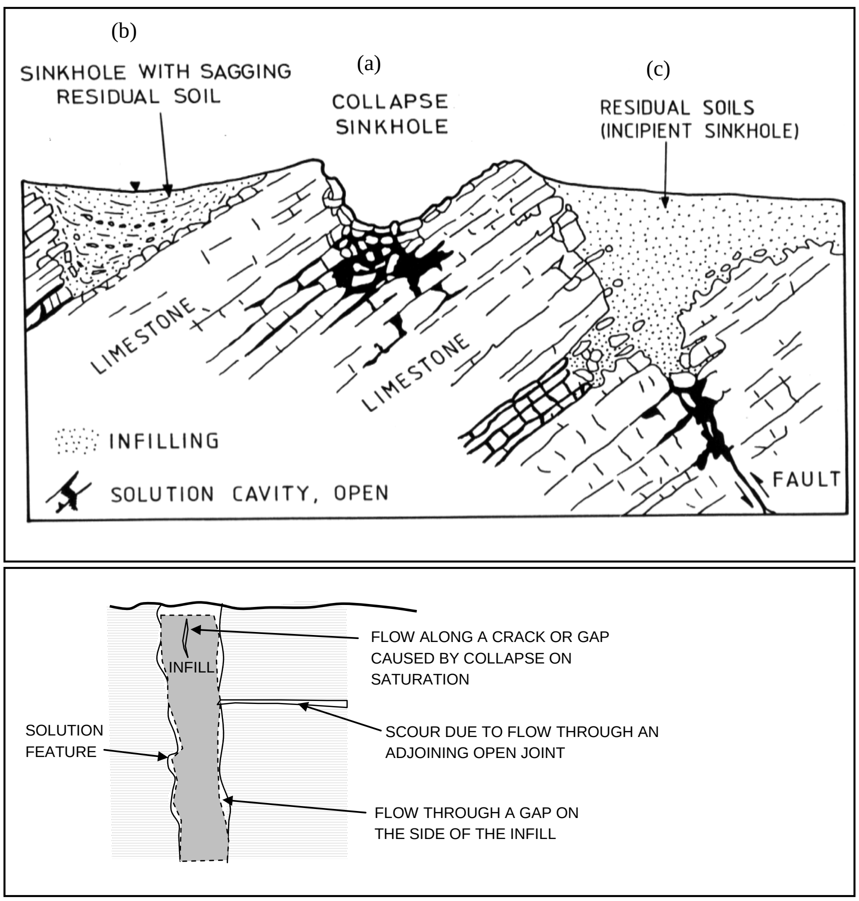


Figure IV-4-B-19. Defects in Rock Foundations (Fell et al. 2008)

Hydraulic Shear Stress

The hydraulic shear stress in a crack or pipe for the reservoir level under consideration is based on the geometry of the embankment core, the assumed pipe or crack dimensions, and the location of the pipe or crack relative to the reservoir level so that the flow gradient and velocity can be determined. According to Wan and Fell (2004b), the hydraulic shear stress can be estimated by the following equation:

$$\tau = \rho_w \, g \, (\Delta H/L) \, A / P_w$$

where ρ_w = density of water; g = acceleration due to gravity; ΔH = hydraulic head difference; L = length of pipe or crack over which the hydraulic head difference occurs; A = cross-sectional area of pipe or crack; and P_w = wetted perimeter of pipe or crack. Since the unit weight of water, $\gamma_w = \rho_w g$ and the hydraulic gradient, $i = \Delta H/L$, then the expression can be simplified to the following:

$$\tau = \gamma_w \, i \, A / P_w$$

Using this basic equation and the estimated geometry of the pipe or crack, the following approximations can be derived for the hydraulic shear stress.

The assumptions for the estimation of the hydraulic shear stress are:

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

Cylindrical Pipe

For cylindrical pipe, the equation for hydraulic shear stress is the following:

$$\tau = \frac{\rho_w g\ (H/L)\left(\pi D^2/4\right)}{\pi D}$$

$$\tau = \frac{\rho_w g\ HD}{4L}$$

where τ = hydraulic shear stress; ρ_w = density of water; g = acceleration due to gravity H = hydraulic head at upstream end; L = length of pipe at mid-depth of reservoir level under consideration; and D = diameter of pipe.

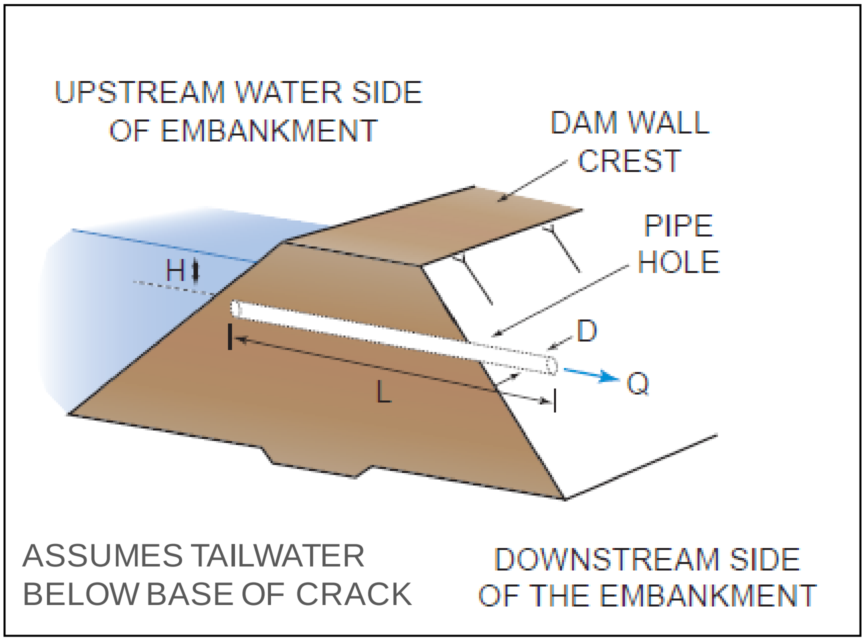


Figure IV-4-B-20. Cylindrical Pipe Geometry
(Fell et al. 2014)

Since $\gamma_w = \rho_w g$ and i = H/L, then

$$\tau = \gamma_w\ i\ (D/4)$$

Vertical Parallel-Sided Transverse Crack

For vertical parallel-sided transverse crack, the equation for hydraulic shear stress is the following:

$$\tau = \frac{\rho_w g\ (H/L)(HW)}{2(H+W)}$$

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

where τ = hydraulic shear stress; ρ_w = density of water; g = acceleration due to gravity H = hydraulic head at upstream end; L = length of crack at mid-depth of reservoir level under consideration; and W = width of crack.

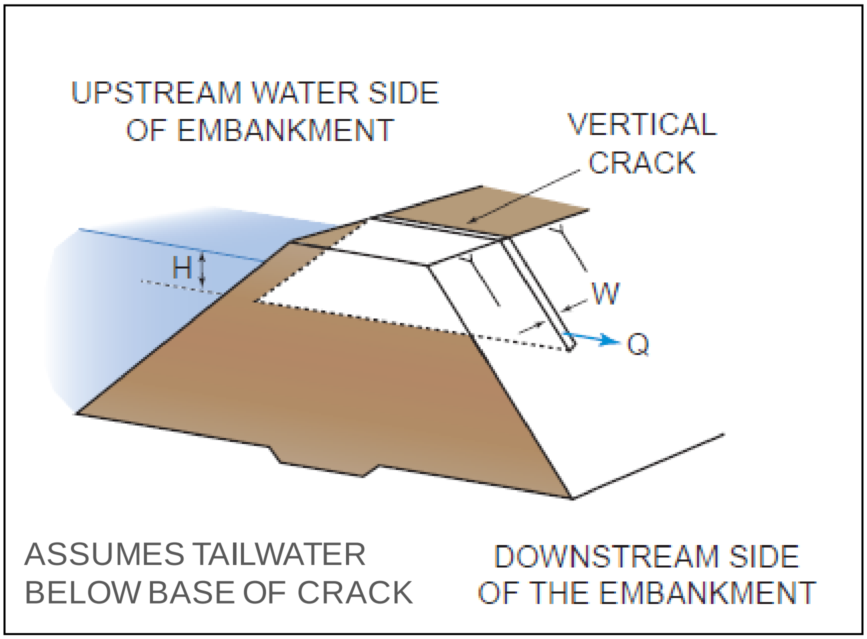


Figure IV-4-B-21. Vertical Parallel-Sided Transverse Crack Geometry (Fell et al. 2014)

Since H + W ≈ H (because H >> W), $\gamma_w = \rho_w g$ and I = H/L, then the equation can be simplified to the following:

$$\tau \approx \gamma_w\ i\ (W/2)$$

Vertical Uniformly Tapered Transverse Crack

For vertical uniformly tapered transverse crack, the equation for hydraulic shear stress is the following:

$$\tau = \frac{\rho_w g\,(H/L)(HW_H/2)}{2\left(H^2 + W_H^2/4\right)^{0.5} + W_H}$$

where τ = hydraulic shear stress; ρ_w = density of water; g = acceleration due to gravity H = hydraulic head at upstream end; L = length of crack at mid-depth of reservoir level under consideration; and W_H = width of crack at H.

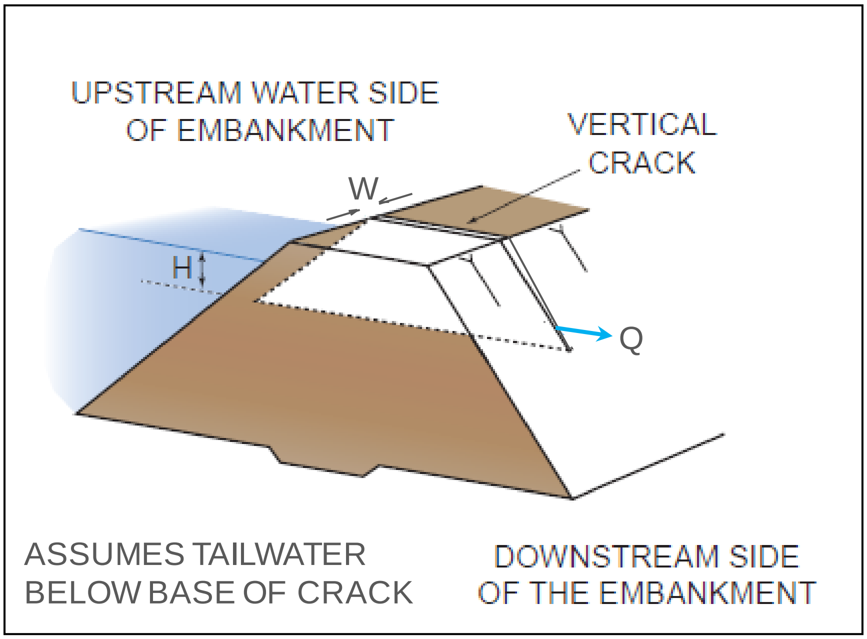


Figure IV-4-B-22. Uniformly Tapered Transverse Crack Geometry (adapted from Fell et al. 2014)

Since $2(H^2 + W_H^2/4)^{0.5} + W_H \approx 2H$ (because H >> W_H), $\gamma_w = \rho_w g$ and I = H/L, then the equation can be simplified to the following:

$$\tau \approx \gamma_w\, i\,(W_H/4)$$

Critical Crack Width

The above approximate relationships can also be used in a reverse manner to estimate the critical continuous pipe diameter or crack width for initiation, which the team can then use as a more likely or less likely factor in assessing the likelihood of a flaw and initiation of concentrated leak erosion:

$$D_c = 4\tau_c/(i\ \gamma_w) \text{ for cylindrical pipe}$$

$$W_c = 2\tau_c/(i\ \gamma_w) \text{ for vertical parallel-sided transverse crack}$$

$$W_c = 4\tau_c/(i\ \gamma_w)(D/H) \text{ for vertical uniformly tapered transverse crack}$$

Initiation

The critical shear stress (τ_c) can be compared to the estimated hydraulic shear stress for the reservoir level under consideration (τ) to help assess the likelihood of initiation of concentrated leak erosion. The factor of safety can be estimated as:

$$FS = \tau_{cr} / \tau$$

Sensitivity or uncertainty analysis is recommended. In addition to a best estimate, a range of values should be considered from a reasonable low estimate to a reasonable high estimate. Probability distributions can also be assigned for the crack geometry and critical shear stress to be used in a Monte Carlo simulation to assess the probability of a factor of safety against initiation of concentrated leak erosion less than one.

Exceeding the limit-state condition simply provides an indication of the likelihood for concentrated leak erosion to initiate and progress. Analytical results should be used to help inform judgment and develop a list of more likely and less likely factors during an elicitation to develop actual probabilities with due consideration for uncertainty.

An example of portrayal of analytical results with sensitivity analysis is shown in Figure IV-4-B-23. In this example, a best estimate for critical shear stress was estimated by a risk team during an elicitation, along with reasonable low and reasonable high estimates. The hydraulic shear stress was then estimated for a range of pipe diameters and reservoir levels. Based upon the estimated pipe diameter or range of pipe diameters for the flaw, this figure can be used to help develop a list or more likely and less likely factors for initiation of concentrated leak erosion as a function of reservoir level.

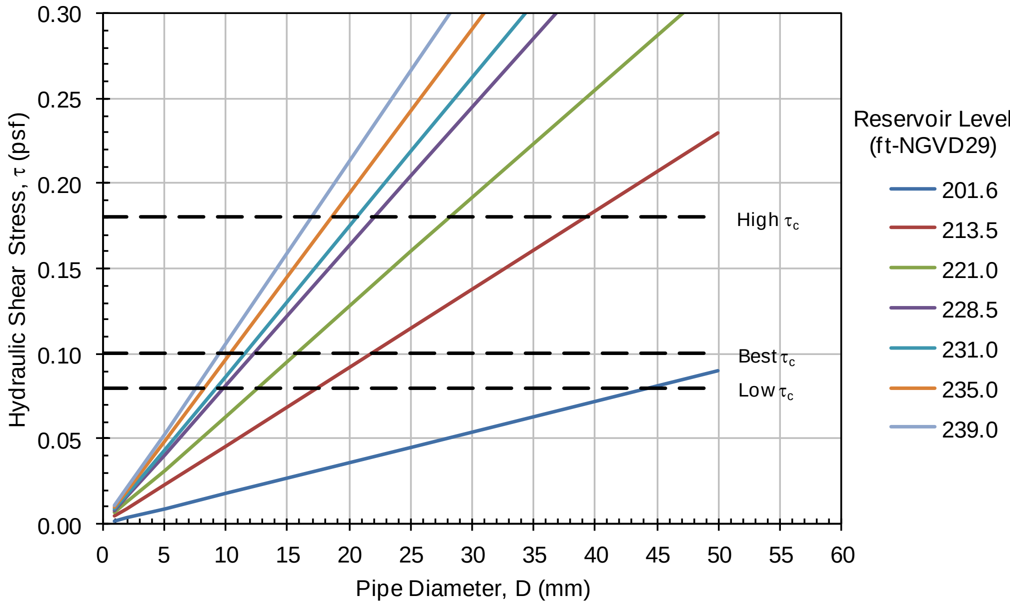


Figure IV-4-B-23. Sample Portrayal of Analytical Results for Initiation of Concentrated Leak Erosion

Appendix IV-4-C: Critical Gradients for Evaluation of Backward Erosion Piping

Critical Gradient for Initiation of a Pipe

Backward erosion piping will initiate when a “heave” or zero effective stress condition occurs in soils subject to upward through-seepage. The heave equation or critical exit gradient from Terzaghi (1943) is given by:

$$i_{cr} = \gamma_b / \gamma_w$$

where γ_b = buoyant unit weight of the soil and γ_w = unit weight of water.

Several researchers have evaluated seepage exiting sloping surfaces, where lower exit gradients are required for initiation of erosion. Two such methods are described below. Both reduce to the “classical” Terzaghi heave equation for vertical upward seepage with horizontal exit faces.

Kovács (1981) Method for Sloping Exit Faces

Kovács (1981) performed a limit equilibrium evaluation for forces acting on a rectangular soil element at the sloping exit face: soil weight reduced by the uplift force, hydrodynamic force created by the percolating water on the soil element, and friction along the base of the soil element (assuming no drained cohesion). The limit equilibrium equation can be solved for the critical exit gradient for initiation of a pipe:

$$i_{cr} = \left(\frac{\gamma_b}{\gamma_w}\right)\left(\frac{\tan(\varphi')\cos(\beta) - \sin(\beta)}{\cos(\beta - \alpha) + \tan(\varphi')\sin(\beta - \alpha)}\right)$$

where γ_b = buoyant unit weight (pcf) of the soil subject to backward erosion; γ_w = unit weight of water (62.4 pcf); ϕ' = drained angle of internal friction (deg); β = slope angle (deg); and α = seepage angle (deg) as defined by Figure IV-4-C-1.

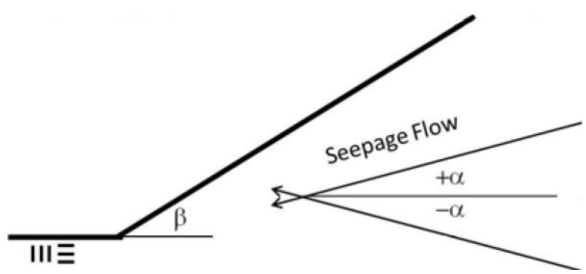


Figure IV-4-C-1. Seepage Angle for Critical Gradient (O’Leary et al. 2013)

Based on the above general relationship, Kovács (1981) derived “special” equations to characterize simplified cases for given values of slope angle and seepage direction, which were summarized in O’Leary et al. (2013):

- For a horizontal exit face (i.e., $\beta = 0°$) and vertical seepage at the downstream toe (i.e., $\alpha = -90°$), the flow lines are perpendicular to the surface (i.e., $\beta - \alpha = 90°$). The critical exit gradient reduces to:

$$i_{cr} = \gamma_b / \gamma_w$$

- For a sloping exit face (i.e., $\beta > 0°$) that is partially covered by tailwater, the flow lines are perpendicular to slope (i.e., $\beta - \alpha = 90°$ and hence $\alpha < 0°$). The critical exit gradient for initiation of a pipe along the submerged slope is given by:

$$i_{cr} = \cos(\beta)\left(\frac{\gamma_b}{\gamma_w}\right)\left(1 - \frac{\tan(\beta)}{\tan(\varphi')}\right)$$

- For a sloping exit face (i.e., $\beta > 0°$) that is entirely free (i.e., not in contact with tailwater) and the seepage field is underlain by a horizontal impervious boundary, the flow line is horizontal (i.e., $\alpha = 0°$ and hence $\beta - \alpha = \beta$) at the toe of the slope. The critical exit gradient for initiation of a pipe at the toe of the slope is given by:

$$i_{cr} = \left(\frac{\gamma_b}{\gamma_w}\right)\left(\frac{\tan(\varphi') - \tan(\beta)}{1 + \tan(\varphi')\tan(\beta)}\right)$$

An example of portrayal of analytical results for two seepage angles is shown in Figure IV-4-C-2. In this example, the critical gradient for a piping path along a geotextile-lined toe trench which had an excavated side slope 1.5H:1V ($\alpha = 33.7°$) was considered as well as vertical upward seepage at the downstream toe ($\alpha = -90°$). Based upon the estimated exit gradients from a seepage analysis, this figure can be used to help develop a list or more likely and less likely factors for initiation of backward erosion piping as a function of reservoir level.

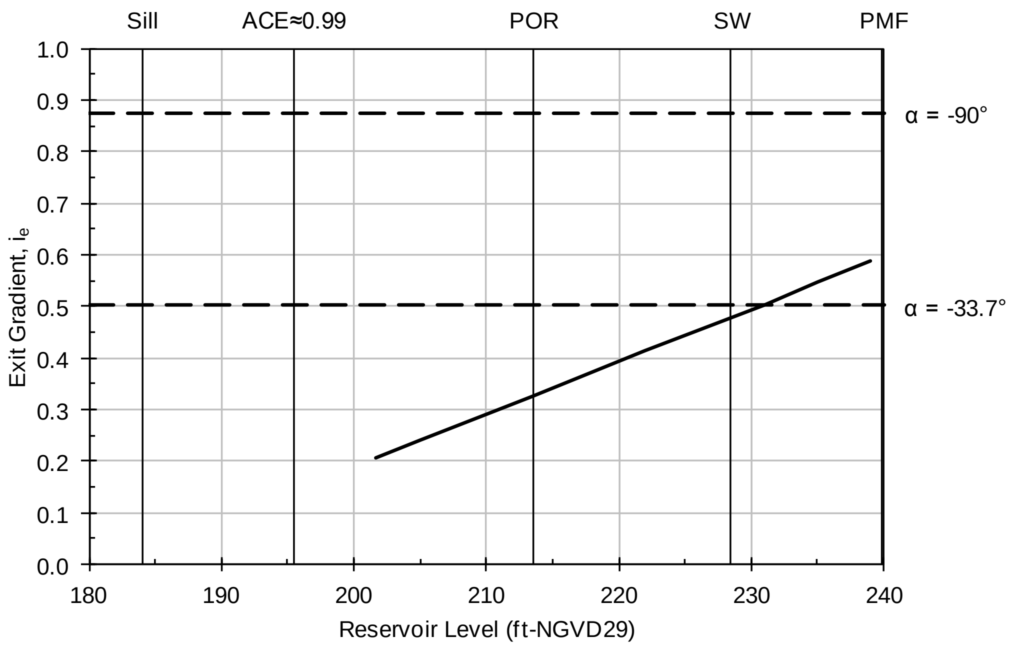


Figure IV-4-C-2. Sample Portrayal of Analytical Results for Initiation of Backward Erosion Piping

van Rhee and Bezuijen (1992) Method for Sloping Exit Faces

The critical exit gradient under outward seepage perpendicular to a slope can also be estimated using the following equation proposed by van Rhee and Bezuijen (1992):

$$i_{cr} = (1 - n)\Delta\left(\frac{\sin(\varphi' - \beta)}{\sin(\varphi')}\right)$$

where n = porosity; and Δ = relative grain density (G_s – 1); ϕ′ = drained angle of internal friction (deg); and β = slope angle (deg).

For a horizontal exit face (i.e., β = 0°), the critical exit gradient reduces to:

$$i_{cr} = (1 - n)\Delta = (1 - n)(G_s - 1)$$

Taylor Series Method of Reliability Analysis

A Taylor series method of reliability analysis can be performed for selected random variables such as foundation layer thickness, permeability, unit weight, anisotropy, etc. The Taylor series provides probabilities of a factor of safety against heave of less than one for the reservoir level under consideration (i.e., probabilistic seepage analysis). For levees, USACE has performed such analyses in conjunction with blanket theory calculations since the 1990s.

Exceeding the limit-state condition simply provides an indication of the likelihood for backward erosion to initiate. Analytical results should be used to help inform judgment

and develop a list of more likely and less likely factors during an elicitation to develop actual probabilities with due consideration for uncertainty.

The methodology is described in ETL 1110-2-561 (31 January 2006). Estimates of the mean and standard deviations are required, and can be developed through an elicitation and application of the six-sigma rule, where the standard deviation is estimated:

$$\sigma = (HCV - LCV)/6$$

where HCV = highest conceivable value; and LCV = lowest conceivable value. For the mean plus or minus three standard deviations, 99.73% of the area under the normal distribution is included. Therefore, essentially all of the values represented by the normal distribution curve are included.

An example of portrayal of Taylor series results for a given reservoir level is shown in Figure IV-4-C-3. In this example, four random variables were considered: horizontal permeability of upper layer (Kha), horizontal permeability of lower layer (Khb), thickness of upper layer (Ta), thickness of lower layer (Tb), and P_I = probability of a factor of safety against heave less than one. For a given reservoir level, nine separate seepage analyses were performed for the combination of random variables shown to obtain an estimate of the exit gradient at the downstream toe of the embankment dam. The process was then repeated for the other reservoir levels.

Case	Random Variables			
	Kha (fpd)	Khb (fpd)	Ta (feet)	Tb (feet)
Mean, μ	40	500	10	80
Standard Deviation, $\sigma = (HCV-LCV)/6$	7.5	75	0	20

Critical gradient for particle detachment, i_{cr} 0.875

Run Case	Random Variables				i_e	FS_e	$Var(FS_e)$
	Kha (fpd)	Khb (fpd)	Ta (feet)	Tb (feet)			
1	40	500	10	80	0.705	1.242	
2	32.5	500	10	80	0.793	1.103	0.019
3	47.5	500	10	80	0.635	1.378	
4	40	425	10	80	0.625	1.399	0.018
5	40	575	10	80	0.775	1.129	
6	40	500	10	80	0.705	1.242	0.000
7	40	500	10	80	0.705	1.242	
8	40	500	10	60	0.601	1.455	0.028
9	40	500	10	100	0.781	1.121	

$\sigma_{FS} = [\Sigma Var(FS)]^{0.5}$ 0.255

$V_{FS} = \sigma_{FS} / E(FS)$ 0.205

$\beta = \ln[E(FS)/(1+V_{FS}^2)^{0.5}]/[\ln(1+V_{FS}^2)]^{0.5}$ 0.96

$P_I = P(FS < 1) = \Phi(-\beta)$ 1.68E-01

Figure IV-4-C-3. Sample Taylor Series Results for Probabilities of a Factor of Safety against Heave Less than One

Critical Gradient for Progression of a Pipe

Bligh (1910) and Lane (1935)

Line-of-creep methods such as Bligh (1910) and Lane (1935) are still in use by some practitioners. They can be used for screening-level assessment of the critical gradient for progression of a pipe. Both empirical methods involve estimating the seepage path length beneath concrete structures (weirs) including cutoff walls. For application to embankment dams and levees, the seepage path length would be beneath the roof-forming material including upstream and downstream blankets or berms, cutoff walls, cutoff or inspection trenches, etc. The creep ratio is calculated as the total seepage path length divided by the hydraulic head difference. For Lane's method, the horizontal seepage path lengths are weighted 3 times less than the vertical seepage path lengths. Hence, it is often referred to as a "weighted creep" method.

$$C = (L_1 + W + L_2 + 2D) / h \text{ for Bligh}$$

$$C_w = [(L_1 + W + L_2)/3 + 2D)] / h \text{ for Lane}$$

where L_1 = length of upstream blanket or berm; W = width of base of embankment; L_2 = length of downstream blanket or berm; and d = depth of vertical structure (e.g., cutoff or weir).

To assess the likelihood of progression of backward erosion piping, the creep ratio for the reservoir level under consideration is compared to the minimum (or safe) creep ratio for the piping material in Table IV-4-C-1. Progression of backward erosion would be expected if the creep ratio is less than the minimum creep ratio.

Table IV-4-C-1. Minimum Creep Ratios

Piping Material	Bligh (1910)	Lane (1935)
Very fine sand or silt	18	8.5
Fine sand	15	7.0
Medium sand	#N/A	6.0
Coarse sand	12	5.0
Fine gravel	#N/A	4.0
Medium gravel	#N/A	3.5
Gravel and sand	9	#N/A
Coarse gravel, including cobbles	#N/A	3.0

The creep ratio is the reciprocal of the average gradient in the foundation for the reservoir level under consideration (i_{avf}), and the minimum creep ratio is the reciprocal of the critical gradient for progression of a pipe ($i_{adv} = 1/C$ or $i_{adv} = 1/C_w$).

Hoffmans (2014)

Hoffmans' analytical model is based on Ohm's law (or Darcy's law) and utilizes the critical Shields' gradient (or the critical mean energy slope in the pipes) to represent the critical overall pipe resistance and the critical Darcy's gradient to represent the seepage resistance. Because the method is relatively new, neither USACE nor Reclamation has experience with its use.

Sellmeijer et al. (2011)

Sellmeijer et al. at Delft University of Technology (TU Delft) in The Netherlands developed a mathematical model for piping based on laboratory flume tests: Sellmeijer (1988), Sellmeijer and Koenders (1991), and Koenders and Sellmeijer (1992). The tests were performed mostly on fine to medium, uniform sands uniform ($1.58 \leq c_u \leq 3.53$) with some medium to coarse sands. Sellmeijer et al. (2011) extended and updated the piping model based on the results of several small-scale, seven medium-scale, and four large-scale field (IJkdijk) tests by Deltares / TU Delft reported in van Beek et al. (2009-10). The critical gradient for progression of a pipe is estimated as:

$$i_{adv} = (F_R)(F_S)(F_G)$$

where F_R = resistance factor (strength of the layer subject to backward erosion); F_S = scale factor (relating pore size and seepage size); and F_G = geometrical shape factor. ***The methodology is only applicable within the limits of the testing parameters shown in Table IV-4-C-2.*** Mean values were used in the equations.

Table IV-4-C-2. Parameter Limits during Piping Model Testing (Sellmeijer et al. 2011)

Parameter	Minimum	Maximum	Mean
Relative Density, RD (percent)	34	100	72.5
Coefficient of Uniformity, U	1.3	2.6	1.81
Roundness, KAS (percent)	35	70	49.8
Particle Size, d_{70} (mm)	0.150	0.430	0.208

<u>Resistance Factor</u>

The resistance factor (F_R) is calculated as:

$$F_R = \eta \cdot (G_s - 1) \cdot \tan(\vartheta)(RD/72.5)^{0.35}(U/1.81)^{0.13}(KAS/49.8)^{-0.02}$$

where KAS = roundness of the particles, which can be visually obtained using Figure IV-4-C-4; RD = relative density (percent); U = coefficient of uniformity; G_s = specific gravity of soil particles; ϑ = bedding angle (deg); and η = White's constant.

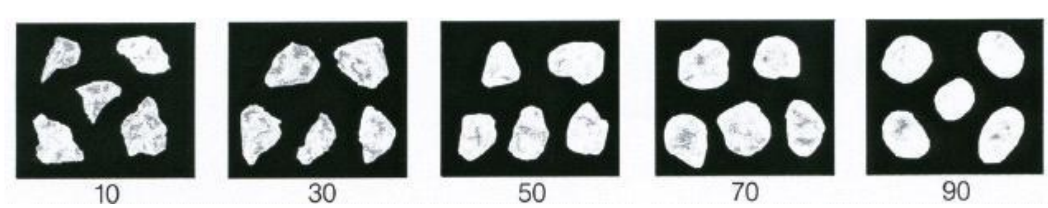


Figure IV-4-C-4. KAS Indication of Angularity (van Beek et al. 2010)

The bedding angle has been found to increase with decreasing particle size and departure from sphericity. For applicability to Sellmeijer's piping rule (where d_{70} ranged from

0.150 to 0.430 mm), it ranges from about 37 to 45 degrees and can be estimated by the following equation from Li (1985):

$$\vartheta = 21.50(d_{70})^{-0.17}$$

where d_{70} = particle size (cm) for which 70 percent is finer (by weight).

For applicability to Sellmeijer's piping rule, White's constant ranges from about 0.25 to 0.33 and can be estimated by the following equation derived from White (1940) Yalin and Karahan (1979) assuming spherical particles:

$$\eta = (6/\pi)\cdot Y_{cr}\cdot \cot(\vartheta)$$

where Y_{cr} = dimensionless critical stress that ranges from about 0.10 to 0.17 for applicability to Sellmeijer's piping rule. Additional research is likely needed for bedding angle and White's constant, and a parametric evaluation is recommended at this time.

Van Beek et al. (2010) indicate that KAS and U appear to be of less importance than the other sand characteristics, and have a weak influence on the critical gradient. Therefore, the U and KAS terms in the equation for F_R are sometimes ignored.

<u>Scale Factor</u>

The scale factor (F_S) is calculated as:

$$F_S = [d_{70} / (\kappa L)^{1/3}](0.000208 / d_{70})^{0.6}$$

where d_{70} = particle size (m) for which 70 percent is finer (by weight); L = seepage path length (m) through the piping layer (measured horizontally); and κ = intrinsic permeability (m^2) of the piping layer which can be estimated by:

$$\kappa = (k_h)(\mu/\gamma_w)$$

where μ = dynamic viscosity of water (N s/m^2); and k_h = permeability of the piping layer in the horizontal direction (m/sec).

Van Beek et al. (2012) adapted Sellmeijer's piping rule to multi-layer foundations to assess the influence of a coarse layer beneath the piping layer. The intrinsic permeability in the above equation is replaced with a layer-weighted average calculated as follows:

$$k_{h,avg} = \sum_{i=1}^{n} \frac{k_{h,i} D_i}{D}$$

where D = total aquifer thickness.

<u>Geometrical Shape Factor</u>

The geometrical shape factor (F_G) is calculated as:

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

where D = thickness of the piping layer (m); and L = seepage path length (m) through the piping layer (measured horizontally).

Schmertmann (2000)

Schmertmann (2000) carried out backward erosion piping tests in flumes at the University of Florida. The tests were carried out on a range of soils from fine to medium sands, up to coarse sand and fine gravel mixes. The soils were mostly fairly uniform ($1.5 \leq c_u \leq 6.1$). He also plotted the Delft tests and found a similar correlation. Since the test geometries used at University of Florida and Delft were not the same, correction factors for geometry were applied in order to plot all of the results together as shown in Figure IV-4-C-5. The methodology requires quite large corrections for scale effects and foundation geometry.

Most of the tests (i.e., 32 out of 39) were performed on sands with a coefficient of uniformity less than 3.2. The other 7 tests were performed on gap-graded soils and a well-graded soil with a coefficient of uniformity up to about 6. Two of the data sets with a coefficient of uniformity up to about 6 were gap-graded, and the applicability of the test results is dubious. The methodology is based on limited data with a coefficient of uniformity between 3 and 6, and some of those test results may have been affected by internal instability. ***Therefore, the relationship should only be used for $1 \leq c_u \leq 3$.***

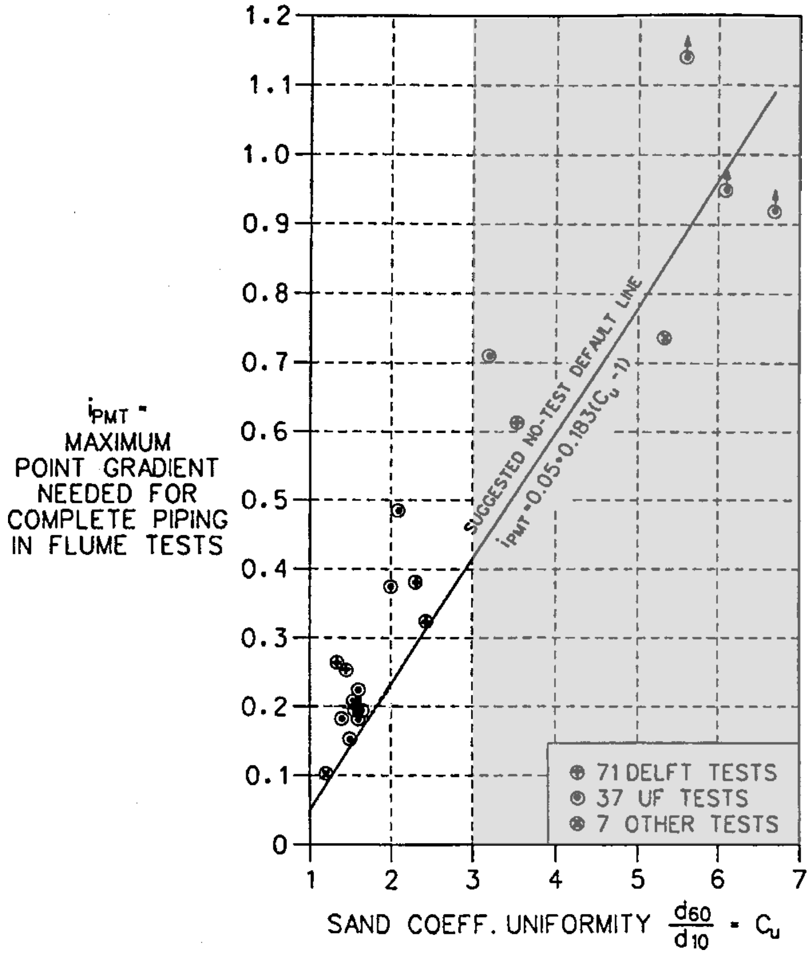


Figure IV-4-C-5. Maximum Point Gradient Needed for Complete Piping (Schmertmann 2000)

Schmertmann (2000) proposed a methodology that gives an average factor of safety against piping that is based around the results of the flume tests in Figure IV-4-C-5. Several corrections for a number of factors to relate the laboratory tests to field conditions are required. The critical gradient for progression of a pipe is estimated as:

$$i_{adv} = (i_{pmt})_{corrected} = [(C_D)(C_L)(C_S)(C_K)(C_Z)(C_\gamma)(C_\alpha) / C_R](i_{pmt})$$

where C_D = correction factor for depth/length ratio; C_K = correction factor for anisotropic permeability of layer subject to backward erosion; C_L = correction factor for total pipe length; C_S = correction factor for grain-size; C_R = correction factor for dam axis curvature; C_Z = correction factor for high-permeability underlayer; C_α = adjustment for pipe inclination; C_γ = correction factor for density; and i_{pmt} = maximum point seepage gradient needed for complete piping in the flume test based on the soil's coefficient of uniformity (from flume tests or Figure IV-4-C-5). Additional information about the correction factors is provided below. ***Some errors contained in Schmertmann (2000) were corrected.***

The laboratory testing essentially used clean sands. No sands with silty fines and no sand-gravel mixtures were apparently used. These materials could behave differently than the limited range of sands that were used in the flume tests. In addition, controlled laboratory testing may not adequately account for actual field variability, and the large number of correction factors that are applied for field conditions suggest the tests may not adequately cover cases encountered in the field. ***Careful evaluation of the appropriateness of the method for a specific dam is needed.***

Depth/Length Ratio Factor

The D/L_f factor (C_D) can be determined from Figure IV-4-C-6, where D = thickness of the piping layer measured perpendicular to the flow lines (i.e., perpendicular to pipe inclination, α); and L_f = direct (not meandered) length between ends of a completed pipe path, from downstream to upstream exit, measured along the pipe path on a transformed section. For a vertical flow path, $D/L_f = \infty$ and $C_D = 0.715$. For horizontal flow paths, D = the vertical thickness of the piping layer. For steeply inclined flow paths, interpolate between these limits. Schmertmann (2000) amended the Weijers and Sellmeijer (1993) theory to obtain the relationship for the D/L_f factor shown in Figure IV-4-C-6 and calculated as:

$$C_D = \frac{\left(\frac{D}{L_f}\right)^{\frac{0.2}{\left(\frac{D}{L_f}\right)^2 - 1}}}{1.4}$$

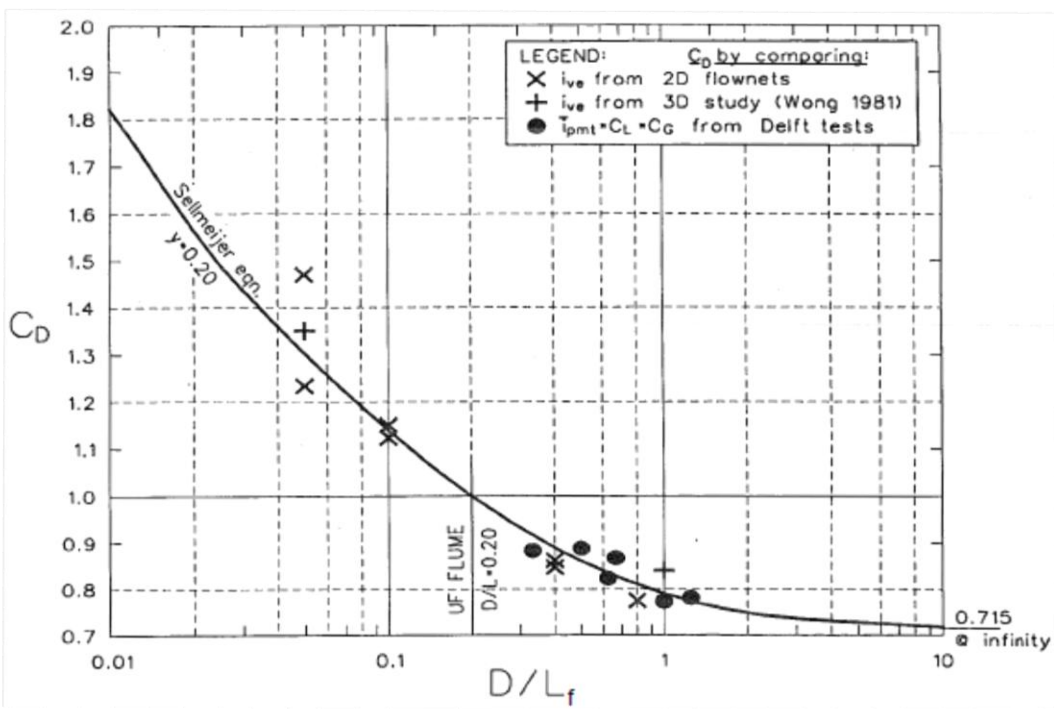


Figure IV-4-C-6. Correction Factor for Depth/Length Ratio (Schmertmann 2000)

Length Factor

The length factor (C_L) is calculated as:

$$C_L = (L_t / L_f)^{0.2}$$

where L_t = flume model length (L_t = 5 feet if Figure IV-4-C-5 is being used to estimate i_{pmt}); and L_f = direct (not meandered) length (feet) between ends of a completed pipe path, from downstream to upstream exit, measured along the pipe path on a transformed section, where

$$L_f = L / (k_h/k_v)^{0.5}$$

where k_h = permeability of the piping layer in the horizontal direction; and k_v = permeability of the piping layer in the vertical direction; and L = direct (not meandered) length (feet) between ends of a completed pipe path, from downstream to upstream exit, measured along the pipe path.

Grain-Size Factor

The grain-size factor (C_S) is calculated as:

$$C_S = (d_{10f} / 0.20 \text{ mm})^{0.2}$$

where d_{10f} = particle size (mm) of the (field) piping layer for which 10 percent of the total weight is finer.

Anisotropic Permeability Factor

The anisotropic permeability factor (C_K) is calculated as:

$$C_K = (1.5 / R_{kf})^{0.5}$$

where R_{kf} = anisotropy of the piping layer (k_h/k_v), where k_h = permeability of the piping layer in the horizontal direction; and k_v = permeability of the piping layer in the vertical direction.

Underlayer Factor

If the layer susceptible to piping is underlain by a high-permeability underlayer, Figure IV-4-C-7 is used to determine the underlayer factor (C_Z), where D = thickness of the underlayer (feet); k_p = permeability of the piping layer (feet/sec); k_u = permeability of the underlayer (feet/sec); and r = equivalent radius (feet) of the developing pipe cross section (prior to gross enlargement). Schmertmann used small radii in his tests (0.3 inch and 0.6 inch). For practical purposes, r is very small, and D/r is very large, so it is suggested that C_Z = 1. If very thin erodible layers are being considered, use radii of 2.5 to 10 mm. For thin alternating layers of erodible and non-erodible soil modeled as a homogenous layer with high anisotropy, use C_Z = 1.

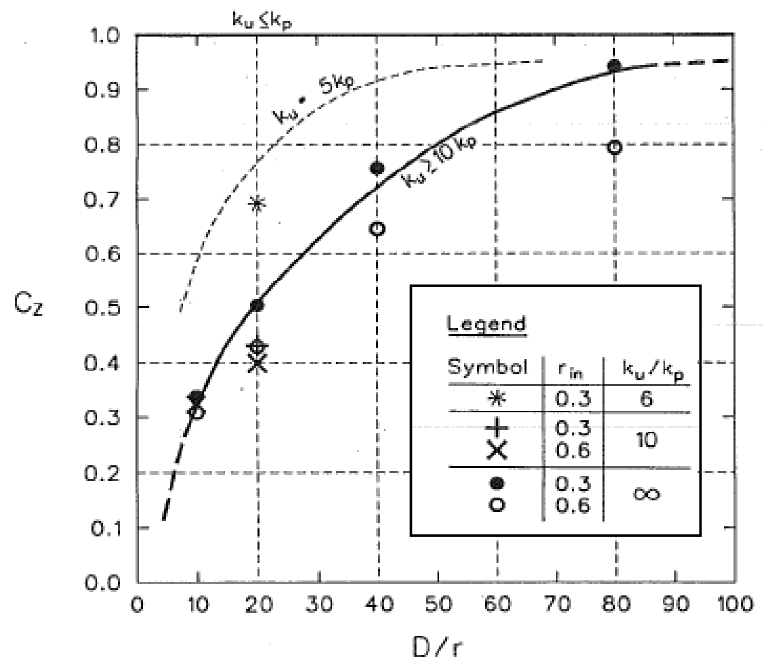


Figure IV-4-C-7. Correction Factor for High-Permeability Underlayer (Schmertmann 2000)

Density Factor

The density factor (C_γ) is calculated as:

$$C_\gamma = 1 + 0.4(D_{rf}/100 - 0.6)$$

where D_{rf} = relative density of soil layer subject to backward erosion.

Correction Factor

The correction factor for dam axis curvature (C_R) is calculated as:

$$C_R = 1.0 \text{ for a straight dam alignment}$$

or

$$C_R = (R_1 + R_0) / 2R \text{ for a curved dam axis}$$

where R = radius to point on the pipe path in a dam with curved axis (i.e., radius of curvature in the dam); R_0 = shortest radius to an end of completed pipe path (i.e., distance from the center of curvature to the upstream toe; and R_1 = longest radius to an end of completed pipe path (i.e., distance from the center of curvature to the downstream toe).

Pipe Inclination Adjustment

The pipe inclination adjustment (C_α) is calculated as:

$$C_\alpha = i_{p\alpha} / i_{po}$$

where i_{po} = field horizontal critical gradient (obtained by making all corrections to i_{pmt} obtained from Figure IV-4-C-5) and calculated as:

$$i_{po} = [(C_D)(C_L)(C_S)(C_K)(C_Z)(C_\gamma) / C_R](i_{pmt})$$

and $i_{p\alpha}$ = field critical gradient using i_{po} and the angle (α) of the pipe path (progressing in the upstream direction). If the pipe path progresses upward, α is positive, where as α is negative if the pipe path progresses downward. For a horizontal seepage exit, $\alpha = 0$ degrees, and for a vertical seepage exit, $\alpha = –90$ degrees. Figure IV-4-C-8 can be used as a guide for determination of the sign for α.

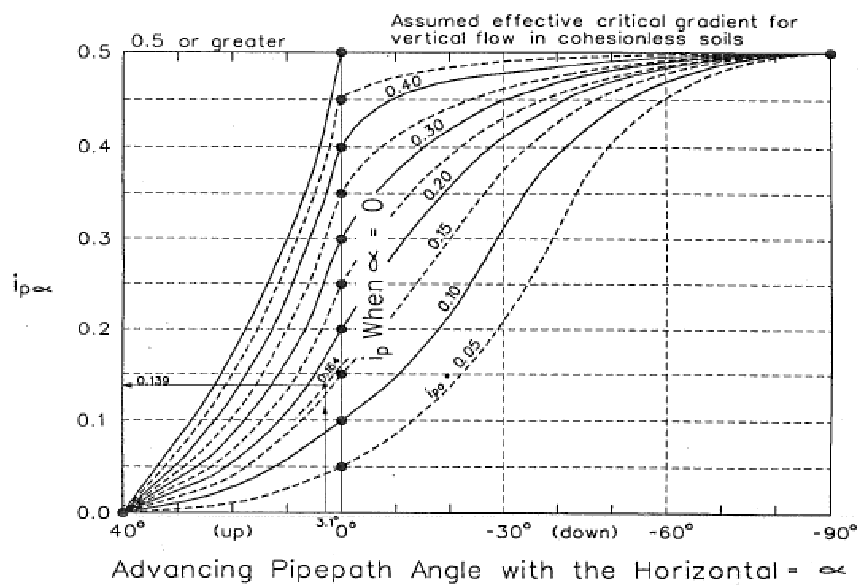


Figure IV-4-C-8. Field Critical Gradient (Schmertmann 2000)

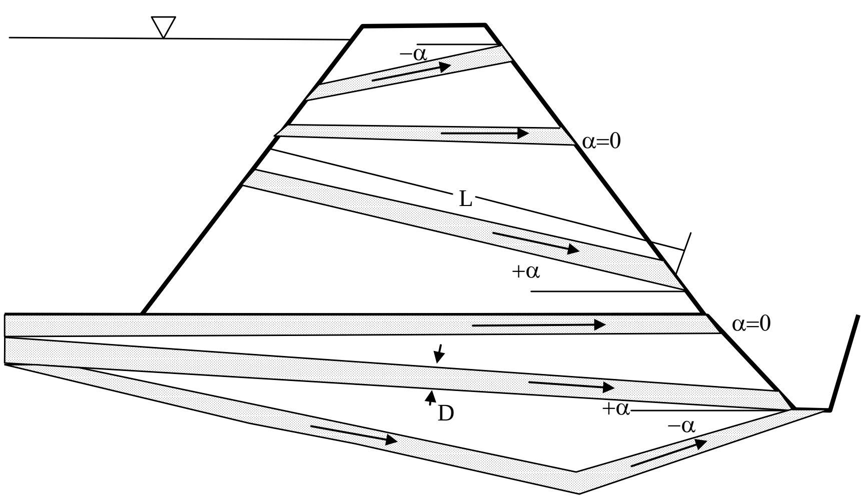


Figure IV-4-C-9. Pipe Path Inclination Geometry

Hydraulic Condition for Progression of a Pipe

To assess the likelihood of progression of a pipe (hydraulic condition), the average gradient for the reservoir level under consideration is compared to the critical gradient for progression of a pipe. The factor of safety against progression of the pipe (hydraulic condition) can be estimated as:

$$FS = i_{adv} / i_{avf}$$

where i_{adv} = critical gradient for progression of a pipe; and i_{avf} = average gradient in the foundation for the reservoir level under consideration.

Sensitivity or uncertainty analysis is recommended. In addition to a best estimate, a range of values should be considered from a reasonable low estimate to a reasonable high estimate. Probability distributions can also be assigned for the various input parameters to be used in a Monte Carlo simulation to assess the probability of a factor of safety against progression of the pipe (hydraulic condition).

Exceeding the limit-state condition simply provides an indication of the likelihood for backward erosion to progress. Analytical results should be used to help to help inform judgment and develop a list of more likely and less likely factors during an elicitation to develop actual probabilities with due consideration for uncertainty.

An example of portrayal of analytical results for multiple methods is shown in Figure IV-4-C-10. In this example, the critical gradient for progression of a pipe was evaluated using four methods. Based upon the estimated average gradients in the foundation from a seepage analysis as well as considering the hydraulic head difference over seepage path length, this figure can be used to help develop a list of more likely and less likely factors for the hydraulic condition for progression of backward erosion piping as a function of reservoir level. The methods shown may not be given equal weight by the risk team in assessing the probability.

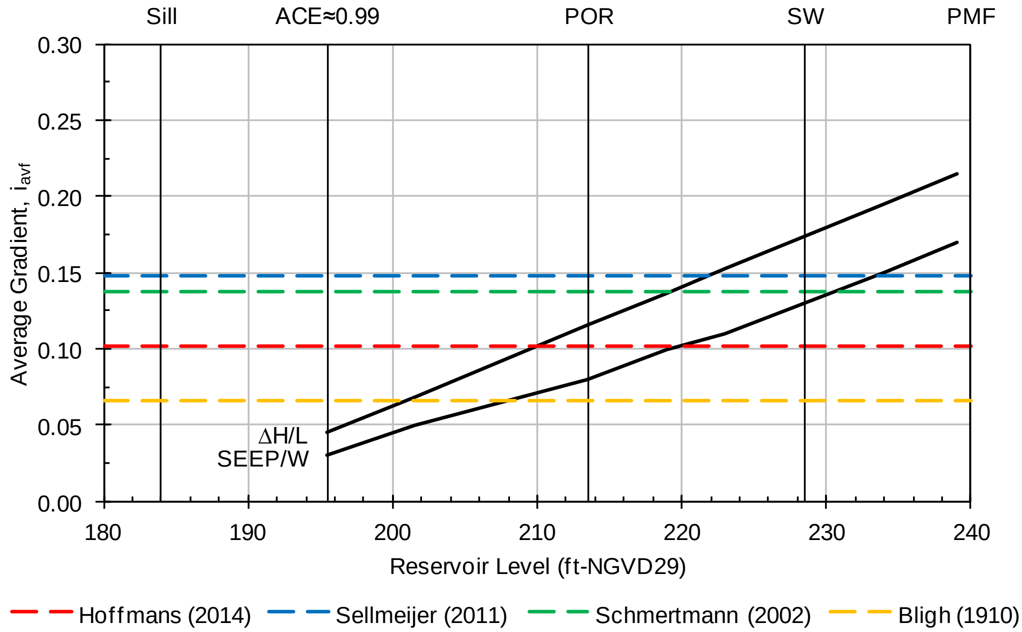


Figure IV-4-C-10. Sample Portrayal of Analytical Results for Likelihood of Progression of a Pipe (Hydraulic Condition)

Appendix IV-4-D: Internal Instability (Suffusion)

Burenkova (1993)

Based on the results of laboratory testing on cohesionless sand-gravel soils with maximum particle sizes up to 100 mm and coefficients of uniformity up to 200, Burenkova (1993) proposed a geometric condition for internal stability of a soil that depends on the conditional factors of uniformity ($h' = d_{90}/d_{60}$ and $h'' = d_{90}/d_{15}$) as shown in Figure IV-4-D-1.

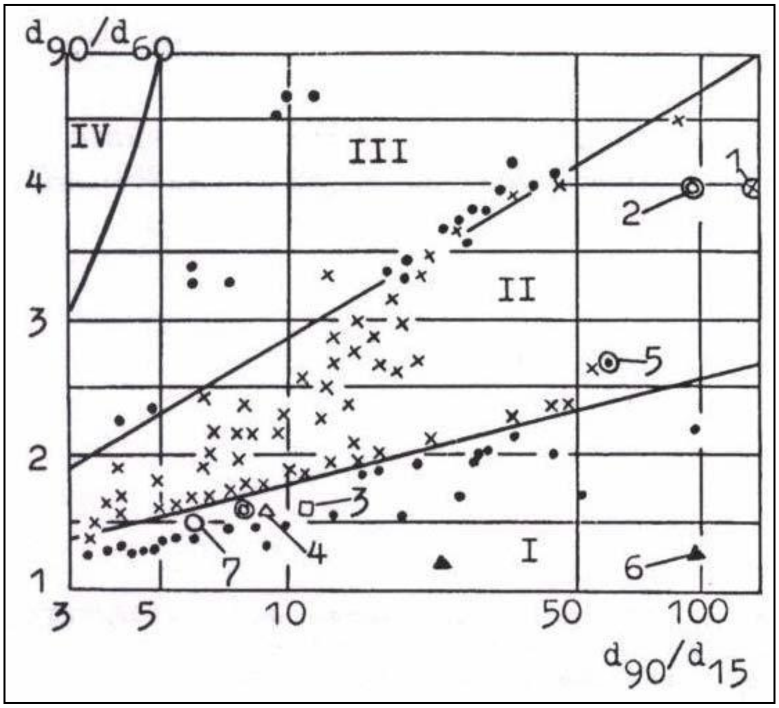


Figure IV-4-D-1. Materials Susceptible to Internal Instability (Burenkova 1993)

Boundaries were defined separating “suffusive soils” from “non-suffusive soils”. Zones I and III represent zones of suffusive compositions; Zone II represents a zone of non-suffusive compositions; and Zone IV represents a zone of artificial soils. Zone II (non-suffusive) boundaries are defined as follows: $0.76 \cdot \log(h'') + 1 < h' < 1.86 \cdot \log(h'') + 1$.

Wan and Fell (2004a, 2008)

According to Wan and Fell (2004a, 2008), the Burenkova (1993) method did not give a clear boundary between internally stable and unstable soils in the data set. Therefore, they developed contours for predicting the probability of internal instability by logistic regression of h′ and h″. Their “modified Burenkova method” for broadly graded and gap-graded soils is shown in Figure IV-4-D-2 for silt-sand-gravel and clay-silt-sand-gravel mixtures of limited plasticity and clay content (i.e., PI ≤ 12 and less than 10 percent clay-size fraction, defined as the percentage finer than 0.002 mm) and Figure IV-4-D-3 for sand-gravel mixtures with a non-plastic FC < 10 percent. The contours in Figure IV-4-D-3 predict higher probabilities of internal instability than those in Figure IV-4-D-2 because the more erosion resistant clayey and silty soil samples were excluded from the data set.

The probability contours are represented by the following equations (Wan and Fell 2004a):

$$P_I = e^Z / (1 + e^Z)$$

For silt-sand-gravel soils and clay-silt-sand-gravel soils percent of limited clay content and plasticity,

$$Z = 2.378 \cdot \log(h'') - 3.648(h') + 3.701$$

For sand-gravel soils with less than 10 percent non-plastic fines,

$$Z = 3.875 \cdot \log(h'') - 3.591(h') + 2.436$$

The probabilities should not be used directly in a risk assessment, but rather used to help develop a list of more likely and less likely factors during an elicitation of probability estimates.

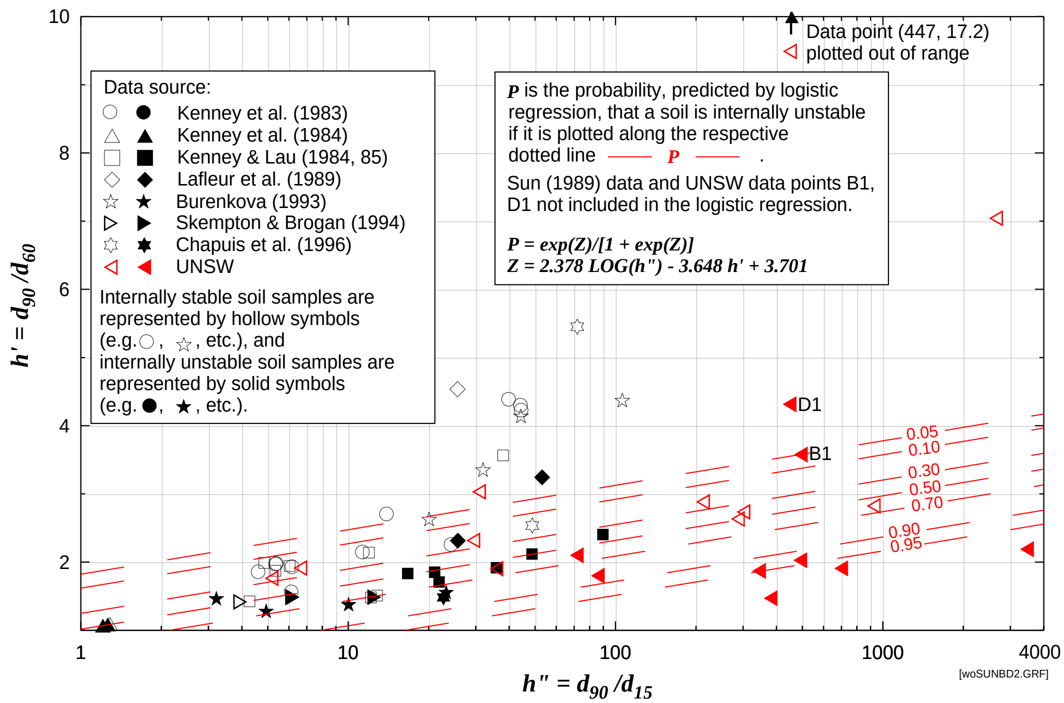


Figure IV-4-D-2. Probability of internal instability for silt-sand-gravel soils and clay-silt-sand-gravel soils of limited clay content and plasticity (Wan and Fell 2004a)

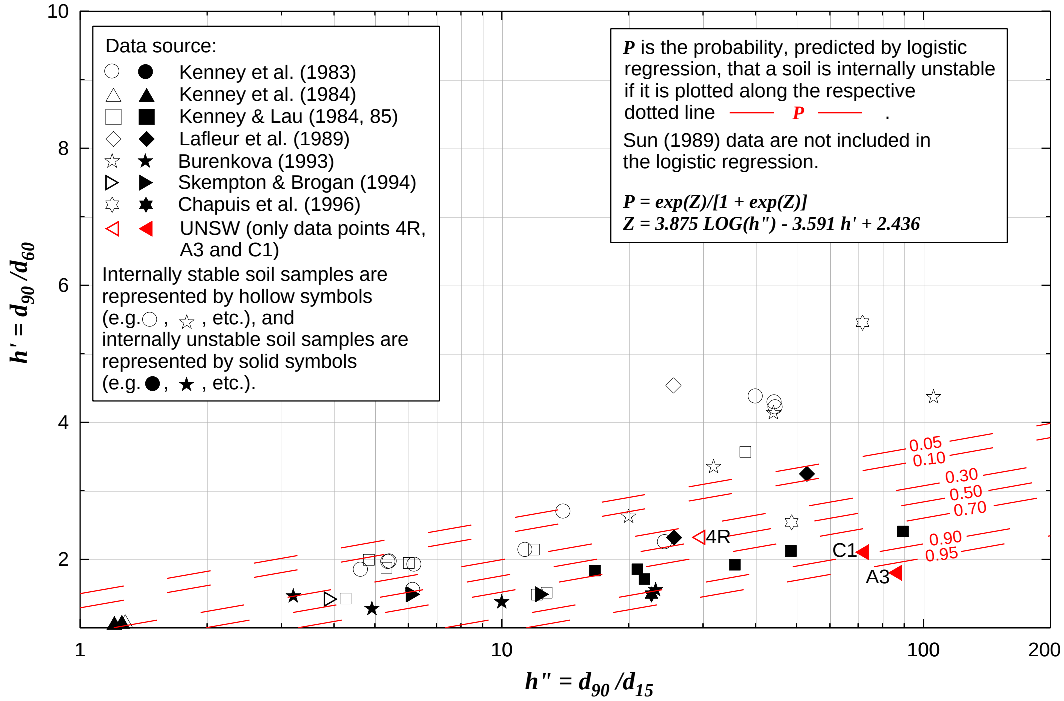
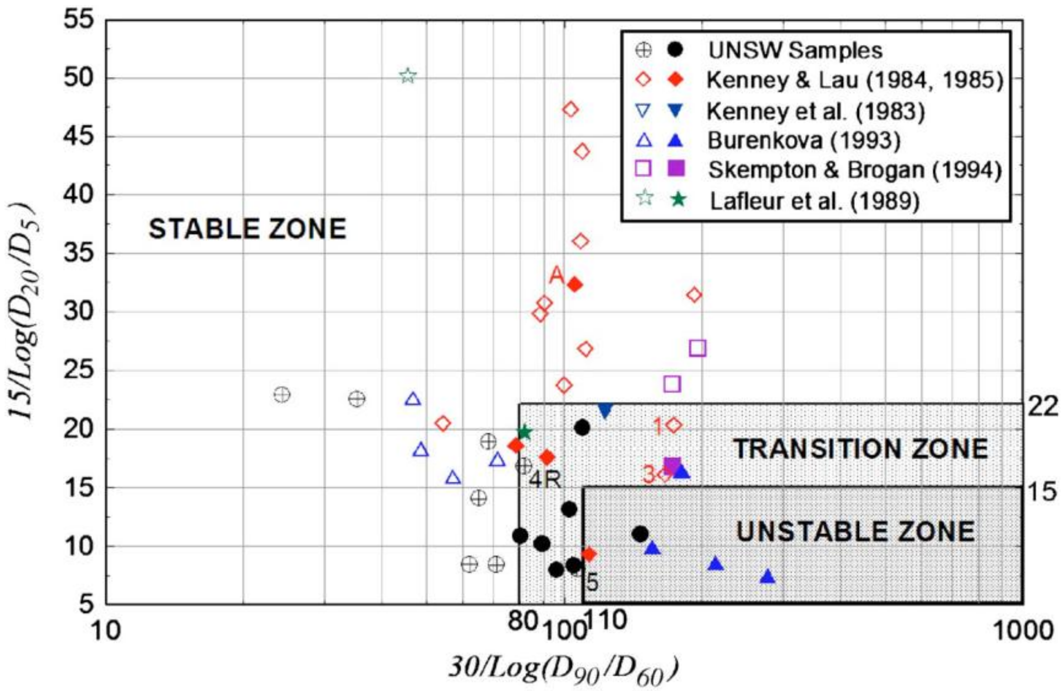


Figure IV-4-D-3. Probability of internal instability for sand-gravel soils (Wan and Fell 2004a)

Wan and Fell (2008)

Wan and Fell (2008) also proposed an alternative method for broadly graded silt-sand-gravel soils as a function of d_{90}/d_{60} and d_{20}/d_5. Boundaries shown in Figure IV-4-D-4 were proposed for likelihood of internal instability. This method is not applicable to gap-graded soils.



Notes:
1. Hollow symbol represents internally stable sample.
2. Solid symbol represents internally unstable sample.

Figure IV-4-D-4. Alternative Method for Assessing Internal Instability (Wan and Fell 2008)

Li and Fannin (2008)

Li and Fannin (2008) reviewed two commonly used methods to determine the susceptibility to internal instability: Kézdi (1979) and Kenney and Lau (1985, 1986). Kézdi divided a soil into a coarse fraction and a fine fraction at one point along its particle-size distribution curve and applied Terzaghi's (1939) rule for designing protective filters (D'_{15}/d'_{85}) to the two fractions, with the fine fraction as the "base" and the coarse fraction as the "filter," to assess if the soil would self-filter and be internally stable. The mass increment (H) over D'_{15} and d'_{85} is constant and equal to 15 percent, resulting in a criterion for instability of H less than 15 percent.

Kenney and Lau calculated an H/F stability index over the increment D to 4D, which increases in magnitude with progression along the gradation curve, where H is the mass fraction between D and 4D and F is the mass passing. They originally proposed a criterion in 1985 for internal instability of $H/F < 1.3$, applicable within $F \leq 30$ percent (and $c_u \leq 3$) for narrowly graded soils and within $F \leq 20$ percent (and $c_u > 3$) for widely graded soils. This criterion was subsequently revised in 1986 to $H/F < 1.0$. This method is commonly used for cohesionless sand-gravel soils (e.g., Reclamation's "4x" line).

An example of converting a particle-size distribution curve to H-F space (referred to as the "shape curve") is shown in Figure IV-4-D-5:

4D = 4 x D

$H = F_{4D} - F$

Gradation Curve

D (mm)	F (%)	4D (mm)	F_{4D} (%)	H (%)	H/F
182.288	99.0	729.152	99.0	0.0	#N/A
108.342	93.3	433.368	99.0	5.7	0.061
61.327	77.2	245.308	99.0	21.8	0.282
40.184	51.3	160.736	97.4	46.0	0.897
16.432	32.1	65.728	78.7	46.7	1.456
5.265	20.1	21.060	35.8	15.8	0.785
2.748	17.2	10.992	26.2	9.0	0.523
1.053	16.0	4.212	18.9	2.9	0.183
0.596	14.7	2.384	16.9	2.3	0.154
0.424	12.2	1.696	16.4	4.3	0.351
0.282	8.4	1.128	16.0	7.6	0.912
0.191	5.5	0.764	15.2	9.6	1.740
0.129	3.6	0.516	13.5	9.9	2.717
0.075	2.4	0.300	8.9	6.5	2.731
0.055	1.1	0.220	6.4	5.3	4.804

4(40.184) = 160.736

16 – 8.4 = 7.6

Figure IV-4-D-5. Example of Obtaining the Shape Curve

Li and Fannin (2008) combined aspects of these two methods for assessing the susceptibility to internal instability. They concluded that the Kenney and Lau criterion is more conservative at F > 15 percent, but the Kézdi criterion is more conservative at F < 15 percent. The combined criteria are shown in Figure IV-4-D-6, where the respective values of H and F are plotted at $(H/F)_{min}$.

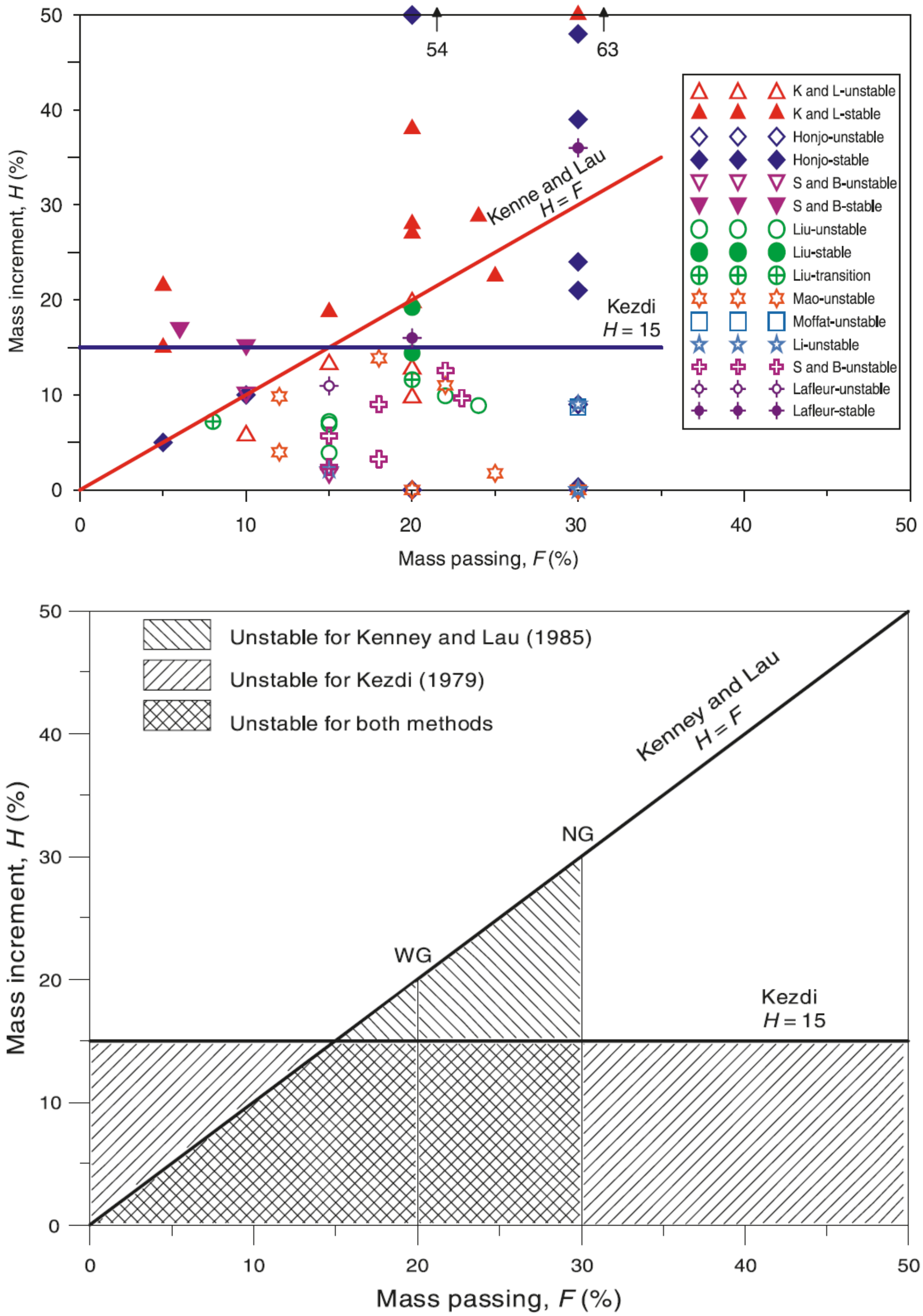


Figure IV-4-D-6. Criteria for Internal Instability
(Li and Fannin 2008)

Appendix IV-4-E: Contact Erosion

Geometric Condition

Experimental results of contact erosion for non-plastic soils for the geometric and hydraulic conditions for the detachment and transport of particles resulted in the domains shown in Table IV-4- E-1. For the "geometric domain" where D_{15}/d_{85} is less than the thresholds in the third column, initiation of contact erosion is very unlikely to occur because there is geometrical filtration regardless of the hydraulic loading.

Table IV-4-E-1. Domain of Geometric and Hydraulic Influence for Non-Plastic Soils (Bonelli 2013)

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

Hydraulic Condition

The hydraulic condition for contact erosion depends of the configuration of the fine and coarse layers. The influence of the coarse layer on the initiation of contact erosion can be neglected if D_{15}/d_{85} is greater than the values listed in the fifth column of Table IV-4-E-1 for the "hydraulic condition" domain. For those situations, the hydraulic loading condition controls, and there is no filtration effect. In the "geometrical and hydraulic condition" domain, the critical velocity is also a function of the coarse soil grading, and the hydraulic loading to initiate contact erosion is higher than the "hydraulic condition" domain.

Brauns (1985) proposed an expression for critical velocity which provides a good approximation for sand below gravel:

$$U_{crit}\ (m/s) = 0.65 n_F \sqrt{\left(\frac{\rho_s - \rho_w}{\rho_w}\right) g d_{50}} = 0.65 n_F \sqrt{(G_s - 1) g d_{50}}$$

where n_F = porosity of the coarse soil (gravel); ρ_s = density of the base soil (sand) particles (kg/m^3); ρ_w = density of water (1,000 kg/m^3); G_s = specific gravity of the sand particles; g = acceleration of gravity (9.81 m/s^2); and d_{50} = mean grain size of the base soil (sand).

Guidoux et al. (2010) measured critical velocities and critical hydraulic gradients for various base soils and recommended using the effective grain diameter (d_H) of Koženy (1953) instead of d_{50} for a more representative particle-size description for the base soil to predict the critical velocity:

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

where d_j (mm) = particle-size of the fraction j of the base soil gradation curve; and F_j (-) = mass fraction of the fraction j. For a well-graded soil, $d_H \approx d_{50}$. Their expression for critical velocity can be used for sands, silts, or sand/clay mixtures below gravel:

$$U_{crit}\ (m/s) = 0.65 n_F \sqrt{\left(\frac{\rho_s - \rho_w}{\rho_w} \right) g d_{50}} = 0.65 n_F \sqrt{(G_s - 1) g d_{50}}$$

where n_F = porosity of the coarse soil (gravel); ρ_s = density of the base soil particles (kg/m^3); ρ_w = density of water (1,000 kg/m^3); g = acceleration of gravity (9.81 m/s^2); G_s = specific gravity of the sand particles; d_H = effective grain diameter of the base soil; and β = empirical coefficient. Several parameters influence the coefficient β, which was estimated by Guidoux et al. (2010) by fitting the above equation to the experimental data and assuming it did not vary among the tested soils. The best fit obtained for β was 5.3E-09 m^2.

The relationships for critical velocity for both methods are shown in Figure IV-4-E-1. Both methods give the same results for sand below gravel. Since the D_{50} of sand can be readily assessed from the gradation curves, the Brauns (1985) method is the simplest to use and provides a good approximation for sand below gravel. For other "base" soils, the Guidoux et al. method must be used.

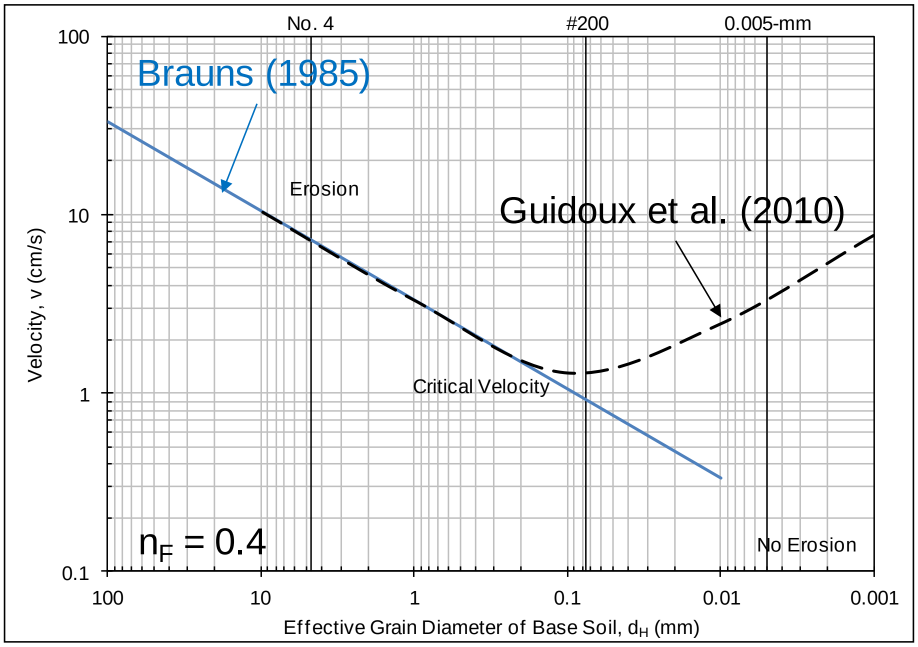


Figure IV-4-E-1. Critical Velocity for Initiation and Progression of Contact Erosion

Schmitz (2007) conducted testing for erosion of silt layers above coarse layers. In contrast to the configuration of fine soil below coarse soil, he noticed an influence of the confining stress on the critical velocity. For higher vertical stresses on the sample, he measured higher critical velocities. Generally, the critical velocities measured were of the same order of magnitude as the reverse configuration, between 1 and 10 cm/s but lower than the critical velocities proposed by Guidoux et al. (2010).

Initiation

The critical velocity can be compared to the estimated Darcy velocity for the reservoir level under consideration to help assess the likelihood of initiation and progression of contact erosion. The factor of safety can be estimated as:

$$FS = U_{crit} / (k_h \, i)$$

where k_h = hydraulic conductivity (horizontal) of the coarse layer; and i = seepage gradient for the reservoir level under consideration. Note this is Darcy velocity and does not need adjustment for porosity.

Sensitivity or uncertainty analysis is recommended. In addition to a best estimate, a range of values should be considered from a reasonable low estimate to a reasonable high estimate. Probability distributions can also be assigned for the mean grain size of the base soil (sand), effective grain diameter (d_H) of the base soil, and hydraulic conductivity (horizontal) of gravel to be used in a Monte Carlo simulation to assess the probability of a factor of safety against initiation of contact erosion less than one.

Exceeding the limit-state condition simply provides an indication of the likelihood for contact erosion to initiate and progress. Analytical results should be used to help to help inform judgment and develop a list of more likely and less likely factors during an elicitation to develop actual probabilities with due consideration for uncertainty.

An example of portrayal of analytical results with sensitivity analysis is shown in Figure IV-4-E-2. In this example, a range of hydraulic conductivity and effective grain diameter of the base soil were estimated by a risk team during an elicitation. Based on the estimated Darcy velocities, this figure can be used to help develop a list or more likely and less likely factors for initiation of and progression of contact erosion as a function of reservoir level.

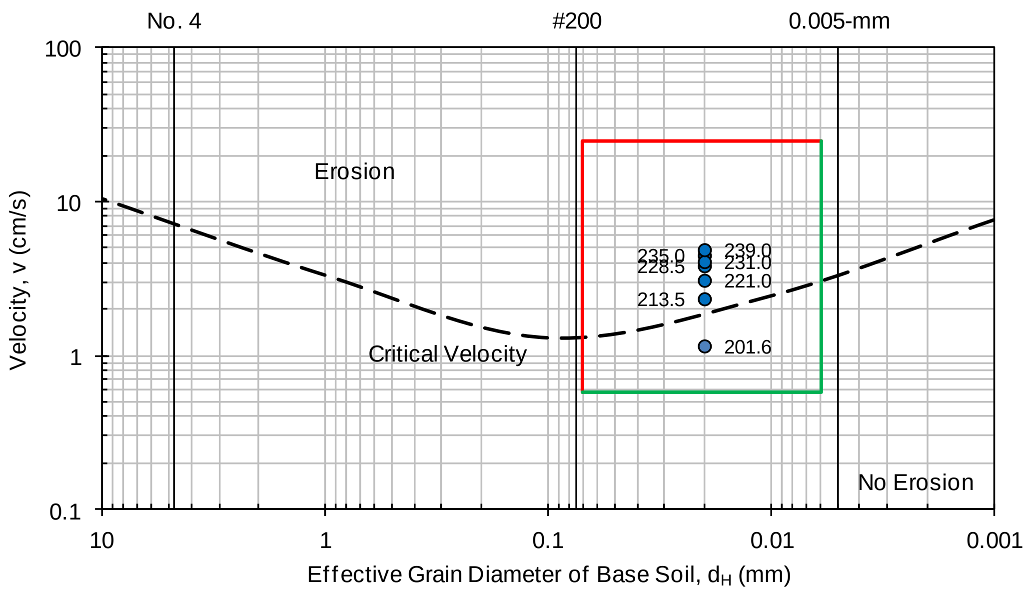


Figure IV-4-E-2. Example Portrayal of Analytical Results for Initiation of Contact Erosion

Appendix IV-4-F: Continuation

Susceptibility to Cracking

The ability of a filter material to hold a crack generally depends on the fines content, cementation, or the presence of plastic fines. Filters with low fines content and non-plastic fines are generally less likely to sustain a crack than filters with a high fines content comprised of plastic fines. Criteria have been developed to decrease the amount of fines and thus the chance of a filter cracking and it is important to include these when evaluating an existing filter. For example, Reclamation and USACE filter design criteria require a minimum D_5F equal to 0.075 mm (i.e., non-plastic fines content less than or equal to 5 percent) in the final in-place product to help ensure these filters will not hold a crack. To achieve the maximum allowable fines content after compaction, the "off the belt" at the quarry/crusher stockpile typically had about 3 percent fines to account for breakdown during handling, transportation, placing, and compacting. In some circumstances for critical modern designs, the maximum in-place fines content has been limited to 3 percent. Reclamation and USACE filter design criteria also require the portion of the filter material passing the No. 40 (0.425 mm) sieve be non-plastic (i.e., PI = 0). Cementation increases the likelihood of cracking. Typical cementing agents include carbonate materials (e.g., limestone or dolomite), gypsum, sulfide materials, and volcanic (pyroclastic) ash, particularly for sand-sized particles. Even small amounts of silt in broadly graded, silty sandy gravel transition zones or filters may result in cracking. As suggested by Terzaghi and Peck, a dense well-graded transition zone with a slight amount of silt fines can crack. There is some laboratory evidence that thin (less than 5 feet thick), vertical, clean, partially saturated and compacted filters subject to severe cracking may hold a crack, and a gravel zone downstream of a cracked filter allowed for healing of the cracked filter (Redlinger et al. 2011). Table IV-4-1, which is based on laboratory testing conducted by Park (2003) and field performance data from Foster (1999) and Foster and Fell (1999), provides guidance on assessing the likelihood of a filter material holding a crack. ***The descriptors should be used to help develop a list of more likely and less likely factors during a team elicitation of probability estimates.***

Table IV-4-1. Likelihood of a Material Holding a Crack (adapted from Fell et al. 2004)

Plasticity of Fines	Fines Content, FC (percent)	Likelihood of Holding a Crack	
		Well Compacted	Not Compacted
Non-plastic (and no cementing present)	5 to 7	Unlikely	Very Unlikely
	7 to 15	Likely	Unlikely to Likely
	≥ 15	Very Likely	Likely
Plastic (or fines susceptible to cementing)	5 to 7	Likely	Unlikely to Likely
	7 to 15	Very Likely	Likely
	≥ 15	Virtually Certain	Very Likely

The evaluation for cracking of a filter or transition zone also needs to consider the effect of stress conditions and the presence of flaws or defects in this zone along with consideration of "common causes" for a flaw in the impervious zone. Consideration of "common causes" using the bullet lists of conditions that may lead to an increased likelihood of a flaw existing <u>through the dam</u> (including considerations for conduits through the dam), a flaw through the foundation or from the embankment into the foundation contained earlier in this chapter should be included.

Susceptibility to Segregation

Segregation is the tendency of large particles in a given mass of aggregate to gather together whenever the material is being stockpiled, loaded, transported, placed or otherwise disturbed. Segregation of filter material can cause pockets of coarse zones that may not be filter-compatible with the material being protected. For segregation to be a significant contributor to the likelihood of continuation of internal erosion, an entire lift of the filter zone has to be segregated from upstream to downstream, which is very unlikely except for very narrow zones, and the segregated layer has to correspond with a flaw or concentrated seep in the embankment. For narrow filter zones placed upstream to downstream in one pass, it may be necessary to evaluate the potential for segregation. A common cause of segregation is improper material handling. Material placed in a pile off of a conveyor, or loaded from a chute, or from a hopper segregates because the larger particles roll to the sides of the stockpiles or piles within the hauling unit. Material dumped from a truck, front loader, or other placing equipment almost always segregates, with the severity of the segregation corresponding to the height of the drop, moisture content, and the maximum size of the particles. Soils which are susceptible to internal instability are also susceptible to segregation during placement which aggravates the problem as coarse particles become nested in a matrix of finer particles.

Based on laboratory testing, Kenney and Westland (1993) concluded that all dry soils consisting of sands and gravels segregate in the same general way, independent of grain size and grain size distribution. Dry soils containing particle sizes smaller than 0.075 mm segregate to a smaller extent than soils not containing fines, and water in sandy soils (mean size finer than 3 mm to 4 mm) inhibits segregation but has little influence on the segregation of gravels (mean size coarser than 10 to 12 mm). To minimize segregation during construction, Reclamation and USACE filter design criteria, which limits the amount of fines and oversize material, as shown in Table IV-4-F-2, can be used to help evaluate existing filter/transition zones. Although a minimum D_5F size of 0.075 mm may have been specified in the final in-place product, breakdown may occur during placement and compaction. The filter design criteria also limits the maximum allowable $D_{90}F$ size based on the minimum $D_{10}F$ size, as shown in Table IV-4-F-3.

Table IV-4-F-2. Minimum and Maximum Particle Size Criteria for Filters (adapted from FEMA 2011)

Base Soil Category	Minimum D_5F	Maximum $D_{100}F$
All Categories	≥ 0.075 mm (No. 200 sieve)	≤ 2 inches (75 mm)
Note: USACE (2005) sets maximum $D_{100}F$ at 3 inches (75 mm), maximum FC of 5 percent, and PI of zero.		

Table IV-4-F-3. Segregation Criteria for Filters (adapted FEMA 2011)

Base Soil Category	If Minimum $D_{10}F$ is: (mm)	Then Maximum $D_{90}F$ is: (mm)
All Categories	< 0.5	20
	0.5 – 1.0	25
	1.0 – 2.0	30
	2.0 – 5.0	40
	5.0 – 10	50
	> 10	60

Estimated Gradation after Segregation or Washout

Fell et al. (2008) recommended an approximate method for estimating the $D_{15}F$ of filter materials after segregation or washout. The procedure assumes 50 percent of the finer soil fraction is segregated out or 50 percent of the unstable or erodible soil fraction is washed out. The approximate method for estimating the $D_{15}F$ of the filter material after segregation or washout involves the following steps (shown as red circles in Figure IV-4-F-1):

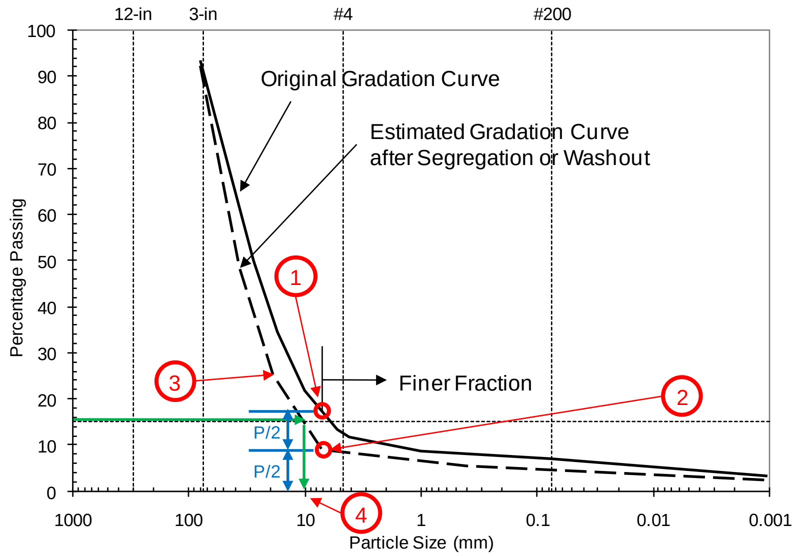
- Step 1: Select the point of maximum curvature on the original gradation curve. For broadly graded soils, the point of maximum curvature is the point of maximum inflection of the gradation curve, as shown in Figure IV-4-F-1a. For gap-graded soils, this point corresponds to the particle size that is missing (i.e., the gap location), as shown in Figure IV-4-F-1b.

- Step 2: Adjust the point of maximum curvature downward by one-half (i.e., locate the midpoint below the point of maximum curvature) because the procedure assumes 50 percent of the finer soil fraction is segregated out or 50 percent of the unstable or erodible soil fraction is washed out.

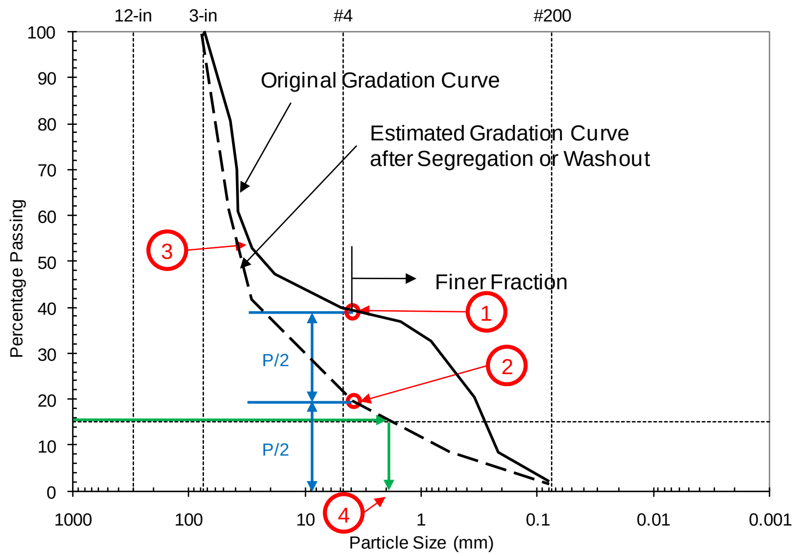
- Step 3: Approximate the shape of the estimated gradation curve after segregation or washout by passing through the midpoint.

- Step 4: Estimate the $D_{15}F$ after segregation or washout using the adjusted gradation curve.

- Step 5: Consider where the fines may have migrated (e.g., plugging something up or lying at the bottom of the layer) and the consequences.



a) Broadly Graded Filter Material



b) Gap-Graded Filter Material

Figure IV-4-F-1. Approximate Method for Estimating $D_{15}F$ after Segregation or Washout

Evaluation of Filters (or Adjacent Materials) not Meeting Modern Particle Retention Criteria

Filter zones and adjacent materials which are coarser than required by modern design methods based on particle size will often be quite effective in controlling internal erosion (Foster and Fell 1999, 2001). Downstream rockfill and sand/gravel zones which were not designed as filters may provide some protection against continuation of internal erosion. In addition, foundation soils can also provide some protection against continuation. Depending on the ratio of particle and pore sizes, the erosion will either:

- Not continue (i.e., no erosion); or
- Stop after only minor erosion (i.e., some erosion); or

- Stop only after a significant amount of erosion (i.e., excessive erosion); or
- Continue (i.e., continuing erosion)

These erosion filter erosion boundaries are conceptually shown in Figure IV-4-F-2.

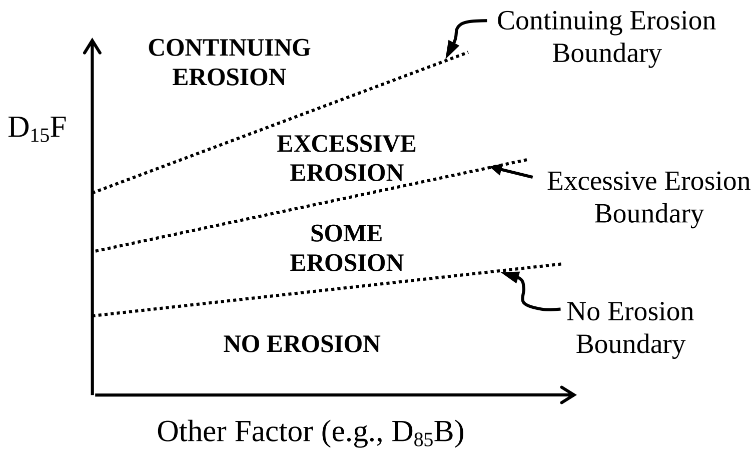


Figure IV-4-F-2. Conceptual Filter Erosion Boundaries (Foster 1999 and Foster and Fell 2001)

The filter evaluation relies heavily on the work of Foster and Fell (2001) to determine no erosion, some erosion, excessive erosion, and continuing erosion boundaries for the base soil. Continuing erosion indicates the base soil could be eroded through the filter without plugging off, and this is the primary focus of the evaluation of the likelihood of continuation of internal erosion. Although internal erosion is expected to initiate for some erosion and excessive erosion, it would eventually plug-off, given time under conditions in the laboratory. The filter testing performed was setup for a vertical downward flow regime. Different orientations in the field need to be considered with caution, especially into the sides of conduits.

Dividing the event tree into branches leading to breach for each of the erosion categories can be considered, particularly if the excessive-continuing erosion portion is high. In addition, if the likely breach mechanism cannot be judged with confidence during the PFMA, estimating the breach probability later could be difficult if the understanding of the mechanism for each erosion category is widely different.

Although Fell et al. (2008) suggest that each of the erosion categories be carried through the event tree, Reclamation and USACE practice has been to come up with one estimate of the likelihood of an unfiltered exit (as discussed earlier), for which a filter evaluation is just one aspect. It is typical to assign the probability of an unfiltered exit based on not just the likelihood of the continuing erosion (CE) boundary, but also *considering* the likelihood of the excessive erosion (EE) boundary, as well as *considering* how far the material is from no erosion (NE) boundary. Reclamation and USACE also consider the variability of the gradations (from fine to coarse extremes), how thick the filtering unit is, how continuous it is likely to be and whether it extends to a free or open face.

Although fairly prescriptive, the assessment is similar to traditional filter evaluation but with more steps, and it can provide a better indication of the likelihood of the core material being filtered even when modern “no erosion” filter criteria are not met in all cases. An example is provided at the end of this chapter. If sufficient gradation exists, the filter evaluation involves the steps described below. If gradation data does not exist or is limited, gradations can be estimated based on the likely source of the materials and any processing, as described in Fell et al. (2008).

- Select representative gradations of the original (or re-graded) base soil (i.e., coarse, average, and fine base soil gradations) based on the fine and coarse base soil envelopes from all gradation tests. For example, if the representative base soil gradation corresponds to 80 percent of all gradation tests, then the fine base soil gradation is indicative of the coarser 10 percent of the base soils, and the fine base soil gradation is indicative of the finer 10 percent of the base soils.

- Assess the no erosion (NE) boundary based on the original (or re-graded) base soil for the coarse, average, and fine base soil gradations using Table IV-4-F-4. For highly dispersive soils (pinhole classification D1 or D2 or Emerson Class 1 or 2), it is recommended to use a lower $D_{15}F$ for the no erosion boundary, as shown in Table IV-4-F-5 based on modern particle retention criteria.

Table IV-4-F-4. Criteria for No Erosion Boundary for Non-Dispersive Soils (adapted from FEMA 2011)

Base Soil Category	Fines Content (percent)	Criteria for No Erosion Boundary
1	FC > 85	$D_{15}F \le 9(D_{85}B)$
2	40 < FC ≤ 85	$D_{15}F \le 0.7$ mm
3	15 < FC ≤ 40	$D_{15}F \le (4(D_{85}B) - 0.7)\left(\frac{40 - FC}{25}\right) + 0.7$ If $4(D_{85}B) < 0.7$ mm, use $D_{15}F \le 0.7$ mm.
4	FC ≤ 15	$D_{15}F \le 4(D_{85}B)$
Notes: The fines content is the percentage finer by weight than 0.075 mm after the base soil is adjusted to a maximum particle size of 4.75 mm.		

Table IV-4-F-5. Criteria for No Erosion Boundary for Dispersive Soils (adapted from FEMA 2011)

Base Soil Category	Fines Content (percent)	Criteria for No Erosion Boundary
1	FC > 85	$D_{15}F \le 6.5(D_{85}B)$
2	35< FC ≤ 85	$D_{15}F \le 0.5$ mm
3	15 < FC ≤ 35	$D_{15}F \le (4(D_{85}B) - 0.5)\left(\frac{40 - FC}{25}\right) + 0.5$ If $4(D_{85}B) < 0.5$ mm, use $D_{15}F \le 0.5$ mm
4	FC ≤ 15	$D_{15}F \le 4(D_{85}B)$
Notes: The fines content is the percentage finer by weight than 0.075 mm after the base soil is adjusted to a maximum particle size of 4.75 mm.		

- Assess the excessive erosion (EE) boundary based on the original (or re-graded) base soil for the coarse, average, and fine base soil gradations using Table IV-4-F-6.

Table IV-4-F-6. Criteria for Excessive Erosion Boundary (adapted from Foster and Fell 1999, 2001)

Base Soil	Criteria for Excessive Erosion Boundary
$D_{95}B \leq 0.3$ mm	$D_{15}F > 9(D_{95}B)$
$0.3 < D_{95}B \leq 2$ mm	$D_{15}F > 9(D_{90}B)$
$D_{95}B > 2$ mm and FC $\leq$ 15 percent	$D_{15}F > 9(D_{85}B)$
$D_{95}B > 2$ mm and 15 percent < FC $\leq$ 35 percent	$D_{15}F > 2.5\left((4(D_{85}B) - 0.7)\left(\frac{35 - FC}{20}\right) + 0.7\right)$
$D_{95}B > 2$ mm and FC > 35 percent	$D_{15}F$ > ($D_{15}F$ value for erosion loss of 0.25g/cm^2 in the CEF test, as shown in Figure IV-4-F-3, can be estimated as $D_{15}F \approx 0.34(1.07)^{fm}$ by curve-fit)
Notes: Criteria are directly applicable to soils with $D_{95}B$ up to 4.75 mm. For soils with coarser particles, determine $D_{85}B$ and $D_{95}B$ using gradation curves adjusted to give a maximum size of 4.75 mm.	

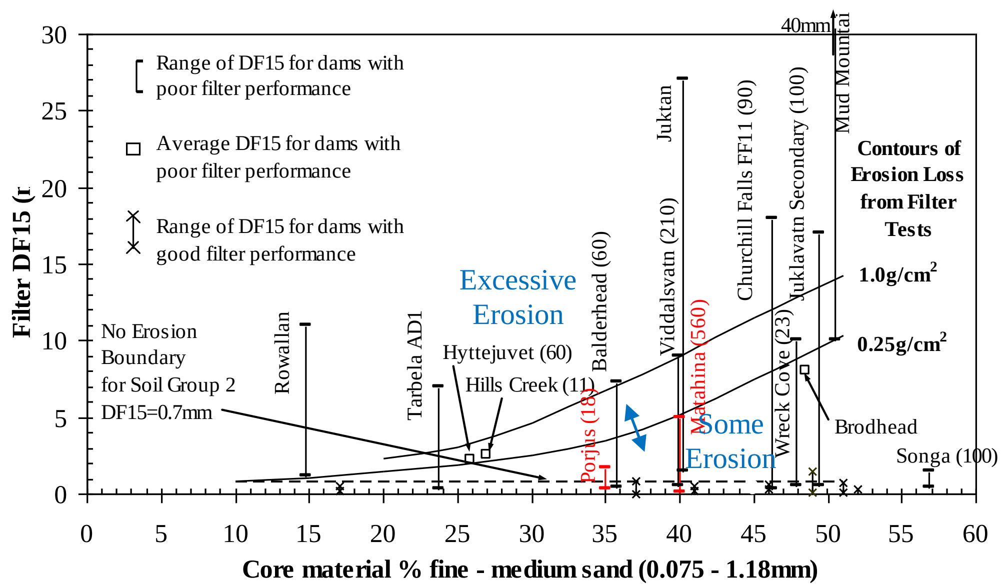


Figure IV-4-F-3. Criteria for Excessive Erosion Boundary (adapted from Fell et al. 2008)

- Assess the continuing erosion (CE) boundary based on the actual (or re-graded) base soil for the coarse, average, and fine base soil gradations. For all soils, this is estimated as $D_{15}F > 9(D_{95}B)$ (Foster and Fell 1999, 2001).

- Plot the erosion boundaries on the original filter gradation curves (and the adjusted filter gradation curves for segregation or washout) on the D_{15} line.

- Estimate the proportion of the original filter gradation (and filter gradation after segregation or washout) within each of the erosion categories for the coarse, average, and fine base soil gradations. The suggested approach is to estimate the proportions for the continuing, excessive, and some erosion categories first and then calculate the proportion for the no erosion category by subtracting the sum of the other proportions from one.

Coarse base soil gradation: $P_{NE,\ coarse} = 1 - (P_{CE,\ coarse} + P_{EE,\ coarse} + P_{SE,\ coarse})$
Average base soil gradation: $P_{NE,\ average} = 1 - (P_{CE,\ average} + P_{EE,\ average} + P_{SE,\ average})$
Fine base soil gradation: $P_{NE,\ fine} = 1 - (P_{CE,\ fine} + P_{EE,\ fine} + P_{SE,\ fine})$

- Make an initial estimate of the probabilities of no erosion, some erosion, excessive erosion, and continuing erosion by calculating the sum-product of the percentage of base soil gradations and the estimated percentage of no erosion, some erosion, excessive erosion, and continuing erosion for the coarse, average, and fine base soil gradations. The calculations are as follows, where N corresponds to the representative base soil gradation (i.e., as a percentage of all gradation tests) and $n = (100 - N)/2$ corresponds to the percentage finer or coarser of the base soil:

$P_{NE} = (n/100)(P_{NE,\ coarse}) + (N/100)(P_{NE,\ average}) + (n/100)(P_{NE,\ fine})$
$P_{SE} = (n/100)(P_{SE,\ coarse}) + (N/100)(P_{SE,\ average}) + (n/100)(P_{SE,\ fine})$
$P_{EE} = (n/100)(P_{EE,\ coarse}) + (N/100)(P_{EE,\ average}) + (n/100)(P_{EE,\ fine})$
$P_{CE} = (n/100)(P_{CE,\ coarse}) + (N/100)(P_{CE,\ average}) + (n/100)(P_{CE,\ fine})$

- If the filter gradation is finer than the continuing erosion boundary, Fell et al. (2008) suggest using Table IV-4-F-7 to estimate the probabilities of continuing erosion (based on how much finer the gradations are compared to the continuing erosion boundary) to allow for the possibility of the gradations being coarser than indicated by the available information. ***The probabilities should not be used directly in a risk assessment, but rather used to help develop a list likely of more likely and less likely factors during an elicitation of probability estimates.***

Table IV-4-F-7. Probability of Continuing Erosion when the Actual Filter Gradation Is Finer than the Continuing Erosion Boundary (adapted from Foster and Fell et al. 2008)

$D_{15}F$	P_{CE}
$< 0.1(D_{15}F_{CE})$	0.0001
$< 0.2(D_{15}F_{CE})$	0.001
$< 0.5(D_{15}F_{CE})$	0.01 – 0.05

Example Filter Evaluation

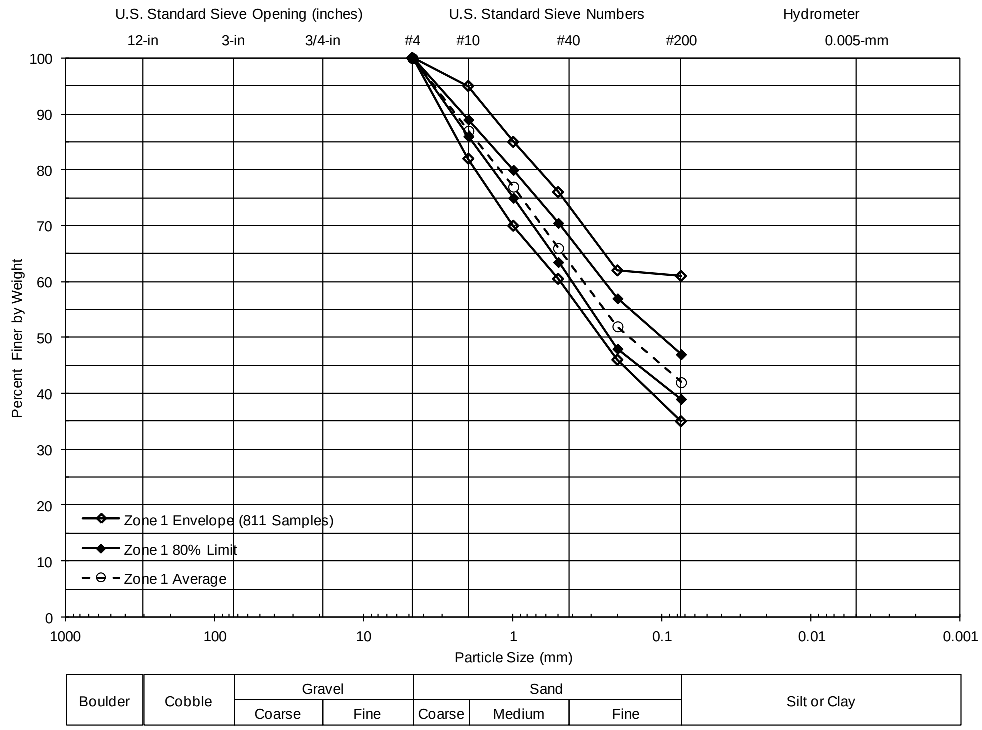


Figure IV-4-F-4. Example Re-Graded Base Soil

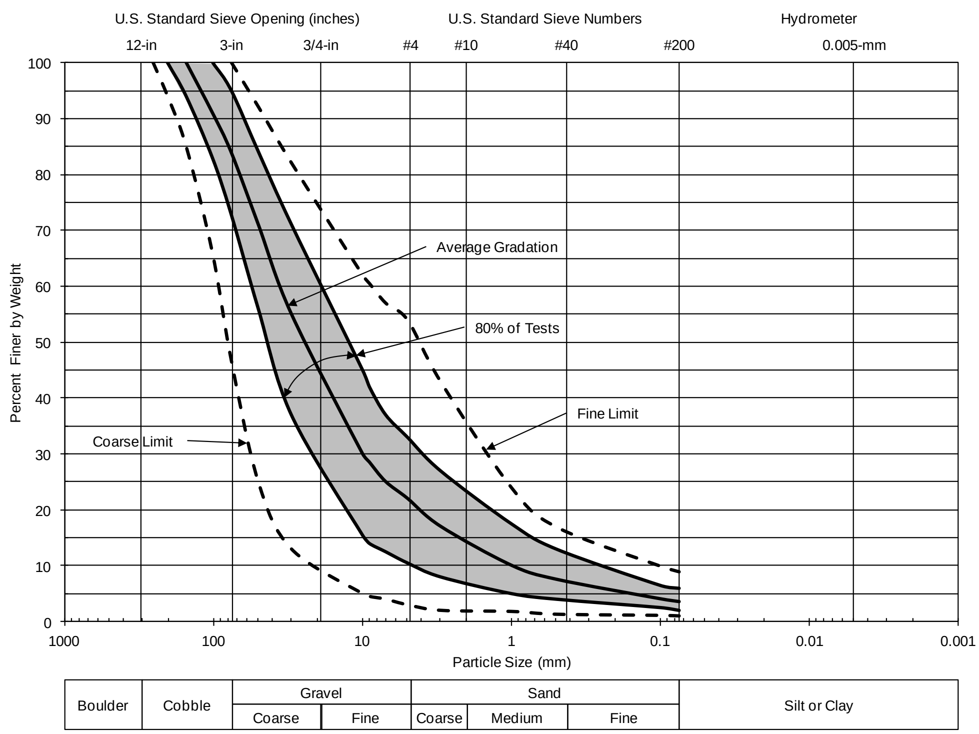


Figure IV-4-F-5. Example Filter Gradations

- Assess if the filter materials are susceptible to cracking. The fines content of the representative boundary filter gradations in Figure IV-4-F-5 is between about 2 and 6 percent. Based on Table IV-4-F-1, the likelihood of the filter material holding a crack would be small, especially for non-plastic fines.

- Assess if the filter materials are susceptible to segregation. Based on Table IV-4-F-3, the boundary filter gradations in Figure IV-4-F-5 indicate the limits to prevent segregation are not met by a large margin. The minimum $D_{10}F$ of about 0.25 mm correspond to a maximum $D_{90}F$ to prevent segregation of 20 mm. However, the maximum $D_{90}F$ is actually about 120 mm. Using the average filter gradation, the minimum $D_{10}F$ is about 1 mm, and the maximum $D_{90}F$ is about 95 mm. The maximum allowable $D_{90}F$ is more like 30 mm, and again the criteria to limit segregation are not met.

- Assess if the filter materials are susceptible to internal instability. Based on Reclamation criteria, the ratio of say the $D_{80}F$ to the $D_{10}F$ particle sizes is much greater than 4, and the likelihood of internal instability appears to be small. However, the filter gradation curve was judged to have a flat tail of fines, which may be susceptible to internal instability.

- Estimate the gradation after segregation or washout using the procedure of Fell et al. (2008). The adjusted gradation curve is shown on Figure IV-4-F-7.

- Assess the no erosion (NE) boundary based on the original (or re-graded) base soil for the coarse, average, and fine base soil gradations. The re-graded fines content of the base soil in Figure IV-4-F-4 is between 35 and 61 percent. Based on Table IV-4-F-4, the fines content corresponds to Base Soil Category 2 and a no erosion boundary of $D_{15}F < 0.7$ mm.

- Assess the excessive erosion (EE) boundary based on the original (or re-graded) base soil for the coarse, average, and fine base soil gradations. Based on Table IV-4-F-6, the base soil is best classified as a soil with $D_{95}B > 2$ mm and FC > 35 percent. This requires determining the excessive erosion boundary from Figure IV-4-F-3 using the percentage of material between 0.075 and 1.18 mm (defined as fine to medium sand). The results are summarized in Table IV-4-F-8.

- Assess the continuing erosion (CE) boundary based on the actual (or re-graded) base soil for the coarse, average, and fine base soil gradations as $D_{15}F < 9(D_{85}B)$. The results are summarized in Table IV-4-F-8.

Table IV-4-F-8. Erosion Boundaries for Example Base Soil

Core Gradation	Base Soil Characteristics			No Erosion	Excessive Erosion	Continuing Erosion
	$D_{95}B$ (mm)	FC (%)	f-m Sand (%)	$D_{15}F$ (mm)	$D_{15}F$ (mm)	$D_{15}F$ (mm)
Coarse	4.0	35	39	0.7	5	36
Average	3.5	42	39	0.7	5	32
Fine	2.0	61	28	0.7	2	18

- Plot the erosion boundaries on the original filter gradation curves (and the adjusted filter gradation curves for segregation or washout) on the D_{15} line. The erosion boundaries are shown on Figure IV-4-F-6 for the original filter gradation and Figure IV-4-F-7 for the adjusted filter gradation after segregation or washout.

- Estimate the proportion of the original filter gradation (and filter gradation after segregation or washout) within each of the erosion categories for the coarse, average, and fine base soil gradations. By inspection, the approximate proportions of the gradation band within each erosion boundary are summarized in Table IV-4-F-9. The proportions for the no erosion category are calculated below.

  For the original filter gradation:
  Coarse base soil gradation: $P_{NE,\ coarse} = 1 - (0 + 0.30 + 0.60) = 0.10$
  Average base soil gradation: $P_{NE,\ average} = 1 - (0 + 0.30 + 0.60) = 0.10$
  Fine base soil gradation: $P_{NE,\ fine} = 1 - (0.05 + 0.45 + 0.40) = 0.10$

  For the adjusted filter gradation after segregation or washout:
  Coarse base soil gradation: $P_{NE,\ coarse} = 1 - (0.02 + 0.58 + 0.40) = 0$
  Average base soil gradation: $P_{NE,\ average} = 1 - (0.04 + 0.56 + 0.40) = 0$
  Fine base soil gradation: $P_{NE,\ fine} = 1 - (0.10 + 0.80 + 0.10) = 0$

Table IV-4-F-9. Proportions for Example Filter Material

Base Soil Gradation	NE	SE	EE	CE
Original Filter Gradation				
Coarser (10%)	0.10	0.60	0.30	0.00
Average (80%)	0.10	0.60	0.30	0.00
Finer (10%)	0.10	0.40	0.45	0.05
Adjusted Filter Gradation after Segregation or Washout				
Coarser (10%)	0.0	0.40	0.58	0.02
Average (80%)	0.0	0.40	0.56	0.04
Finer (10%)	0.0	0.10	0.80	0.10

- Make an initial estimate of the probabilities of no erosion, some erosion, excessive erosion, and continuing erosion by calculating the sum-product of the percentage of base soil gradations and the estimated percentage of no erosion, some erosion, excessive erosion, and continuing erosion for the coarse, average, and fine base soil gradations. The calculations are as follows, where N corresponds to the representative base soil gradation (i.e., as a percentage of all gradation tests) and n = (100 –N)/2 corresponds to the percentage finer or coarser of the base soil:

For the original filter gradation:
P_{NE} = (10/100)(0.10) + (80/100)(0.10) + (10/100)(0.10) = 0.1
P_{SE} = (10/100)(0.60) + (80/100)(0.60) + (10/100)(0.40) = 0.58
P_{EE} = (10/100)(0.30) + (80/100)(0.30) + (10/100)(0.45) = 0.315
P_{CE} = (10/100)(0.00) + (80/100)(0.00) + (10/100)(0.05) = 0.005

For the adjusted filter gradation after segregation or washout:
P_{NE} = (10/100)(0.00) + (80/100)(0.00) + (10/100)(0.00) = 0
P_{SE} = (10/100)(0.40) + (80/100)(0.40) + (10/100)(0.10) = 0.37
P_{EE} = (10/100)(0.58) + (80/100)(0.56) + (10/100)(0.80) = 0.586
P_{CE} = (10/100)(0.02) + (80/100)(0.04) + (10/100)(0.10) = 0.044

- The probability of continuation without considering other factors (e.g., filter thickness, continuity of coarse zones, presence of a free face, etc.) could be estimated on the low side as the probability of continuing erosion or 0.005 for the original filter gradation and 0.044 for the adjusted filter gradation after segregation or washout. If it were judged that there was a 10 percent chance of the segregated or washed out filter being in contact with the core, the minimum probability of continuation could be estimated as 0.1(0.044) + 0.9(0.005) ≈ 0.01. The maximum probability of continuation is based on examining the excessive and some erosion boundaries in Figures IV-4-F-6 and IV-4-F-7. For example, if it were judged that there was about a 50 percent chance that soil within the excessive erosion category would not eventually plug off but practically no chance that soil within the some erosion category would not plug off, then the probability of continuation for the original filter gradation could be estimated as 0.005 + 0.5(0.315) ≈ 0.16. Similarly, the probability of continuation for the adjusted filter gradation could be estimated as 0.044 + 0.5(0.586) ≈ 0.34. The weighted maximum probability of continuation is 0.1(0.34) + 0.9(0.16) ≈ 0.2. Therefore, probability of continuation is ranges from about 0.01 to 0.2.

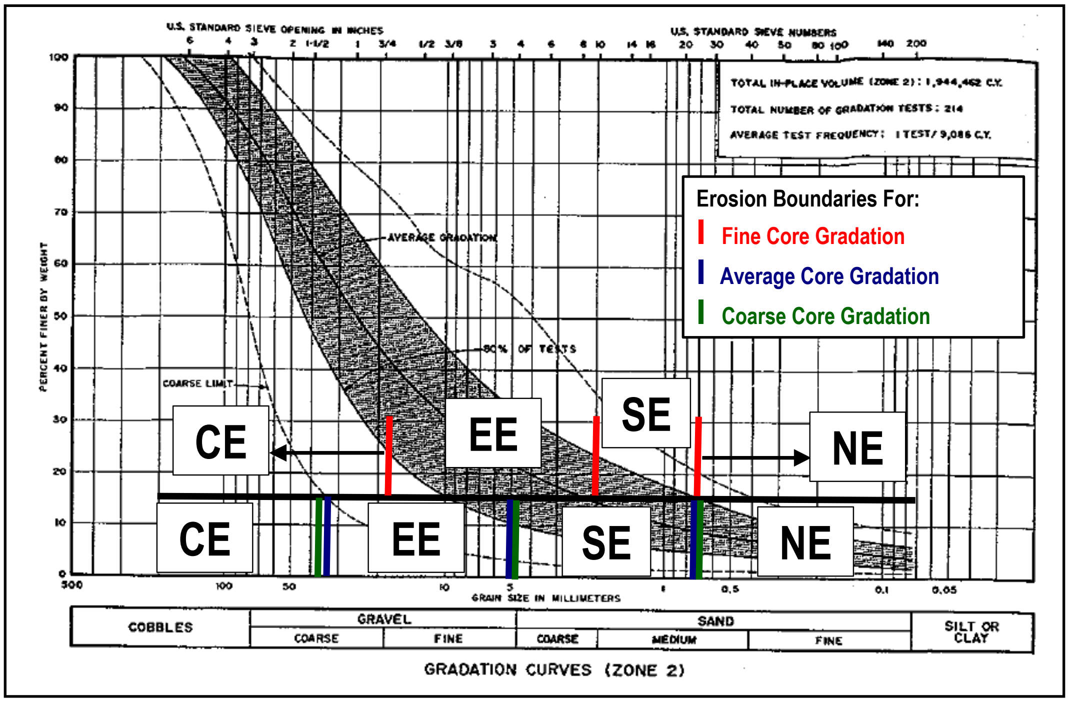


Figure IV-4-F-6. Erosion Boundaries on $D_{15}F$ of Original Filter Gradation

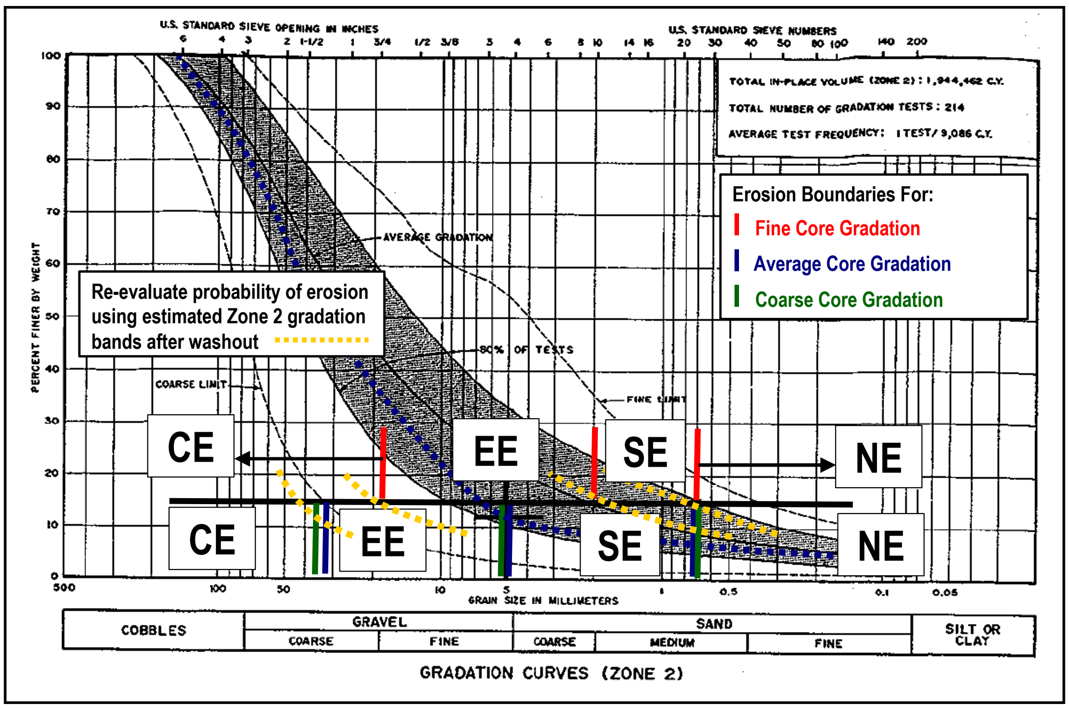


Figure IV-4-F-7. Erosion Boundaries on $D_{15}F$ of Adjusted Gradation after Segregation or Washout

Appendix IV-4-G: Rate of Enlargement of a Pipe

The rate of enlargement of a pipe in the progression phase can be estimated using methods described in Wan and Fell (2002) for a circular pipe. The rate of erosion per unit surface area at time t is given by:

$$\dot{\varepsilon}_t = \frac{1}{\psi_t}\frac{dV_t}{dt} = k_d\left(\tau - \tau_c\right) \text{ for volume erosion}$$

$$\dot{m}_t = \frac{1}{\psi_t}\frac{dM_t}{dt} = C_e\left(\tau - \tau_c\right) \text{ for mass erosion}$$

where $\Psi_t = P_{w,t}$ L = surface area of the pipe at time t; dV_t/dt = rate of soil volume removal due to erosion at time t; dM_t/dt = rate of soil mass removal due to erosion at time t; τ = hydraulic shear stress for the reservoir level under consideration; τ_c = critical shear stress for initiation of erosion; and k_d = erodibility coefficient; and C_e = coefficient of soil erosion..

Using the above equations, the erosion loss (per unit length) can be rewritten as:

$$dV_t = \dot{\varepsilon}_t\ \Psi_t\ dt = k_d(\tau - \tau_c)(P_w)\ dt = k_d(\tau - \tau_c)\ (\pi\phi_t)\ dt \text{ for volume erosion}$$

$$dM_t = \dot{m}_t\ \Psi_t\ dt = C_e(\tau - \tau_c)(P_w)\ dt = C_e(\tau - \tau_c)\ (\pi\phi_t)\ dt \text{ for mass erosion}$$

The change in pipe diameter at time t is given by:

$$d\phi_t = 2[dV_t/(\ \pi\phi_t)] \text{ for volume erosion}$$

$$d\phi_t = 2[dM_t/(\rho_d\ \pi\phi_t)] \text{ for mass erosion}$$

These equations can be readily setup in a spreadsheet to estimate the pipe diameter for user-specified time increments or steps based on estimates of hydraulic shear stress and erodibility parameters previously described and the following assumptions:

- Linear head loss from upstream to downstream
- Steady uniform flow along the pipe
- Zero pressure head at the downstream end
- Shape of the pipe remains circular
- Enlarging pipe can sustain a roof
- Uniform frictional resistance along the surface of the pipe or crack
- Driving force = frictional resistance
- Reservoir remains constant with time

An example of portrayal of analytical results is shown in Figure IV-4-G-1. In this example, an initial pipe diameter was assumed, and the critical shear stress, erodibility coefficient, and pipe diameter at failure were estimated by a risk team during an elicitation. Based on the estimated pipe diameter as a function of time, this figure can be used to help develop a list or more likely and less likely factors for the potential time available for intervention or full breach development as a function of reservoir level.

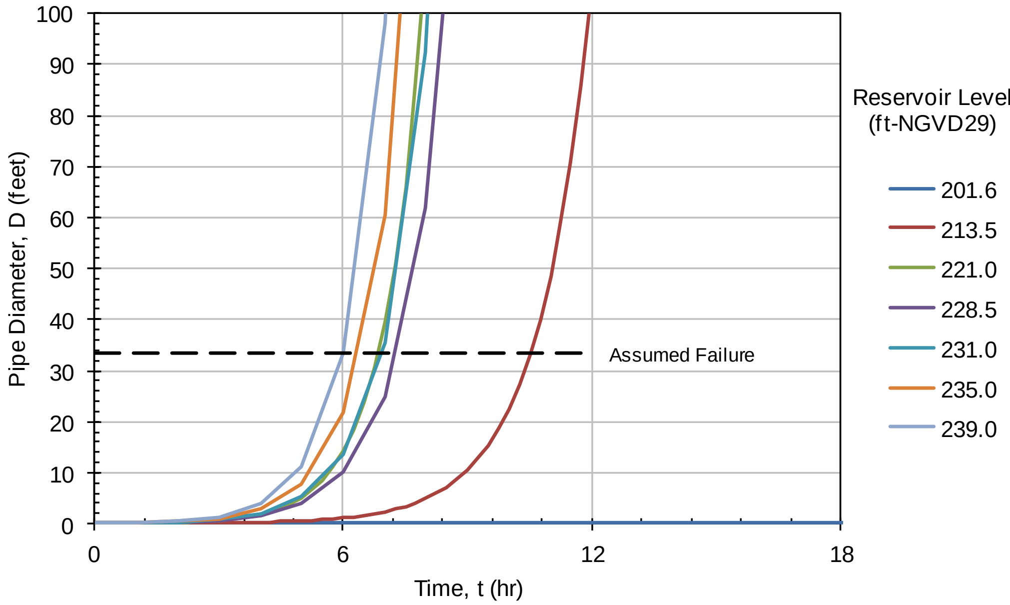


Figure IV-4-G-1. Example Portrayal of Analytical Results for Rate of Enlargement of a Pipe

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