USSD

21st Century Dam Design — Advances and Adaptations

31st Annual USSD Conference
San Diego, California, April 11-15, 2011

Hosted by
Black & Veatch Corporation
GEI Consultants, Inc.
Kleinfelder, Inc.
MWH Americas, Inc.
Parsons Water and Infrastructure Inc.
URS Corporation

On the Cover

Artist's rendition of San Vicente Dam after completion of the dam raise project to increase local storage and provide a more flexible conveyance system for use during emergencies such as earthquakes that could curtail the region's imported water supplies. The existing 220-foot-high dam, owned by the City of San Diego, will be raised by 117 feet to increase reservoir storage capacity by 152,000 acre-feet. The project will be the tallest dam raise in the United States and tallest roller compacted concrete dam raise in the world.

U.S. Society on Dams

Vision

To be the nation's leading organization of professionals dedicated to advancing the role of dams for the benefit of society.

Mission — USSD is dedicated to:

- Advancing the knowledge of dam engineering, construction, planning, operation, performance, rehabilitation, decommissioning, maintenance, security and safety;
- Fostering dam technology for socially, environmentally and financially sustainable water resources systems;
- Providing public awareness of the role of dams in the management of the nation's water resources;
- Enhancing practices to meet current and future challenges on dams; and
- Representing the United States as an active member of the International Commission on Large Dams (ICOLD).

Printed in the United States of America
Library of Congress Control Number: 2011924673
ISBN 978-1-884575-52-5

U.S. Society on Dams
1616 Seventeenth Street, #483
Denver, CO 80202
Telephone: 303-628-5430
Fax: 303-628-5431
E-mail: stephens@ussdams.org
Internet: www.ussdams.org

EVALUATING THE RISKS OF AN INTERNAL EROSION FAILURE AT AMISTAD DAM

William O. Engemoen[1]
Randel Mead[2]
Luis Hernández[3]

ABSTRACT

Amistad Dam is an important storage and flood control facility on the Rio Grande operated by the United States and Mexico Sections of the International Boundary and Water Commission. This large dam, which contains a concrete section and two very long embankment sections, was constructed on a karstic foundation of soluble limestone and marl. Since construction, extensive seepage has been measured downstream and sinkholes have been discovered and treated in the reservoir area. Although the embankment sections have performed satisfactorily in the 41-year operational history of the dam; the karstic foundation conditions, high seepage flows, and sinkhole formation pose a real concern for the potential of ongoing or future internal erosion of the embankment and foundation. A joint team of engineers from both Mexico and the United States was convened to discuss the various methods in which the dam may fail and to estimate the risks of such a failure. This process included a review and evaluation of the design, construction, and past performance of the facility; the development of potential failure modes; an estimation of the probability of dam failure under each failure mode; and the evaluation of potential consequences in Mexico and the United States in the event of a dam failure. This process has resulted in a better understanding of the threat posed by internal erosion at this dam with a karstic foundation, and has led to a plan for the future investigations and actions necessary to better evaluate and protect the structure against various types of internal erosion failure modes.

INTRODUCTION

A Joint Technical Advisor group consisting of representatives from the Corps of Engineers (USACE) and the National Water Commission (CONAGUA) of Mexico, and from the United States and Mexican Sections of the International Boundary and Water Commission (IBWC/CILA) conducted the 5-year Safety of Dam's (SOD) inspection of Amistad Dam in April 2007. The Joint Technical Advisors used a risk-informed dam safety based action classification that is currently being used by the USACE to assign the Dam Safety Action Classification (DSAC)-II rating. The DSAC II rating indicates the dam is Potentially Unsafe. The Joint Technical Advisor group recommended that a joint geotechnical expert panel of consultants be established to guide the studies,

[1]Geotechnical Engineer - Bureau of Reclamation, Denver, Colorado, wengemoen@usbr.gov
[2]Geotechnical Engineer – US Army Corps of Engineers, Tulsa, Oklahoma, Randel.J.Mead@usace.army.mil
[3]Civil Engineer – International Boundary and Water Commission – US Section, El Paso, Texas, Luis.Hernandez@ibwc.gov

investigations, analyses, and evaluation for the dam. The IBWC established a Joint Panel of Experts (Panel) in June 2008 to conduct an evaluation of Amistad Dam. In addition, a Panel of Consultants was engaged to assist in the evaluation of the Amistad Dam. This working group generated a consensus report that summarized the results of the initial evaluation of Amistad Dam and offered a number of recommendations for future study and actions.

DESCRIPTION OF THE FACILITY

General

Amistad Dam was constructed during the years from 1964 to 1969, and is operated and maintained by the International Boundary and Water Commission, United States and Mexico. The responsibility for dam safety is shared by personnel of both the United States and Mexican Sections of the Commission. Amistad Dam is located on the Rio Grande nineteen kilometers north of the International cities of Del Rio, Texas USA, and Ciudad Acuña, Coahuila, Mexico. It is the second of two major International storage dams constructed jointly by the United States and Mexico pursuant to the Water Treaty of 1944. The dam provides for water conservation, flood control, hydroelectric generation, recreation, and the regulation of flows in the Rio Grande to ensure the continuance of existing water uses and allow for the development of feasible projects.

The total length of the dam is 9,760 meters and consists of a 665-meter-long concrete gravity section in the river channel, which is flanked by 2,591 meters of earth embankment in the United States and 6,504 meters of earth embankment in Mexico. The concrete gravity section consists of a 290-meter ogee weir spillway equally divided on each side of the International boundary. The concrete dam has a non-overflow transition section 28 meters long on either side of the spillway. There is a power intake section 68 meters long adjacent to both of the transition sections. The remainder of the concrete dam consists of a 91-meter non-overflow section between the power intake monoliths and the earth embankment on either bank. The maximum structural height of the concrete section is approximately 77 m, while the maximum structural height of the embankments is about 37 m.

Key Aspects of Dam Design and Construction

Both the United States and the Mexico portions of the dam appeared to have been designed and constructed in reasonably similar manners. Following is a bullet listing of key aspects of the dam design, with differences between the United States and Mexico side listed when appropriate. As the focus of the evaluation of Amistad Dam deals primarily with the potential for internal erosion in the embankment and karstic foundation, relatively few comments deal with the concrete portion.

- The embankments on both the United States and Mexico sides have a similar configuration, consisting of a central core of zone 1 material, flanked by an upstream semi-pervious zone 2. The upstream and downstream shells are comprised of zone 3 sands and gravels. The embankment cross section is shown on Figure 1. The design was somewhat unusual given the upstream transition zone. It is more typical to see a downstream transition zone between core and shell materials. The intent of the upstream zone 2 was apparently to introduce a semi-pervious zone upstream of the core to increase seepage head losses through the embankment, in essence slightly increasing the width of the core. As shown on Figure 1, the majority of the embankment was founded on bedrock, with a relative small amount of overburden left beneath the upstream and downstream toes. Although not shown on the figure, a single line grout curtain was constructed from the bottom of the cutoff trench, extending through the bedrock to variable depths depending on rock conditions. The reported maximum depth of the grout curtain on the United States side was 100 m.

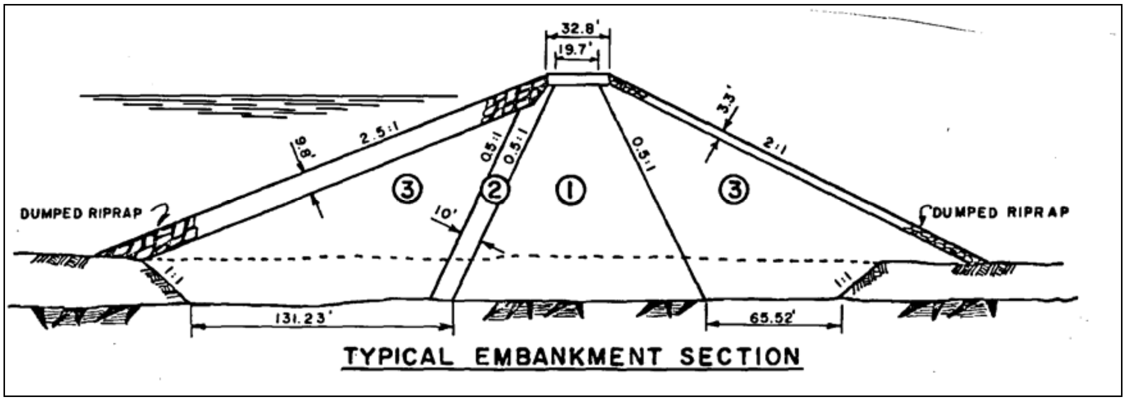

Figure 1. Embankment Cross Section
Note: Dimensions shown in feet instead of meters

• There were problems with embankment material availability on the United States side, as the specified gradations of all three embankment zones had to be adjusted during construction. As a result of shortages of good clay soils, the core of the dam had a variable combination of soils, as shown in Figure 2.

• In general, these shortages of preferred materials led to the use of less desirable materials in the United States embankment section. The zone 1 core and the zone 2 semi-pervious zone were constructed of less plastic materials that are generally more erodible than more plastic soils. In addition, the zone 3 shells consisted of dirtier gravels which would be less pervious (and somewhat more likely to sustain a crack) than the initially specified clean gravels and sands.

• The embankment zones on the Mexico side appear to be comprised of better materials compared to the United States side. The zone 1 core (and potentially the semi-pervious zone 2 as well) was constructed of plastic clay, which should form an impermeable core with reasonable erosion resistance. The zone 3 shells appear to be relatively clean, providing good drainage potential and low potential for sustaining a crack.

• The embankments were not designed with an engineered internal filter (chimney filter/drain), whereas a modern embankment would include this design element. When comparing the gradations of the embankment zones as derived from the Mexico data reported above, the average downstream zone 3 material does not satisfy Sherard and Dunnigan filter criteria for the average zone 1 core, suggesting that zone 1 material has the potential to migrate into zone 3 shell material.

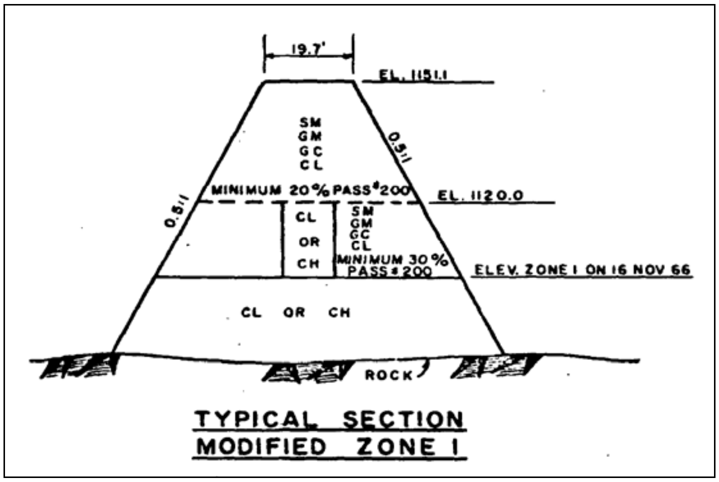

Figure 2. Configuration of United States Side Zone 1 Core
Note: Dimensions shown in feet instead of meters

• Solution features were encountered during construction in the foundations for embankments on both sides of the border. These features appear to have been carefully treated by cleaning out the solution cavities or channels in the rock and replacing the infilling with concrete. However, there is little evidence that modern foundation treatment measures were incorporated in the remaining portions of the foundation. There is no mention, outside of solution features, of the use of slush grouting or foundation filters.

• Because one of the functions of Amistad Dam is to provide flood protection, there is a large amount of normal freeboard at this dam. The normal top of conservation pool is at reservoir elevation 340.5 m, while the dam crest is at elevation 351.2 m. This means the normal freeboard is at least 10.7 m, which generally leads to lower gradients through the embankment and foundation than at other dams with less freeboard.

Site Geology and Foundation Conditions

The Amistad Dam site consists of relatively flat-lying bedrock covered by a layer of Quaternary age overburden. The overburden was generally differentiated as two types of materials – discontinuous alluvial terrace deposits consisting of silt, sand, and gravel occurring in abandoned stream channels; and more recent colluvial deposits of silty/sandy clay, caliche and limestone fragments overlying the alluvium or lying directly on bedrock where alluvium was not present. The terrace alluvium was predominantly sand and gravel, but also included some cobbles and small boulders. Thicknesses of this unit ranged from a few centimeters up to 5 m. The colluvial deposits consisted of primarily silty/sandy clay with some gravel. The thickness of the colluvium ranged from a minimum of about 1 m to a maximum of about 6 m. The maximum overall thickness of overburden encountered during construction was reported to be 8 m. All overburden was removed beneath the zone 1 core of the embankment, and in fact most overburden was also removed from beneath the outer shells of the dam. Thus, the overburden is not particularly important to the performance of the embankments.

Bedrock in the vicinity of the dam is composed of Cretaceous Age sedimentary rocks, which makes them more than 65 million years old. The only two formations involved in the dam foundation are the Del Rio clay and the older Georgetown limestone. The Del Rio clay is found only on the right end of the dam, and thus constitutes a small part of the foundation. This Del Rio formation, estimated to have an average thickness of about 20 m, has been described as a fossiliferous clayey to shale-like unit with limestone beds. The upper portion is likely weathered and thus predominantly clay-like. For the Mexico embankment, the Del Rio unit apparently served as the borrow source for the zone 1 clay core.

The bulk of the dam is founded on the Georgetown limestone, which has a maximum thickness of approximately 150 m. The Georgetown limestone has been described as a light to bluish gray, moderately hard, fine grained, slightly fossiliferous limestone containing scattered shale seams that range from 1 mm to 50 cm in thickness. Karstic features exist in the Georgetown limestone in the vicinity of Amistad Dam. The uppermost 10 to 15 m of the Georgetown unit consists of a transitional, argillaceous limestone and marl that caps the limestone (and toward the right abutment grades into the overlying Del Rio formation). The marl is generally missing from the dam foundation on the United States foundation, presumably eroded away in past geologic time. This marl is widely present on the Mexico side, and forms the foundation for much of the embankment.

Without question, the most important aspect of the Georgetown limestone is weathering and solutioning, which has been observed throughout the dam site. The intensity and depth of weathering and solutioning is variable, but is generally believed to be limited to the upper portion of the Georgetown, with the river elevation of approximate elevation 275 m being an estimated lower limit. Solution features observed at the site and encountered during foundation excavation include channels, caverns, and sinkholes. These features are reported to have generally developed along joints and faults associated with regional deformations during the early Tertiary or possibly later Cretaceous time. The solutioning is believed to follow the jointing pattern downward and the bedding planes in lateral directions. Observed solution features included tubular pipes as well as elongated irregular fissures. The features observed during construction were generally small, ranging from a few centimeters to one meter or so in diameter, and rarely extended to a depth of more than 8 m below the surface. Although some features were open in places, most were filled with clayey material. Of note, the largest known subsurface solution feature in the area is a cavern located approximately 1 km downstream from the dam. This cavern is over 1.8 km in length with a floor elevation of 290 m, or about 15 m above the present level of the river.

When encountered during foundation excavation for the embankment, all solutioning features were carefully cleaned of infilling materials and then replaced with concrete. In addition to the concrete backfilling of large features, available records indicate that smaller solution features and weathered joints were also treated. Overall, the standard of treatment of solution features appeared to be thorough.

DESCRIPTION OF PAST PERFORMANCE

During the approximate 41-year operational history of Amistad Dam, there has been a history of seepage related issues including high seepage flows and the presence of sinkholes. Two general topics related to the past performance of the dam with respect to seepage will be considered; past incidents/observations, and instrumented behavior.

Past Incidents/Observations

• Extensive seepage has been reported downstream of Amistad Dam since initial reservoir impoundment in 1968. Currently, seepage is measured at 35 different monitoring locations and totals approximately 5 m^3/s at conservation pool elevation 340.5 m. It is generally believed that the vast majority of this seepage is traveling through the karstic Georgetown limestone formation beneath the embankment sections.

• Measured total seepage is much more extensive on the Mexico side, with the seepage quantity on the United States side only a fraction of the seepage on the Mexico side. Most of the seepage on the Mexico side is measured in two general areas, the Carmina area and the Arroyo Jaboncillos.

• Throughout the operational history, observations of the seepage have indicated that the flowing seepage waters appear clear, and there is typically no evidence of particle transport. However, sand was observed in the seepage waters at the Carmina springs in October 1993. Detailed descriptions of the amount of sand observed or the duration of the observed particle transport were not located in the records search conducted to date by the Panel. Furthermore, proper particle detection equipment has never existed at Amistad Dam; thus, soil transport could have occurred but not observed.

• A year after the observation of sand at Carmina, the reservoir pool had receded to near elevation 328 m, which is more than 12 m below the normal high pool level of elevation 340.5. The pool had been this low only once since initial filling. In October 1994, with the pool at this approximate low level, a sinkhole was observed in the reservoir area on the Mexico side. This feature was observed approximately 250 meters upstream of the upstream dam toe at dam station 7+100. Ten more sinkholes were observed in the same general area over the next three months. Two more sinkholes were then observed in June 1996 and an additional five in 1997. Most of these sinkholes were located between dam stations 6+500 and 7+200, and none of the reported sinkholes was closer than 200 meters from the upstream toe of the dam. The general location of these features is shown in Figure 3. During this period of sinkhole observations, the pool had remained at low levels and downstream seepage appeared to be at normal levels. A possible explanation for the occurrence of these features is that the sinkholes are at the location of ancient "paleo-chimneys" or collapse features which formed in geologic time and were subsequently infilled. Upon reservoir impoundment, infilling materials within these collapse features became saturated. Once the reservoir was drawn down for a long period in the 1990's, effective stresses increased in the infilling material (buoyant weights went to total saturated weights), and the infilling material collapsed under its own weight. An alternate explanation would be that the sinkholes are manifestations of active internal erosion.

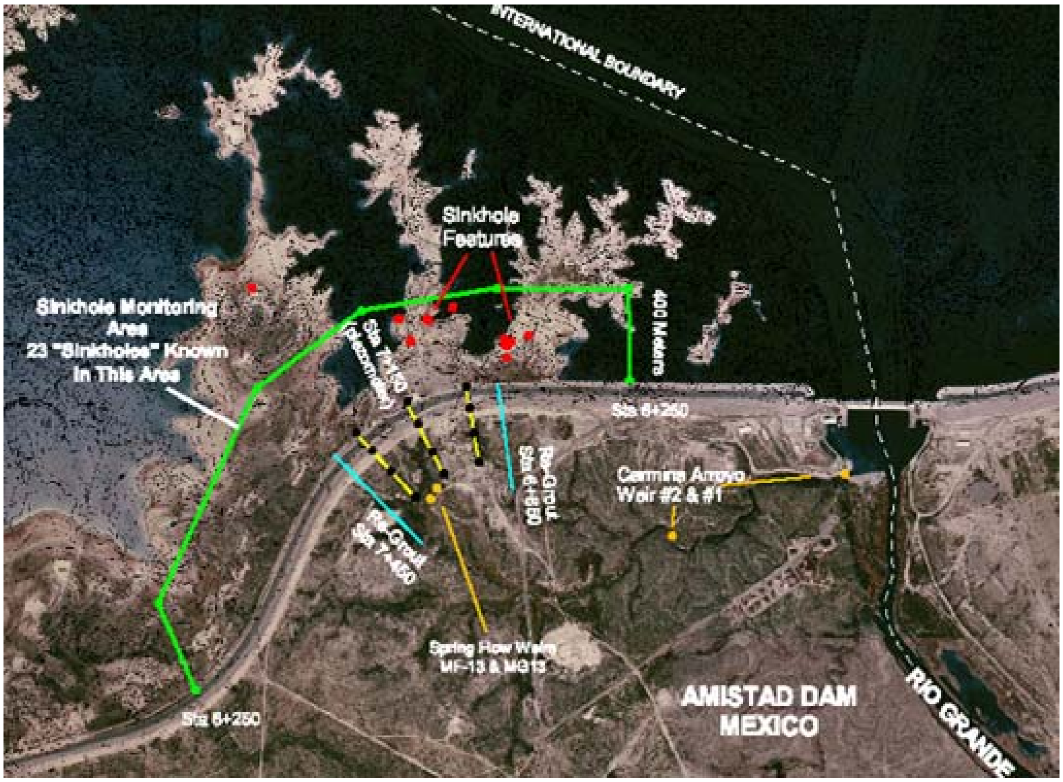

Figure 3. Location of Sinkholes on Mexico Side

• Dye tests were performed in the sinkhole area in the 1994-95 timeframe. In one reported application, dye was introduced into Sinkhole No. 1, and appeared in the Carmina seeps approximately 9 hours later. The distance traveled (in a straight line) was estimated at 400 m. The computed seepage velocity would be on the order of 0.01 m/s, which is quite low. In another test, dye was introduced in a drill/piezometer hole in the sinkhole area and arrived at Arroyo Jaboncillos approximately 22 hours later. That distance was estimated at 3.5 km, making that computed seepage velocity 0.04 m/s.

• In August 1995, grouting was initiated in the area of the sinkholes, between dam stations 6+850 and 7+450. The grouting program essentially consisted of two components – re-grouting of the curtain along the dam axis, and grouting at the upstream toe of the dam. The intent of the grouting was to intercept potential seepage paths near the foundation contact, fill any voids that might have developed from the seepage pathways, and thus minimize the potential for internal erosion of foundation and embankment materials within the dam footprint. As shown on Figure 4, voids were encountered at some locations and likely filled with grout.

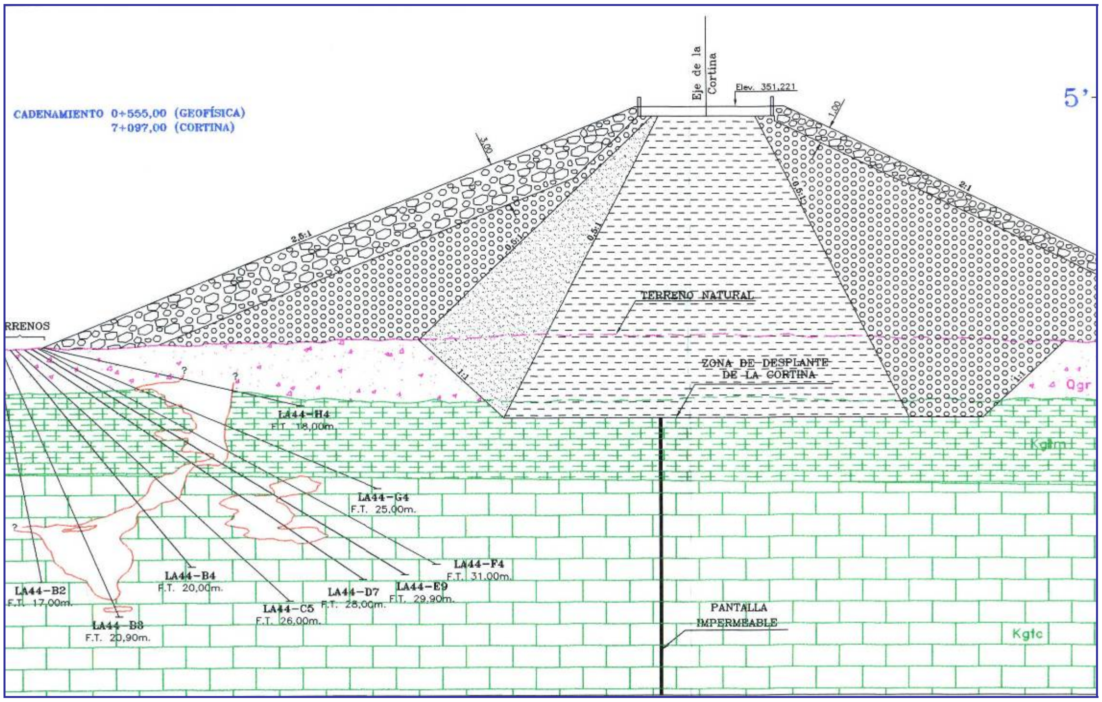

Figure 4. Sinkhole Grouting at Upstream Toe

• Most of the downstream seepage at Amistad occurs at considerable distances downstream from the dam toe (100 m or more). However, it is worth noting that there is one area on the Mexico side where surface seepage occurs at the downstream toe of the dam. This area, at the location of weir V-4, is located between approximate dam stations 6+350 and 6+460. A French drain was constructed at the toe in 1973 to help drain seepage from the area. Reportedly, prior to construction of the drain, seepage exited from the base of the riprap slope protection at the toe. At high pools (approximate reservoir elevation 338 m or higher), measureable seepage is observed.

• In 1996, a potential area of movement in the upstream riprap slope protection was reported between dam stations 6+850 and 7+400. Interestingly, this location corresponds to the general location of the sinkholes on the Mexico side. The general perception at the time of discovery was that the movement in the upstream slope was likely due to wave action. However, the circular shape of the anomaly suggests the possibility that it may be related to subsidence due to internal erosion or collapse of paleo sinkholes in the karst. Additional riprap was added to this area, and subsequent movements have not been reported.

Instrumented Behavior

Key observations with the instrumented behavior at Amistad Dam are included below.

• In general, seepage behavior has been reasonably consistent over the operational history of Amistad Dam, with a few significant exceptions. There are indications that seepage is decreasing at the Jaboncillos monitoring site, although this may be due to a landowner diverting a portion of the seepage flows for irrigation. Weir readings at the Carmina and V-4 monitoring stations indicate that seepage is increasing, which would suggest changes in the foundation seepage pathways or seepage entrance points. However, there are other factors (such as alterations to the weirs) that may explain some of the behavior.

• Piezometer installations within the embankment sections are somewhat problematic. Piezometers at different depths and locations within the embankment at some installations indicate near-identical readings, which suggest a connection or faulty seal between the different instruments. As such, limited reliable data appears to exist as far as pore pressures and phreatic surfaces within the embankment sections.

• There are several piezometer installations within the foundation, although coverage is still rather limited given the extreme length of the dam. Piezometers have been installed in key areas such as the location of sinkholes and V-4 seepage. In general, these installations show that head losses are occurring upstream, and that hydraulic gradients within the foundation are low. However, these types of data are always subject to uncertainty as there is no way of knowing if the instruments are located in critical seepage pathways or even reflect an accurate gradient (if not in the same flow path).

POTENTIAL FAILURE MODES

During a Potential Failure Mode Analysis (PFMA) meeting held at Amistad Dam, the team developed a list of the most plausible potential failure modes (PFM) for the embankment portions of the dam. The most plausible failure modes were judged to occur under static conditions, or normal operations. Each of these potential failure modes is also possible under flood loading, but the annual probability of failure during flood loading is limited due to the remote likelihood of the flood loading. The following paragraphs list the descriptions developed for these potentially critical static failure modes.

PFM 1 - Stoping of a Preexisting Sinkhole Connected to a Seepage Path

Seepage flows along a discontinuity in the marl or weathered limestone with sufficient velocity and capacity to erode materials from a pre-existing foundation cavern. Cavern begins to stope or erode upward due to gravity or along an existing vertical discontinuity. Bedrock, overburden, and/or clay filling are removed until base of embankment is encountered.

- If sinkhole occurs at dam centerline, dam could breach by overtopping from crest loss or internal erosion due to cracking.
- If sinkhole occurs on upstream slope, dam could breach by flow from the reservoir through sinkhole.
- If sinkhole occurs on downstream slope, dam could fail from slope stability.

A sketch of this potential failure mode is included as Figure 5.

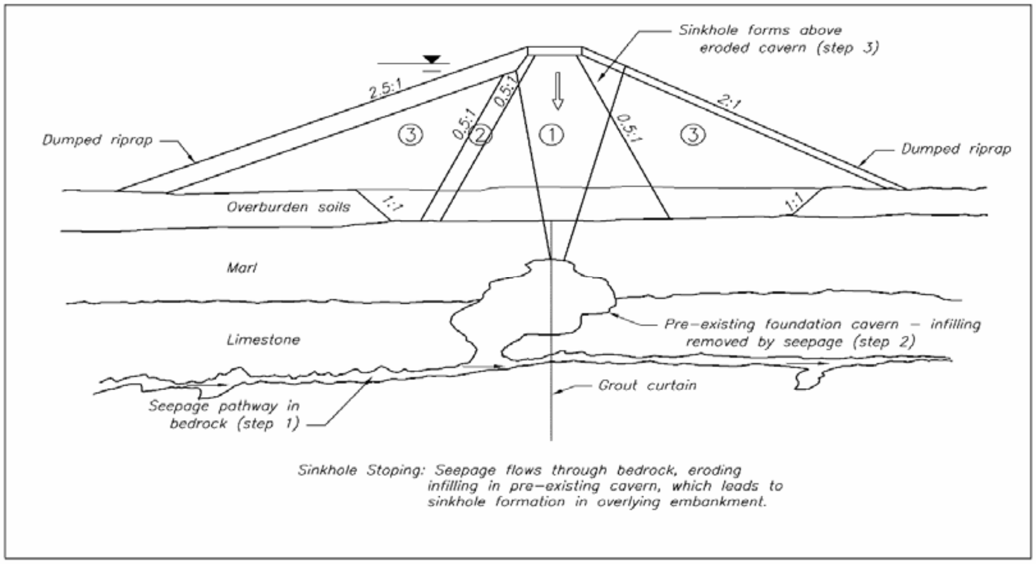

Figure 5. PFM1 - Sinkhole Stoping Failure Mode

PFM 2 - Scour at Foundation Contact

Concentrated seepage flows through the uppermost portion of the marl or weathered limestone. Seepage has sufficient velocity/capacity to begin erosion of unprotected embankment core at base of cutoff trench. As cutoff trench embankment material is removed, settlement and transverse cracking occurs in dam embankment. Internal erosion continues to progress through damaged/cracked core material ultimately leading to dam breach by erosion. A sketch of this potential failure mode is included as Figure 6.

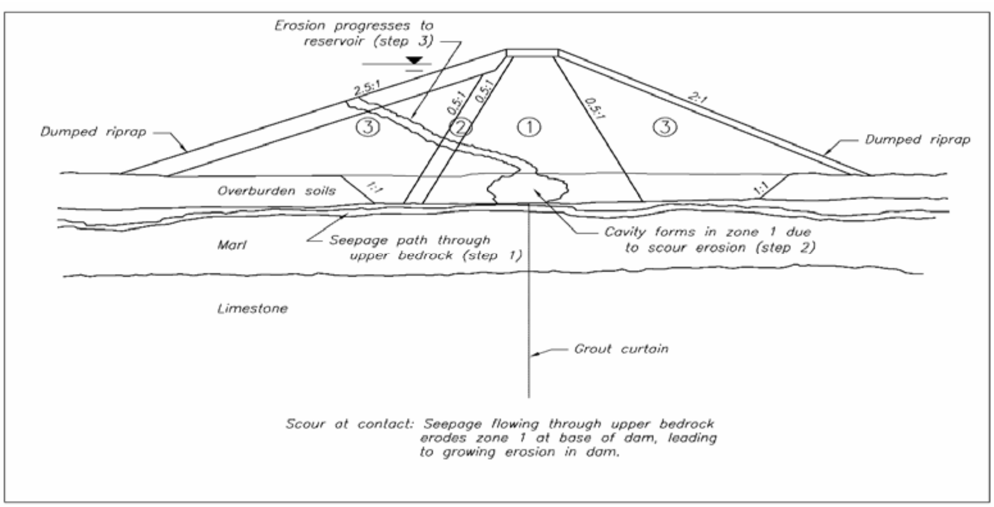

Figure 6. PFM2 - Scour at Contact Failure Mode

PFM 3 - Filter Incompatibility

Concentrated seepage, or seepage under sufficiently high gradients, flows through embankment and encounters downstream embankment or foundation material that allows an unfiltered exit. Erosion of core material begins at unfiltered exit. Backwards erosion piping leads to core material being transported through coarser downstream zones. Internal erosion progresses, which leads to dam failure by uncontrolled erosion or progressive slope failure. A sketch of this potential failure mode is included as Figure 7.

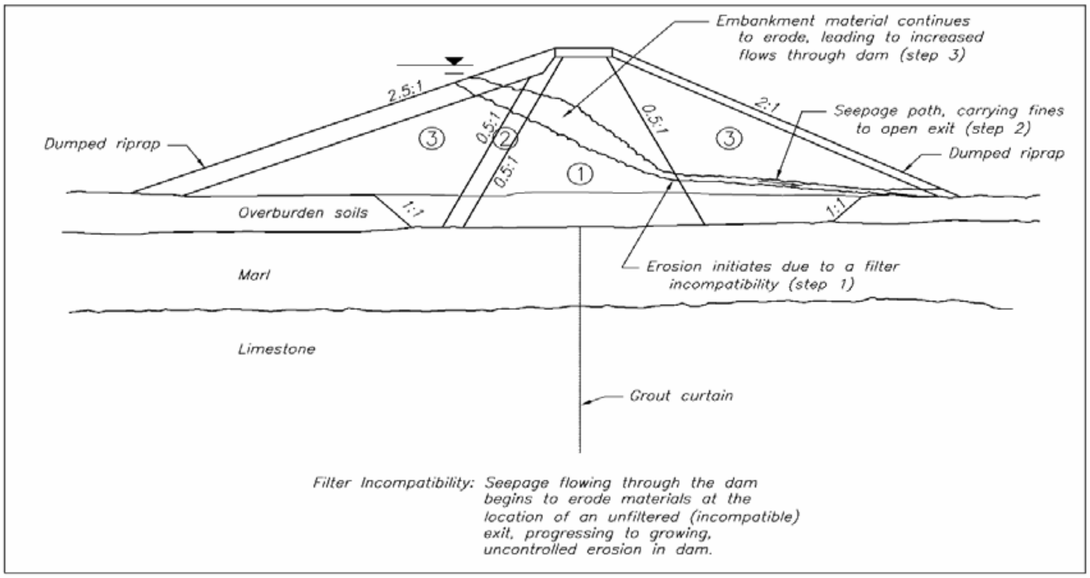

Figure 7. PFM3 - Filter Incompatibility Failure Mode

PFM 4 - Uncontrolled Foundation Seepage

Seepage develops or enlarges in the foundation due to erosion or unplugging of clay infilling in existing discontinuity/flow path or network. Seepage continues to increase, leading to uncontrolled release of reservoir (no failure of embankment). A sketch of this potential failure mode is included as Figure 8.

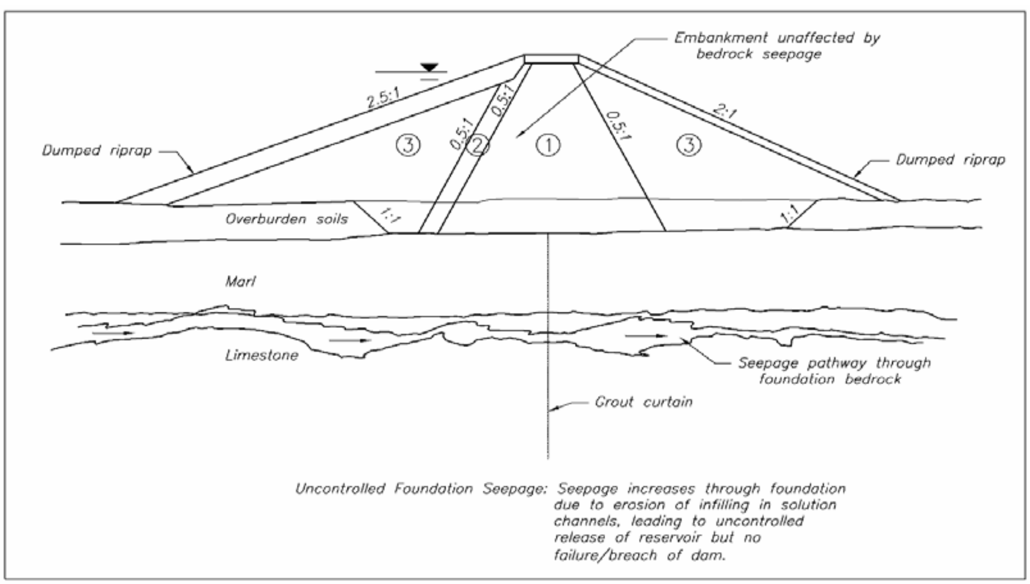

Figure 8 – PFM4 - Uncontrolled Foundation Seepage Failure Mode

PFM 5 - Stoping of a Sinkhole by Gravity

There exists beneath the embankment a large and unfilled solution cavity or one in which cavity infilling has been gradually removed by seepage that was not encountered or remediated during construction. Through gravity, bedrock and overburden stope into the cavity up to base of embankment. Stoping continues upward in embankment materials. If sinkhole occurs on dam centerline, the dam could breach by overtopping due to crest loss. If sinkholes occur on the upstream or downstream slopes, slope failure might occur. A sketch of this potential failure mode is included as Figure 9.

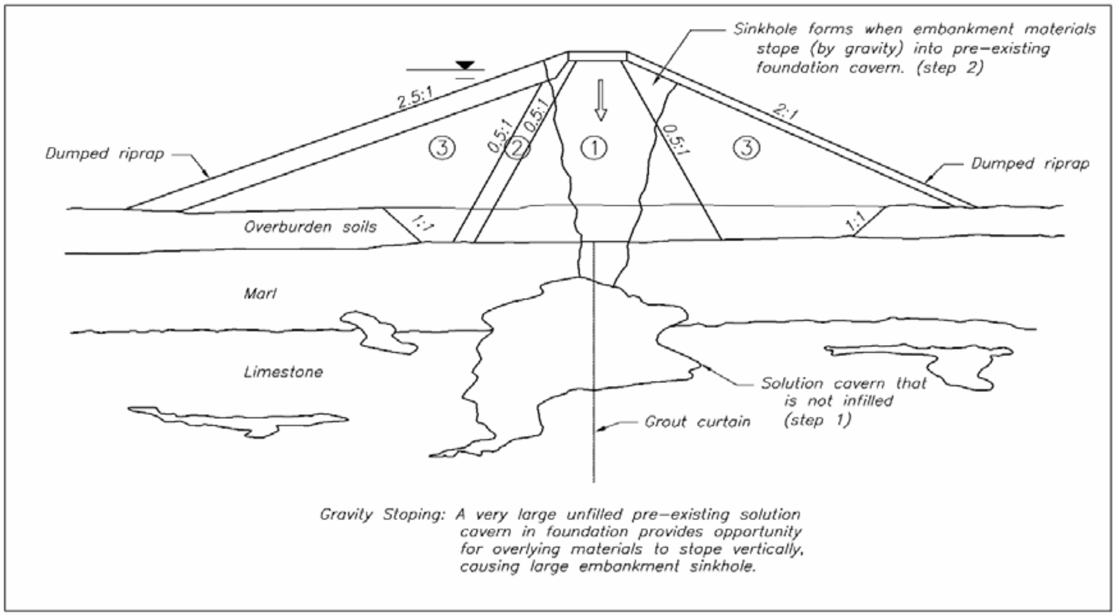

Figure 9. PFM5 - Gravity Stoping Failure Mode

PFM 6 - Slope Failure Resulting from High Pore Pressures

Water flow through an existing discontinuity in the upper portion of the marl or limestone undergoes a change such that pore pressures increase. This could be due to pathway enlargement and less head or plugging/unplugging along discontinuity. These high pore pressures are transmitted to the embankment materials resulting in lowered effective stress, reduced strength, and slope instability. Slope failure creates potential for large crest loss and dam breach. A sketch of this potential failure mode is included as Figure 10.

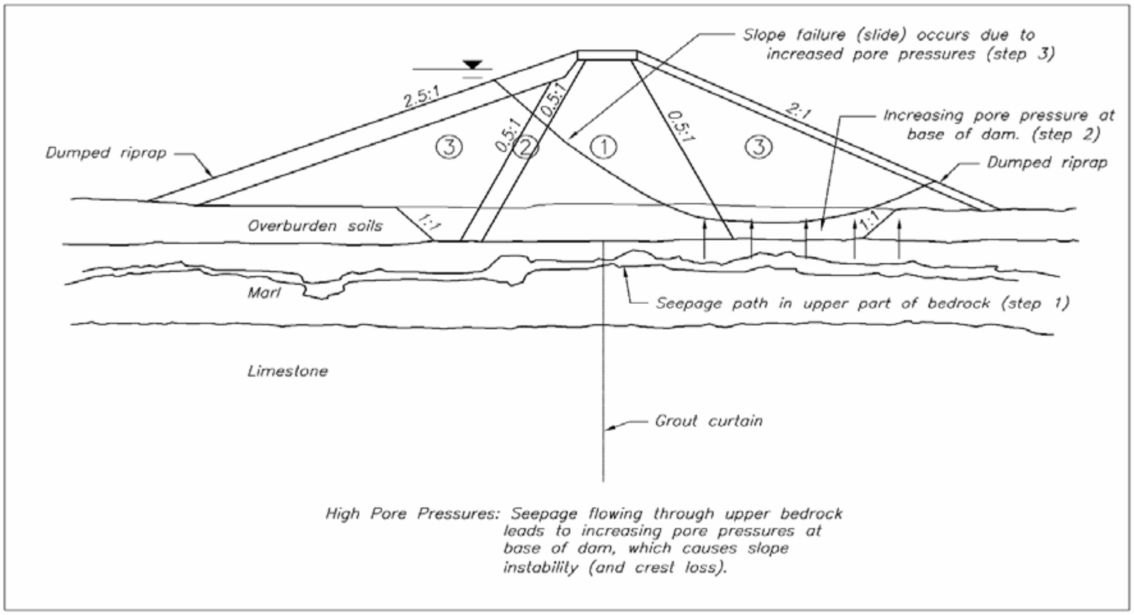

Figure 10. PFM6 - High Pore Pressures Failure Mode

PFM 7 - Internal Erosion of Embankment Along Contact With Concrete Dam

Concentrated seepage flows along the interface of the embankment and concrete dam structures, possibly due to poor compaction in the area of separation of embankment soils from the concrete. Core materials are eroded along this pathway through unfiltered shell materials. Erosion pathway continues to enlarge, leading to dam breach by excessive erosion or slope failure. A sketch of this potential failure mode is included as Figure 11.

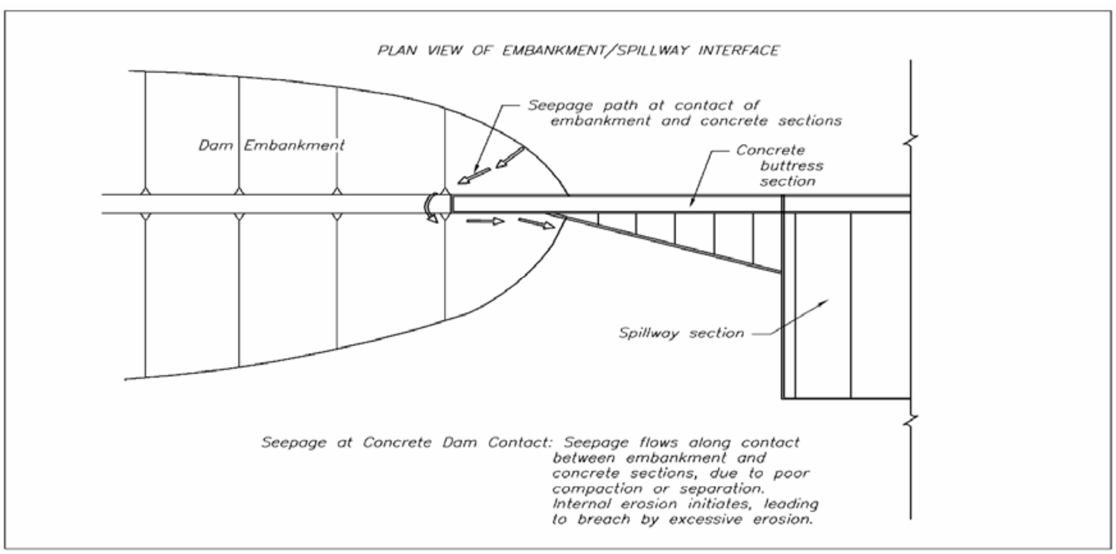

Figure 11. PFM7 - Seepage at Concrete Dam Contact Failure Mode

ESTIMATED RISK OF DAM FAILURE

Once the failure modes had been defined and thoroughly understood, the team began the process of discussing each step of the failure mechanisms and assessing probabilities. A risk analysis was held at Amistad Dam the week of April 5-9, 2010. Risks were evaluated using methodologies followed by the Bureau of Reclamation (Reclamation) and the USACE. The team that estimated the risks was comprised of personnel from Reclamation, the USACE, a joint board of consultants, and representatives from both Sections of the IBWC.

Each of the seven failure potential failure modes was developed into an event tree which listed the steps that would have to occur in order for the dam to fail. For example, the event tree for PFM 1 consisted of the following steps:

1. Reservoir rises to a critical level
2. Continuous seepage path exists beneath embankment
3. Seepage path is connected to a preexisting sinkhole
4. Erosion initiates in the preexisting sinkhole infilling
5. An open or unfiltered exit for the seepage exists
6. Erosion stoping in the sinkhole progresses to the top of the foundation rock
7. Stope is located in critical location within embankment footprint
8. Both stoping & particle transport continues without self-healing or plugging
9. Intervention is unsuccessful
10. Dam breaches

The risk team proceeded to list and discuss all factors that would be relevant to each step of node of the event tree; an example is shown in Table 1.

Table 1. List of Factors for Given Event Tree Node - PFM 1

Dam: Amistad	**Failure Mode:** PFM 1	Table No. 3
Event Tree Pathway: Erosion initiates in the pre-existing sinkhole infilling		

More Likely Factors	**Less Likely Factors**
Limited capability to measure sediment transport	Limited evidence of sediment transport in past
Seepage may be increasing	Dye tests indicated low velocities along seepage paths, as well as low gradients
	Highest measured gradient was 0.06 at Sta 7+350 (with the pool at high level)
	Not a large number of solution features observed on surface during construction excavation
	Infilling reported to be a compact (cemented?) mix of clay, sand, and gravel.

After thorough discussions, probabilities were assigned to each step, and then all individual probabilities were multiplied together to get an annual probability of failure. For example, the probabilities for PFM 1 under normal operations are shown in Table 2.

Note that most probabilities are shown as a range, or probability distribution function. A Monte Carlo analysis consisting of 10,000 iterations was used to generate the mean estimates of annual probability of failure.

Table 2. Annual Probability of Failure Estimates for PFM 1

Mean Annual Probability of Failure (Normal Operations) Stoping of a Pre-existing Sinkhole Connected to a Seepage Path	
Event Tree Branch	**Probability Estimate**
Reservoir rises to critical level	1.0
Continuous seepage path exists beneath embankment	0.9 – 0.999 (best estimate = 0.999)
Seepage path connected to pre-existing sinkhole	0.6 – 0.9 (best estimate = 0.7)
Erosion initiates in the preexisting sinkhole infilling	0.001– 0.09 (best estimate = 0.05)
An open or unfiltered exit for the seepage exists	0.5 – 0.99 (best estimate = 0.75)
Erosion stoping in sinkhole progresses to top of rock	0.1 – 0.99 (best estimate = 0.5)
Stope located in critical location beneath embankment	0.1 – 0.5 (best estimate = 0.3)
Stoping and particle transport continue with no self-healing or plugging	0.001– 0.1 (best estimate = 0.01)
Intervention is unsuccessful	0.01 – 0.5 (best estimate = 0.2)
Dam breaches	0.3 – 0.9 (best estimate = 0.7)
Annual Probability of Failure (Mean Estimate – Monte Carlo Simulation)	**$2x10^{-5}$**

A summary table of the estimated annual probability of failure for each failure mode is shown in Table 3.

Separately from the team effort to estimate annual probability of failures for each potential failure mode, life loss from dam failure was estimated with procedures used by Reclamation and the USACE. The estimated life loss will not be a discussion topic in this paper, but the life loss did enter in to the consideration of risks posed by a dam failure.

It is worth noting, however, that potential failure mode 4, since it does not involve a dam breach, is judged to have a low likelihood to threaten downstream lives. This particular failure mode involves a slowly increasing, and ultimately uncontrolled loss, of reservoir seepage. However, it is almost certain that this scenario will take months or even years to develop, and not envisioned to create any potential sudden large increase in flows that would result in significant life loss.

Table 3. Summary of Annual Probability of Failure Estimates

Summary of Annual Probability of Failure Estimates	
Failure Mode Loading Condition	**Annual Probability of Failure**
PFM1 - Stoping of a Pre-existing Sinkhole Connected to a Seepage Path	
Normal operations	$2x10^{-5}$
Flood RWS – El 345	$1x10^{-6}$
Flood RWS – El 349	$2x10^{-8}$
PFM2 - Scour at Foundation Contact	
Normal operations	$1x10^{-5}$
Flood RWS – El 345	$3x10^{-7}$
Flood RWS – El 349	$1x10^{-9}$
PFM3 - Filter Incompatibility	
Normal operations	$2x10^{-6}$
Flood RWS – El 345	$7x10^{-7}$
Flood RWS – El 349	$3x10^{-9}$
PFM4 - Uncontrolled Foundation Seepage	
Normal operations	$7x10^{-6}$
Flood RWS – El 345	$9x10^{-7}$
Flood RWS – El 349	$3x10^{-9}$
PFM5 - Stoping of a Sinkhole by Gravity	
Normal operations	$2x10^{-7}$
Flood RWS – El 345	$1x10^{-7}$
Flood RWS – El 349	$5x10^{-10}$
PFM6 - Slope Failure Resulting from High Pore Pressures	
Normal operations	$6x10^{-8}$
Flood RWS – El 345	$1x10^{-8}$
Flood RWS – El 349	$1x10^{-10}$
PFM7 - Internal Erosion of Embankment Along Contact With Concrete Dam	
Normal operations	$1x10^{-7}$
Flood RWS – El 345	$8x10^{-9}$
Flood RWS – El 349	$6x10^{-11}$

CONCLUSIONS

After conducting a review and evaluation of design and construction details, foundation conditions, and past behavior, the Joint Expert Panel participated in a Potential Failure Modes Analysis and quantitative risk analysis to evaluate the potential risks associated with internal erosion failure modes at Amistad Dam. Both of these processes forced the team to closely evaluate all data and observations, identify vulnerabilities, and consider the series of steps that would have to occur to lead to dam failure. Although the annual probabilities of failure for individual failure modes estimated by the Panel are not alarmingly high, once the potential life loss associated with dam failure is considered, the resulting annualized life loss risks are judged to justify actions to reduce risk to the downstream public. The concerns with the safety of Amistad Dam are due in large part to the large amounts of downstream seepage, the presence of upstream sinkholes, and indications of changing behavior.

Of the most plausible failure modes identified by the Panel, the following three are judged to be of greatest concern and pose risks that justify additional actions:

- PFM 1 – Stoping of a pre-existing sinkhole connected to a seepage path
- PFM 2 – Scour at foundation contact
- PFM 3 – Filter incompatibility

Four other failure modes evaluated by the Panel were judged to have risks that do not provide strong justification to take actions to reduce at this time:

- PFM 4 – Uncontrolled foundation seepage
- PFM 5 – Stoping of a sinkhole by gravity
- PFM 6 – Slope failure resulting from high pore pressures
- PFM 7 – Internal erosion of embankment along contact with concrete dam

For failure modes PFM 1 and PFM 2, the Panel believes that additional explorations and engineering analyses are unlikely to refine risk estimates sufficiently to bring the risks below tolerable guidelines. Therefore, the focus of the future actions for these two failure modes deal with gathering design data and developing potential structural modifications to make the dam safer. However, for failure mode PFM 3, the Panel believes that additional investigations and engineering analyses may provide useful information that could lower risks to tolerable levels. Hence, future actions for this failure mode will deal with gathering additional data, conducting explorations, and performing additional engineering analyses.

ACKNOWLEDGMENTS

The authors would like to thank all the participants from both Sections of IBWC; consultants Francisco Silva, Steve Poulos, Donald Bruce, and Luis Ramirez; and dam safety experts from Mexico's National Water Commission for their valuable involvement in the evaluation process for Amistad Dam.