INVESTIGATION AND EVALUATION OF SEEPAGE CONDITIONS AND POTENTIAL FAILURE MODES AROUND OUTLET CONDUITS

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ABSTRACT

Evaluation of potential seepage failure modes in embankment dams and in particular around outlet works penetrations is a very important aspect of dam safety evaluations. Assessing potential seepage failure modes typically requires designing and evaluating investigation and instrumentation programs. Experience has shown that adverse seepage and piping conditions can develop and remain difficult to detect until the failure mode is in an advanced stage of the continuation phase of the failure mode development process (Appendix O, ER 1110-2-1156). In this paper both the theoretical considerations and practical observations of seepage conditions around conduits will be supported with case history information from two large outlet works conduits through embankment dams on relatively deep alluvial foundations: Lake Darling Dam, North Dakota, and Lake Isabella Auxiliary Dam, California. In both cases, large twin barrel cast-in-place reinforced concrete outlet conduits were constructed with partial cut and cover methods. The conduits were placed on relatively thick alluvial foundation soil deposits and operated for extended periods of time before detailed safety evaluation studies were initiated. These case histories reveal how the identification of small unfiltered defects in and around the conduits is critical to the assessment of the potential for the initiation of seepage and piping failure modes. Once initiation has occurred, the small and insidious nature of erosion pipes that develop during the early stages of the failure mode makes direct detection almost an impossible task. Even with extensive investigation and instrumentation monitoring programs, the assessment of the safety of these structures is a very difficult task and requires extensive experience and keen engineering insight and judgment.

INTRODUCTION

<u>Seepage Failure Mode Continuum</u>

Fell et al (2003) describes a four phase process for the development of internal erosion and piping failures in embankments initiated from a concentrated leak. The phases include 1) Initiation, 2) Continuation, 3) Progression, and 4) Breach Failure. These phases have proved to be an effective partitioning of the failure mode development process required for the development of system response event trees and quantitative risk analysis. They have been established as part of the US Bureau of Reclamation, Best Practices for Dam Safety Risk Analysis (2010).

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The author, working as part of a six member team of experts for the Peer Review of six of the initial DSAC-1 (Dam Safety Action Classification) dams under the Corps risk informed dam safety program in 2005 and 2006, identified some important considerations and clarification of the Continuation and Progression phases of the failure mode development process. These clarifications were summarized in a "continuum" figure used by the Peer Review team in assessing the DSAC-1 dams and has been included by the Corps in Appendix O of ER 1110-2-1156. A version of this figure that was first published in the ASDSO Dam Safety Journal (Halpin and Ferguson, 2007) is shown on Figure 1. The failure mode continuum illustrated by the arrow pointing from left to right also indicates the stages of failure mode development and the corresponding types of intervention strategies that may be available to arrest further development in most circumstances. As a general rule, the rate of failure mode development in different dams is not the same, and in fact, can vary significantly. In same cases, the development of a failure mode may take 10 to 20 years or more, whereas in other cases, the failure mode may develop rapidly on first filling of the reservoir. For large flood control structures, it may take 50 to 100 years for a complete first filling to take place. In those instances where the failure mode development process is slow, it may take many cycles of reservoir filling. Initially, the failure mode develops only episodically. However, the rate of development can begin to increase rapidly in the later stages of the continuation phase and then through the progression and breach formation phases.

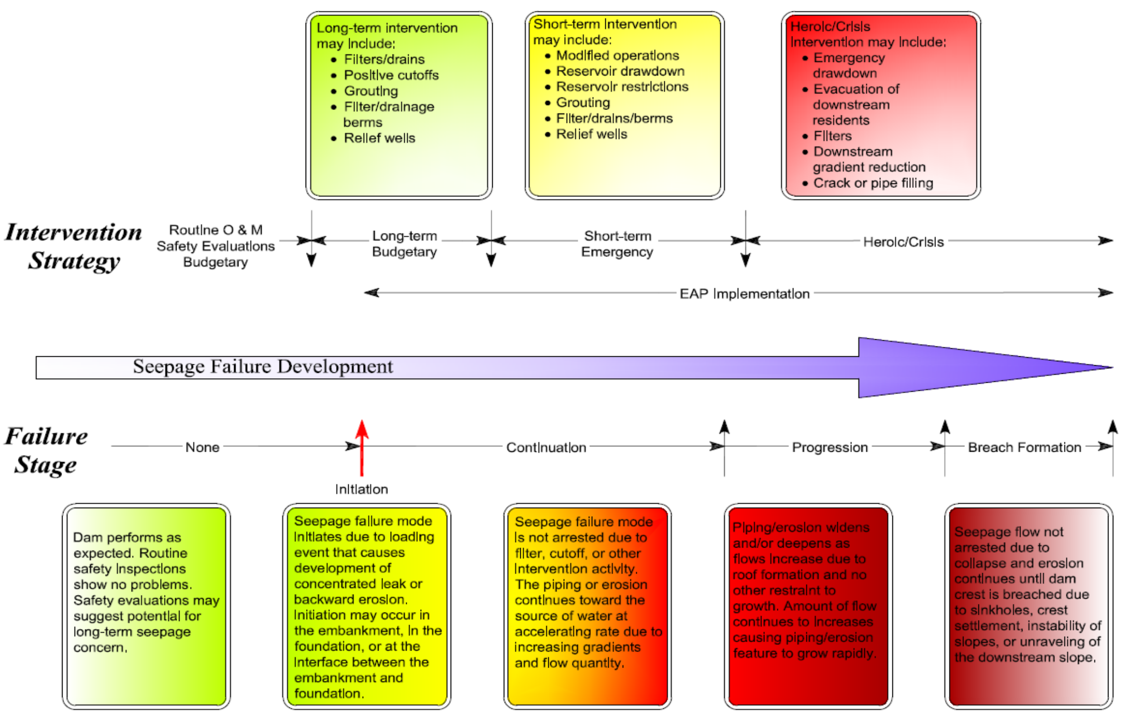

Figure 1. Seepage Failure Mode Continuum.

The factors that may affect the overall rate of failure mode development include:

- Erodibility of the foundation or embankment materials– fine uniform sands, to sandy silts to silts, and dispersive clays are the most significant concern
- Seepage gradients at the point of initiation and at the advancing front of the piping/erosion feature. Gradients sufficient to move soil particles are required to sustain the erosion and piping process
- The permeability of the material where the erosion is occurring and the corresponding volume/velocity of flow.
- The presence of layers of materials that can sustain a roof or otherwise contribute to the failure mode development process. Likewise, the swelling and/or collapse of material above and around a developing piping feature can significantly reduce the rate at which the failure mode develops over time.

Some further discussion of these factors, with illustrations from a generalized seepage model is presented in a later section of the paper.

Potential Failure Modes to be Considered

The types of failure modes that can develop in foundation and/or embankment materials around an outlet conduit have been described in a variety of study reports and publications (FEMA, 2005). The author suggests that the following seepage related potential failure modes should always be considered as a starting point of all safety evaluations of conduits penetrating embankment dams. It should be noted that each site and the corresponding loading conditions (normal, flood, and seismic); slope stability; and design, construction, and operational details will require the engineer to consider potential variations within these PFM's as part of the evaluation and risk analysis process.

PFM #1 – Piping/erosion along backfill/conduit Interface

PFM #2 – Piping/erosion along backfill/native soil or bedrock Interface

PFM #3 – Piping/erosion into unfiltered defects in the conduit or related conduit structures/drain pipes

PFM #4 - Piping/erosion through vertical or horizontal cracks developed through differential settlement and hydraulic fracturing processes

PFM #5 - Piping/erosion as a result of a combination of the mechanisms identified in PFM's #1 through #4 above that combine to create a continuous pathway for the initiation, continuation and progression phases of the failure mode development process.

PFM #6 - General piping/erosion failure modes in embankment and/or foundation materials in areas adjacent to the outlet conduit that may combine with the failure mode processes associated with #1 through #4 above.

CRITICAL CONSIDERATIONS FOR ASSESSING SEEPAGE CONDITIONS AROUND OUTLET PENETRATIONS

There are a number of factors that may make the development of the seepage related PFM's around a conduit described in the preceding section more likely to occur. The potential for each of these factors should be considered prior to the development of any investigation and instrumentation program. These factors include:

a) The potential for, and location of unfiltered defects that contribute to development of concentrated leaks
   - Cracks in conduit floors and walls (a variety of settlement, seismic response or other loading conditions can lead to formation of cracks or can damage water stops in joints with the same end result)
   - Cracks or defects in drain pipes along side or under the conduit or terminal structure
   - Structural configurations that leads to concentration of seepage flows
   - Toe drain pipes in the vicinity of conduit

b) Site Geology
   - Foundation conditions that contribute to differential settlement in the vicinity of the conduit
   - Continuous layers of erodible materials such as uniform medium to fine sand, medium to fine silty sand, sandy silt and silt.
   - Bedrock defects such as relatively open joints, fractures, or Karst. These defects can transmit relatively high water pressures and flows to vulnerable areas under the dam and adjacent to the conduit. If such defects were left untreated, water pressure and seepage flows can attack vulnerable soils at the interface.
   - Dispersive clays in the foundation, or that may have been used for the central core, homogeneous embankment construction, or structural fills around the conduits
   - Internally unstable grading of embankment or foundation soils

c) Design and construction details
   - The configuration of any embankment closure activities around the outlet conduit that can contribute to the potential for differential settlements/movements that cause reduction in principal stresses on vertical or horizontal planes that can be a root cause for hydraulic fracturing
   - The presence of seepage collars around the conduit
   - The shape of the excavation for the conduit and the characteristics of the exposed foundation materials,

- Treatment of the excavation and backfill interface
- Compaction of backfill materials including the contact between the backfill and the exterior walls of the conduit
- Presence or lack of filters, drains, and water stops at critical locations

d) General settlement under and around the conduit and the related potential for reduction in principal stresses that increase the potential for hydraulic fracturing immediately around the conduit

e) Presence of multiple factors described above that can combine to create failure mode pathways

SOME THEORETICAL ASPECTS OF THE FAILURE MODE DEVELOPMENT PROCESS

There are two topics areas related to seepage conditions around conduits that lend themselves to generalized modeling that can improve our understanding of potential failure mode development processes. This modeling can therefore help clarify how these topics should be considered in the development of investigation/instrumentation programs, and related safety evaluations. These topics include 1) the characteristics of piping features and the rate of their formation, and 2) settlement below and around conduits and the related effects on principal stresses and the potential for hydraulic fracturing. These two topics are discussed further in the following sections.

Formation of Piping Features

Results from the analysis of a simplified two dimensional seepage model can be used to illustrate a number of key factors related to the formation of piping features and the overall failure mode development process. Further, these results can be used to help clarify the differences in the Continuation and Progression phases. The simplified model is illustrated on Figure 2 and consists of a homogeneous embankment dam constructed of a clayey and silty, sand with gravel over a 100-foot-deep, two layered soil foundation consisting of an upper 15 foot thick layer of material ranging from a silty fine sand to sandy silt, and a lower layer of clayey and silty sand with gravel. The upper foundation layer is relatively loose while the lower layer is dense (over consolidated). An outlet penetration was constructed by excavating down to the top of the over consolidated clayey and silty sand with gravel. The outlet conduit consists of two side-by-side, cast-in-place reinforced concrete conduits. The conduits are separated by a reinforced concrete dividing wall, are about 10 feet high and about 6 feet wide each (inside dimension). The material properties for the embankment and foundation layers in the model are summarized in Table 1.

The simplified model was created to help understand the seepage and general water pressure characteristics that may be associated with the development of a piping feature in the upper silty sand and sandy silt at the interface with the outlet penetration excavation backfill (PFM #2 described above). Additional details on this actual case history are presented in a later section. The purpose here is to look at the results of a simplified model and what the results help us to understand about the failure mode development process. This in turn can be used to guide the selection of methods that could be used to explore, instrument, and evaluate the potential for a defect to be developing in an area around the conduit.

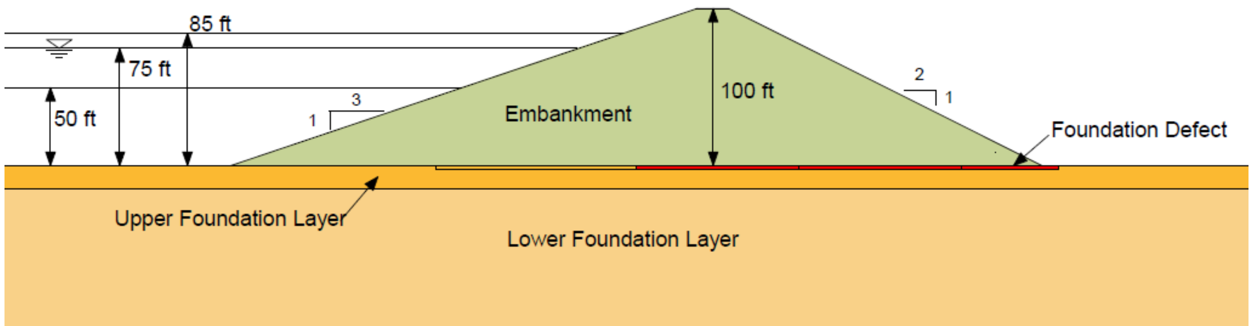

Figure 2. Generalized Seepage Model

Table 1. Summary of Material Properties Used in Seepage Model

Material	**Horizontal Permeability (cm/sec)$^{(1)}$**	**$K_h/K_v^{(2)}$**
Embankment	1×10^{-5}	9
Upper Foundation silty sand to sandy silt	1×10^{-4} 1×10^{-3} (high)	4
Lower Foundation silty and clayey sand with gravel	1×10^{-5}	9
Erosion Defect	1×10^{-2} 1×10^{-1} (high)	1

Notes: (1) cm/sec – centimeters per second
(2) K_h/K_v – Ratio of horizontal (K_h) to vertical (K_v) permeability

The variables considered in the analysis include the reservoir level, and the extent to which a defect has developed in the upper foundation materials. The embankment height was set at 100 feet and reservoir levels of 85, 75, and 50 feet or 85, 75, and 50 percent of the embankment height were evaluated. Seepage conditions at the downstream embankment toe prior to initiation of a potential failure mode along with defects extending from the downstream toe to a location that is 10, 30, 50, and 75 percent of the length of the base of the dam toward the reservoir were considered. It should be noted that this is a 2- dimensional model while the actual development of the piping or erosion feature, and the corresponding regime of water pressures (equipotential lines) and gradients that would develop in the vicinity of the defect is a three dimensional problem. Hence the prediction of gradients and flow quantities from a 2-dimensional model may not be conservative. Flow lines and equipotential lines are converging to the advancing face of the pipe not only in the upstream-downstream direction as simulated by the 2-dimensional model, but from the sides of the developing defect. Likewise, the flow into such a defect would not only occur at the advancing face as predicted by a 2-dimensional model, but also flow into the pipe from each side for some distance downstream. Hence the actual gradients and flow quantities into the active front and upstream portion of a pipe or erosion defect could be substantially higher than the 2-dimensional model results indicate.

The seepage model was created using SEEPW (Geo-Slope International, Ltd, version 6.17). A defect was simulated by including a relatively thin, high permeability layer with various amount of penetration under the dam. The permeability of the defect was set to be two to three orders of magnitude higher than the surrounding materials.

The results of the analyses showing the estimated seepage gradients in the embankment and foundation materials for different extents of defect development are illustrated on Figure 3. The extent of the defects is shown on each cross-section in red. A summary of the estimated seepage gradients at the active face of the defect along with the estimated seepage quantity flowing into the upstream end of the defect are summarized in Table 2.

Table 2. Summary of Seepage Analysis Results

Extent of Seepage Defect Development	Reservoir Level 85 feet		Reservoir Level 75 feet		Reservoir Level 50 feet	
	Avg. Gradient at Entrance to Defect (i_{ave})	Est. Flow into Defect (Q gpm/ft)	Avg. Gradient at Entrance to Defect (i_{ave})	Est. Flow into Defect (Q gpm/ft)	Avg. Gradient at Entrance to Defect (i_{ave})	Est. Flow into Defect (Q gpm/ft) (3)
Downstream Toe	0.6 to 0.7 (1)	.002	0.4 (1)	.002	0.3 (1)	.001
10%	0.4 (2)	.005	0.3 (2)	.004	0.2 (2)	.002
30%	0.5 (2)	.005	0.4 (2)	.005	0.3 (2)	.003
50%	0.8 (2)	.007	0.7 (2)	.006	0.4 (2)	.004
75%	0.9 (2)	.010	0.8 (2)	.009	0.6 (2)	.006

Notes: (1) Gradient is primarily vertical immediately downstream of the dam toe
(2) Gradient is predominantly horizontal at entry to the simulated defect.
(3) gpm/ft – gallons per minute per foot of dam width

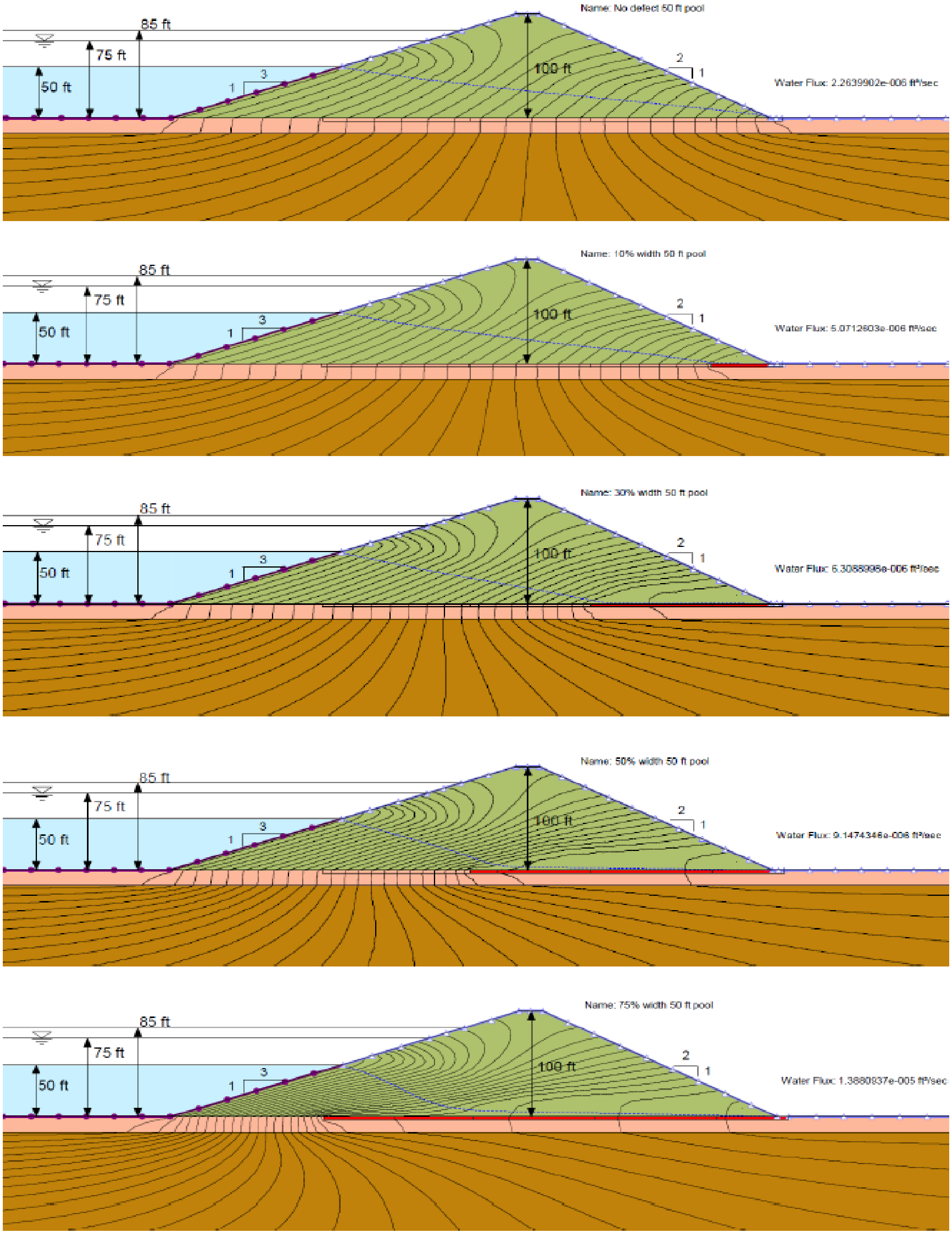

Figure 3. Equipotential Line Plots for Zero, 10, 30, 50, and 75 Percent Defect Extension under the Dam.

Some general observations and conclusions from this analysis are as follows:

1. Prior to initiation of a pipe or erosion feature, the seepage exit gradients at the downstream toe are relatively high but not high enough to cause initiation of a potential failure mode unless an unfiltered defect is present that allows for the development of a concentrated leak where sufficient gradient, flow volume and velocity develop to cause particle movement.
2. Once initiated, sufficient gradients, flow velocity, and flow quantity can exist at the active face of a developing pipe or erosion defect under higher reservoir levels to sustain the erosion process in the highly erodible soils. Initially, the erosion process would activate at only the higher reservoir water levels, and would be highly episodic during those high reservoir water elevations. The erosion would stop once the reservoir level drops and the gradients and seepage quantities reduce. If a roof forming material is present above the erosion feature, the defect would likely remain open but dormant until the next period of sustained higher reservoir levels occurs.
3. The seepage gradients at the active front of the pipe are increasing as the defect advances upstream. As shown in Table 2, the gradients increase from about 0.4 to 1.0 as the defect moves from about 10% to 75% of the distance along the base of the dam. Consequently, the reservoir level required to reactivate the erosion and pipe development process decreases over time. For example, a gradient of 0.5 or higher develops at the active front of the pipe when the feature has advance to 30% of the distance toward the upstream toe with the reservoir at 85% of the embankment height. A similar gradient exists at a reservoir level of about 75% when the defect has advance from 40 to 50% of the distance to the upstream toe and a reservoir elevation of only 50% is required to cause a 0.5 or higher gradient once the active front of the defect when it has advanced to about 75% of the distance toward the upstream toe. Consequently, the rate of development/advancement would generally increase over time.
4. However, the other interesting observation is that for the conditions modeled, the rate and corresponding velocity of seepage flow in the defect, while increasing, remains relatively small, even when the defect has developed up to 75% of the distance toward the upstream toe. This suggests that while the rate of advancement is increasing, the size of the defect, which would be a function of the volume and velocity of flow, is not changing substantially. If the circumference and hence the effective diameter of the pipe is a direct function of the quantity of flow, the model results indicate that the circumference and effective diameter would have only increased by a factor of about five from the time of onset of the continuation phase to the point where the defect has advance 75% of the distance toward the upstream toe of the dam. This would likely be the case until the defect reaches a point when the amount of flow into the defect would significantly increase. In this case, that would not occur, until the defect has reached the reservoir at the upstream toe.

5. Another important observation has to do with the nature of the equipotential lines downstream of the active front of the defect. The model shows that the overall seepage gradient would be very flat relative to the gradient in the vicinity of the active front of the defect. Downstream of the active front, the gradients do not change substantially for significant changes in the reservoir level. This observation suggests that a sufficient number of instruments at appropriate locations are required to properly characterize seepage conditions and evaluate the initiation and extent of potential failure mode development. In other words, a couple of instruments at the downstream toe and perhaps a couple of instruments installed through the dam at the crest will not be sufficient to detect if a failure mode has developed and is active.
6. The final observation that could be made has to do with the rate at which the failure mode would develop once the break through to the reservoir occurs. At this time, unless there was some lateral constraint, or unless the collapse of the material above the piping feature were to occur that would arrest or limit the failure mode development, the amount of time to failure would likely be relatively small. There would likely be some form of additional breakout of the seepage along the downstream toe and the ability to mitigate the failure mode would be substantially reduced or not possible.

Based on these representative results and the experience of the author with a variety of evaluations of seepage conditions in embankment dams and their foundations, including the special considerations around conduit penetrations, it is recommended that the following distinction be made in the Continuation and Progression phases of potential seepage related failure modes:

> **Continuation:** begins immediately following initiation and lasts for the period when the piping feature remains relatively small, when the rate and size of the erosion feature is controlled by the permeability of the material where the erosion is occurring. The primary considerations for assessing the continuation phase (and the corresponding considerations for designing any investigation program including instrumentation) includes 1) continuity and erodibility of the layer where the pipe/erosion will occur, 2) presence of roof forming materials that supports and sustains the development of the pipe/erosion feature without collapse that may disrupt the erosion process, and 3) ability to sustain gradients that cause erosion at the active front of the pipe/erosion feature.
>
> **Progression:** begins immediately when a relatively small pipe/erosion feature in the continuation phase breaks through to a high permeability material or water source that results in a significant increase in the flow volume and velocity in the pipe/erosion feature. The increase in flow volume and velocity causes gross enlargement of the feature. The primary considerations for assessing the progression phase includes 1) any lateral or vertical restraint that would limit the amount of flow that can occur and the

process of gross enlargement, and 2) presence of material in the foundation or dam above the erosion feature, that upon collapse could clog the erosion feature and arrest the progression process.

Hydraulic Fracturing Considerations

The phenomenon of hydraulic fracturing in embankment dams has been investigated and described by a number of well known authors since the 1940's but began receiving increased attention in the early 1970's (Sherard et al, 1972, and Sherard, 1985).

The locations in dams that are vulnerable to hydraulic fracturing are those where differential settlement causes elongation of embankment material, internal stress transfer, and a corresponding reduction in principal stresses. Stress reduction, in certain instances can become tensile and cause cracks to form under self stresses prior to filling of the reservoir. Alternatively, stress reduction may reduce the principal stresses to a point where water pressures and corresponding stress changes that develop in the dam or foundation during first reservoir filling exceed the strength of the materials resulting in failure and the formation of cracks. Once the cracks form, water pressures can easily jack these fractures open allowing a significant increase in the flow volume and velocity to occur and increasing the potential for the failure mode development process to advance. The locations where this phenomenon is known to have occurred include:

1. Adjacent to steep abutments
2. Significant or abrupt changes in the foundation surface profile
3. Around outlet penetrations where settlement of adjacent embankment and foundation materials is larger than the settlement of the conduit.

An example where this can occur relative to an outlet conduit is illustrated on Figure 4.

Hydraulic fracturing can occur even in dams without unusually large or concentrated differential settlement. Case histories as well and finite element studies indicate that hydraulic fractures may develop on near vertical transverse planes caused by the longitudinal stretching of the embankment and/or foundation materials in the region of differential settlement, near horizontal planes caused by arching of self weight forces. This phenomenon can occur

- across the core as a result of the differential settlement between the central core and shells of the dam,
- across a relatively narrow but deep cutoff trench,
- around localized foundation defects that traverse the foundation under the low permeability cutoff,
- other orientations due to a more complex combination of actions

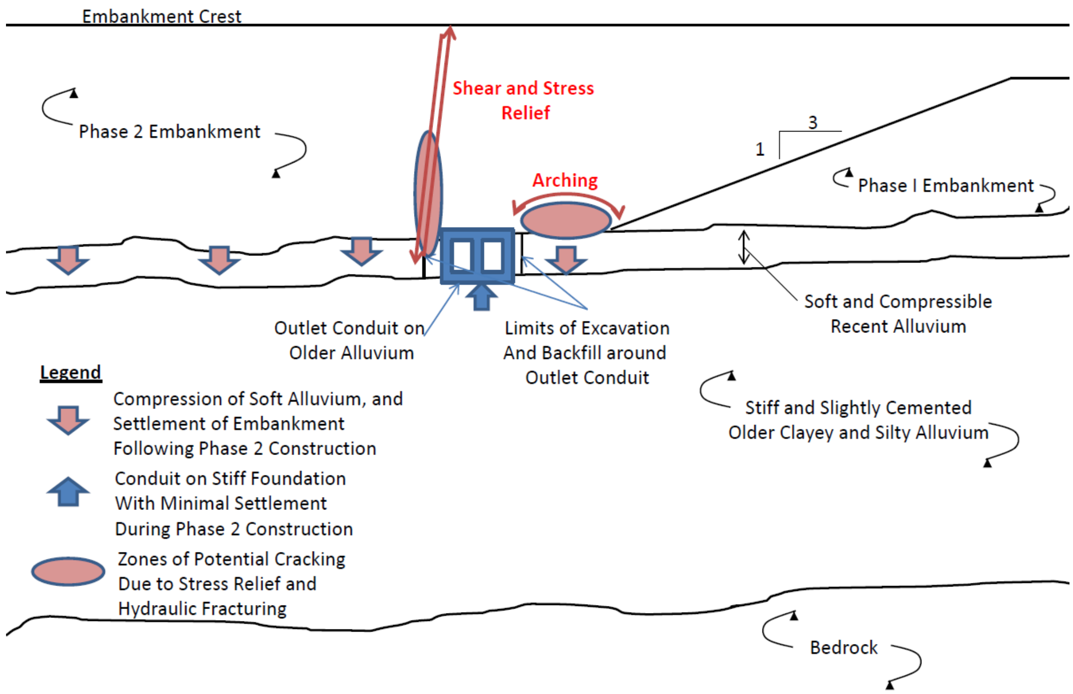

Figure 4 – Locations where Cracking and Hydraulic Fracturing Could Develop Around an Outlet Conduit

Failures associated with hydraulic fracturing typically occur in embankment without internal filters. Failure happens much more easily and commonly in dams built of highly erodible dispersive clays. Other more highly erodible soils are also a special concern.

Boreholes drilled for explorations, installation of instrumentation, or for remedial grouting present a special consideration related to hydraulic fracturing. Specifically when drilling in areas that have experienced significant stress transfer as described above, the potential for drilling to cause hydraulic fractures to form is high. Precautions required to safely drill in dams, particularly the low permeability core or embankment materials include (ER 1110-1-1807):

1. Hollow stem or sonic drilling methods (note that special precautions are required to prevent embankment damage or disturbance due to differential water pressures between the inside and outside of casing when removing center plugs or casing from the borehole)
2. Cased rotary wash with mud only in special circumstances
3. Never drill with water or air!

OTHER CONSIDERATIONS FOR INVEWSTIGATIONS AND PFM EVALUATIONS

There are a number of general considerations when evaluating PFM's around conduits and penetrations in embankment dams and developing investigation and instrumentation programs to support these evaluations.

Potential for Initiation

i. As discussed above initiation of a seepage/ piping failure mode will most likely occur at any unfiltered seepage discharge location (an open joint or crack), where a concentrated leak develops. Investigation programs must first and foremost focus on finding such defects or where seepage concentration can occur that results in an unfiltered concentrated leak.
ii. Flowing water in conduits and structures can magnify the potential for initiation and also mask the fact that a concentrated leak has developed carrying soil particles. The only way to find some critical defects will be to dewater the conduit(s) and terminal structures for careful inspection and assessment.

Continuation

iii. Initial piping and erosion features will be small and will remain relatively small until the volume and velocity of flow is sufficient to significantly increase their size.
iv. Piping and erosion are episodic. Seepage may appear to be clear the majority of time (corresponding to only small traces of particle movement) until the continuation phase is nearly complete. Detection of particle migration will require seepage to be collected and discharged through a weir box with sufficient retention time to allow soil particles to settle and be observed.
v. The assessment of the continuation phase must consider the potential for concentration of seepage gradients and flow at the active face of the erosion feature. At this location, it is possible for the seepage gradient to be many times greater than that required to carry soil particles.
vi. Erosion will follow the path of least erosion resistance. Unless there is a preexisting defect or pathway, the location of the erosion feature will likely be insidious and very difficult to directly encounter.
vii. Detection of, and the assessment of the corresponding extent of continuation that has occurred will require a well designed system of instrumentation and a keen eye for identifying trends in the instrumentation readings.

CASE HISTORY #1 – LAKE DARLING DAM, ND

A detailed description of the Lake Darling Dam outlet works investigation and related rehabilitation program that was completed by the US Fish and Wildlife Service in 1988 was previously presented by the author (Ferguson, 1994). Lake Darling Dam is a high hazard zoned embankment dam located on the Souris River above the towns of Burlington and Mino, North Dakota. It was constructed in 1935, has a structural height of 35 feet, a normal storage capacity of 112,000 acre-feet, a crest length of over 3000 feet and a tributary drainage basin of over 9,000 square miles. The primary outlet works at the time consisted of twin 10 ft. x 14 ft. cast-in-place concrete conduits with cast iron roller gates under the maximum section of the dam. Foundation conditions below the outlet works consisted of over 70 feet of interbedded alluvial sand, silt and clay soils in turn over a claystone bedrock.

The Figure 5 below, taken from Figure 2 in that publication shows some of the details of the outlet conduit. Examination of the information in this figure indicates that the conduit is relatively large compared to the height of the embankment. There was a significant possibility of differential settlement caused by differential loading of slightly compressible foundation (C_c, Compression Index approx. 0.05 to 0.1). The differential load between the base of the conduit and the adjacent base of the embankment was about 1200 psf. This likely resulted in a total differential settlement along the profile of the dam under the crest in the area adjacent to the conduit of perhaps about 4 to 5 inches. From a cross-sectional perspective, the total maximum differential settlement of the conduit was about 4 inches (the difference in settlement under the crest and at the ends of the structure near the upstream or downstream toes of the dam).

Minimal precautions were taken in the design and during construction to minimize the adverse foundation conditions below the conduit. Specifically, camber or flexible construction joints with water stops were not included to address potential settlement of the structure. Likewise, the embankment did not include any chimney filter/drain system that would address hydraulic fracture or cracking potential that may occur as a result of differential settlement in the vicinity of the conduit. During installation of an isolated tip piezometer in Boring B-106 drilled from the crest of the dam and adjacent to the outlet conduit, a significant loss of backfill grout occurred. The loss of grout appeared to occur at or below about the midpoint of the conduit (about elevation 1580). The stratigraphy encountered in the borings indicated that the embankment material was a low plastic clay (CL) and the foundation material immediately below the dam was a variety of uniform fine sand (SP), well graded sand (SW) silty sand (SM, SW-SM) to sandy silt (ML), to clayey sand (SC) and sandy clay (CL). Some of the foundation material would classify as highly erodible.

The characteristics of the grout loss and the actions required to complete installation of the instrument suggested that the loss was associated with a pre-existing void that had formed adjacent to the conduit. The discussion of hydraulic fracturing presented above indicates that hydraulic fracturing that had previously occurred or was caused by the column of backfill grout in the borehole may have also been a contributing factor.

The conduit was subsequently dewatered and a detailed evaluation of the structure was completed. The investigation program was completed with a sequence of activities aimed at determining the root cause of the grout loss in the adjacent boring. The investigation program included a concrete condition and settlement survey, and a series of non-destructive geophysical tests. Once this information was evaluated, a series of core holes were drilled through the floor and exterior walls of the conduit. The concrete condition survey indicated that there was significant settlement of the conduit that lead to the formation of a series of uncontrolled cracks in the conduit floor and lower portions of the walls. Cracks up to one-inch wide were documented and were flowing water at some locations at the time of the investigation. The observed cracking was typical of conduits constructed during this period where precautions such as camber and periodic construction joints with water stops were not included. Cracking was typically spaced on intervals of 10 to 20 feet. The maximum measured settlements of the conduit ranged from 0.2 to 0.3 feet under the maximum section of the dam. Geophysical surveys included Impulse Response (IR) to evaluate support conditions and Impact Echo (IE) to evaluate the condition of the concrete. The IR tests along the floor suggested numerous locations where void/poor or questionable subgrade conditions existed. Coreholes were drilled at a series of both good and poor subgrade locations. The core samples were carefully inspected and dimensions were recorded to the nearest one-sixteenth of an inch. The depth to subgrade was then measured and compared to the length of the core sample to determine the size (if any) of the voids beneath the floor or adjacent to the walls. Voids up to 2-inches were found at several of the corehole locations. Clear water was flowing from the holes where voids were detected. In general, the results of the geophysical (IR) tests were confirmed. However, one sample location that showed poor subgrade response did not have a void and likewise one location showing good support conditions had a void of about ½-inch. The voids were typically found where fine sand, silty sand, sandy silt and silt foundation materials were found. At one of the void locations upgradient of a large crack, course sand and some small gravels to about three-eights of an inch in size were found at the base of the void.

A remediation program was subsequently designed and constructed that included a comprehensive program of low pressure grouting around the conduit. The program was carefully observed and the work progressed from downstream to upstream along the floor and walls of the conduit to maximize the potential to intersect voids and completely fill the voids with what would classify today as a moderately stable and mobile grout material. Additional details of the grouting and overall remediation program that included relief wells and installation of a new floor with proper filters for the cracks that had formed is provide in the reference publication above.

Significant grout takes occurred at many locations (significant is defined as more than 2 cubic feet). In several instances, grout flow occurred from as many as four or five open holes ahead of the grouting operations in the floor. A total of about 20 cubic feet of grout was pumped into voids around the conduit. The most significant grout take occurred in the downstream portion of the west wall, a location where the exploration program did not suggest the potential for voids to occur. In addition, grouting along the east wall in the vicinity of boring B-106 where the previous grout loss occurred did not encounter any large or significant voids.

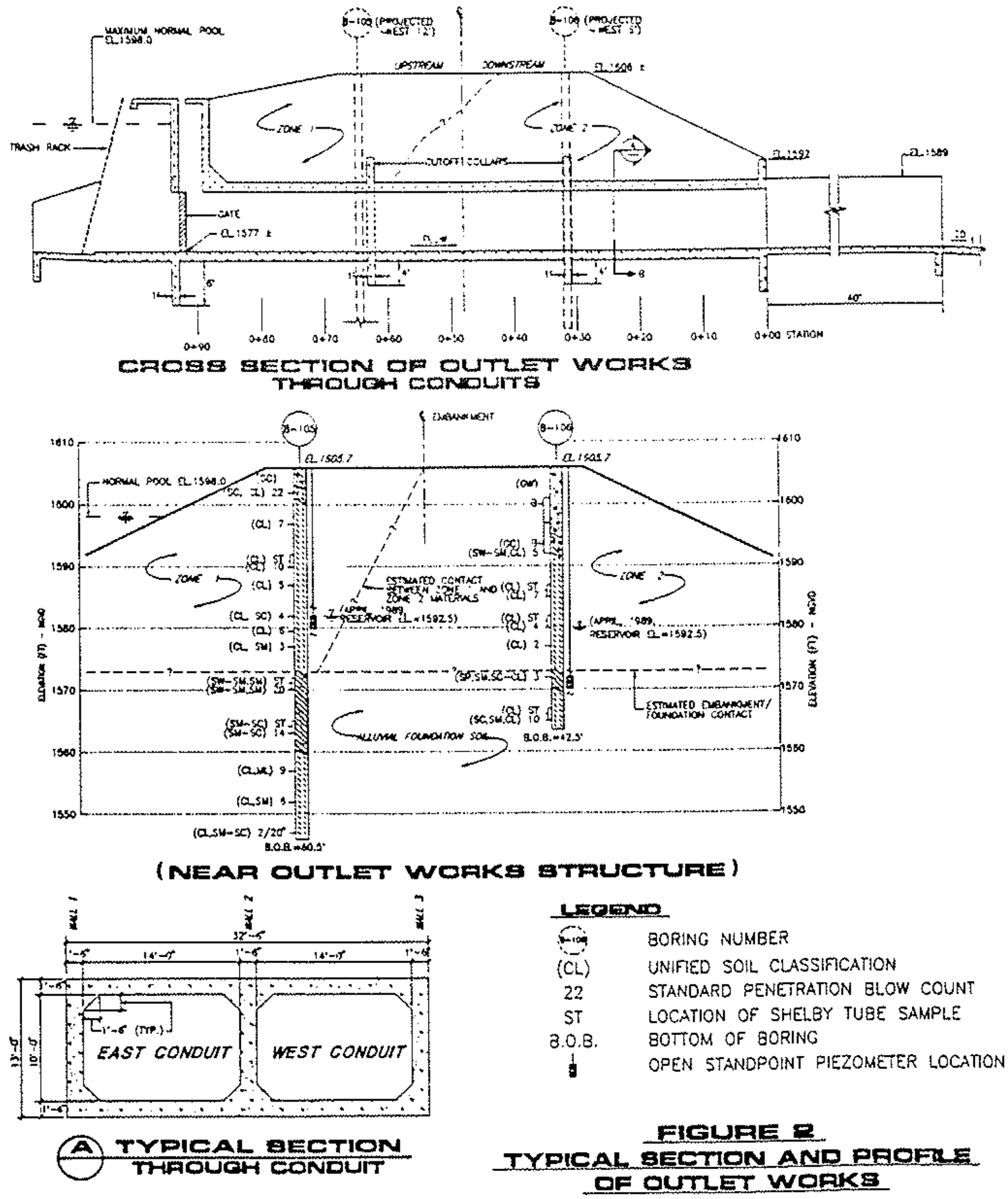

Figure 5. Lake Darling Dam Primary Outlet Works at the time of the 1988 Investigations and Dam Safety Evaluation Studies

While there may still be some questions regarding the root cause of the initial grout loss and the role that hydraulic fracturing may have played, the subsequent exploration and grouting program encountered voids below the conduit and along the lower conduit wall (one location). The overall conclusion would be that the cracking of the conduit floor and lower walls provided the critical mechanism for the development of concentrated leaks at a location of highly erodible foundation soils and that initiation of a seepage failure mode had occurred. The pattern of interconnected grout discharge from open grout holes indicated that the failure mode was in the continuation phase and may have advanced to a location under the crest of the dam.

CASE HISTORY #2 – ISABELLA AUXILIARY DAM AND BOREL CONDUIT, CA

Figure 6. Isabella Auxiliary Dam

A description of the Isabella Main and Auxiliary dams and reservoir along with details regarding the construction of the Auxiliary Dam and the Borel Canal conduit penetration beneath it have been presented previously by Ferguson et al (2007), Ferguson (2010), and Serefini et al (2012).

The configuration of the Auxiliary Dam and the location of the Borel Canal conduit are shown on Figure 6. The Auxiliary Dam is a homogeneous, rolled earthfill structure with a crest length of 3,257 feet, a top width of 20 feet, a crest elevation of 2633.5 feet and also has 6.5 feet of freeboard above the original Spillway Design Flood elevation. The foundation of the Auxiliary Dam consists of heterogeneous valley fill alluvium with a maximum depth of about 130 feet over highly and deeply weathered granitic bedrock.

The Auxiliary Dam was built in two phases and the typical plan view showing the two phases along with a simplified profile and cross-section of the embankment at the conduit is shown on Figure 7.

The Borel Canal was originally constructed between 1897 and 1904 and ran through the reservoir area prior to project construction. The Auxiliary Dam was built over the canal and provisions to keep the canal and associated hydroelectric power plant in operation were made by the construction of the Borel Canal Conduit. The conduit consists of a 525-foot long doublebarrel cast-in-place reinforced-concrete structure. The two rectangular gate chambers have opening dimensions of 5 feet - 8 inches x 10 feet separated by a 1 foot - 8 inch thick reinforced concrete wall. A vertical control tower was constructed on top of the conduit's gate chamber 55 feet upstream of the centerline of the dam. A profile and cross-section of the conduit is shown on Figure 8.

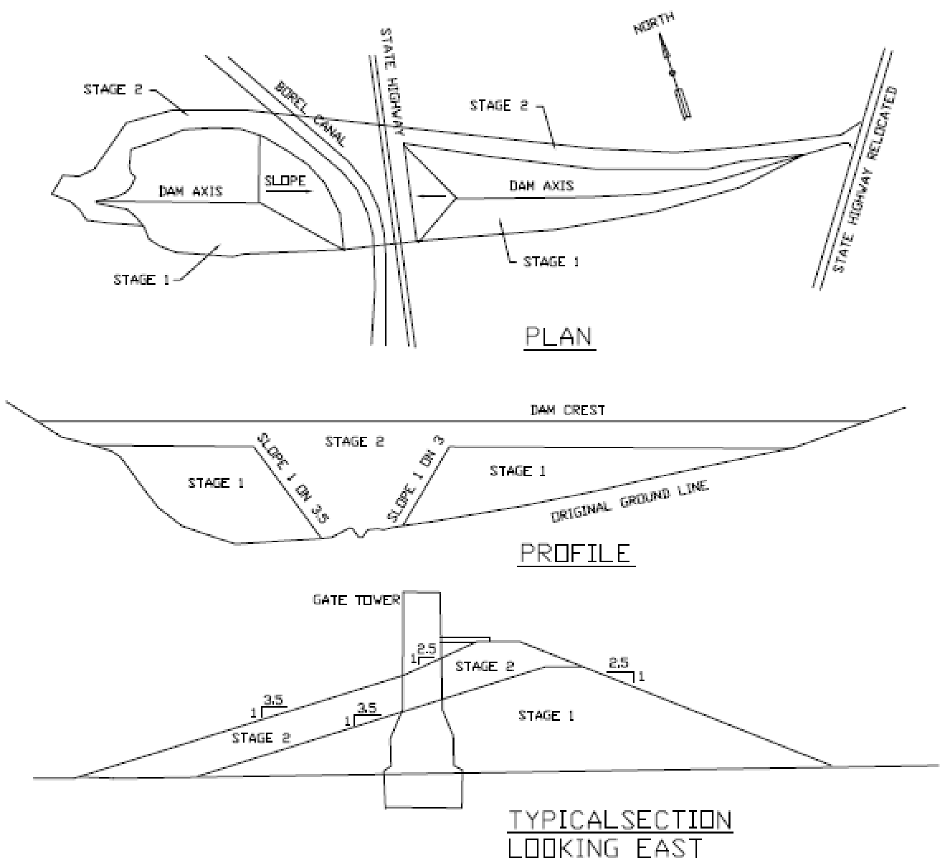

Figure 7. Plan View and Typical Cross Section of the Isabella Auxiliary Dam

The Corps had observed a number of distress indicators during exploration programs along the downstream toe of the dam, and in instrumentation readings in the vicinity of the conduit. They subsequently requested an evaluation of the conduit as part of a dam safety evaluation program.

The Borel Canal conduit is a relatively smaller conduit in relation to the height and cross section of the embankment when compared to the Lake Darling Dam outlet described in the preceding section. The conduit itself was founded on a relatively stiff material (lower foundation layer), and the adjacent embankment was founded on an upper foundation layer having a thickness of about 10 to 15 feet and a compression index, C_c of around 0.1 to 0.2. As noted above, the embankment was built in two phases with the closure section and dam raise being the second phase. The left side of the conduit (looking downstream) was located near the end of the eastern Phase 1 embankment. The Phase 1 embankment was constructed and the foundation beneath the embankment was allowed to consolidate for over 6 months before the Phase 2 embankment and closure section was completed.

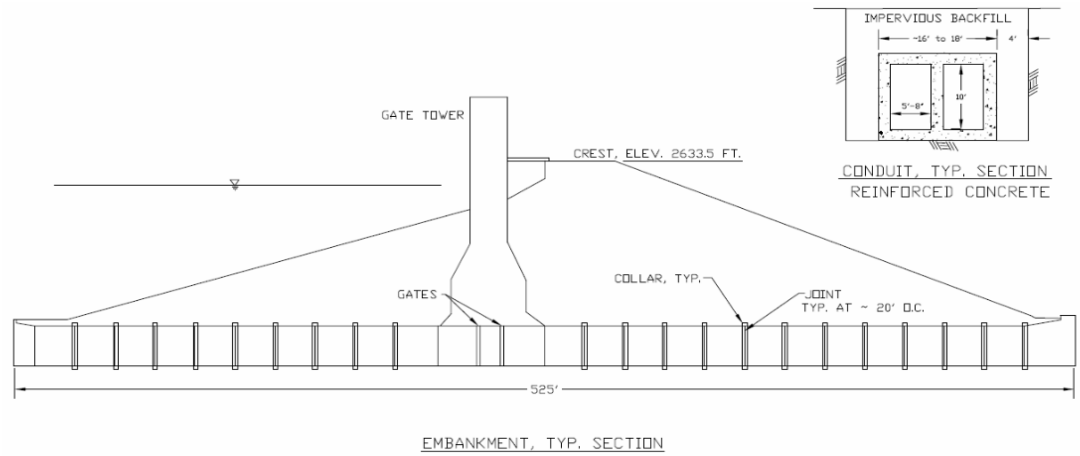

Figure 8. Profile and Cross-section View of the Borel Canal Conduit

The configuration and sequence of construction of the conduit and Auxiliary dam embankment created a complex settlement and related stress history in the area in and around the conduit. Differential settlement would have occurred and would have been a complicated three-dimensional influence on stresses in the embankment. In addition to the issues associated with differential settlement, a number of other important considerations in each of the categories summarized in Section 2, above were identified as potentially influencing seepage conditions around the outlet conduit. These considerations included:

- A series of seepage cutoff collars had been installed around each of the construction joints in the conduit. The construction joints included provisions to minimize the potential for differential settlement or offsets at the joints and had flexible water stops around the entire perimeter.
- Construction drawings indicated that there was a small (4-inch diameter) drain pipe installed outside of the conduit that ran from the control tower to the downstream toe of the dam. This drain pipe provided a means of dewatering small quantities of water that may accumulate in the lower portion of the tower. At the downstream toe of the dam, this tower drain pipe had been connected to a open joint (bell and spigot) drain pipe that turned and ran nearly parallel to the toe of the dam for several hundred feet and was then connected to an outfall that discharged into the upper end of a toe drain system extending along the downstream toe of the Auxiliary dam. The condition of the tower drain pipe, the open-jointed drain pipe, and the toe drain system was unknown.
- There were reports of concentrated leaks through the monolith joints of the conduit

- Previous investigations suggested that a highly erodible foundation silty sand, sandy silt material existed in the upper foundation soils under the dam and around the conduit.
- Construction photographs of the conduit excavation and construction suggested that the shape and treatment of the excavation as well as the ability to properly place and compact backfill materials in the excavation created a potentially disturbed pathway from a seepage failure mode standpoint. In other words, there were many aspects of the conduit that needed to be considered in performing an investigation and assessment of seepage safety.

Prior to designing and completing an investigation of the conduit, members of the study team identified eleven different potential failure mode pathways around the conduit. Using these PFM's, a 3 phase investigation program was designed and completed:

Phase 1; Vertical borings along both sides of the conduit from the dam crest, midslope, and downstream toe areas. Open standpipes with vibrating wire piezometers were installed in the borings in the silty sand/sandy silt upper foundation layer, along with a few instruments in the upper portions of the lower foundation layer.

Phase 2; Dewatering and detailed condition assessment of the conduit including careful inspection and evaluation of each monolith joint, along with ground penetrating radar surveys and evaluation of backfill and foundation conditions from within the conduits.

Phase 3; Coreholes were drilled through the walls and floor of the conduits, and soil samples of the backfill and adjacent foundation soils were taken. Seepage conditions around the conduit were observed and vibrating wire piezometers were installed in these borings to also measure water pressures in the foundation silty sand/sandy silt material.

The layout of the exploration program is generally shown on Figure 9. As part of the investigation two significant locations were identified where concentrated seepage were discharging and creating the potential for initiation of a seepage failure mode.

Location #1: Tower and toe drain piping system – a video inspection of the open-jointed portion of this system identified not only the open joints, but what appeared to be a break in this pipe. Debris and sediment was also observed in the pipe.

Location #2: Defects in the Canal lining immediately downstream of the conduit discharge structure. A number of pin boils were observed in the bottom of the canal lining when it was dewatered for the inspection program. Examination of these locations suggested there was unfiltered discharge occuring through the openings. A later investigation of the lining where a more complete dewatering and cleanup was performed indicated a number of distress locations including the damaged lining at the location immediately downstream of the conduit discharge structure shown on Figure 10. In fact, cloudy water was observed discharging from this location during the time the water in the canal and conduit was being drawn down for the investigation.

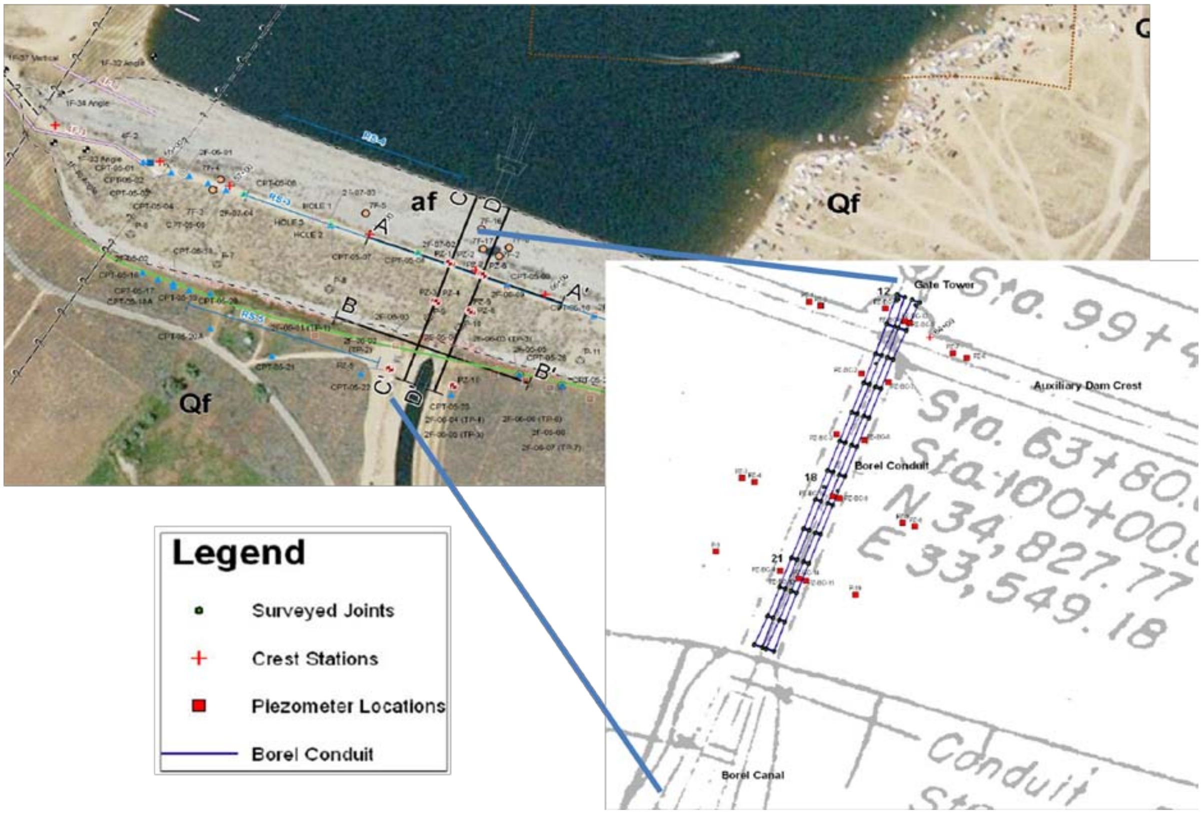

Figure 9. Layout of Investigation and Instruments for assessment of the Borel Canal Conduit

Figure 10. Unfiltered Openings in Canal Lining Immediately Downstream of Conduit Discharge Structure

A total of 26 vibrating wire piezomenter had been installed and connected to an automated data acquisition system. Eighteen of these piezometers were installed in the upper silty sand and sandy silt layer of concern. Upon completion of the investigation, the conduit was re-watered and instrument reading were taken to characterize the distribution of water pressures in the foundation soils in the area around the conduit. Readings taken from the instruments installed in the upper silty sand and sandy silt layer adjacent to the conduit was evaluated on both an individual and collective basis. One of the collective data evaluations involved a contouring of the water pressures to identify any trends that would be instructive. The results of this contouring effort is shown on Figure 11.

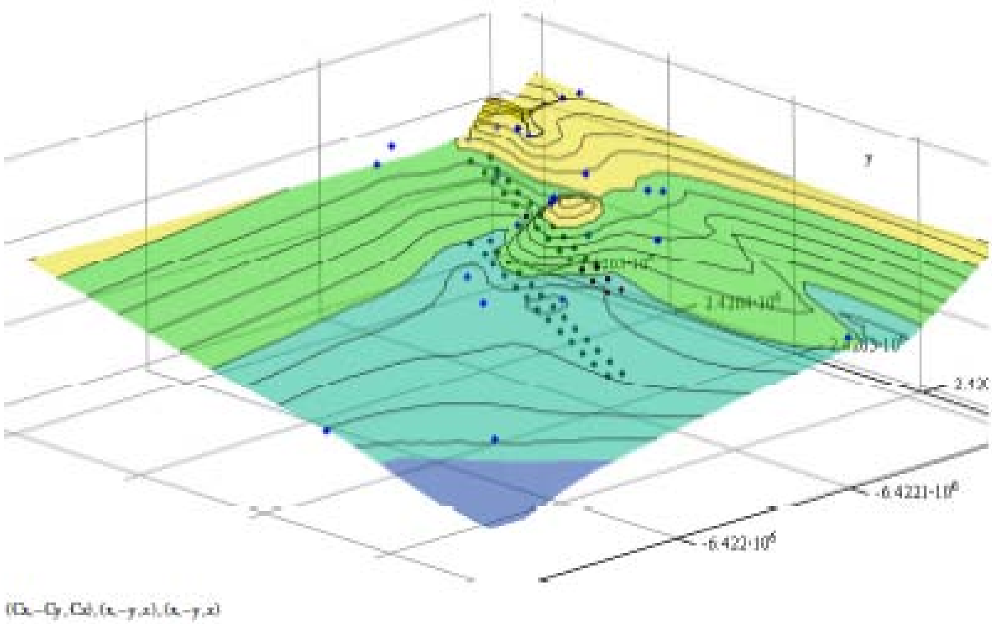

Figure 11. Plot of Water Pressure Contours in the Upper Silty Sand and Sandy Silt Foundation Layer Adjacent to the Borel Canal Conduit

The series of black dots shown on the 3-dimentional plot of water pressure contours on Figure 11 represent the survey point installed along the upper and lower portions of the conduit at each end of each monolith extending from the downstream toe of the dam (the black dots in the lower right of the plot) to the outlet control tower (the dots in the upper central portion of the figure). The blue dots on this figure represent the locations of the various vibrating wire piezometers used for the contouring exercise. The perspective of this plot is looking from the lower toward the upper end of the conduit from an oblique “birds eye” view.

The plot of water pressure contours was very instructive. Each contour line on this plot represents a change of 1-foot in the water pressure within the upper foundation layer. You will note that there are a series of very flat contours along both sides of the conduit extending from the discharge end toward the control tower. The very flat contours extend to the tower on the right side of the conduit (looking downstream), and about 2/3's the distance to the tower along the left side. From those points, the water pressure contours then increase at a relatively steep rate. These results are consistent with the equipotential lines shown on Figure 3 for a defect that has advanced between 50 and 75% of the distance under the dam.

This represents a relatively abbreviated discussion of a complex investigation and evaluation of the seepage conditions around the conduit. A number of direct observations made, along with supporting evaluation of field, laboratory, and instrumentation data, and a detailed risk analysis began to paint a rather convincing story that seepage related PFM's had initiated at several possible locations and may have advanced for a considerable distance under the dam. The effectiveness of the investigation and analysis program in identifying these concerns was due primarily to the understanding of potential failure modes and all of the considerations associated with conditions in and around conduits that went into the design and execution of the investigation program.

CONCLUSIONS

The evaluation of seepage conditions around outlet conduits in dams is a challenging problem. It requires a keen understanding of potential failure modes tied to site geology, design features, and construction methods. To identify whether or not potential failure modes have developed begins with the identification of locations where concentrated and unfiltered seepage leaks can develop. Continuation of a failure mode will be difficult to identify and track. It will require development of a relatively detailed geologic model of the dam foundation, a clear understanding of design and construction details, and conduct of a sufficient number of borings and instrument installation at appropriate locations to properly characterize conditions around the conduit and site response to changes in reservoir levels. In other words, a couple of borings and related instruments at the downstream toe and perhaps a through the dam at the crest will likely not be sufficient to detect if a failure mode has developed and is active. Seepage gradients at the active front of a developing erosion feature will be significantly different than downstream of the active front. Understanding the development of seepage gradient around an active erosion process is critical to the evaluation process. Potential cracking of embankment and foundation materials as a result of differential settlement and possibly hydraulic fracturing will also be an important element of an overall safety evaluation.

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