Restoration of the Salton Sea

Volume 2: Embankment Designs and Optimization Study

Appendix 2D: Risk Analysis

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1.0 Scope and Participants

1.1 Scope

This report presents the results of a risk analysis of the optimized designs for the Salton Sea restoration project Embankment Alternatives Optimization Study conducted jointly by Kleinfelder and representatives from the Bureau of Reclamation (Reclamation). This report forms Appendix 2D in Kleinfelder's complete report for the Salton Sea restoration project. The requirements for this work were outlined by Reclamation under Task 13 of Order No. 04B8810942 of Contract No. 04CA810942, dated April 21, 2006 between the Bureau of Reclamation and Samuel Engineering, Inc. of Englewood, Colorado. Kleinfelder has performed the work summarized in this report under subcontract agreement with Samuel Engineering. Four different alternatives, including: the mid-Sea dam and perimeter dike; the mid-Sea barrier and habitat pond embankment; the concentric lakes dikes; and the north-Sea dam, were assessed using Reclamation's team-based risk methodology.

1.2 Participants

A risk evaluation team (RET) was convened from August 7th through August 11th, 2006 to evaluate the risk of various embankment failures under static and seismic loadings. The meeting was held at Reclamation's Technical Service Center (TSC) in Denver, Colorado. The personnel listed in Table 2D.1 participated in the risk analysis.

Table 2D.1
Risk Analysis Participants

Participant	RA Role	Group
Keith Ferguson	Team Leader and Author	Kleinfelder, Engineer
Karl Dise	Facilitator	TSC – Geotechnical Engineer – 86-68311
Elena Sossenkina	@Risk Operator	Kleinfelder, Engineer
Scott Shewbridge	Team Member	Kleinfelder, Engineer
Paul Weghorst	Team Member	TSC – Hydraulic Engineer – 86-68520
Richard Wiltshire	Team Member	TSC – Geotechnical Engineer – 86-68311

2.0 Risk Analysis Methodology

Estimates of dam failure risk require a quantification of the likelihood of the loadings, the likelihood of structural response of the dam and appurtenant structures given the load, and the adverse consequences (loss of life) given that failure occurs. In addition, the uncertainty surrounding each factor is quantified. The RET estimated risks in terms of Annual Probability of Failure (APF) and Annualized Loss of Life (ALL), which are defined as follows:

> *Annual Probability of Failure = (Probability of the Loading) x (Probability of Failure given the Loading)*
>
> *Annualized Loss of Life = (Probability of the Loading) x (Probability of Failure given the Loading) x (Adverse Consequences given the Failure)*
>
> Where:
>
> < *Probability of the Loading* is the annual probability that the chosen load range responsible for a failure will occur.
>
> < *Probability of Failure given the Loading* is the likelihood that the dam will fail under the specific loading (ranges from 0 to 1.0).
>
> < *Adverse Consequences given the Failure* is typically expressed in terms of the estimated number of lives lost given a dam failure

The estimated Annual Probability of Failure and Annualized Loss of Life developed during a risk analysis are then compared to Reclamation's Public Protection Guidelines (Reclamation, 2003), which state that:

> *Annual Probability of Failure* $\geq$ 0.0001 ($1x10^{-4}$) indicates increasing justification to take actions to reduce the probability of failure.
>
> *Annualized Loss of Life Risks* $\geq$ 0.001 ($1x10^{-3}$) indicates increasing justification to take actions to reduce risk.

The *Probability of Failure given the Loading* portion of the above equation typically consumes the largest portion of a risk analysis effort. The approach most often followed to develop the probability of adverse response involves a team thoroughly breaking down a failure mode into a detailed "event tree" that includes the individual steps or components that sequentially lead to dam failure. Thorough discussions are held on the factors that affect each branch of the event tree, and then the RET estimates the associated probabilities for those branches.

In establishing the estimates of probabilities for each branch, team members made estimates with the aid of a scale of verbal descriptors of probability ranging from "virtually impossible" to "virtually certain." These verbal descriptors and their associated probability are shown on
Table 2D.2.

Table 2D.2
Verbal Descriptors

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

Individual estimates of probability for each branch are given a range of probability to reflect the team's level of uncertainty. This range is expressed in the form of a function having a probable low, best estimate, and probable high. The computer program @RISK was utilized to perform the computation for the Annual Probability of Failure and compute the Annualized Loss of Life. The @RISK program uses a simulation called a Monte Carlo analysis to simulate the range of distributions and results of each branch of the event tree, and to combine all branches to show the overall range of risk for a given failure mode. The resulting values were then plotted on graphs showing the comparison to Reclamation's Public Protection Guidelines (Reclamation, 2003).

3.0 Description of Alternatives And Design Features

3.1 General

This chapter discusses the embankment features to document the current designs for the various alternatives developed for the Salton Sea restoration project at the time of the risk analysis. A team of engineers experienced in embankment dam design, analysis, and construction had collaborated to formulate these designs. The designs are considered preliminary due to limitations of available site investigation data and due to the limited extent of engineering analysis that could be performed.

For the purposes of this report, the following is a summary list of alternatives being studied by the Bureau of Reclamation.

Alternative No. 1 – Mid-Sea Dam/North Marine Lake

Alternative No. 2 – Mid-Sea Barrier/South Marine Lake

Alternative No. 3 – Concentric Lakes Dikes

Alternative No. 4 – North-Sea Dam/Marine Lake

Alternative No. 5 – Habitat Enhancement without Marine Lake

The general configurations of each of these alternatives are described in the main report and a brief summary is provided below.

3.2 Alternative No. 1 — Mid-Sea Dam/North Marine Lake (Salton Sea Authority Alternative)

This alternative would provide both elevation and salinity control. An impervious mid-Sea dam embankment would be constructed so the water north of the embankment would be maintained at a higher elevation than the brine pool on the south side. The area south of the embankment would serve as an outlet for water and salt from the north and would rapidly shrink in size and increase in salinity to form a brine pool. The north marine lake would have a water surface area of up to 140 square miles at elevation –230 feet mean sea level (msl). The estimated long-term elevation of the brine pool is –270 msl.

The general layout of this alternative is shown on Figure D.A.1. In addition to the north marine lake, a smaller south marine lake would be created by the construction of a south-Sea dam. These two bodies of water would be connected along the western edge of the Sea by the construction of a western perimeter dike. It also includes a perimeter dike along a portion of the east side and a 6-mile-long canal

The combination of the mid-Sea dam, south-Sea dam and the perimeter dikes comprise the primary embankments needed for this alternative that must comply with Reclamation's Public Protection Guidelines (Reclamation, 2003). In general, the mid-Sea and south-Sea dam embankments would be designed and constructed to meet Reclamation's design criteria for "high hazard" structures with an estimated APF of less than or equal to 1×10^{-4}. The perimeter dike embankment would be would be designed and constructed to meet Reclamation's design criteria for "significant hazard" structures.

This alternative also includes the construction of 16,000 acres of shallow habitat ponds adjacent to the canal along the southeast side of the exposed Sea bed/brine pool and at the north end of the Sea. The habitat ponds would be impounded by low earthfill embankments

The mid-Sea dam, south-Sea dam, and perimeter dike embankments would be constructed in the wet using over-water, conveyor, truck haul, or a combination of these placement methods.

The habitat pond embankments would be constructed in the dry. Because of their low-height and "low hazard" classification, they would be constructed of homogenous soil fill with no filters or internal zoning. No erosion protection would be placed on the outer slopes of these embankments.

3.3 Alternative No. 2 — Mid-Sea Barrier/South Marine Lake

This alternative would provide salinity control but no elevation control, and up to 21,700 acres of shallow habitat ponds (URS, 2004a). The water entering the Sea from the south into the south marine lake would support marine habitat. The estimated long-term elevation of the marine lake is –258 msl. The area north of the barrier embankment would serve as an outlet for water and salt from the south side to form a brine pool. As the main body of the Sea shrinks, dikes would be constructed to create impoundments to provide freshwater marsh and shallow water shoreline. As the main body of the Sea shrinks, the 21,700 acres of habitat ponds would be constructed on the exposed Seabed at the locations shown on Figure D.A.2 to take advantage of the gently sloping Seafloor for different habitat.

Unlike the mid-Sea dam, which can support differing water elevations on each side, the barrier would not experience a differential head of more than 5 feet. The current barrier concept calls for a seepage barrier in the embankment and a design that would have to meet Reclamation's Public Protection Guidelines (Reclamation, 2003). The structure would likely be classified as a significant hazard structure based on consideration of the loss of significant wildlife benefits and the significant costs associated with repair/replacement of the barrier should failure occur. The anticipated maximum embankment section for the barrier would have a structural height of up to 60 feet. The estimated length of the barrier would be 7.3 miles.

As described above, this alternative also includes the construction of habitat ponds that would be impounded by low earthfill embankments as described under Alternative No. 1 in sub-section 3.2.

3.4 Alternative No. 3 – Concentric Lakes Dikes

This alternative provides both elevation and salinity control and involves forming four concentric annular 5- to 6-foot-deep pools within the Sea. Inside these pools, a brine pool would develop as shown on Figure D.A.3.

Several alternative design approaches are being considered for the concentric lakes dikes based upon consideration of hazard, loss of potential benefits should one of the outer most rings fail, and replacement costs. An optimized concept for the concentric lakes dike embankments, meeting Reclamation's Public Protection Guidelines (Reclamation, 2003) for "significant hazard" structures have been developed. A second cross-section that considers only static design considerations has also been developed.

For structures meeting the static and seismic design criteria, the maximum embankment sections would have a structural height of up to 20 to 40 feet depending on the depth of the soft Seafloor and soft lacustrine and upper alluvium materials. The static design and "low hazard" embankments would have a structural height of 10 to 20 feet depending on the thickness of the Seafloor deposits that would be removed for embankment construction.

3.5 Alternative No. 4 — North-Sea Dam/Marine Lake

This alternative would provide both elevation and salinity control. An impervious dam embankment would be constructed so the water north of the embankment would be maintained at a higher elevation than the brine pool on the south side as shown in Figure D.A.4. The area south of the embankment would serve as an outlet for water and salt from the north and would shrink in size to achieve equilibrium with inflows from the south and discharges from the north marine lake. The brine pool would increase in salinity through time. The north marine lake would have a water surface area of up to 17,000 acres at elevation -228 msl. An emergency spillway on the dam crest would be required to regulate pool level and to pass design flood discharges from the Whitewater River basin.

In addition to the north marine lake, 37,200 acres of shallow habitat ponds would be created in the southern end of the Sea. As the main body of the Sea shrinks, these habitat ponds would be constructed on the exposed Seabed to take advantage of the gently sloping Seafloor for different habitat.

The combination of north-Sea dam and the habitat pond embankments comprise the primary embankments needed for this alternative. The north-Sea dam embankment would need to comply with Reclamation's Public Protection Guidelines (Reclamation, 2003). These guidelines require that the north-Sea dam embankment be designed and constructed to meet Reclamation's design criteria for "high hazard" structures with an estimated annual probability of failure of less than or equal to 1×10^{-4}. The habitat pond embankments would be "low hazard" structures.

3.6 Alternative No. 5 — Habitat Enhancement without Marine Lake

Saline habitat complexes would be constructed at the south and north ends of the Sea. Five separate complexes would be constructed with a combined surface area of 42,200 acres as shown on Figure D.A.5. As a whole, the complexes would average about 60 percent land (levees, berms, islands, etc.) and 40 percent water. About 25 percent of the habitat would be open water with little land development and deep water (up to 10 feet) for fisheries. These deep-water areas would be constructed through excavation, with the excavated material used to create islands behind non-deep water cell embankments. The remaining 75 percent of the habitat would be divided into areas suitable for different species and their use, with the ratio of land to water varying from 70:30 to 30:70. The majority of these shallow water habitats would be less than 3 feet deep.

The habitat pond embankments comprise the primary embankments needed for this alternative. These ponds would be impounded by low earthfill embankments as described under Alternative No. 1 in sub-section 3.2. The habitat pond embankments would be constructed in the dry. Because of their low-height, they would be constructed of homogenous soil fill with minimal filters or internal zoning. No erosion protection would be placed on the outer slopes of these embankments.

3.7 Summary

The five alternatives described in the preceding sub-sections each require different embankments to achieve the desired water storage and management objectives. A summary of the required embankments and the design criteria is provided in Table D.3.

Table 2D.3
Summary of Project Alternatives

Component	**Mid-Sea Dam/North Marine Lake (1 – Salton Sea Authority Alternative)**	**Mid-Sea Barrier/South Marine Lake (2)**	**Concentric Lakes Dikes (3)**	**North-Sea Dam/Marine Lake (4)**	**Habitat Enhancement without Marine Lake (5)**
Mid-Sea Dam	X				
Mid-Sea Barrier		X			
Perimeter Dikes	X				
South-Sea Dam	X				
North-Sea Dam				X	
Concentric Lakes Dikes			X		
Habitat Pond Embankments	X	X		X	X
Annual Probability of Failure (APF – max)	$\geq 1x10^{-4}$	Non – Seismic $< 1x10^{-4}$, Seismic $\geq 1x10^{-4}$	Seismic, $\geq 1x10^{-4}$	$\geq 1x10^{-4}$	"Low hazard"

The embankments listed in Table 2D.3 are described in detail in Chapters 4.0 and 5.0 of the main report and shown on Figures D.A.6 through D.A.11

4.0 Geology and Foundation Site Conditions

Considerable engineering challenges are presented by the unfavorable geologic and environmental conditions at the Sea. Soft sediments in the Seafloor present a very poor foundation to build upon. The poor foundation conditions are aggravated by the large earthquakes that frequently occur in this seismically active area. Additional challenges arise from difficult environmental conditions. The Sea often experiences strong winds and high waves. The high salt content of the water is corrosive to steel structures and equipment, and forms salt crusts upon materials that are splashed by the water. The Seafloor sediment contains potentially toxic substances such as hydrogen sulfide and selenium, and its high water content complicates excavation, transport, and disposal.

The Sea is located in an inter-mountain basin that is a terminal sink for the New, Alamo, and Whitewater Rivers. Several active faults have been identified in this area that frequently experiences large earthquakes. The Seafloor contains unconsolidated soil deposits that are estimated to be up to 18,000 feet thick. Preliminary investigations have been conducted to identify the composition and strength of the upper layers of the Seafloor deposits. Borings and cone penetration test (CPT) soundings on approximately 1-mile spacing across and around the Sea have been made and samples from the borings have been tested. The details of this study are contained in the Preliminary In-Sea Geotechnical Report prepared by URS Corporation in 2004 (URS, 2004a). This study shows the presence of up to 25-feet of Seafloor deposits (very soft organic-rich clay) underlain by up to 20-feet of soft lacustrine deposits, and below that an upper stiff lacustrine deposit. Upper alluvial deposits of variable thickness lie above the upper stiff lacustrine deposits toward the west side and underlay some of the soft lacustrine deposits. Reclamation has reviewed the available geotechnical data and has determined that:

- With only one boring and/or CPT test per mile, the Seafloor deposits have not yet been fully characterized with respect to the variation in the deposits thicknesses and strengths.

- The seismic behavior of the Seafloor deposits is not fully understood. Additional study is required to better define the nature of seismic ground motions and to determine if the Seafloor deposits magnify or attenuate seismic forces. Additional details of this issue are discussed in the Risk Analysis Report (Reclamation, 2005d).

- The organic-rich Seafloor deposits are an unacceptable foundation material and should be removed prior to any embankment construction.

- Portions of the upper alluvial deposits are likely to liquefy in response to a seismic event. This issue is discussed in detail in the Evaluation of CPT and SPT Site Data section of this report. Treatment of portions of the Upper Alluvial Deposits will be required to provide a stable embankment foundation and prevent slope failure during construction and in the event of a moderate to large earthquake. This issue is discussed further in the Dam Embankment Stability Analysis-Post Earthquake Case sub-section of this report.

- Portions of the soft lacustrine deposits are likely to liquefy in response to a seismic event. This issue is discussed in detail in the Evaluation of CPT and SPT Site Data sub-section of this report. Treatment of portions of the soft lacustrine deposits will be required to provide a stable embankment foundation during construction and prevent seismic induced slope failure and excessive settlement. This issue is discussed further in the Dam Embankment Stability Analysis-Post Earthquake Case sub-section of this report.

Available environmental data regarding the climate, water quality, and the chemistry of the Seafloor deposits has been reviewed. Reclamation has determined that:

- The Sea often experiences strong winds and high waves. This will slow construction and increase costs. The embankments should be designed for a minimum of five feet of freeboard and require armoring to resist wave action. For this reason, the embankments subjected to deep water are designed with outer slopes of coarse rockfill (riprap).

- The high salinity of the Seawater is likely to increase construction costs. The water is corrosive to steel structures and equipment; it forms salt crusts upon materials that are splashed by the water. Grout and soil-cement placements will require imported water for construction placement and in-place curing is likely to suffer strength reductions due to contact with Seawater and saline sediments during curing.

- The Seafloor deposits contain potentially toxic substances such as hydrogen sulfide and selenium. The concentrations of these substances will need to be more thoroughly evaluated for the construction areas. Special precautions for worker protection may be required.

The high water content of the Seafloor deposits complicates excavation, transport, and disposal activities. It is assumed that suction dredges are needed to excavate the Seafloor deposit materials and that in-Sea disposal may require placement at significant distances from the excavation because the material is not likely to stand at a steep angle and current circulation of Seawaters may transport the material back towards the excavation and/or all around the Sea. This will increase material handling costs.

5.0 Previous Risk Studies

The first Risk Analysis Study for the Salton Sea restoration project was conducted by Reclamation in 2005 as part of the appraisal-level studies of the remedial alternatives (Reclamation, 2005d). This risk analysis addressed all alternatives considered by Reclamation at that time. The evaluation was based on limited field data, slope stability and deformation analyses performed to date, and used standard Reclamation risk analysis methodologies.

A total of nine alternatives were evaluated as part of the first Risk Analysis Study, including the following:

- Mid-Sea Dam with North Marine Lake
- Mid-Sea Dam with South Marine Lake
- Concentric Lakes Dikes with Cascading Reservoirs
- Revised Salton Sea Authority Alternative
- Mid-Sea Barrier with North Marine Lake
- Mid-Sea Barrier with South Marine Lake
- Mid-Sea Barrier with South Marine Lake and Habitat Ponds
- Revised Evolving Sea
- No Sea – Reclaimed to Agriculture

All but the last alternative were further divided into two categories and evaluated with and without foundation treatment. The Risk Analysis addressed four types of impoundment structures in the study alternatives that had a potential for loss of life: mid-Sea dam, mid-Sea barrier, lakes dikes, and habitat pond embankments.

Concluding statements were prepared for static, hydrologic, and seismic failure modes for major impoundment features of the alternatives. A brief summary of the risk assessment results is presented below.

5.1 Static Failure Modes

Potential static failure modes were grouped into three categories: internal erosion of the embankment, internal erosion of the foundation, and internal erosion of the embankment into foundation. All estimated APF and ALL for static failure modes for the mid-Sea dam, mid-Sea barrier, lakes dikes, and habitat pond embankments were below Reclamation guidelines for public safety. The risks associated with a failure of the mid-Sea barrier and habitat pond embankments were judged negligible. The highest APF and ALL equal to 4.2E-05 were estimated for the lakes dike embankments. APF and ALL for the mid-Sea dam were estimated at 2.1E-06.

5.2 Hydrologic Failure Modes

The RET judged the risk for hydrologic failure modes for all impoundment structures to be low because of the long warning period, low population at risk and the embankment designs that meet stability requirements for the probable maximum flood, which has a return period greater than 50,000 years. Accordingly, no detailed evaluations of the hydrologic failure modes were performed for the Risk Analysis Study.

5.3 Seismic Failure Modes

Seismic failure modes for all structures were evaluated based the risk assessment of the mid-Sea-dam with and without foundation treatment. Thirteen different potential seismic failure modes, including failures due to liquefaction of various foundation layers and liquefaction of the embankment itself, failures by overtopping and due to fault displacement, were identified for the mid-Sea dam. Without foundation treatment, the mean seismic APF for the mid-Sea dam was estimated at 9.0E-03, exceeding the accepted Reclamation guideline of 1.0E-04 by almost one hundred times. This result would equate to about a 36% chance of failure in 50 years. In addition, the computed ALL value of 9.0E-2 was higher than the accepted Reclamation guideline of 1.0E-03. A risk analysis of the mid-Sea dam with treated foundation indicated that foundation treatment is an effective method to reduce the risks to acceptable levels. The computed APF and ALL values were 3.5E-05 and 3.5E-04, respectively. Evaluation of other structures yielded similar results. In general, alternatives with treated foundations met Reclamation's Public Protection Guidelines (PPG) criteria (Reclamation, 2003) and alternatives with untreated foundations did not meet the PPG criteria. For the mid-Sea barrier, lakes dikes, and habitat pond embankments without foundation treatment, the mean seismic APF were estimated at 3.5E-03, 2.0E-3, and 2.3E-2, respectively. The RET therefore determined that foundation treatment would be necessary to decrease the likelihood of liquefaction, control seismic deformations, and bring the structures to an acceptable risk.

6.0 Embankment Loading Conditions

6.1 General Assumptions

This risk analysis is based on the assumption that no liquefiable layers exist in the upper stiff lacustrine deposit. To date no equivalent $N_{1\text{-}60}$ blowcounts of less 16.5 have been measured in that deposit. However, the team recognizes that the data set is very limited. Failure probabilities estimated as part of this risk analysis, would substantially increase if such a condition were evaluated.

Large uncertainty is assumed in the earthquake loading parameters including not just PGA, but spatial variability, and duration of the time histories. The team noted that higher frequency events might not necessarily have a shorter duration but may actually have a longer duration and result in more damage.

The site is located at a tectonic plate boundary and annual fault movements are relatively large. Evidence is growing that fault offsets may actually propagate to the ground surface. The rupture at the bedrock surface under the site is potentially large (such as 20 feet horizontal and 1 to 2 feet vertical).

6.2 Static Loading

For each of the static failure modes, the RET assumed that the reservoir retained by the embankment was maintained at a relatively constant level near the peak pool level. The following table summarizes the expected reservoir level and associated freeboard.

Table 2D.4
Design Reservoir Level and Embankment Freeboard

Component	Embankment Crest Elevation, MSL	Normal Pool Elevation, MSL	Minimum Embankment Freeboard, ft
Mid-Sea Dam	-225	-230	5
Mid-Sea Barrier	-245	-258	13*
Perimeter Dike	-225	-230	5
South-Sea Dam	-225	-230	5
North-Sea Dam	-223	-228	5
Concentric Lakes Dikes	-226, -236, -251, -261	-230, -240, -255, -265	4

*This is maximum. Actual will vary from 0 to 13 feet until design objective is reached.

6.3 Seismic Loading

A probabilistic seismic hazard analysis, including all relevant seismic sources, was completed for the Salton Sea restoration project site by Reclamation (Reclamation, 2005b). The team reviewed these loadings during the risk assessment brainstorm session. Based on this review, the existing seismic loadings were judged acceptable for the use in this risk analysis. The four seismic loads considered in the risk assessment are presented in Table 2D.5 below.

Table 2D.5
Seismic Loads

Load Range	Earthquake Frequency	Range of Estimated PGAs
1	< 500 yr	0 to 0.26 g
2	500 to 5,000 yr	0.26 to 0.7 g
3	5,000 to 20,000 yr	0.7 to 0.9 g
4	> 20,000 yr	> 0.9 g

7.0 Potential Failure Modes

To ensure risk team members had a clear and similar understanding of the failure mechanisms, each failure mode was discussed and defined prior to estimating the risk for that failure mode.

The team considered whether each alternative embankment would have distinctly different failure modes. After review of the site conditions and proposed embankment configurations, the team concluded that some failure modes were likely to be common to all the proposed structures, with many similarities due to similar foundation geology and the selection of a common seismic design standard (yield acceleration equal to or greater than 0.17g, see Appendix 2B, Seepage and Stability Analyses for more details). Differences were generally attributable to differences in embankment configurations and/or detection, mitigation or removal of problematic foundation geologic materials.

Table 2D.6 includes failure modes evaluated during this risk analysis. Because of the similarity of failure modes for all of the alternative structures, the team adopted an approach of evaluating a set of “common” failure modes using the mid-Sea dam and south-Sea dam configuration for the base assessments. Then the team assessed how the other alternative conditions differed from those configurations, leading to either fewer or additional needed conditions for failure (i.e., branches) and/or increased or decreased likelihood of each individual condition.

Table 2D.6
Failure Modes Evaluated During Risk Analysis

Component	Sub-component	Static - Internal Erosion (Piping) of Embankment	Static - Internal Erosion of Foundation Materials	Seismic - Deformation and Overtopping of Embankment	Seismic - Deformation and Internal Erosion of Embankment	Seismic - Deformation and Internal Erosion of Foundation Materials	Seismic - Liquefaction of Stiff Lacustrine, Deformation and Overtopping of Embankment	Seismic - Offset and Translation of Embankment
Mid-Sea Dam	Sand dam with stone columns	√ FM1	√ FM2	√ FM3	√ FM4	√ FM5	√ FM6	
	Rock notches with maximum seismic filters	• FM7	√ FM8	√ FM9	• FM10	√ FM11	√ FM6	
Mid-Sea Barrier				•				
Perimeter Dike		•	•	•	•	•	•	•
South-Sea Dam		•	•	•	•	•	•	√ FM12
North-Sea Dam		•	•	•	•	•	•	
Lakes Dikes		•	•	•	•	•	•	•

- ✓ Indicates that an event tree and estimate of the failure probability was developed during the risk analysis
- • Indicates that an estimate of the failure probability was developed based on the results of the analyses performed on the other failure modes (√)

For each structure, the RET evaluated risks associated with static and seismic failure modes. No hydrologic failure modes were considered in this risk assessment. In previous studies (Reclamation, 2005d), Reclamation had evaluated the possibility of hydrologic failure modes and determined that they were unlikely to impossible. Members of the current risk evaluation team reviewed operational conditions for each of the alternatives. Since the inflows for each of the alternatives will be highly controlled, the risk of hydrologic loading leading to overtopping, spillway, or outlet structure failures is unlikely to impossible. Given these factors, the RET concluded that there are no plausible hydrologic failure modes expected to pose any appreciable risk. Accordingly, no detailed evaluations of the hydrologic failure modes were performed for the Risk Analysis Study.

In general, the team evaluated three categories of static failure modes: internal erosion of the embankment, internal erosion of the foundation, and internal erosion of the embankment into the foundation. Seismic failure modes included failure due to overtopping, seismic cracking (through seepage), seismic under seepage, liquefaction of the foundation, and failures due to fault displacement. The following sub-sections describe the failure modes identified for each structure.

7.1 Mid-Sea Dam

The RET considered two mid-Sea dam design alternatives, the sand dam with stone columns and the rockfill dam with rock notches, shown on Figures D.A.6 and D.A.7 in Attachment A. The rockfill dam design considered for this risk assessment incorporated maximum seismic filters.

7.1.1 Sand Dam with Stone Columns

7.1.1.1 FM No. 1 Static - Internal Erosion (Piping) of Embankment

The specific description of this failure mode is a defect in the embankment's SCB slurry wall that allows concentrated seepage paths to form within the dam embankment, with sufficient velocities to begin the erosion and transport of Type A soil particles. Assuming Type B does not serve as a filter for Type A, the erosion would progress and lead to the development of a "pipe" within the Type A material. As the pipe enlarges, additional seepage and higher velocities would result in more erosion in the Type A materials and the SCB slurry wall. Ultimately, the developing piping pathway could progress to the reservoir and lead to a complete erosion failure, or collapse and create a sinkhole which leads to toppling of the SCB wall, continued erosion, crest loss, and overtopping.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by means of an event tree discussed later (Chapter 8.0).

7.1.1.2 FM No. 2 Static - Internal Erosion of Foundation Materials

The specific description of this failure mode is that an erodible, homogeneous silty sand layer within the upper stiff lacustrine exists that is hydrologically constrained downstream of the dam core, but is connected to the reservoir, leading to high head beneath the downstream toe of the dam. The shape and likelihood of the existence of the silty sand layer within the upper stiff lacustrine is highly dependent on how deep the SCB slurry wall extends down through the dam and into the upper stiff lacustrine material. Due to the presence of small, isolated holes from this layer through the upper stiff lacustrine, flow from the silty sand initiates, leading to concentrated seepage paths in the silty sand layer with sufficient velocities to begin the erosion and transport of the silty sand layer. Assuming Type B does not serve as a filter for this silty sand, or that the hole in the stiff lacustrine leads to direct ejection of silty sand downstream of the Type B material, the erosion would progress and lead to the development of a “pipe” within the silty sand material. As the pipe enlarges, additional seepage and higher velocities would result in more erosion. Ultimately, the developing piping pathway could progress to the reservoir and lead to a complete erosion failure, or collapse and create a sinkhole, which leads to crest loss and overtopping. The RET members also considered a variation of the same failure mode, postulating that the development of a pipe may initiate if high gradients push fines from the silty sand layer into the base of a stone column. As voids in the stone column matrix fill with fines and the local gradient decreases, the piping pathway proceeds to the next stone column, ultimately progressing to the reservoir on the upstream side and Type B material shell on the downstream side. If Type B material is not filter-compatible with the silty sand layer, the erosion would continue and lead to failure as discussed above.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by means of an event tree discussed later (Chapter 8.0).

7.1.1.3 FM No. 3 Seismic - Deformation and Overtopping of Embankment

This failure mode postulates that seismic shaking occurs, leading to transient slope failures. These failures accrue deformation leading to crest loss, resulting in overtopping.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by means of an event tree discussed later (Chapter 8.0).

7.1.1.4 FM No. 4 Seismic - Deformation and Internal Erosion of Embankment

This failure mode postulates that seismic shaking occurs, leading to transient slope failures. These failures accrue deformation not sufficient to overtop the dam, but sufficient to cause a defect in the embankment’s SCB slurry wall. This defect allows concentrated seepage paths to form within the dam embankment, with sufficient velocities to begin the erosion and transport of Type A soil particles. Assuming Type B does not serve as a filter for Type A, the erosion would progress and lead to the development of a “pipe” within the Type A material. As the pipe enlarges, additional seepage velocities would lead to higher velocities and result in more erosion in the Type A materials and the SCB wall. Ultimately, the developing piping pathway could progress to the reservoir and lead to a complete erosion failure, or collapse and create a sinkhole which leads to toppling of the SCB wall, continued erosion, crest loss, and overtopping.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by means of an event tree discussed later (Chapter 8.0).

7.1.1.5 FM No. 5 Seismic - Deformation and Internal Erosion of Foundation Materials

The specific description of this failure mode is a silty sand layer exists within the upper stiff lacustrine exists that is hydrologically constrained downstream of the dam core, but is connected to the reservoir, leading to high head beneath the downstream toe of the dam. Seismic shaking occurs, leading to cracking or a small hole in the upper stiff lacustrine material. Flow of water through the hole or crack from the silty sand initiates, leading to concentrated seepage paths in the silty sand layer with sufficient velocities to begin the erosion and transport of the silty sand layer. Assuming Type B does not serve as a filter for this silty sand, or that the hole in the upper stiff lacustrine leads to direct ejection of silty sand downstream of the Type B material, the erosion would progress and lead to the development of a “pipe” within the silty sand material. As the pipe enlarges, additional seepage velocities would lead to higher velocities and result in more erosion. Ultimately, the developing piping pathway could progress to the reservoir and lead to a complete erosion failure, or collapse and create a sinkhole, which leads to crest loss and overtopping. The shape and likelihood of the existence of the silty sand layer within the upper stiff lacustrine is highly dependent on how deep the SCB slurry wall extends down through the dam and into the upper stiff lacustrine material.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by means of an event tree discussed later (Chapter 8.0).

7.1.1.6 FM No. 6 Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment

We assumed that subsurface explorations fail to find a liquefiable layer in the foundation. This failure mode postulates that seismic shaking occurs, leading to liquefaction of a silty sand layer within the upper stiff lacustrine foundation of the dam, leading to transient slope failures. These failures accrue deformation leading to crest loss, resulting in overtopping. This failure mode is very similar to FM No. 3. However, because the sand dam and the rockfill dam with rock notches embankments do not incorporate mitigation of potentially liquefiable materials in the upper stiff lacustrine, neither embankment would meet the design criteria of a yield acceleration of 0.17g. Yield acceleration with liquefied foundation would likely be approximately 0.03 to 0.05g. Therefore, while the structure of the event tree is the same as described in FM No. 3 for the sand dam, the probability of failure is likely to be different.

7.1.2 Rockfill Dam with Rock Notches and Maximum Seismic Filters

7.1.2.1 FM No. 7 Static - Internal Erosion of Embankment

The description of this failure mode for the rockfill dam with rock notches design alternative (Figure D.A.7) is similar to FM No. 1 for the sand dam alternative. It postulates that a defect exists in the embankment's SCB slurry wall that allows concentrated seepage paths to form within the dam embankment, with sufficient velocities to begin the erosion and transport of Zone A (sand/gravel core) soil particles. Assuming Fine Rockfill does not serve as a filter for Zone A, the erosion would progress and lead to the development of a "pipe" within the Zone A material. As the pipe enlarges, additional seepage and higher velocities would result in more erosion in the Zone A materials and the SCB slurry wall. Ultimately, the developing piping pathway could progress to the reservoir and lead to a complete erosion failure, or collapse and create a sinkhole which leads to toppling of the SCB wall, continued erosion, crest loss, and overtopping.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by comparing to the event tree developed for FM No. 1 for the sand dam, and evaluated it by means of an event tree discussed later (Chapter 8.0).

7.1.2.2 FM No. 8 Static - Internal Erosion of Foundation Materials

In general, this failure mode for the rockfill dam with rock notches alternative is the same as described in FM No. 2 for the sand dam, with a shorter seepage path.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by developing an event tree, as discussed in Chapter 8.0.

7.1.2.3 FM No. 9 Seismic - Deformation and Overtopping of Embankment

In general, because of the consistent design criteria (i.e., yield acceleration equal to 0.17g), this failure mode for the rockfill dam with rock notches alternative is the same as described in FM No. 3 for the sand dam.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by developing an event tree, as discussed in Chapter 8.0.

7.1.2.4 FM No. 10 Seismic - Deformation and Internal Erosion of Embankment

In general, this failure mode for the rockfill dam with rock notches alternative is the same as described in FM No. 4 for the sand dam. It postulates that seismic shaking causes sufficient deformations to develop a defect in the embankment's SCB slurry wall. This defect allows concentrated seepage paths to form within the dam embankment, with sufficient velocities to begin the erosion and transport of sand/gravel core (Zone A) soil particles. If Fine Rockfill does not serve as a filter for Zone A, the erosion would progress. As it progresses, additional seepage velocities would lead to higher velocities and result in more erosion in the Zone A materials and the SCB wall. Ultimately, the developing piping pathway could progress to the reservoir and create a sinkhole which leads to toppling of the SCB wall, continued erosion, crest loss, and overtopping.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by considering how it would compare to the event tree developed for FM No. 4 for the sand dam (Chapter 8.0).

7.1.2.5 FM No.11 Seismic - Deformation and Internal Erosion of Foundation Materials

This failure mode for the rockfill dam with rock notches design alternative is the same as FM No. 5 for the sand dam, with a shorter seepage path.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by means of an event tree discussed later (Chapter 8.0).

7.1.2.6 FM No. 6 Seismic - Liquefaction of Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment

Failure mode FM No. 6 described for the sand dam also applies to the rockfill dam with rock notches.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by means of an event tree discussed later (Chapter 8.0).

7.2 Mid-Sea Barrier

The mid-Sea barrier would be a relatively low head structure, designed to allow considerable amounts of seepage (Figure D.A.8). The risk team reviewed expected seepage performance and considered it highly unlikely that the structure would be subject to static and/or seismically induced seepage failures and did not evaluate the risks associated with these types of failures.

In general, because of the consistent design criteria (i.e., yield acceleration equal to 0.17g), this structure is likely to have a seismic overtopping failure mode that is similar to FM No. 3 for the sand dam.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by considering how it would compare to the event tree developed for FM No. 3 for the sand dam (Chapter 8.0).

7.3 Perimeter Dikes

The perimeter dikes would be relatively low to medium head structures (Figure D.A.10). The risk team reviewed expected seepage performance and considered it likely that while the structures could be subject to static and/or seismically-induced seepage failures, that those risks would be lower than for the mid-Sea dam embankments. In general, because of the consistent design criteria (i.e., yield acceleration equal to 0.17g), this structure is likely to have a seismic overtopping failure mode that is similar to FM No. 3 for the sand dam.

The risk team considered these mechanisms to be plausible failure modes, and evaluated them by considering how they would compare to the event trees developed for FM Nos. 3, 4, 5, and 6 for the sand dam (Chapter 8.0).

7.4 South-Sea Dam

The south-Sea dam will essentially be the same embankment configuration as the mid-Sea dam (Figure D.A.9). The risk team reviewed information available regarding subsurface conditions and other than the exception to be described below, they found no significant differences in the geology and/or uncertainty about the geology for this alignment. Both sites appeared to be equally challenging. Therefore, the team considered it likely this structure could be subject to static and/or seismically induced failures and that those risks would be similar to those for the mid-Sea dam embankments.

The risk team considered these mechanisms to be plausible failure modes, and evaluated them by considering how they would compare to the event trees developed for FM Nos. 1, 2, 3, 4, 5, and 6 for the sand dam (Chapter 8.0).

7.4.1 FM No. 12 Seismic - Offset and Translation of Embankment

One significant difference for the south-Sea dam is geologic and seismological evidence of a fault transition zone along the dam alignment. Evaluations by Reclamation suggest that fault offsets as high as 5 meters could occur in the vicinity of the Sea. Further, they believe that it is likely that a surface expression of this fault offset could occur under the south-Sea dam, as currently proposed. The fault offset is bounded by Imperial fault behavior on the south and possibly San Andreas fault behavior near Bombay Beach. The characteristic deformation would be horizontal with a minor component of vertical movement.

The specific description of this failure mode is an earthquake causes fault offsets up to 5 m along the San Andreas/Imperial Fault Zone and up to 2 meters along the fault transition zone at south-west corner of the Sea that propagate to the ground surface, causing offsets in the embankment. The fault offsets damage and cause displacements of zones and cutoffs within the dam that are either large enough to cause direct seepage increases or lead to formation of defects that can progress to piping. High velocity flow leads to erosion of the embankment, crest loss, and results in overtopping. Note, this failure mode assumes no effort to determine fault offsets history or to mitigate dam design.

The risk team considered this mechanism to be a plausible failure mode, and evaluated it by means of an event tree discussed later (Chapter 8.0).

7.5 North-Sea Dam

The north-Sea dam would essentially be the same embankment configuration as the mid-Sea dam (Figure D.A.9). The risk team reviewed information available regarding subsurface conditions and found no significant differences in the geology and/or uncertainty about the geology for this alignment. Both sites appeared to be equally challenging. Therefore, the team considered it likely this structure could be subject to static and/or seismically induced failures and that those risks would be similar to those for the mid-Sea dam embankments.

The risk team considered these mechanisms to be plausible failure modes, and evaluated them by considering how they would compare to the event trees developed for FM Nos. 1, 2, 3, 4, 5, and 6 for the sand dam (Chapter 8.0).

7.6 Concentric Lakes Dikes

The concentric lakes dikes would be relatively low to medium head structures. The risk team reviewed expected seepage performance and considered it likely that while the structures could be subject to static and/or seismically-induced seepage failures, that those risks would be lower than for the mid-Sea dam embankments. In general, because of the consistent design criteria (i.e., yield acceleration equal to 0.17g), this structure is also likely to have a seismic overtopping failure mode that is similar to FM No. 3 for the sand dam. Finally, because the lakes dikes cross over both the Imperial / San Andreas Fault alignment and the transition zone at the southwest corner of the Sea, they are likely to be subject to a potential failure mode that is similar to FM No. 12 for the south-Sea dam.

The risk team considered these mechanisms to be plausible failure modes, and evaluated them by considering how they would compare to the event trees developed for FM Nos. 3, 4, 5, and 6 for the sand dam and FM No. 12 for the south-Sea dam (Chapter 8.0).

8.0 Estimation of Annual Probability of Failure

As each of the failure modes was defined and then understood, the RET then began the process of discussing each step of the failure mechanism and assessing probabilities. For all of the above plausible failure modes, event trees were utilized to assess the overall probability of embankment failure. These event trees are provided in Attachment B.

8.1 Mid-Sea Dam

8.1.1 Sand Dam with Stone Columns

8.1.1.1 FM No. 1 Static - Internal Erosion (Piping) of Embankment

The following events must take place in order for dam failure to occur:

Initiation of Internal Erosion – Erosion initiates at a defect in the SCB slurry wall

a. Continuation – Filtered exit of seepage from Type A to Type B is deficient
b. Progression – Material capable of supporting a roof
c. Progression – Erosion can occur and flows are not limited
d. Intervention – Intervention is unsuccessful, leading to failure

The following is a discussion of the factors affecting the likelihood of occurrence for each of the above events.

Event FM No. 1a Erosion initiates at a defect in the SCB slurry wall

This critical node involved a detailed discussion of whether a defect would exist, and whether sufficient velocity to begin the erosion of individual soil grains would develop through the defect. Table 1 in Attachment C includes a summary of the factors contributing to this condition and the team's considerations that make this event more or less likely.

For this project, the team considered the defect to be an opening in the SCB wall equal to the width of a panel (6 to 10 feet) and 1 to 3 feet in height. The team felt this necessary to develop sufficient quantities of seepage to allow the failure mode to initiate. Construction defects that could cause this size of a defect were considered to be: 1) caving during trench excavation; 2) bad grout mix and; 3) movement of surrounding soil that causes SCB wall to offset.

Some of the key observations suggesting this node is likely are listed below:

- Construction must take place over 8 miles and would require installation of more than 7,000 panels. With such a large number of SCB wall panels to construct, even strict quality control/quality assurance procedures may miss a defect.
- A magnitude 5 earthquake is likely at some point in the more than 400 days required to build the SCB wall. Such an earthquake may damage newly placed SCB material that did not have time to gain strength.
- Several mechanisms for a defect to develop during construction are plausible, such as caving of trench side walls during SCB slurry placement, improper soil cement bentonite mix, loss of trench fluid, not advancing the SCB wall deep enough at certain areas, unexpected interruption of placement due to weather conditions or other circumstances, etc.

Some of the key observations suggesting this node is unlikely include:

- Because of the extensive exploration program, depth to which the SCB wall should be installed would be well known
- Construction practice includes well-established quality control procedures and with good past construction performance records
- SCB slurry sets within 24 hours and gains 70% strength in 7 days
- SCB wall would be constructed in panels, which would constrain the length of a potential defect

Based on the above discussion, the team estimated that the probability of this node was within a range from 0.0001 to 0.01. After further discussion, the team considered that due to cost of this project, it was likely that quality control measures would be strict and therefore considered that the probability of this node would be controlled to a range from 0.0001 to 0.001(uniform distribution).

Event FM No. 1b Filtered exit of seepage from Type A to Type B is deficient

In this node, the team discussed the probability of whether the Type B would serve as a filter for finer Type A particles. A high probability indicates the Type B does not serve as an effective filter, and a low number indicates that it does. Table 2 in Attachment C presents factors and considerations discussed by the team that make this event more or less likely.

Type B material will be placed through several feet of water. Such method of placement is hard to impossible to control. Material characteristics quality control will occur while stockpiling and handling of the material, but not during the actual placement. As particles are dropped through water, some segregation can occur due to pluviation and lenses of a coarser fraction may develop within the Type B shell. These observations suggest that this node is likely. However, the team concluded that the likelihood of these lenses being interconnected over a long distance to create a continuous channel from the Type A/B interface to the downstream slope is low.

Furthermore, Types A & B materials would most likely be processed from the same borrow source and would have similar gradations, with Type B being slightly coarser. The specifications would require that Type A material have less than 10% fines. Relatively high gradients and velocities are necessary to move soil particles in a granular material. In addition, permeability of the material with less than 10% fines would be high, so hydraulic head would drop off quickly, decreasing local gradients and further reducing the potential for erosion.

Based on the above discussion, the team estimated that the probability of this node was within a range from 0.005 to 0.02 (uniform distribution).

Event FM No. 1c Materials are capable of supporting a roof

This branch of the event tree addresses the probability that the Type A within the seepage path is capable of supporting a roof. Table 3 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

The team found little reason to expect the Type A to support a roof. The Type A consists of clean sand with less than 10 percent fines. This type of material exhibits no cohesion, when saturated and would not be expected to be able to sustain a crack and support a roof. On the other hand, some apparent cohesion may exist in the partially saturated portion of Type A above the phreatic surface. Partially saturated sand may be able to support a roof; however, gradients above phreatic surface are likely to be significantly smaller than required to initiate erosion. For these reasons, the team estimated the probability of roof support at 0.001

Event FM No. 1d Erosion can occur and flows are not limited

For this node, the team evaluated the probability that some feature, or combination of features, would serve to limit the seepage flows. (A high probability indicates that the material is susceptible to erosion and there is little to limit the flow, while a low probability indicates the presence of features that would serve to throttle flows along a seepage path). Table 4 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

The team believed there were potential limiting factors for seepage. For one thing, the SCB is not easily erodible, and at least initially, flows would be limited by the size of the opening/defect through the SCB wall. Type B consists of cohesionless materials with a range of particle sizes from gravel to sand. This zone may well serve as a crackstopper, supplying particles to a seepage path that would help clog a developing erosion pathway within the Type A core. In addition, the stone column reinforcement within Type A may further reduce potential for erosion.

For these reasons, the team estimated that the probability that erosion would occur and flows would not be limited would have a probability of 0.01 to 0.1 (uniform distribution).

Event FM No. 1e Intervention is unsuccessful

At this point in the event tree, a stable roofed tunnel has formed through the Type A and the soil is being actively eroded by the flow of water. This particular node then addresses whether early intervention can halt the erosion process. (A high number indicates that the failure process is not likely to either be detected or be stopped, while a low number indicates it is likely that the ongoing failure would be recognized and effectively halted.) Table 5 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

Factors making early intervention less likely to succeed include potential difficulties in detecting erosion if downstream water is high, such as during first filling, and infrequent monitoring. Conversely, the team saw some reasons that would suggest early intervention would succeed. Key factors included the potential that the failure may develop slowly, relatively simple remedial measures could be undertaken, and relatively easy visual detection of erosion (boils, flume of deposited material, etc.) if the downstream water level is low. Based on these observations, the team estimated that the probability that early intervention would be unsuccessful would be 0.1.

FM No. 1 Estimated annual probability of failure

The above branches of the event tree were multiplied together, in a Monte Carlo analysis consisting of 5,000 iterations, to determine the mean annual probability of failure. Based on this analysis, the mean annual probability of failure for this failure mode is 3.8E-11. A summary of this calculated probability is shown in Table 2D.7 below.

Table 2D.7
Sand Dam Embankment Static Through Seepage Failure

Event Tree Branch	Probability
Defect in SCB wall	6.0E-04
Unfiltered exit at leak	1.3E-02
Material can support roof	1.0E-03
Material is erodible and flow not limited	5.5E-02
Intervention unsuccessful	1.0E-01
Annual Probability of Failure	**3.8E-11**

Two key branches of the event tree are the probabilities dealing with existence of a large defect in the SCB slurry wall and the ability of Type A material to support a roof. The low probabilities for these two factors play a large role in defining the overall low annual failure probability. A robust quality control and quality assurance program would detect the vast majority of construction related defects, and they would be repaired before the structure is put in service. A good performance records for SCB walls installed in California levees over a long distance provide a strong justification for the lack of any sizable defects capable of producing concentrated flow with sufficient velocity to erode the cohesionless Type A core reinforced with stone columns. In addition, based on the proposed gradation requirements there is a very high likelihood that the Type B would serve as an effective filter for Type A. Based on these two branches, a relatively low failure probability appears reasonable.

8.1.1.2 FM No. 2 Static - Internal Erosion of Foundation Materials

The following events must occur in order for dam failure to occur:

a. Necessary condition – A constrained, high-head silty sand inclusion exists, undetected in the upper stiff lacustrine.
b. Necessary condition – Erosion initiates at a hole in the upper stiff lacustrine from the inclusion.
c. Initiation – Velocity is sufficient to start erosion in the inclusion.
d. Progression – Material capable of supporting a roof, erosion can occur and flows are not limited.
e. Intervention – Intervention is unsuccessful, leading to failure.

The following is a discussion of the factors affecting the likelihood of occurrence for each of the above events.

Event FM No. 2a A constrained, high-head silty sand inclusion exists, undetected in the upper stiff lacustrine

This critical node involved a detailed discussion of whether a constrained, high-head silty sand inclusion exists, undetected in the upper stiff lacustrine. Table 6 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

Explorations to date with CPT indicate the presence of coarse-grained inclusions within the upper stiff lacustrine and previous reports (URS, 2005) describe silty sand lenses in this layer. Accordingly, the team believed that the existence of a layer with permeability of at least two orders of magnitude higher than the surrounding upper stiff lacustrine is likely. Another necessary condition for this node is that this inclusion, located below Seafloor deposits and soft lacustrine (or upper alluvial) layers is connected to the reservoir on the upstream side. The possible mechanisms to expose the inclusion include desiccation, ancient erosion channels, and sand dunes that could have existed in the Seabed when it was dry. On the other hand, the team considered that depositional environment of upper stiff lacustrine implies that fat clay has been placed continuously for long periods, making a connection to the reservoir less likely and that if cracks existed upstream, there is no reason they wouldn't exist downstream. Further, proposed explorations on close centers would likely identify these inclusions and design can be adjusted to address the conditions.

Based on the above discussion, the team estimated that the probability of this node in a range from 0.0001 to 0.005 (uniform distribution).

Event FM No. 2b Erosion initiates at a hole in the upper stiff lacustrine from the inclusion

In this node, the team discussed the likelihood that the downstream constraint would be breached into a single small isolated defect in the downstream clay layer. In addition, the team considered the probability that this isolated hole would be large enough to allow seepage velocities to develop and start erosion in the inclusion, but small enough to maintain high head in the inclusion to allow continued piping progression. (A high probability indicates such a hole could exist and a low number indicates it would not.) Table 7 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

The fact that the upper stiff lacustrine layer is between 4 and 31.5 feet thick and is located below the soft lacustrine (or alluvial) and Seafloor deposits layers make this condition unlikely. However, the team identified several factors that increase the likelihood of this node. In particular, natural and man-made penetrations may exist in the Seabed and extend deep enough to be connected to the silty sand inclusion in the upper stiff lacustrine. Natural penetrations include animal burrows, roots, and old sand boils or and mud holes developed during previous seismic activity in the area. Man-made penetrations may consist of relief wells, old foundations, and other remnants of pre-Sea human activity.

Based on the above discussion, the team estimated that the probability of this node was within a range from 0.0001 to 0.007 (uniform distribution).

Event FM No. 2c Velocity is sufficient to start erosion in the inclusion

In this node, the team discussed the probability of whether seepage velocity would be sufficient to start erosion in the inclusion. (A high probability indicates erosion would occur and a low number indicates it would not.) Table 8 in Attachment C presents factors and considerations discussed by the team that make this event more or less likely.

The team believed there were potential limiting factors for seepage. The sand dam cross-section has a wide footprint and a seepage path from the reservoir to the downstream exit point would be approximately 1,200 feet, while the total head that would be dissipated over this distance is approximately 50 feet. Accordingly, average gradient for this flow path would be low. Typical permeability of silty sand is in the range from 10^{-3} to 10^{-5} cm/sec. The expected low permeability of the inclusion and even lower permeability of the surrounding clay, combined with the low hydraulic gradients may imply low seepage velocities and limited flows. On the other hand, the homogeneous silty sand may be highly erodible, and velocities on the order of 1 to 2 ft/sec may initiate erosion.

Based on the above discussion, the team judged that the development of velocities high enough to start erosion is unlikely and estimated the probability of this node to be in the range from 0.05 to 0.5 (uniform distribution).

Event FM No. 2d Materials are capable of supporting a roof, erosion can occur and progression is not limited

This branch of the event tree addresses the probability that the inclusion within the seepage path would be capable of supporting a roof and would erode, resulting in unlimited progression. Table 9 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

The team found little reason to expect that silty sand inclusion itself and the surrounding upper stiff lacustrine would not be able to support a roof. Upper stiff lacustrine is described as highly plastic, mostly stiff to very stiff clay, although locally firm. Based on limited consolidation test data, this stratum appears to be normally consolidated. This type of material would be expected to be able to sustain a crack and support a roof. Depositional environment of stiff lacustrine indicates that a continuous layer of silty sand (particle size range of 0.1 to 0.5 mm) can be uniformly graded over extensive distances and there is virtually unlimited supply of water in the reservoir to sustain erosion progress. However, the thickness of such layers is likely to be limited to a couple feet. As silty sand is eroded away, overlying upper stiff lacustrine clay would gradually sag into the void and, since the inclusion layers are believed to be relatively thin, may be able to close it off completely without developing vertical cracks or shearing off. The team hypothesized that erosion would progress laterally, along the centerline, rather than in the upstream direction.

The team judged this node very unlikely to unlikely with reasonable low and reasonable high probability estimates of 0.001 to 0.01 respectively (uniform distribution). This was largely based on the estimated thickness of the silty sand inclusion. If the inclusion were significantly thicker, on the order of several feet, the probability of unlimited progression would be higher.

Event FM No. 2e Intervention is unsuccessful

At this point in the event tree, a stable roofed tunnel would have formed through the silty sand inclusion and the soil is being actively eroded by the flow of water. This particular node addresses whether early intervention can halt the erosion process. (A high number indicates that the failure process is not likely to either be detected or to be stopped, while a low number indicates it is likely that the ongoing failure would be recognized and effectively halted.) Table 10 in Attachment C is a listing of the team’s factors and considerations that make this event more or less likely.

Key considerations for this node include when new and dangerous seepage might be detected and whether or not efforts to stop the erosion process prior to the breach initiation would be successful. With high tailwater, which is likely during the first filling, ongoing erosion and new or changing seepage would be difficult to detect visually. Potential difficulties in detecting erosion in the foundation are also associated with the fact that the magnitude of subsidence due to internal erosion would likely be about the same as typical settlement (if layer is few inches to 1 or 2 feet thick). On the opposite side, the team saw some reasons that would suggest early intervention would succeed. Key factor included slow development time, which would allow for construction of necessary modifications.

In general, the team was concerned that an erosion failure may not be easily detected by observations. Various instruments and remote-sensing technologies could be used to aid in detecting potential problems as early as possible and could be considered as potential risk reduction measures. Based on these observations, the team estimated that the probability that early intervention would be unsuccessful would be neutral with reasonably low and reasonably high probabilities of 0.1 and 0.7 respectively (uniform distribution).

FM No. 2 Estimated annual probability of failure

The above branches of the event tree were multiplied together, in a Monte Carlo analysis consisting of 5,000 iterations, to determine the mean annual probability of failure. Based on this analysis, the mean annual probability of failure for this failure mode is 6.09E-09. A summary of this calculated probability is shown in Table 2D.8 below.

Table 2D.8
Sand Dam Embankment Static Underseepage Failure

Event Tree Branch	Probability
Constrained inclusion exists	2.6E-03
Correct size isolated hole in upper stiff lacustrine exists	3.6E-03
Velocity is sufficient to start erosion	2.8E-01
Material can support a roof, is erodible and flow is not limited	6.0E-03
Intervention unsuccessful	4.0E-01
Annual Probability of Failure	**6.1E-09**

Three key branches of the event tree are the probabilities dealing with the existence of an undetected silty sand inclusion in the upper stiff lacustrine, the existence of a correctly sized and isolated hole, and the likelihood of unlimited progression of erosion failure. The low probabilities for these three nodes play a large role in defining the overall low annual failure probability. Several factors make these nodes very unlikely. The silty sand inclusion needs to be undetected despite an extensive exploration on close centers. It needs to be isolated, small, and constrained on the downstream side to maintain high exit gradient, and it needs to be connected to the reservoir on the upstream side to apply full reservoir head conditions. Further, the inclusion needs to be relatively thick, so that the overlaying upper stiff lacustrine does not close off the void and stop erosion from progressing upstream. Based on these three branches, a relatively low failure probability appears reasonable.

8.1.1.2 FM No. 3 Seismic - Deformation and Overtopping of Embankment

This failure mode, presented by a single node on the decision tree, has three sub-nodes. The first sub-node addresses uncertainties in the strength parameters of the Type A material (sand core reinforced with stone columns). The second sub-node estimated deformations that would occur in the dam under various seismic loads for a given strength of the Type A material. The third and last sub-node discusses the likelihood the dam would fail by overtopping as a function of residual freeboard.

The following is a discussion of the factors affecting the likelihood of occurrence for each of the above events.

Event FM No. 3a Type A material strength distribution and
Event FM No. 3b Deformations under various seismic loads

Absent improvements to the sand core of the barrier, the RET determined that liquefaction would likely be triggered for characteristic earthquakes on the San Andreas and Imperial valley faults and the barrier would fail. This would result in an annual probability of failure of greater than 1E-02. The risk team then assessed whether drained or undrained strength criteria were appropriate for the barrier sand core if the core was improved with stone columns. The team recommended a minimum performance specification be established for construction requiring that the stone column improved Type A material in the central portion of the sand dam should have an N_{1-60} blowcount of no less than 20. Based on the results of stability and FLAC evaluations (Appendix 2C of the Kleinfelder's complete report), this requirement would ensure that undrained strengths had a lower bound. The group decided that it would then be appropriate to use an equivalent Su convention to represent the lower strength boundary of this Type A material. The lower bound of the strength was set as the lower bound of the Seed and Harder curve (Seed and Harder, 1990) with an equivalent N_{1-60} blowcount of 20 corresponding to an undrained strength of 1,000 psf. A middle bound of 1,600 psf was adopted based on a calculation of the average strength along the failure surface and the calculation: Su = 40 ft x 65 pcf x tan (32^o), representing a drained, but saturated strength. The upper bound was set at 3,000 psf based on a similar calculation: Su = 40 ft x 125 pcf x tan (32^o), which would be equivalent to a drained strength for the material if it did not liquefy and was not saturated. Then, based on these anticipated strengths, the results of the slope stability analyses were used to estimate likely yield accelerations for the embankment.

The RET then estimated likely deformations for a particular seismic load based on the results of the simplified Newmark deformation and FLAC analyses (Appendix 2C). The deformations depend on the estimated yield accelerations, which depend on the strength of the sand core. The corresponding deformations estimated with FLAC suggest that for an embankment constructed with a material with a minimum undrained strength of 1,000 psf and a yield acceleration of at least 0.17g, little to no deformations are expected under Load 1 (0 to 0.26g PGA). In addition, no deformations are expected under any earthquake loads for the upper bound strength estimates of 3,000 psf. If the Type A material has strength between 1,000 and 2,000 psf, the team estimated deformations to be in the ranges from 0.01 to 1 foot, from 0.5 to 4 feet, and from 1 to 6 feet for Load 2 (0.26g to 0.70g), Load 3 (0.70g to 0.90g), and Load 4 (>0.90g), respectively. Table 2D.9

shows estimated deformation as a function of seismic load and material strength. Table 11 in Attachment C presents factors considered by the RET in development of this relationship.

Table 2D.9
Deformation versus Type A material strength

Type A material strength, psf	Deformation, ft						
	Load 4		Load 3		Load 2		Load 1
	max	min	max	min	max	min	expected
1000	6	4	4	2	1	0.1	0
2000	2	1	0.8	0.5	0.1	0.01	0
3000	0	0	0	0	0	0	0

Event FM No. 3c Overtopping potential as a function of residual freeboard

The risk team developed a "fragility" curve to represent the relationship between the amount of residual freeboard in feet, and the possibility of dam failure by overtopping. Table 12 in Attachment C presents factors and considerations discussed by the team that make this event more or less likely. The following table summarizes the results.

Table 2D.10
Probability of failure versus freeboard

Probability of failure at this residual freeboard	Minimum freeboard, ft	Maximum freeboard, ft
0	1.5	4
0.1	1	3
0.5	-0.1	1.5
0.9	-0.85	1
0.95	-0.75	0.75
1	-1	0.5

The key reasons supporting the above estimate include:

- SCB slurry wall would not deform and would block transverse, open, deep cracks from developing.
- Wind that can produce significant waves is relatively frequent in the area and wave run up can be several feet.

- Without additional erosion protection, central portion of the sand dam comprised of sandy Type A material may be prone to erosion under conditions of overflow that would be more than 3 to 6 inches.
- Sand bag or Geotube® intervention is included in estimates, but potential mitigation measures such as additional crest armoring with rock and reinforcement of the upper portion of the SCB wall are not included. Depending on the outcome, these measures could be included in the design to reduce the risk of failure due to overtopping.

FM No. 3 Estimated annual probability of failure

The above sub-nodes were analyzed together, in a Monte Carlo analysis consisting of 5,000 iterations, to determine the mean annual probability of failure. Based on this analysis, the mean annual probability of failure for this failure mode, calculated as the maximum probability of failure under any seismic load, is 3.80E-06. Estimated probabilities of failure due to seismic Loads 1 through 4 are presented in Table 2D.11 below. As shown in the table, the sand dam would most likely fail by overtopping due to an earthquake with PGA of 0.9g or higher.

Table 2D.11
Sand Dam Embankment Seismic Overtopping

Seismic Load	Probability
Load 1	1.0E-10
Load 2	6.5E-14
Load 3	7.8E-07
Load 4	3.8E-06
Annual Probability of Failure	**3.8E-06**

8.1.1.4 FM No. 4 Seismic - Deformation and Internal Erosion of Embankment

The following events would need to take place in order for dam failure to occur:

a. Initiation of Internal Erosion – Erosion initiates at a defect in the SCB slurry wall
b. Continuation – Filtered exit of seepage from Type A to Type B is deficient
c. Progression – Material capable of supporting a roof
d. Progression – Erosion can occur and flows are not limited

e. Intervention – Intervention is unsuccessful, leading to failure

The following is a discussion of the factors affecting the likelihood of occurrence for each of the above events.

Event FM No. 4a Erosion initiates at a defect in the SCB slurry wall

This critical node involved a detailed discussion of the likelihood the SCB slurry wall would be damaged by an earthquake such that large seepage quantities flow through the wall. Table 13 in Attachment C includes likely and unlikely factors contributing to this conditions and considerations that make this event more or less likely.

FLAC results (Appendix 2C) suggest that the highest shear strains in the SCB wall would be at the contact between the dam and upper stiff lacustrine material. Strains would be large enough to induce cracking but not a complete offset of the SCB wall. Due to cracking, permeability of the SCB within the defect would increase approximately by 2 orders of magnitude (i.e., from $1x10^{-6}$ to $1x10^{-4}$ cm/sec). Defect development was discussed at three possible locations:

- Shear near crest of the dam
- Shear at base
- Shear in weak area or defect constructed in SCB wall

Overall, the group postulated the most likely location for a significant defect was at the base of the SCB wall. If the base of the wall was damaged enough to cause two orders of magnitude change in permeability, the unit rate of seepage through the wall could change from about 0.0001 to 0.01 cfs. Over a three-mile length, the leakage would increase from about 2 cfs to 200 cfs. This amount of seepage was considered by the team to be a failure of the sand dam system.

The likelihood of the SCB wall being damaged by an earthquake greatly depends on the seismic load. Based on estimated strains and deformation characteristics of the SCB material, the team theorized that the wall should exhibit elastic behavior under seismic Load 1 (PGA less than 0.26g) and therefore would sustain no damage. Increasingly more severe fracturing is expected for Loads 2 and 3. As the load and shear increases, the block size would decrease and the aperture would increase. The team estimated that one mile of the SCB wall, or 1/8th of the total structure length, could be damaged under Load 2, and up to 3 miles of the wall could be damaged under Load 3. Load 4 (PGA greater than 0.9g) should cause extensive damage to the SCB wall.

Based on the above discussion, the team estimated that the probability of this node as follows:

Table 2D.12
Probability of defect in SCB wall due to seismic load

Seismic Load	Probability (uniform distribution)		
	Reasonable Low	Best Estimate	Reasonable High
Load 1	-	0	-
Load 2	0.01	-	0.1
Load 3	0.9	-	0.99
Load 4	-	1	-

Event FM No. 4b Filtered exit of seepage from Type A to Type B is deficient

In this node, the team discussed the probability of whether the Type B would serve as a filter for finer Type A particles. Table 14 in Attachment C presents factors and considerations discussed by the team that make this event more or less likely. This node is similar to Event FM No.1b, for which the estimated probability that Type B is not filter compatible with Type A was between 0.005 and 0.02. Seismic shaking would likely cause transient failures in the outer shells and some (or all) Type B material may slide away, reducing the distance from the Types A/B interface to the seepage exit face. This would make the possibility of an unfiltered exit more likely. Based on the above discussion, the team estimated that the probability of this node would increase by the factor of 2, compared to event FM No. 1b and estimated it to be within a range from 0.01 to 0.04 (uniform distribution).

Event FM No. 4c Materials are capable of supporting a roof

This branch of the event tree addresses the probability that the Type A within the seepage path is capable of supporting a roof. Table 15 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely. This node is similar to event FM No.1c. After reviewing the mechanisms and factors influencing the ability of Type A to form and support a roof, the team did not see a reason to change the probability of this node from static to seismic conditions. Accordingly, the probability of roof support for this failure mode was considered to be the same as for Event FM No.1c and equal to 0.001

Event FM No. 4d Erosion can occur and flows are not limited

For this node, the team evaluated the probability that some feature, or combination of features, would serve to limit the seepage flows. (A high probability indicates that the material is susceptible to erosion and there is little to limit the flow, while a low probability indicates the presence of features that would serve to throttle flows along a seepage path). Table 16 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

The team believed that all factors restricting seepage under static conditions (FM No. 1) would also apply to this failure mode. Type B consists of cohesionless materials with a range of particle sizes from gravel to fines. This zone may well serve as a crackstopper, supplying particles to a seepage path that would help clog a developing erosion pathway within the Type A core. The stone column reinforcement within Type A may also reduce potential for erosion. In addition, the intact SCB would not be easily erodible, and at least initially, flows would be limited by the size of the opening/defect through the SCB wall. However, during an earthquake SCB material may crack or crush, making it less resistant to erosion. As discussed in Event FM No. 4a of this failure mode, the extent of cracking would be largely dependent on the seismic loading experienced by the SCB wall. The higher the load, the more likely progression would be unlimited. At the end, the team judged that the positive factors outweigh the negative factors and, although increased compared to static conditions, the probability of unlimited progression would be low. The team estimated the probability of progressive erosion as follows:

Table 2D.13
Probability of unlimited erosion

Seismic Load	Probability (uniform distribution)	
	Reasonable Low	Reasonable High
Load 1	0.01	0.1
Load 2	0.01	0.1
Load 3	0.01	0.1
Load 4	0.02	0.15

Event FM No. 4e Intervention is unsuccessful

This node addresses whether early intervention could halt the erosion process. (A high number indicates that the failure process is not likely to either be detected or to be stopped, while a low number indicates it is likely that the ongoing failure would be recognized and effectively halted.) Table 17 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

The team saw some reasons that would suggest early intervention would succeed. Key factors included slow development and relatively simple remediation. However, an earthquake could cause damage to the project infrastructure, and other remedial measures may take priority, limiting resources and attention available to this failure mode. Other factors making early intervention less likely to succeed include potential difficulties in detecting erosion if downstream water is high and difficult/restricted access immediately after an earthquake, due to road damage or a failure of another portion of the dam. The team judged that the intervention would be more likely to be successful for lower seismic loads.

Table 2D.14
Intervention unsuccessful

Seismic Load	Probability (uniform distribution)	
	Reasonable Low	**Reasonable High**
Load 1	Not considered	
Load 2	0.2	0.4
Load 3	0.4	0.6
Load 4	0.7	0.9

FM No. 4 Estimated annual probability of failure

The above branches of the event tree were multiplied together, in a Monte Carlo analysis consisting of 5,000 iterations, to determine the mean annual probability of failure. Based on this analysis, the mean annual probability of failure for this failure mode, calculated as the maximum probability of failure under any seismic load, is 3.06E-11. A summary of this calculated probability is shown in Table 2D.15 below.

Table 2D.15
Sand Dam Embankment Seismic Through Seepage Failure

Seismic Load	Probability
Load 1	0
Load 2	1.5E-11
Load 3	2.5E-11
Load 4	3.1E-11
Annual Probability of Failure	**3.1E-11**

The key branch of this event tree is the likelihood of a large defect developing in the SCB slurry wall due to an earthquake. The probability of this node largely would depend on the size of an earthquake and ranges from 0 for an earthquake with PGA less than 0.26g to 1 for an earthquake with PGA greater than 0.9g. Compared to the associated static failure mode (FM No. 1), all nodes of this event tree are as or more likely to occur. However, taking into account the annual probability of an earthquake, this failure mode has the same overall annual probability of failure as FM No. 1.

8.1.1.5 FM No. 5 Seismic - Deformation and Internal Erosion of Foundation Materials

This failure mode is similar to the static failure mode FM No. 2. The following events must take place in order for dam failure to occur:

a. Necessary condition – A constrained, high-head silty sand inclusion exists, undetected in the upper stiff lacustrine
b. Necessary condition – Erosion initiates at a hole in the upper stiff lacustrine from the inclusion
c. Initiation – Velocity is sufficient to start erosion in the inclusion
d. Progression – Material capable of supporting a roof, erosion can occur and flows are not limited
e. Intervention – Intervention is unsuccessful, leading to failure

The following is a discussion of the factors affecting the likelihood of occurrence for each of the above events.

Event FM No. 5a A constrained, high-head silty sand inclusion exists, undetected in the upper stiff lacustrine

This critical node involved a detailed discussion of whether a constrained, high-head silty sand inclusion could exist, undetected in the upper stiff lacustrine. Table 18 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

This node is similar to Event FM No. 2a. After considering how a silty sand inclusion may develop within the upper stiff lacustrine stratum and discussing factors influencing the likelihood of its existence, the team did not see a reason to change the probability of this node from static to seismic conditions. Accordingly, the probability of inclusion existence for this failure mode was considered to be the same as for Event FM No. 2a and range from 0.0001 to 0.005 (uniform distribution).

Event FM No. 5b Erosion initiates at a hole in the upper stiff lacustrine from the inclusion

For this event, the team discussed the likelihood that the downstream constraint would be breached into a single small isolated defect in the downstream clay layer. In addition, the team considered the probability that this isolated hole would be large enough to allow seepage velocities to develop and start erosion in the inclusion, but small enough to maintain high head in the inclusion to allow continued piping progression. (A high probability indicates such a hole could exist and a low number indicates it would not.) Table 19 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

The team believed that all factors contributing to this condition under the static failure mode Event FM No. 2b would also apply to this failure mode. The fact that the upper stiff lacustrine layer is between 4 and 31.5 feet thick and is located below the soft lacustrine (or alluvial) and Seafloor deposits layers would make the existence of a small isolated defect connecting the inclusion to the ground surface very unlikely. However, several factors would increase the likelihood of this node. In particular, natural and man-made penetrations may exist in the Seabed and extend deep enough to be connected to the silty sand inclusion in the upper stiff lacustrine. Natural penetrations include animal burrows, roots, old sand boils and mud holes developed during previous seismic activity in the area, and man-made penetrations may consist of relief wells, old foundations, and other remnants of pre-Sea human activity, and poorly grouted exploration borings. Seismic shaking may cause development of additional defects, further increasing probability of this event. For example, an earthquake may damage grouted exploration holes. Seismic loading could also increase pore water pressure in the silty sand layer, which could lead to a blow out failure, if excess pore water pressure is higher than overburden pressure. Accordingly, the team judged that this node is ten times more likely to occur under seismic conditions than under static and estimated its probability to be within a range from 0.001 to 0.07 (uniform distribution).

Event FM No. 5c Velocity is sufficient to start erosion in the inclusion

In this node, the team discussed the probability of whether seepage velocity would be sufficient to start erosion in the inclusion. (A high probability indicates erosion would occur and a low number indicates it would not.) Table 20 in Attachment C presents factors and considerations discussed by the team that make this event more or less likely.

This node is similar to Event FM No. 2c. The team did not see a reason to change the probability of this node from static to seismic conditions and judged that the probability of seepage velocities being sufficient to start erosion was in the range from 0.05 to 0.5 (uniform distribution).

Event FM No. 5d Materials are capable of supporting a roof, erosion can occur and progression is not limited

This branch of the event tree addresses the probability that the inclusion within the seepage path would be capable of supporting a roof and would erode, resulting in unlimited progression. Table 21 in Attachment C presents factors and considerations that make this event more or less likely.

This node is similar to Event FM No. 2d. The team found little reason to expect that silty sand inclusion itself and the surrounding upper stiff lacustrine would not be able to support a roof under static or seismic conditions. Upper stiff lacustrine is described as highly plastic, mostly stiff to very stiff clay, although locally firm. Based on limited consolidation test data, this stratum appears to be normally consolidated. This type of material would be expected to be able to sustain a crack and support a roof. Depositional environment of upper stiff lacustrine indicates that a continuous layer of silty sand (particle size range of 0.1 to 0.5 mm) can be uniformly graded over extensive distances and there is virtually unlimited supply of water in the reservoir to sustain erosion progress. However, the thickness of such layers would likely be limited to a couple feet. As silty sand would be eroded away, overlying upper stiff lacustrine clay would gradually sag into the void and, since the inclusion layers are believed to be relatively thin, may be able to close it off completely without developing vertical cracks or shearing off. The team hypothesized that erosion would progress laterally, along the centerline, rather than in the upstream direction.

The team did not see a reason to change the probability of this node from static to seismic conditions and judged this node to be very unlikely to unlikely with reasonably low and reasonably high probability estimates of 0.001 to 0.01 respectively (uniform distribution). This was largely based on the estimated thickness of the silty sand inclusion. If the inclusion were significantly thicker, in an order of several feet, the probability of unlimited progression would be higher.

Event FM No. 5e Intervention is unsuccessful

This node addresses whether early intervention could halt the erosion process. (A high number indicates that the failure process is not likely to either be detected or to be stopped, while a low number indicates it is likely that the ongoing failure would be recognized and effectively halted.) Table 22 in Attachment C is a listing of the team’s factors and considerations that make this event more or less likely.

The erosion failure would likely take a long time to develop and could be detected through routine visual observations (murky water downstream, sand boils etc), if tailwater is low. The slow development time could allow for construction of necessary modifications. On the other hand, the magnitude of subsidence due to internal erosion would likely be about the same as typical settlement (if layer is few inches to 1 or 2 feet thick), making it potentially difficult to detect. In addition, an earthquake may cause damage to the project infrastructure, and other remedial measures may take priority, limiting resources and attention available to this failure mode. Immediately after an earthquake, the access to the problem area may be restricted due to road damage or a failure of another portion of the dam. The team judged that intervention under seismic conditions would be less likely to be successful under seismic conditions than under static.

Based on these observations, the team estimated that the probability that early intervention would be unsuccessful would have reasonably low and reasonable high probabilities of 0.5 and 0.9, respectively (uniform distribution).

FM No. 5 Estimated annual probability of failure

The above branches of the event tree were multiplied together, in a Monte Carlo analysis consisting of 5,000 iterations, to determine the mean annual probability of failure. Based on this analysis, the mean annual probability of failure for this failure mode is 8.0E-08. A summary of this calculated probability is shown in Table 2D.16 below.

Table 2D.16
Sand Dam Embankment Seismic Underseepage Failure

Event Tree Branch	Probability
Constrained inclusion exists	1.6E-07
Correct size isolated hole in upper stiff lacustrine exists	6.3E-05
Velocity is sufficient to start erosion	1.8E-03
Material can support a roof, is erodible and flow is not limited	6.4E-03
Intervention unsuccessful	1.2E-00
Annual Probability of Failure	**8.0 E-08**

Three key branches of the event tree are the probabilities dealing with the existence of an undetected silty sand inclusion in the upper stiff lacustrine, the existence of a correctly sized and isolated hole and the likelihood of unlimited progression of erosion failure. The low probabilities for these three nodes play a large role in defining the overall low annual failure probability. Several factors make these nodes very unlikely. The silty sand inclusion would need to be undetected despite the expected extensive exploration on close centers. It would need to be constrained on the downstream side to maintain high exit gradient, and it would need to be connected to the reservoir on the upstream side to apply full reservoir head conditions. The hole on the downstream side would need to be isolated and small enough to maintain high head and erosive gradients in the inclusion. Further, the inclusion would need to be relatively thick, so that the overlaying upper stiff lacustrine does not close off the void and stop erosion from progressing upstream. Based on these three branches, a relatively low failure probability appears reasonable.

Compared to the associated static failure mode (FM No. 2), all nodes of this event tree are as or more likely to occur. However, taking into account the annual probability of an earthquake, this failure mode is two orders of magnitude lower overall annual probability of failure than FM No. 2.

8.1.1.6 FM No. 6 Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment

This failure mode postulates that seismic shaking would cause liquefaction of a silty sand layer within the upper stiff lacustrine foundation of the dam, leading to transient slope failures. These failures accrue deformation leading to crest loss, resulting in overtopping.

It was previously noted that this risk analysis has been based on the assumption that there are no liquefiable layers in the upper stiff lacustrine deposit. This failure mode, on the other hand, is based on the assumption that an undetected and unmitigated liquefiable layer does exist. We assumed that such layer was not found by the subsurface exploration or, if it was found, no remedial actions were taken to address the issue. The probability of failure was then evaluated for each of the four seismic loads. The team reasoned that the following events would need to take place for a failure to occur:

a. Necessary Condition – Silty sand layer with $N_{1\text{-}60}$ blowcounts of 8 or less exists undetected and unmitigated in the upper stiff lacustrine
b. Initiation – Silty sand inclusion/layer in upper stiff lacustrine liquefies
c. Continuation – Significant deformations of embankment
d. Progression – Failure by overtopping

The following is a discussion of the factors affecting the likelihood of occurrence for each of the above events.

Event FM No.6a Silty sand inclusion/layer with $N_{1\text{-}60}$ of 8 or less exists undetected and unmitigated in the upper stiff lacustrine

The likelihood of a similar condition was explored in detail in FM No. 2a. That event was considered to be virtually impossible with the probability in the range from 0.0001 to 0.005 (uniform distribution). The key difference between FM No.2a and FM No.6a is the number of necessary conditions required for these nodes. In addition to the existence of a silty sand inclusion in the upper stiff lacustrine, Event FM No.2a postulated that this inclusion was subjected to high head and was constrained on the downstream side. The last two conditions are not necessary for initiation of the FM No.6. Accordingly, The RET estimated the probability of FM No.6a to be five times higher than that estimated for FM No. 2a (0.0005 to 0.01)

Event FM No. 6b Silty sand inclusion/layer in upper stiff lacustrine liquefies under seismic load

The team judged that silty sand material with equivalent N_{1-60} blowcount of 8 would almost certainly liquefy under an earthquake with PGA greater than 0.26g. Accordingly, the probability of liquefaction under Loads 2, 3, and 4 was estimated at 1.0. The probability of liquefaction under Load 1 was estimated at 0.01.

Event FM No. 6c Significant deformations of embankment and
Event FM No. 6d Failure by overtopping

These two events, described in Tables 23 and 24 of Attachment C, are presented by a single node on the failure mode event tree.

Failure mode FM No. 6 is similar to FM No. 3 with one key difference. The embankment yield acceleration with liquefied upper stiff lacustrine would be approximately 0.03 to 0.05g, which is significantly lower than the design criteria of 0.17g, assumed to be met for FM No. 3. The team judged that embankment deformations due to liquefaction in the upper stiff lacustrine would be at least 5 feet for all seismic loads. The design freeboard for the mid-Sea dam is set at 5 feet. Therefore, these deformations would result in a residual freeboard of zero, leading to crest loss and initial overtopping. Actual deformations may be significantly higher. Based on the Newmark analysis, deformations for a yield acceleration of 0.05g would be between 15 and 30 feet.

Event FM No.6c is similar to Event FM No. 3c. The team reviewed “fragility” curve developed for FM No. 3c and concluded that it would also apply to FM No. 6c. This fragility curve, described in Table D.10, represents the relationship between the amount of residual freeboard in feet, and the possibility of dam failure by overtopping. The key reasons supporting the above estimate are presented in Table 24, Attachment C and discussed in detail for FM No. 3c.

FM No. 6 Estimated annual probability of failure

The above sub-nodes were analyzed together, in a Monte Carlo analysis consisting of 5,000 iterations, to determine the mean annual probability of failure. Based on this analysis, the mean annual probability of failure for this failure mode, calculated as the maximum probability of failure under any seismic load, is 3.95E-05. Assuming unmitigated liquefaction in the upper stiff lacustrine, the sand dam's estimated probabilities of failure due to seismic Loads 1 through 4 are presented in Table 2D.17 below.

Table 2D.17
Sand Dam Embankment Seismic Overtopping with Liquefaction in Upper Stiff Lacustrine

Seismic Load	Probability
Load 1	4.0E-05
Load 2	3.2E-06
Load 3	2.1E-07
Load 4	1.2E-07
Annual Probability of Failure	**4.0E-05**

8.1.2 Rockfill Dam with Rock Notches

8.1.2.1 FM No. 7 Static - Internal Erosion of Embankment

The following events would need to take place in order for dam failure to occur:

a. Initiation of Internal Erosion – Erosion initiates at a defect in the SCB slurry wall

b. Continuation – Filtered exit of seepage from sand gravel core to fine rockfill, to coarse rockfill is deficient

c. Progression – Material capable of supporting a roof

d. Progression – Erosion can occur and flows are not limited

e. Intervention – Intervention is unsuccessful, leading to failure

As can be seen from the list of events above, this potential failure mode is similar to FM No. 1 for the sand dam, with a few key differences. The rockfill dam would have three progressively coarser internal zones, instead of two. Gradation of the sand/gravel core would be filter compatible with the adjacent fine rockfill zone, and outer rockfill shells would serve as a filter for fine rockfill, which would make node b less likely. Further, materials comprising the rockfill dam are coarser than those used in the sand dam construction. Accordingly, higher velocities and gradients would be required to initiate and sustain internal erosion of the rockfill dam zones, further reducing the probability of failure. Based on these considerations the team concluded that the overall annual probability of failure of rockfill dam with rock notches due to internal erosion is lower than the 3.8E-11 estimated for the sand dam. Because it is well below Reclamation's criteria, a detailed estimation of the risk was not performed.

8.1.2.2 FM No. 8 Static - Internal Erosion of Foundation Materials

This failure mode is similar to FM No. 2. The following events would need to take place in order for dam failure to occur:

a. Necessary condition – A constrained, high-head silty sand inclusion exists, undetected in the upper stiff lacustrine
b. Necessary condition – Erosion initiates at a hole in the upper stiff lacustrine from the inclusion
c. Initiation – Velocity is sufficient to start erosion in the inclusion
d. Progression – Material capable of supporting a roof, erosion can occur and flows are not limited
e. Intervention – Intervention is unsuccessful, leading to failure

The following is a discussion of the factors affecting the likelihood of occurrence for each of the above events.

Event FM No. 8a A constrained, high-head silty sand inclusion exists, undetected in the upper stiff lacustrine

This critical node involved a detailed discussion of whether a constrained, high-head silty sand inclusion exists, undetected in the upper stiff lacustrine. Table 25 in Attachment C presents factors considered by the team that make this event more or less likely.

Explorations to date with CPT indicate the presence of coarse-grained inclusions/layers within the upper stiff lacustrine and previous reports (URS, 2005) describe silty sand lenses in this layer. The team argued that proposed explorations on close centers would likely identify these inclusions and design could be adjusted to address the conditions; however, the horizontal distance from the upstream rock notch to the downstream rock notch is approximately 400 feet, or one third of that for the sand dam geometry. A smaller inclusion, which is easier to miss with explorations, would be sufficient to satisfy conditions for this event to occur. Accordingly, the team concluded that the existence of a layer with permeability of at least two orders of magnitude higher than the surrounding upper stiff lacustrine is likely. Another necessary condition for this event is that this inclusion, located below Seafloor deposits and soft lacustrine (or alluvial) layers, is connected to the reservoir on the upstream side. The mechanisms to expose the inclusion include desiccation, ancient erosion channels, and sand dunes that could have existed in the Seabed when it was dry. On the other hand, the team considered that depositional environment of upper stiff lacustrine implies that fat clay has been placed continuously for long periods, making a connection to the reservoir less likely. Further, vertical distance from the bottom of the upstream rock notch to a pervious inclusion at the bottom of the downstream rock notch would be approximately 40 feet. A connection of inclusion to the full reservoir head due to mechanisms discussed is unlikely at such depth.

Based on the above discussion, the team estimated that the probability of this node in a range from 0.0005 to 0.01 (uniform distribution).

Event FM No. 8b Erosion initiates at a hole in the upper stiff lacustrine from the inclusion

In this node, the team discussed the likelihood that the downstream constraint would be breached into a single small isolated defect in the downstream clay layer. In addition, the team considered the probability that this isolated hole would be large enough to allow seepage velocities to develop and start erosion in the inclusion, but small enough to maintain high head in the inclusion to allow continued piping progression. (A high probability indicates such a hole could exist and a low number indicates it would not.)

This node is similar to Event FM No. 2b. Table 7 in Attachment C is a listing of the team's factors and considerations that also make this event more or less likely. Natural and man-made penetrations may exist in the Seabed and extend deep enough to be connected to the silty sand inclusion in the upper stiff lacustrine. Natural penetrations include animal burrows, roots, old sand boils, and mud holes developed during previous seismic activity in the area, and man-made penetrations may consist of relief wells, foundations, and other remnants of pre-Sea human activity. The team identified factors that increase the likelihood of this event node compared to Event FM No. 2b. The fact that the inclusion may be directly below the base of the downstream rock notch, rather than several feet below the ground surface, makes this condition more likely.

After considering the differences, the team judged this node was more likely than Event FM No. 2b by a factor of 10. The low and high estimates were therefore 0.001 to 0.07, respectively (uniform distribution).

Event FM No. 8c Velocity is sufficient to start erosion in the inclusion

In this node, the team discussed the probability of whether seepage velocity would be sufficient to start erosion in the inclusion. (A high probability indicates erosion would occur and a low number indicates it would not.) Table 8 in Attachment C presents factors and considerations discussed by the team that also make this event more or less likely.

This event is similar to FM No. 2c, with a few key differences. Typical permeability of silty sand is in the range from 10^{-3} to 10^{-5} cm/sec, which would limit flow velocity and flows. However, the seepage path length from the upstream notch to the downstream notch is approximately 400 feet (instead of

1,200 feet) and full head would dissipate over a shorter distance, resulting in higher gradients. The homogeneous silty sand may be highly erodible, and velocities on the order of 1 to 2 ft/sec may initiate erosion. Therefore, the team judged this event to be more likely for FM No. 8 than for FM No. 2 and estimated probability of this node to be in the range from 0.1 to 0.7 (uniform distribution).

Event FM No. 8d Materials are capable of supporting a roof, erosion can occur and progression is not limited

This branch of the event tree addresses the probability that the inclusion within the seepage path is capable of supporting a roof and would erode, resulting in unlimited progression. Table 9 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

The team found little reason to expect that silty sand inclusion itself and the surrounding upper stiff lacustrine would not be able to support a roof. Stiff lacustrine is described as highly plastic, mostly stiff to very stiff clay, although locally firm. Based on limited consolidation test data, this stratum appears to be normally consolidated. This type of material would be expected to be able to sustain a crack and support a roof. Depositional environment of stiff lacustrine indicates that a continuous layer of silty sand can be uniformly graded over extensive distances and there is virtually unlimited supply of water in the reservoir to sustain erosion progress. However, the thickness of such layers is likely to be limited to a couple feet. As silty sand is eroded away, overlying upper stiff lacustrine clay would gradually sag into the void and, since the inclusion layers are believed to be relatively thin, may be able to close it off completely without developing vertical cracks or shearing off. The team hypothesized that erosion would progress laterally, along the centerline, rather than in the upstream direction.

The seepage path between rock notches is only 400 feet, or one-third of a typical seepage path through the sand dam foundation, making conditions for unlimited progression more likely for FM No. 8 than for FM No. 2. On the other hand, rockfill is less erodible than sand and the failure progression may halt, once the pipe reaches the upstream rockfill shell. Accordingly, probability of this event for FM No. 8 is estimated to be the same as Event FM No. 2d for the sand dam (0.001 to 0.01).

Event FM No. 8e Intervention is unsuccessful

At this point in the event tree, a stable roofed tunnel has formed through the silty sand inclusion and the soil is being actively eroded by the flow of water. This particular node addresses whether early intervention can halt the erosion process. (A high number indicates that the failure process is not likely to either be detected or to be stopped, while a low number indicates it is likely that the ongoing failure would be recognized and effectively halted.) Table 10 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

Factors making early intervention less likely to succeed include potential difficulties in detecting changing or new seepage and the presence of downstream rockfill, which complicates the placement of a weighted filter,

Key considerations for this node include when new and dangerous seepage might be detected and whether or not efforts to stop the erosion process prior to the breach initiation would be successful. With high tailwater, which is likely during the first filling, ongoing erosion, and new or changing seepage would be difficult to detect visually. Transported material would likely be hidden in the rockfill matrix and erosion may continue undetected for a long period of time, even with a low or no tailwater. Potential difficulties in detecting erosion in foundation are also associated with the fact that the magnitude of subsidence due to internal erosion would likely be about the same as typical settlement (if layer is few inches to 1 or 2 feet thick). On the opposite side, the team saw some reasons that would suggest early intervention would succeed. The key factor included slow development time, which could allow for construction of necessary modifications.

In general, the team was concerned that an erosion failure may not be easily detected by observations. Various instruments and remote-sensing technologies could be used to aid in detecting potential problems as early as possible and may be considered as potential risk reduction measures. However, the team also felt that the rock notches dam might be slightly more vulnerable to an undetected failure than the sand dam. Based on these observations, the team estimated that the probability that early intervention would be unsuccessful would be likely with reasonably low and reasonably high probabilities of 0.2 and 0.9, respectively (uniform distribution).

FM No. 8 Estimated annual probability of failure

The above branches of the event tree were multiplied together, in a Monte Carlo analysis consisting of 5,000 iterations, to determine the mean annual probability of failure. Based on this analysis, the mean annual probability of failure for this failure mode is 2.26E-07. A summary of this calculated probability is shown in Table 2D.18 below.

Table 2D.18
Rockfill Dam with Rock Notches Embankment Static Underseepage Failure

Event Tree Branch	Probability
Constrained inclusion exists	5.0E-03
Correct size isolated hole in upper stiff lacustrine exists	3.6E-02
Velocity is sufficient to start erosion	4.0E-01
Material can support a roof, is erodible and flow is not limited	6.0E-03
Intervention unsuccessful	5.5E-01
Annual Probability of Failure	2.3E-07

Three key branches of the event tree are the probabilities dealing with the existence of an undetected silty sand inclusion in the upper stiff lacustrine, the existence of a correctly sized and isolated hole, and the likelihood of unlimited progression of erosion failure. The low probabilities for these three nodes play a large role in defining the overall low annual failure probability. Several factors make these nodes very unlikely. The silty sand inclusion would need to be undetected in spite of exploration on close centers. It would need to be constrained on the downstream side to maintain high exit gradient, and it needs to be connected to the reservoir on the upstream side to apply full reservoir head conditions. Further, the inclusion would need to be relatively thick, so that the overlying upper stiff lacustrine does not close off the void and stop erosion from progressing upstream.

The annual probability of failure due to underseepage is two orders of magnitude higher for the rockfill dam with rock notches than for the sand dam with stone columns. This is largely because the seepage path through the sand dam foundation is three times longer than the seepage path through the upper stiff lacustrine foundation connecting upstream and downstream rock notches. Based on this factor, a higher, but still relatively low failure probability for FM No. 8 appears reasonable.

8.1.2.3 FM No. 9 Seismic - Deformation and Overtopping of Embankment

Similar to FM No. 3, this failure mode, presented by a single node on the decision tree, has three sub-nodes. The first sub-node addresses uncertainties in the strength parameters of the upper stiff lacustrine stratum. The second sub-node estimated deformations that would occur in the dam under various seismic loads for a given strength of the upper stiff lacustrine material. The third and last sub-node discusses the likelihood the dam would fail by overtopping as a function of residual freeboard. The following is a discussion of the factors affecting the likelihood of occurrence for each of the above events.

Event FM No. 9a Upper stiff lacustrine strength distribution and
Event FM No. 9b Deformations under various seismic loads

The group agreed to use an equivalent undrained strength (Su) convention to represent the strength of the upper stiff lacustrine material. The lower bound of the strength was set at 1,000 psf. This value was estimated assuming a linear increase of strength with depth (Su/σ'_v of 0.3) and an average depth of a failure surface of 60 feet. The upper bound value was estimated at 2,700 psf, assuming frictional resistance of 32 degrees: Su =60 ft x 65 psf x tan (32^o). The most likely value was estimated at 1,500 psf. Pert distribution was used to estimate strength between these values. The strength distribution of the upper stiff lacustrine is very similar to that assumed for the Type A material because of the lack of over-consolidation evidence in these materials.

The group assumed that the range of deformations for this rockfill with rock notches embankment configuration would be the same as predicted for the sand dam alternative (FM No. 3b) based on the fact that the cross-section was set to have a yield acceleration of 0.17g; the same as the sand dam. Table 2D.19 shows estimated deformation as a function of seismic load and material strength. Table 26 in Attachment C presents factors considered by the RET in development of this relationship.

Table 2D.19
Deformation versus upper stiff lacustrine material strength

Upper Stiff lacustrine strength, psf	Deformation, ft						
	Load 4		Load 3		Load 2		Load 1
	max	min	max	min	max	min	expected
1000	6	4	4	2	1	0.1	0
2000	2	1	0.8	0.5	0.1	0.01	0
3000	0	0	0	0	0	0	0

Event FM No. 9c Overtopping potential as a function of residual freeboard

The risk team developed a "fragility" curve to represent the relationship between the amount of residual freeboard in feet, and the possibility of dam failure by overtopping. Table 27 in Attachment C presents factors and considerations discussed by the team that make this event more or less likely. The following table summarizes the results.

Table 2D.20
Probability of failure versus freeboard

Probability of failure at this residual freeboard	Minimum Freeboard	Maximum Freeboard
0	1.5	1
0.1	-3.5	-0.5
0.5	-6.5	-3
0.9	-7.5	-4
1.0	-10	-6

The key reasons supporting the above estimate include:

- SCB wall does not deform and would block transverse, open deep cracks from developing.
- Rockfill is on both sides of the SCB wall that would be highly resistant to erosion during overtopping. The exterior portion of the rockfill shells is currently anticipated to range in size from 1 to 4 feet. Flow through capacity of such rockfill would be very large.
- Wind that can produce significant waves is relatively frequent in the area and wave run up can be several feet, though wave action on the rockfill would likely be less damaging than on the sand dam.

- Sand bag or Geotube® intervention is included in the estimates, but potential mitigation measures such as additional crest armoring with rock and reinforcement of the upper portion of the SCB wall are not included. Depending on the outcome, these measures could be included in the design to reduce the risk of failure due to overtopping.

Generally, the team felt that the rockfill dam with rock notches embankment was less likely to be overtopped for a similar level of deformation on the sand dam with stone columns embankment shown in the fragility curve of Event FM No. 3c.

FM No. 9 Estimated annual probability of failure

The above sub-nodes were analyzed together, in a Monte Carlo analysis consisting of 5,000 iterations, to determine the mean annual probability of failure. Based on this analysis, the mean annual probability of failure for this failure mode, calculated as the maximum probability of failure under any seismic load, is 1.0E-15. Estimated probability of failure due to seismic Loads 1 through 4 are presented in Table 2D.21 below.

Table 2D.21
Rockfill Dam with Rock Notches Embankment Seismic Overtopping

Seismic Load	Probability
Load 1	1.0E-15
Load 2	6.5E-19
Load 3	3.9E-20
Load 4	2.3E-20
Annual Probability of Failure	**1.0E-15**

The key reason this potential failure is very unlikely to occur is the choice of construction materials. Rockfill has a high through-flow capacity and would be highly resistant to erosion under overtopping. Even for an earthquake with PGA greater than 0.9g, vertical deformation of the crest would not be expected to exceed 6 feet. Design freeboard at the maximum normal pool is 5 feet. Accordingly, the dam would be overtopped at the most by 1 foot. The team believed that rockfill with rock sizes from 1 foot to 4 feet in diameter could sustain such overtopping without damage.

8.1.2.4 FM No.10 Seismic - Deformation and Internal Erosion of Embankment

The following events must take place in order for dam failure to occur:

a. Initiation of Internal Erosion – Erosion initiates at a defect in the SCB slurry wall

b. Continuation – Filtered exit of seepage from sand gravel core to fine rockfill, to coarse rockfill is deficient
c. Progression – Material capable of supporting a roof
d. Progression – Erosion can occur and flows are not limited
e. Intervention – Intervention is unsuccessful, leading to failure

As can be seen from the list of events above, this potential failure mode is similar to FM No. 4 for the sand dam, with a few key differences. The rockfill dam would have three progressively coarser internal zones, instead of two. Gradation of the sand/gravel core would be filter compatible with the adjacent fine rockfill zone, and outer rockfill shells would serve as a filter for fine rockfill, which would make node b less likely. Further, materials comprising the rockfill dam would be coarser than those used in the sand dam construction. Accordingly, higher velocities and gradients would be required to initiate and sustain internal erosion of the rockfill dam zones, further reducing the probability of failure. Based on these considerations the team concluded that this failure mode has the annual probability of failure equal or lower than the 3.1E-11 estimated for the sand dam (FM No. 4). A detailed estimation of the risk was not performed.

8.1.2.5 FM No. 11 Seismic - Deformation and Internal Erosion of Foundation Materials

This failure mode is similar to FM No. 8. The following events must take place in order for dam failure to occur:

a. Necessary condition – A constrained, high-head silty sand inclusion exists, undetected in the upper stiff lacustrine
b. Necessary condition – Erosion initiates at a hole in the upper stiff lacustrine from the inclusion
c. Initiation – Velocity is sufficient to start erosion in the inclusion
d. Progression – Material capable of supporting a roof, erosion can occur, and flows are not limited
e. Intervention – Intervention is unsuccessful, leading to failure

Event FM No. 11a A constrained, high-head silty sand inclusion exists, undetected in the upper stiff lacustrine

This critical node involved a detailed discussion of whether a constrained, high-head silty sand inclusion could exist, undetected in the upper stiff lacustrine. Table 28 in Attachment C presents factors that make this event more or less likely.

This node is similar to Event, FM No. 8a. After considering how a silty sand inclusion may develop within the upper stiff lacustrine stratum and discussing factors influencing the likelihood of its existence, the team identified no reason to change the probability of this node from static to seismic conditions. Accordingly, the probability of inclusion existence for this failure mode was considered to be the same as for Event FM No. 8a and range from 0.0005 to 0.01 (uniform distribution).

Event FM No. 11b Erosion initiates at a hole in the upper stiff lacustrine from the inclusion

In this node, the team discussed the likelihood that the downstream constraint would be breached into a single small isolated defect in the downstream clay layer. In addition, the team considered the probability that this isolated hole would be large enough to allow seepage velocities to develop and start erosion in the inclusion, but small enough to maintain high head in the inclusion to allow continued piping progression. (A high probability indicates such a hole could exist and a low number indicates it would not.)

This node is similar to Event FM No. 5b. Table 19 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely. Natural and man-made penetrations may exist in the Seabed and extend deep enough to be connected to the silty sand inclusion in the upper stiff lacustrine. Natural penetrations include animal burrows, roots, old sand boils and mud holes developed during previous seismic activity in the area. Man-made penetrations may consist of relief wells, old foundations, and other remnants of pre-Sea human activity.

The team judged the likelihood of this node would not change from static to seismic conditions. The low and high estimates were therefore the same as for Event FM No. 5b and equal to 0.001 to 0.07, respectively (uniform distribution).

Event FM No. 11c Velocity is sufficient to start erosion in the inclusion

In this node, the team discussed the probability of whether seepage velocity would be sufficient to start erosion in the inclusion. (A high probability indicates erosion would occur and a low number indicates it would not.) Table 20 in Attachment C presents factors and considerations discussed by the team that would make this event more or less likely.

This node is similar to FM No. 5c. The team saw no reason the likelihood of seepage velocities being sufficient to start erosion would be different under seismic and static conditions. Accordingly, the low and high probabilities of this node were estimated at 0.1 and 0.7, respectively, same as the probability of Event FM No. 5c (uniform distribution).

Event FM No. 11d Materials are capable of supporting a roof, erosion can occur, and progression is not limited

This branch of the event tree addresses the probability that the inclusion within the seepage path would be capable of supporting a roof and would erode, resulting in unlimited progression. Table 21 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

This node is similar to FM No. 5d. The team saw no reason that the likelihood of unlimited progression would be different under seismic and static conditions. Accordingly, the low and high probabilities of this node were estimated at 0.001 and 0.01, respectively, same as the probability of FM No. 5d (uniform distribution).

Event FM No. 11e Intervention is unsuccessful

This node addresses whether early intervention could halt the erosion process. (A high number indicates that the failure process is not likely to either be detected or to be stopped, while a low number indicates it is likely that the ongoing failure would be recognized and effectively halted.) Table 22 in Attachment C is a listing of the team's factors and considerations that make this event more or less likely.

The erosion failure would likely take a long time to develop and could be detected through routine visual observations (murky water downstream, sand boils etc), if tailwater is low. The slow development time could allow for construction of necessary modifications. However, transported material is likely to be hidden in the rockfill matrix and erosion may continue undetected for a long period of time, even with low or no tailwater. In addition, the magnitude of subsidence due to internal erosion would likely be about the same as typical settlement (if layer is few inches to 1 or 2 feet thick), making it potentially difficult to detect.

The team judged that intervention would be less likely to be successful under seismic conditions. An earthquake may cause damage to the project infrastructure, and other remedial measures may take priority, limiting resources and attention available to this failure mode. Access to the problem area may be restricted immediately after an earthquake due to road damage or a failure of another portion of the dam.

Based on these observations, the team estimated that the likelihood of early intervention being unsuccessful is slightly higher for this failure mode than for the associated static Event FM No. 5e. Accordingly, reasonably low and high probabilities were estimated at 0.5 and 0.9, respectively (uniform distribution).

FM No. 11 Estimated annual probability of failure

The above branches of the event tree were multiplied together, in a Monte Carlo analysis consisting of 5,000 iterations, to determine the mean annual probability of failure. Based on this analysis, the mean annual probability of failure for this failure mode, calculated as the maximum probability of failure for any seismic load, is 3.1E-07. A summary of this calculated probability is shown in Table 2D.22 below.

Table 2D.22
Rockfill Dam with Rock Notches
Seismic Underseepage Failure

Event Tree Branch	Probability
Constrained inclusion exists	5.3E-03
Correct size isolated hole in upper stiff lacustrine exists	3.6E-02
Velocity is sufficient to start erosion	4.3E-01
Material can support a roof, is erodible and flow is not limited	5.5E-03
Intervention unsuccessful	7.0E-01
Annual Probability of Failure	**3.1E-07**

Three key branches of the event tree are the probabilities dealing with the existence of an undetected silty sand inclusion in the upper stiff lacustrine, the existence of a correctly sized and isolated hole and the likelihood of unlimited progression of erosion failure. The low probabilities for these three nodes play a large role in defining the overall low annual failure probability. Several factors make these nodes very unlikely. The silty sand inclusion would need to be undetected even though there would be an extensive exploration on close centers. It would need to be constrained on the downstream side to maintain high exit gradient, and it would need to be connected to the reservoir on the upstream side to apply full reservoir head conditions. Further, the inclusion would need to be relatively thick, so that the overlying upper stiff lacustrine would not close off the void and stop erosion from progressing upstream.

Compared to the associated static failure mode (FM No. 8), all nodes of this event tree are as or more likely to occur. However, taking into account the annual probability of an earthquake, the overall annual probability of this failure mode is the same order of magnitude as FM No. 8.

8.1.2.6 *FM No. 6 Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment*

Failure mode FM No. 6 described for the sand dam also applies to the rockfill dam with rock notches. The team estimated that deformations in the rockfill dam due to liquefaction of the upper stiff lacustrine foundation would be similar to those induced in the sand dam. Accordingly, the RET judged that the probability of failure for the rockfill dam with rock notches should be the same as estimated for the sand dam and equal to 4.0E-05. Assuming unmitigated liquefaction in the upper stiff lacustrine, the sand dam's estimated probabilities of failure due to seismic Loads 1 through 4 would be similar to those presented in Table 2D.17.

The above discussion of FM No. 6 assumes that no geologic/geotechnical investigations would be conducted to detect whether liquefiable layers exist within the upper stiff lacustrine or that the investigation fails to find this layer. Further, it assumes that no changes to the dam design would be made to mitigate the probability of failure presented above. The RET believes that this would be a "worst-case" scenario and can be effectively mitigated through a thorough investigation program and adaptation of the designs to address any areas where potentially liquefiable layers are found in the upper stiff lacustrine. Subsequent risk numbers are presented assuming that these risks are appropriately mitigated.

8.2 Mid-Sea Barrier

The mid-Sea barrier would be a relatively low head structure, designed to allow considerable amounts of seepage. The risk team judged static and/or seismically induced seepage failures of the mid-Sea barrier to be very unlikely and did not evaluate the risks associated with these types of failures.

Risk associated with seismic overtopping was evaluated by comparing the mid-Sea barrier design (with stone columns) to the mid-Sea dam configuration. Because both structures are designed to the same seismic criteria, (i.e., yield acceleration equal to 0.17g), the mid-Sea barrier is likely to have a seismic overtopping failure mode that is similar to FM No. 3 for the sand dam. Then the team evaluated how conditions leading to a mid-Sea barrier failure by overtopping would compare to the event tree developed for FM No. 3. The RET judged that the likelihood of the mid-Sea barrier failure by overtopping is lower or equal to the probability estimated for FM No. 3 for the sand dam.

8.3 Perimeter Dikes

The perimeter dikes would be relatively low to medium head structures. The risk team reviewed expected seepage performance and concluded that risks associated with statically and seismically induced seepage failures of the perimeter dikes would be equal to or lower than for the mid-Sea dam embankment (FM No. 1, FM No. 2, FM No. 4, and FM No. 5).

Because the perimeter dikes are designed to the same seismic criteria as the mid-Sea dam, and have the same design freeboard of 5 feet, the team judged the probability of failure of these structures due to seismic deformations and overtopping will be equal to or lower than the probabilities estimated for the sand dam (FM No. 3 and FM No. 6).

8.4 South-Sea Dam

The south-Sea dam would essentially be the same embankment configuration as the mid-Sea dam. The risk team reviewed information available regarding subsurface conditions and other than the exception to be described below, they found no significant differences in the geology and/or uncertainty about the geology for this alignment. Both sites appeared to be equally challenging. Therefore, the team concluded that risks associated with static and seismic seepage failures would be similar to those for the mid-Sea dam embankments (FM No. 1, FM No. 2, FM No. 4, and FM No. 5). Further, the team concluded that risks associated seismic deformations and failure by overtopping will also be similar to the estimates developed for the mid-Sea dam (FM No. 3 and FM No. 6).

8.4.1 FM No. 12 Seismic - Offset and Translation of Embankment

Evaluations by Reclamation suggest that fault offsets as high as 5 meters may occur in the vicinity of the Sea. Further, based on available information, it is likely that a surface expression of this offset could occur under the south-Sea dam, as currently proposed. The fault offset is bounded by Imperial fault behavior on the south, fault transition zone near the southwest corner of the Sea and possibly San Andreas fault behavior near Bombay Beach. The characteristic deformation would be horizontal with a minor component of vertical movement.

The risk analysis team believes that there is a 100% certainty that the south-Sea dam would fail if a characteristic rupture of the San Andreas Fault occurred in the dam foundation. The initial judgment of the team is that there is better than a 1 in 100 (.01) chance of dam failure, if a characteristic rupture of the Imperial fault occurred in the south-Sea dam foundation and if no measures are incorporated in the design to mitigate potential for direct breach, overtopping, internal erosion through the translated core, and internal erosion through the translated foundation. If design measures are incorporated, the RET believe the risk associated with these failure modes could be reduced by a factor of 100 times or greater.

The team performed further examination of the failure mode in order to evaluate the approximate 0.01 probability estimate. One decision and two nodes were assigned to this event tree:

- Decision - Are translation mitigation design features incorporated into the structure?
- Event 1 – If translation mitigation design features are not incorporated into the structure, what is the likelihood that displacements on the Imperial-San Andreas step-over translation would be greater than 1 meter?
- Event 2 - – If translation mitigation design features are not incorporated into the structure, what is the likelihood the south-Sea dam would fail by translation, if displacements are greater than 1 meter?

Decision FM No. 12 Translation mitigation design features are incorporated into the structure.

As discussed above, absent design features to mitigate the impacts of translation, the RET believes it is 100% certain that the south-Sea dam will fail. With design features, the RET believe the reliability of the structure can be improved by 100 times or greater. Effectively, the first node in the event tree is not specified and involves a decision, leading to the development of two distinct branches. One for the structure without translation mitigation design features incorporated and one with mitigation design features. With the design features incorporated, the RET believe the probability of failure of the structure will be decreased by two orders of magnitude.

Event FM No. 12a Displacements exceeding 1 meter

As discussed above, evaluations by Reclamation suggest that fault offsets as high as 5 meters may occur in the vicinity of the Sea and it is likely that a surface expression of this fault offset could occur under the south-Sea dam, as currently proposed. The fault offset is bounded by Imperial fault behavior on the south, fault transition zone in the southwest corner of the Sea and possibly San Andreas fault behavior near Bombay Beach. The characteristic deformation would be horizontal with a minor component of vertical movement. Table 29 in Attachment C presents factors considered by the team that make this event more or less likely.

Based on information regarding the “characteristic” earthquakes on the San Andreas and Imperial faults, the team judged that there is a 1 in 80 chance that deformations would exceed 1 meter.

Event FM No. 12b Embankment failure by translation

Table 30 in Attachment C presents factors considered by the team that make this event more or less likely. The team concluded that the south-Sea dam failure by translation is likely if design features to mitigate direct breaching, overtopping, post-event core erosion and post event foundation erosion are not included and deformations exceed 1 meter. The RET estimated the probability of this event at 0.9 for the case without translation mitigation design features. The following key factors were considered in the estimate:

- Strike/slip offset has vertical component, crest settlement of 2 to 4 feet is likely
- With this much displacement significant shaking is very likely and Type B material is likely to slide away from Type A, removing filter
- Strike/slip offset movement is oriented approximately 45 degrees to dam alignment and would cause SCB wall to fail in compression.
- Seepage velocities are likely to be sufficient to start erosion at downstream end of Type A material and the SCB wall would no longer be there to limit progression
- Intervention would likely be impossible because of the rapid failure development

FM No. 12 Estimated annual probability of failure

The above branches of the event tree were multiplied together, in a Monte Carlo analysis consisting of 5,000 iterations, to determine the mean annual probability of failure. Based on this analysis, the mean annual probability of failure for this failure mode is 1.1E-02. A summary of this calculated probability is shown in Table 2D.23 below.

Table 2D.23
South-Sea Dam Seismic
Deformations and Failure by Translation
Translation Mitigation Design Features Not Included

Event Tree Branch	Probability
Deformations exceeding 1 m	1.3E-02
Failure by translation	9.0E-01
Annual Probability of Failure	**1.1E-02**

As discussed above, the probability of failure for this failure mode is expected to be decreased by two orders of magnitude if translation mitigation design features are incorporated into the structure.

8.5 North-Sea Dam

The north-Sea dam would essentially be the same embankment configuration as the mid-Sea dam. The risk team reviewed information available regarding subsurface conditions and other than the exception described below, they found no significant differences in the geology and or uncertainty about the geology for this alignment. Both sites appeared to be equally challenging. Therefore, the team concluded that risks associated with static and seismic seepage failures would be similar to those for the mid-Sea dam embankment (FM No. 1, FM No. 2, FM No. 4, and FM No. 5). Further, the team concluded that risks associated seismic deformations and failure by overtopping would also be similar to the estimates developed for the mid-Sea dam (FM No. 3 and FM No. 6).

8.6 Concentric Lakes Dikes

The concentric lakes dikes would be relatively low to medium head structures. The risk team reviewed expected seepage performance and concluded that risks associated with statically and seismically induced seepage failures of the concentric lakes dikes would be equal to or lower than for the mid-Sea dam embankment (FM No. 1, FM No. 2, FM No. 4, and FM No. 5).

Because perimeter dikes would be designed to the same seismic criteria as the mid-Sea dam, and would have the same design freeboard of 5 feet, the team judged the probability of failure of these structures due to seismic deformations and overtopping would be equal to or lower of the probabilities estimated for the sand dam (FM No. 3, and FM No. 6).

In general, because of the consistent design criteria (i.e., yield acceleration equal to 0.17g), this structure is also likely to have seismic overtopping failure modes that are similar to FM No. 3 and FM No.6 for the sand dam. Finally, because the concentric lakes dikes cross over the Imperial / San Andreas Fault Zone transition and the fault transition zone near the southwest corner of the Sea, they are likely to be subject to a potential failure mode that is similar to FM No. 12 for the south-Sea dam.

The above discussion of FM No. 6 assumes that no geologic/geotechnical investigations would be conducted to detect whether liquefiable layers exist within the upper stiff lacustrine and that no changes to the dam design would be made to mitigate the probability of failure presented above. Likewise, the discussion of FM No. 12 assumes that no adaptive design would be implemented to significantly reduce the risks associated with fault translation The RET believes that these would be “worst-case” scenarios and that both the risks associated with liquefaction in the upper still lacustrine, and fault translation can be effectively mitigated through a thorough investigation program and adaptation of the designs.

Subsequent risk numbers are presented assuming that these risks are appropriately mitigated.

8.7 Habitat Ponds

The habitat ponds would be impounded by low earthfill embankments. The embankments constructed of compacted clay and silt material would have exterior slopes of 3H:1V and height of up to 9 feet. Foundation improvements would include excavation of Seafloor deposits and installation of geogrid reinforcement at the bottom of the excavation trench. The RET considered expected performance of the habitat pond embankments under static and seismic loads and concluded that their performance would be similar to levees. Accordingly, the team estimated APF for various modes based on expert opinion using levee performance data. No detailed evaluations were performed for these estimates.

9.0 Loss of Life Estimates

The risk team evaluated the loss of life (LOL) estimates for each of the different alternatives. The following paragraphs summarize the results of their assessment.

9.1 Population at Risk

The RET estimated the population at risk (PAR) considering previous estimates developed by Reclamation and based on the team's judgment and experience with the project conditions. No other filed data or analysis was used to obtain these estimates.

In general, access to the project structures would be closed to general public and there would be no permanent residents downstream that could be exposed to dam failure flooding. Accordingly, the population-at-risk and the potential loss of life for this project are very low.

The team identified five different population groups that would be potentially exposed to dam failure. The first consists of public motorists on project roads. The number of people in this category was previously estimated between 15 (night-time) and 60 (day-time use). However, the RET judged that the number of people in this category should be zero. All roads at risk on the project site, including crest access roads, would be restricted to Operations and Maintenance (O&M) personnel only. With no public access allowed, there should be no public motorists exposed to dam failure flooding.

The second group consists of boaters on the marine lake. Alternatives No. 1 and No. 4, which include construction of mid-Sea dam, south-Sea dam and north-Sea dam, would create larger marine lakes and attract the highest number of recreational boaters. Habitat ponds and mid-Sea barrier structures, on the other hand, would provide no boating opportunities. Accordingly, the number of boaters at risk associated with a failure of these structures was estimated to be zero. In general, for various reasons boating on the Sea is currently limited to daytime use only. The team judged that the same pattern would continue after the restoration and estimated the nighttime population at risk for this group category to be negligible for all structures.

The third group of people potentially at risk consists of O&M personnel performing routine inspections or repairs. O&M personnel would be trained in dam safety and emergency action plan procedures and should be able to recognize a developing failure before formation of a breach. Regular maintenance would be performed during daytime and there should be no O&M personnel at risk at nighttime, unless it is an emergency. In an emergency, the O&M crews would be appropriately equipped and prepared to deal with potentially dangerous conditions.

The fourth group, which has the highest estimated PAR, includes people fishing from the embankments. The team estimated that during daytime up to 10 people in this category could be at risk associated with failure of the mid-Sea dam and south-Sea dam. The PAR for nighttime was estimated between 1 and 3.

The last group includes wildlife observers, hikers, and other users of recreation opportunities provided by the project.

Tables 2D.24 and 2D.25 present estimated daytime and nighttime PAR for various project structures. The differences in the PAR estimates from structure to structure are largely based on the number and value of recreational opportunities provided by each structure.

Table 2D.24
Estimates of Population at Risk, Daytime

Component	Motorists (1)	Boaters	O&M Personnel	Fishing	Other (2)	Total, PAR average
Mid-Sea-Dam	0	4-9	2	5-10	0-1	18
Mid-Sea-Barrier	0	0	2	2-6 (1-3 to 2-6)	0	6
Perimeter Dike	0	2-5	2	2-6 (1-3 to 5-10)	0-1	11
South-Sea Dam	0	2-5	2	2-6 (1-3 to 5-10)	0-1	11
North-Sea Dam	0	4-9	2	2-6 (1-3 to 2-6)	0-1	14
Concentric Lakes Dikes	0	2-5	2	1-3	0	8
Habitat Ponds	0	0	2	0	0-1	3

Notes: a) Roads at risk would be restricted to O&M personnel access only. No public access would be allowed.
b) Includes wildlife observers, hikers, and other users.

Table 2D.25
Estimates of Population at Risk, Nighttime

Component	Motorists (1)	Boaters	O&M Personnel	Fishing	Other (2)	Total, PAR average
Mid-Sea-Dam	0	0	0	1-3	0	2
Mid-Sea-Barrier	0	0	0	1-3	0	2
Perimeter Dike	0	0	0	1-3	0	2
South-Sea Dam	0	0	0	1-3	0	2
North-Sea Dam	0	0	0	1-3	0	2
Concentric Lakes Dikes	0	0	0	1-3	0	2
Habitat Ponds	0	0	0	0	0	0

Notes: a) Roads at risk would be restricted to O&M personnel access only. No public access would be allowed.
b) Includes wildlife observers, hikers, and other users.

9.2. Flood Severity

As discussed in Chapter 7.0, hydrologic risk for this project is negligible. The failure of the various Salton Sea Alternative structures could result in flooding that would have little impact on constructed facilities, but could affect individuals that happened to be in the path of the flood wave. It appears clear that the flood severity in all reaches would be low. Accordingly, consequences of failure under hydrologic failure modes were not considered in this risk analysis.

The different embankments and their different hydraulic heights would produce different "flood severity" outflows if these structures were to fail. The RET used Reclamation's publication "A Procedure for Estimating Loss of Life Caused by Dam Failure, DSO-99-06" (Reclamation, 1999) to estimate the potential loss of life for each structure. The RET assumed that the flood severity for the low hydraulic head structures should be "low", and this category should apply to the habitat pond embankments, the concentric lakes dikes, the perimeter dikes, and the mid-Sea barrier. The "best estimate" and the suggested range (low to high) in fatality rate corresponding to **low** flood severity are: 0.01 and 0 to 0.02, respectively, assuming no "warning time" and vague "flood severity understanding." Similarly, the RET assumed that the flood severity for the higher hydraulic head structures should be "medium", and this category should apply to the mid-Sea, north-Sea, and south-Sea dams. The "best estimate" and the suggested range (low to high) in fatality rate corresponding to **medium** flood severity are: 0.15 and 0.03 to 0.35, respectively assuming no "warning time" and vague "flood severity understanding." These fatality rates are used in estimating the appropriate LOL numbers for these structures, in the event they were to fail due to seismic loading.

9.3. Loss of Life Estimates

Using the methodology outlined in Reclamation's loss of life estimating procedure (Reclamation, 1999), the RET estimated fatality rates and resulting loss of life for the various project structures.

9.3.1 Consequences of Failure for Static Failure Modes

Consistent with previous studies (Reclamation, 2005d), the RET concluded that for static failure modes, the warning time would be longer than 60 minutes, and following Reclamation guidelines (Reclamation, 1999), the fatality rate would be very low, in the order of 1 in 10,000. Using a 1 in 10,000 fatality rate would result in essentially no loss of life for static failure modes.

9.3.2 Consequences of Failure for Seismic Failure Modes

The embankment structures could fail rapidly under seismic loading, breaching the reservoir in a very short time. However, "high severity" flooding was not deemed appropriate, as the failure would not be instantaneous and/or "explosive", which would be more likely for the sudden failure of a concrete dam. Using Reclamation's methodology (Reclamation, 1999), and assuming either "low" or "medium" flood severity (depending on the embankment's hydraulic height), no warning time, and no flood severity understanding, the lower bound, best estimate, and upper bound estimates of fatality rates and resulting potential loss of life for seismic failure modes are summarized in Tables 2D.26 through 2D.28, respectively.

Table 2D.26
Lower Bound Estimates
Fatality Rate and Loss of Life
Seismic Failure Modes

Component and (Estimated Flood Severity)	Fatality Rate	Loss of Life		
		Day	Night	Average
Mid-Sea dam (medium)	0.03	0.5	0.06	0.28
Mid-Sea barrier (low)	0.00	0	0	0
Perimeter dikes (low)	0.00	0	0	0
South-Sea dam (medium)	0.03	0.3	0.06	0.18
North-Sea dam (medium)	0.03	0.4	0.06	0.23
Concentric lakes dikes (low)	0.00	0	0	0
Habitat ponds embankments (low)	0.00	0	0	0

Table 2D.27
Best Estimates
Fatality Rate and Loss of Life
Seismic Failure Modes

Component and (Estimated Flood Severity)	**Fatality Rate**	**Loss of Life**		
		Day	**Night**	**Average**
Mid-Sea dam (medium)	0.15	2.7	0.3	1.5
Mid-Sea barrier (low)	0.01	0.06	0.02	0.04
Perimeter dikes (low)	0.01	0.11	0.02	0.06
South-Sea dam (medium)	0.15	1.7	0.3	1.0
North-Sea dam (medium)	0.15	2.1	0.3	1.2
Concentric lakes dikes (low)	0.01	0.08	0.02	0.05
Habitat pond embankments (low)	0.01	0.03	0	0.02

Table 2D.28
Upper Bound Estimates
Fatality Rate and Loss of Life
Seismic Failure Modes

Component and (Estimated Flood Severity)	Fatality Rate	Loss of Life		
		Day	Night	Average
Mid-Sea dam (medium)	0.35	6.3	0.7	3.5
Mid-Sea barrier (low)	0.02	0.12	0.04	0.08
Perimeter dikes (low)	0.02	0.22	0.04	0.13
South-Sea dam (medium)	0.35	3.9	0.7	2.3
North-Sea dam (medium)	0.35	4.9	0.7	2.8
Concentric lakes dikes (low)	0.02	0.16	0.04	0.10
Habitat pond (low) embankments	0.02	0.06	0	0.03

The team then developed LOL distributions to be used for calculations of the annualized loss of life (ALL) for each embankment structure. The LOL distributions were developed based on the estimates provided above and are summarized below in Table 2D.29.

Table 2D.29
LOL Estimates for Project Structures, Day/Night Averages
Seismic Failure Modes

Component	Static Failure Modes, LOL	Seismic Failure Modes, LOL		
		Lower Bound	Best Estimate	Upper Bound
Mid-Sea dam	0	0.28	1.5	3.5
Mid-Sea barrier	0	0	0.04	0.08
Perimeter dikes	0	0	0.06	0.13
South-Sea dam	0	0.18	1.0	2.3
North-Sea dam	0	0.23	1.2	2.8
Concentric lakes dikes	0	0	0.05	0.10
Habitat pond embankments	0	0	0.02	0.03

The RET reasoned that because the best estimate LOL values (both static and seismic failure modes, day/night averages) for the mid-Sea barrier, perimeter dikes, concentric lakes dikes, and habitat pond embankments are all far below 1.0, the LOL value to be used for those structures should be therefore zero (0).

10. Summary of Risks

As discussed in Chapter 7.0, the RET evaluated a set of "common" failure modes using the mid-Sea dam and south-Sea dam configurations for the base assessments. Failure modes for other structures were evaluated by comparing conditions that would lead to failure to those discussed for the mid-Sea dam and south-Sea dam configurations. The team then estimated the probability of failure of each structure under a given failure mode by considering how it would compare to the probabilities estimated for the sand dam with stone columns, rockfill dam with rock notches and maximum seismic filters, or south-Sea dam. The results of these evaluations are summarized in Table 2D.30

Table 2D.30
Summary of Risk Estimates for All Embankment Structures

Component		Static - Internal Erosion (Piping) of Embankment	Static - Internal Erosion of Foundation Materials	Seismic - Deformation and Overtopping of Embankment	Seismic - Deformation and Internal Erosion of Embankment	Seismic - Deformation and Internal Erosion of Foundation Materials	Seismic - Liquefaction of Upper Stiff Lacustrine Deformation and Overtopping of Embankment	Seismic - Offset and Translation of Embankment
Mid-Sea-Dam	Sand dam with stone columns	FM1	FM2	FM3	FM4	FM5	FM6	
Mid-Sea-Dam	Rockfill dam with rock notches with maximum seismic filters	FM7 ≤ FM1	FM8	FM9	FM10 ≤ FM4	FM11	FM6	
Mid-Sea barrier		≤ FM1 (no stone columns)	≤ FM2 (no stone columns)	≤ FM3 (with stone columns)	≤ FM4 (with stone columns)	≤ FM5 (with stone columns)		
Perimeter dikes		≤ FM1	= FM2	≤ FM3	≤ FM4	≤ FM5	≤ FM6	
South-Sea dam		≤ FM1	≤ FM2	≤ FM3	≤ FM4	≤ FM5	≤ FM6	= FM12
North-Sea dam		≤ FM1	≤ FM2	≤ FM3	≤ FM4	≤ FM5	≤ FM6	
Concentric lakes dikes		≤ FM1[a] 1.0 E–02 [b]	≤ FM2[a] 1.0E–02 [b]	≤ FM3[a] 1.0E–02 [b]	≤ FM4[a] 1.0E–03 [b]	≤ FM5[a]	≤ FM6[a]	= FM12 [a,b]
Habitat pond embankments		1.0E-04 [c]	1.0E–04 [c]	1.0E–02[c]	1.0E–03 [c]			≤ FM12

Notes: a) These values are estimated for cross-section meeting seismic design criteria and require APF of 1.0xE-04. This could include only the outer 1 or 2 lakes.
b) These values are estimated for cross-section that does not meet seismic or seepage design criteria. This could be adopted for the inner lakes.

c) These values are based on expert opinion using levee performance data. No detailed evaluation performed for this estimate.

The APF estimates were then multiplied by the estimated loss of life (LOL) to calculate the annualized loss of life (ALL) risks posed by the potential failure modes. For these evaluations, the RET used the “best estimate” of LOL, rounded up for all integer values greater than 1 and set to 0 for all values less than 0.2. The resulting mean annual probabilities of failure and annualized loss of life estimates for the static and seismic failure modes evaluated for each structure are summarized in Tables 2D.31 through 2D.38. Risks associated with failure of the mid-Sea sand dam and the mid-Sea rockfill dam with rock notches and maximum seismic filters are shown in Tables 2D.31 and 2D.32, respectively. Risks associated with failure of the mid-Sea barrier are provided in Table 2D.33. Risks associated with failure of the perimeter dikes are summarized in Table 2D.34. Risks associated with failure of the south-Sea and north-Sea dams are shown in Tables 2D.35 and 2D.36, respectively. Risks associated with failure of the concentric lakes dikes and habitat pond embankments are summarized in Tables 2D.37 and D.38, respectively. In addition, the mean risks and uncertainty bands for each structure are presented graphically on the f-N plots shown as Figures D.1 through D.7.

Table 2D.31
Summary of Mean Risk Estimates
Mid-Sea Dam Option A, Sand Dam with Stone Columns

Failure Mode	Failure Mode Description	Mean APF	LOL	Mean ALL
FM No. 1	Mid-Sea-Dam, Static - Internal Erosion (Piping) of Embankment	3.8E-11	0	0
FM No. 2	Mid-Sea-Dam, Static - Internal Erosion of Foundation Materials	6.1E-09	0	0
FM No. 3	Mid-Sea-Dam, Seismic - Deformation and Overtopping of Embankment	3.8E-06	2	7.6E-06
FM No. 4	Mid-Sea-Dam, Seismic - Deformation and Internal Erosion of Embankment	3.1E-11	2	6.2E-11
FM No. 5	Mid-Sea-Dam, Seismic - Deformation and Internal Erosion of Foundation Materials	8.0E-08	2	1.6E-07
FM No. 6	Mid-Sea-Dam, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	4.0E-05	2	8.0E-05
Overall Risk (maximum risk for static and seismic FMs)		4.0E-05	2	8.0E-05

Note: LOL values are based on best estimate values from Table 2D.29. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0.

Table 2D.32
Summary of Mean Risk Estimates
Mid-Sea Dam Option D, Rockfill Dam with Rock Notches, Maximum Seismic Filters

Failure Mode	Failure Mode Description	Mean APF	LOL [(a)]	Mean ALL
FM No. 7	Rock Notches, Static - Internal Erosion of Embankment	≤3.1E-11	0	0
FM No. 8	Rock Notches, Static - Internal Erosion of Foundation Materials	2.3E-07 [(b)]	0	0
FM No. 9	Rock Notches, Seismic - Deformation and Overtopping of Embankment	1.0E-15	2	2.0E-15
FM No. 10	Rock Notches, Seismic - Deformation and Internal Erosion of Embankment	≤3.1E-11	2	≤6.2E-11
FM No. 11	Rock Notches, Seismic - Deformation and Internal Erosion of Foundation Materials	3.1E-07 [(b)]	2	6.2E-07
FM No. 6	Rock Notches, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	4.0E-05	2	8.0E-05
Overall Risk (maximum risk for static and seismic FMs)		4.0E-5	2	8.0E-05

Notes: a) LOL values are based on best estimate values from Table 2D.29. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0

b) These values derived through the Risk Analysis are in the range where there is decreasing justification to take action to reduce risk in the long or short term. However, the design configurations do not meet Reclamation design criteria for “full” filters.

Table 2D.33
Summary of Mean Risk Estimates
Mid-Sea Barrier

Based on Failure Mode	Failure Mode Description	Mean APF	LOL	Mean ALL
FM No. 3 (without stone columns)	Mid-Sea-Barrier, Seismic - Deformation and Overtopping of Embankment	>1.0E-02	0	0
FM No. 3 (with stone columns)	Mid-Sea-Barrier, Seismic - Deformation and Overtopping of Embankment	≤ 3.8E-06	0	0
Overall Risk (maximum risk for static and seismic FMs) Without Stone Columns		>1.0E-02	0	0
Overall Risk (maximum risk for static and seismic FMs) With Stone Columns		≤ 3.8E-06	0	0

Note: LOL values are based on best estimate values from Table 2D.29. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0

Table 2D.34
Summary of Mean Risk Estimates
Perimeter Dikes (with Stone Columns)

Based on Failure Mode	Failure Mode Description	Mean APF	LOL	Mean ALL
FM No. 1	Perimeter Dikes, Static - Internal Erosion (Piping) of Embankment	≤ 3.8E-11	0	0
FM No. 2	Perimeter Dikes, Static - Internal Erosion of Foundation Materials	6.1E-09	0	0
FM No. 3	Perimeter Dikes, Seismic - Deformation and Overtopping of Embankment	≤ 3.8E-06	0	0
FM No. 4	Perimeter Dikes, Seismic - Deformation and Internal Erosion of Embankment	≤ 3.1E-11	0	0
FM No. 5	Perimeter Dikes, Seismic - Deformation and Internal Erosion of Foundation Materials	≤ 8.0E-08	0	0
FM No. 6	Perimeter Dikes, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	≤4.0E-05	0	0
Overall Risk (maximum risk for static and seismic FMs)		≤4.0E-05	0	0

Note: LOL values are based on best estimate values from Table 2D.29. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0

Table 2D.35
Summary of Mean Risk Estimates
South-Sea Dam

Based on Failure Mode	**Failure Mode Description**	**Mean APF**	**LOL [(a)]**	**Mean ALL**
FM No. 1	South-Sea Dam, Static - Internal Erosion (Piping) of Embankment	≤ 3.8E-11	0	0
FM No. 2	South-Sea Dam, Static - Internal Erosion of Foundation Materials	≤ 6.1E-09	0	0
FM No. 3	South-Sea Dam, Seismic - Deformation and Overtopping of Embankment	≤ 3.8E-06	1	≤ 3.8E-06
FM No. 4	South-Sea Dam, Seismic - Deformation and Internal Erosion of Embankment	≤ 3.1E-11	1	≤ 3.1E-11
FM No. 5	South-Sea Dam, Seismic - Deformation and Internal Erosion of Foundation Materials	≤ 8.0E-08	1	≤ 8.0E-08
FM No. 6	South-Sea Dam, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	≤4.0E-05	1	≤4.0E-05
FM No. 12	South-Sea Dam, Seismic - Offset and Translation of Embankment (with translation mitigation design features)	1.0E-04	1	1.0E-04
FM No. 12	*South-Sea Dam, Seismic - Offset and Translation of Embankment (without translation mitigation design features)*	*1.1E-02*	*1*	*1.1E-02*
Overall Risk (maximum risk for static and seismic FMs) [(b)]		1.0E-04	1	1.0E-04

Notes:
a) LOL values are based on best estimate values from Table 2D.29. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0
b) Maximum overall risk would be 1.1E-02 and annualized loss of life would be 1.1E-02 if translation mitigation design features are not incorporated in the design.

Table 2D.36
Summary of Mean Risk Estimates
North-Sea Dam (with Stone Columns)

Based on Failure Mode	Failure Mode Description	Mean APF	LOL	Mean ALL
FM No. 1	North-Sea Dam, Static – Internal Erosion (Piping) of Embankment	≤ 3.8E-11	0	0
FM No. 2	North-Sea Dam, Static – Internal Erosion of Foundation Materials	≤ 6.1E-09	0	0
FM No. 3	North-Sea Dam, Seismic - Deformation and Overtopping of Embankment	≤ 3.8E-06	2	≤ 7.6E-06
FM No. 4	North-Sea Dam, Seismic - Deformation and Internal Erosion of Embankment	≤ 3.1E-11	2	≤ 6.2E-11
FM No. 5	North-Sea Dam, Seismic - Deformation and Internal Erosion of Foundation Materials	≤ 8.0E-08	2	≤ 1.6E-07
FM No. 6	North-Sea Dam, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	≤4.0E-05	2	≤ 8.0E-05
Overall Risk (maximum risk for static and seismic FMs)		≤ 4.0E-05	2	≤ 8.0E-05

Note: LOL values are based on best estimate values from Table 2D.29. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0

Table 2D.37
Summary of Mean Risk Estimates
Concentric Lakes Dikes

Based on Failure Mode	Failure Mode Description	Mean APF	LOL [d]	Mean ALL
FM No. 1	Lakes Dikes, Static - Internal Erosion (Piping) of Embankment	≤ 3.8E-11 [a] 1.0E-02 [b]	0	0
FM No. 2	Lakes Dikes, Static - Internal Erosion of Foundation Materials	≤ 6.1E-09 [a] 1.0E-02 [b]	0	0
FM No. 3	Lakes Dikes, Seismic - Deformation and Overtopping of Embankment	≤ 3.8E-06 [a] 1.0E-02 [b]	0	0
FM No. 4	Lakes Dikes, Seismic - Deformation and Internal Erosion of Embankment	≤ 3.1E-11 [a] 1.0E-03 [b]	0	0
FM No. 5	Lakes Dikes, Seismic - Deformation and Internal Erosion of Foundation Materials	≤ 8.0E-08 [a]	0	0
FM No. 6	Lakes Dikes, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	≤ 4.0E-05 [a] ≤ 8.4E-03 [b]	0	0
FM No. 12	South-Sea Dam, Seismic - Offset and Translation of Embankment (with translation mitigation design features)	1.0E-04 [c]	0	0
FM No. 12	*South-Sea Dam, Seismic - Offset and Translation of Embankment (without translation mitigation design features)*	*1.1E-02*[c]	*0*	*0*
Overall Risk (maximum risk for static and seismic FMs)		1.0E-02[c]	0	0

Notes:
a) These values are estimated for improved cross-section meeting seismic design criteria. For example, the outer 1 to 2 lakes would be designed to meet the seismic design criteria.
b) These values are estimated for unimproved cross-sections that do not meet seismic or seepage design criteria. This could be adopted for the remaining inner lakes.
c) Maximum overall risk would be 1.1E-02 and annualized loss of life would be zero if translation mitigation design features are not incorporated in the design for "outer" lakes.
d) LOL values are based on best estimate values from Table 2D.29. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0.

Table 2D.38
Summary of Mean Risk Estimates
Habitat Pond Embankments

Based on Failure Mode	Failure Mode Description	Mean APF	LOL [(b)]	Mean ALL
N/A [(a)]	Habitat Ponds, Static - Internal Erosion (Piping) of Embankment	1.0E-04	0	0
N/A [(a)]	Habitat Ponds, Static - Internal Erosion of Foundation Materials	1.0E-04	0	0
N/A [(a)]	Habitat Ponds, Seismic - Deformation and Overtopping of Embankment	1.0E-02	0	0
N/A [(a)]	Habitat Ponds, Seismic - Deformation and Internal Erosion of Embankment	1.0E-03	0	0
N/A [(a)]	Habitat Ponds, Seismic - Deformation and Internal Erosion of Foundation Materials	≤ 1.1E-02	0	0
N/A [(a)]	Habitat Ponds, Seismic - Liquefaction of Upper Stiff Lacustrine, Seismic Deformation and Overtopping of Embankment	≤ 1.1E-02	0	0
FM No. 12	Habitat Ponds, Seismic - Offset and Translation of Embankment	≤ 1.1E-02	0	0
Overall Risk (maximum risk for static and seismic FMs)		1.0E-02	0	0

Notes: a) These values are based on expert opinion using levee performance data. No detailed evaluation performed for this estimate.

b) LOL values are based on best estimate values from Table 2D.29. Values less than 0.2 were rounded to zero. Values between 0.2 and 1.2 were rounded to 1.0 and values equal or greater than 1.2 were rounded to 2.0

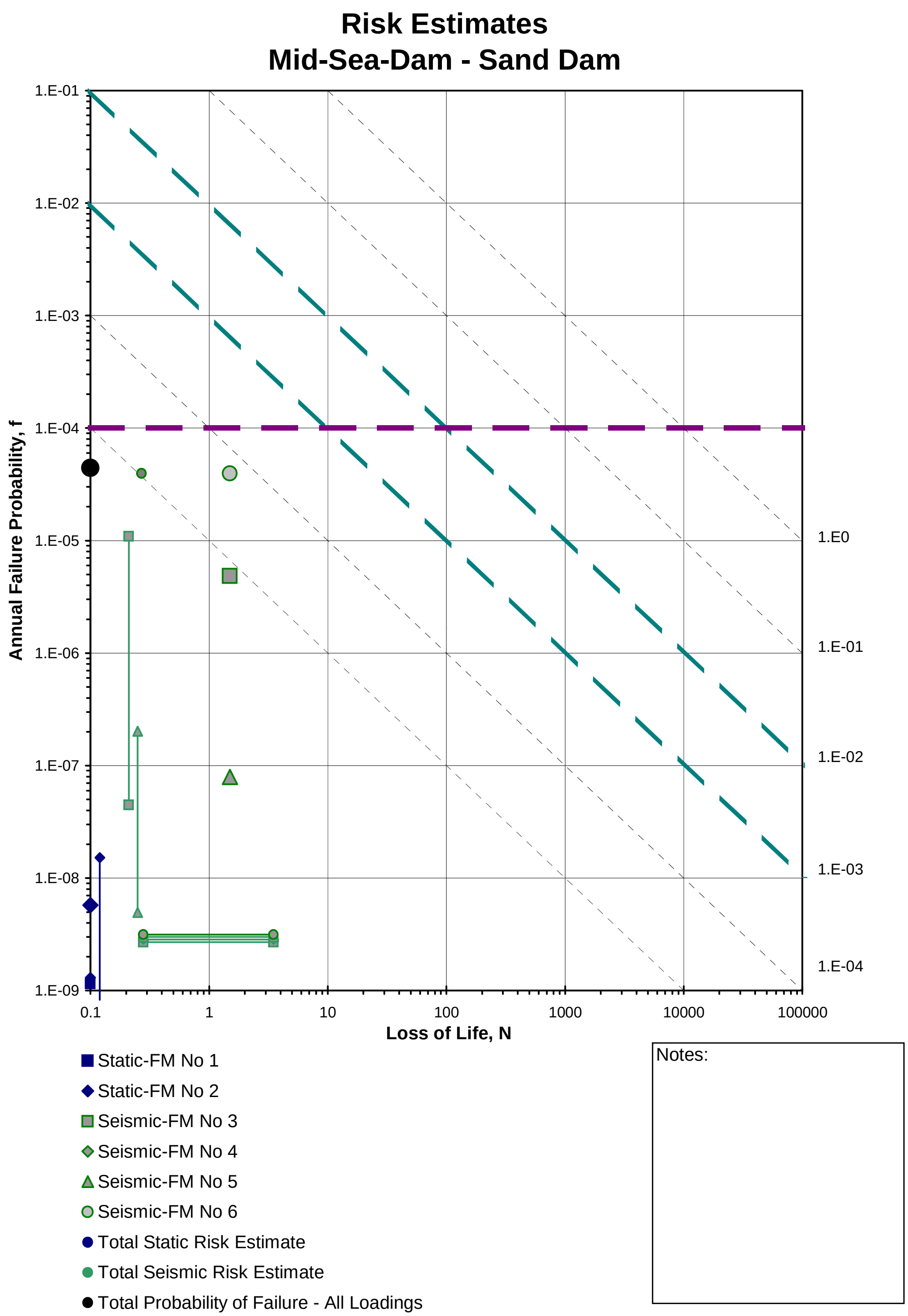


Figure D.1 f-N Chart Portraying Risks for Mid-Sea Dam – Sand Dam with Stone Columns

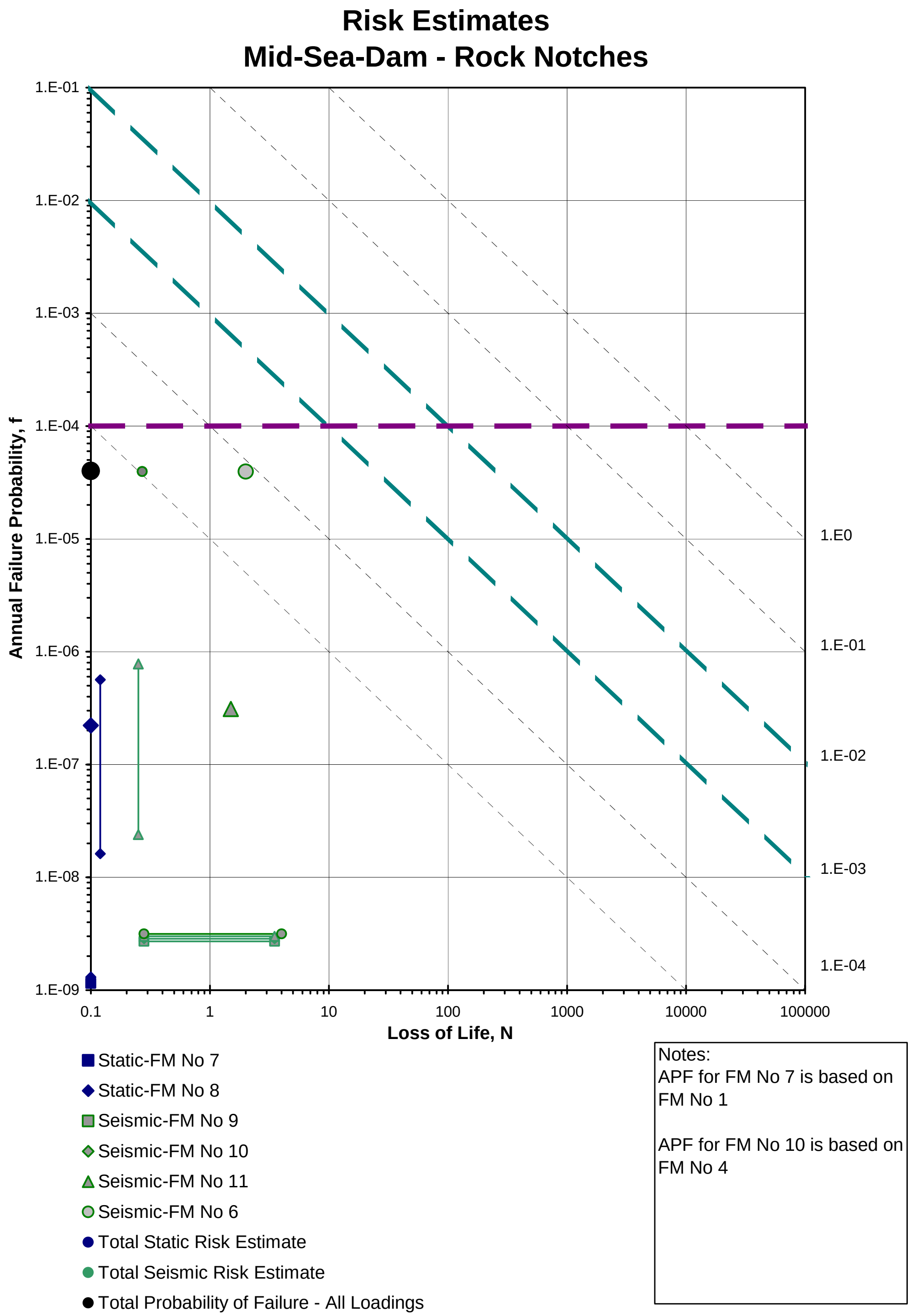


Figure D.2 f-N Chart Portraying Risks for Mid-Sea Dam – Rockfill Dam with Rock Notches and Maximum Seismic Filters

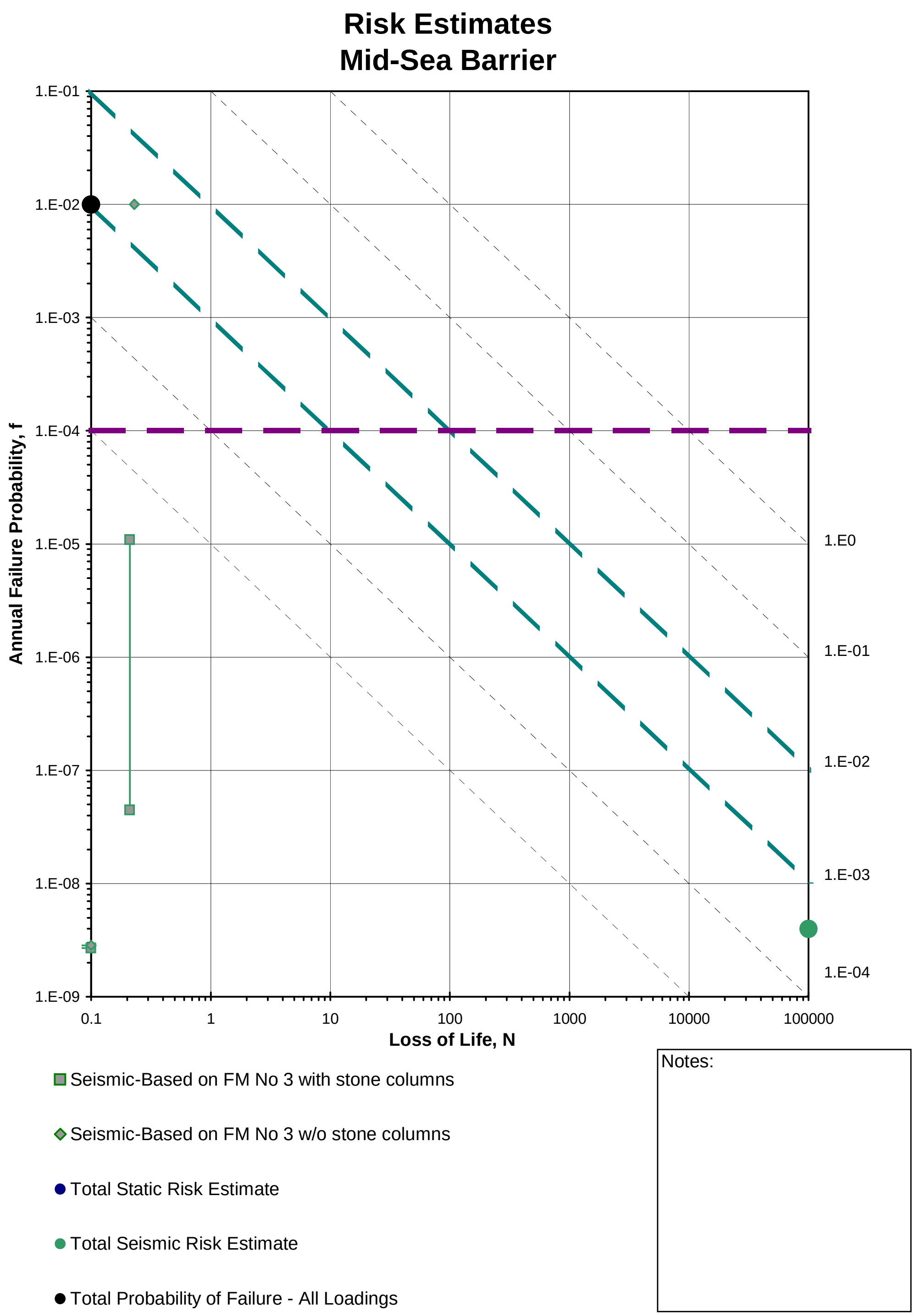


Figure D.3 f-N Chart Portraying Risks for Mid-Sea Barrier with and without Stone Columns

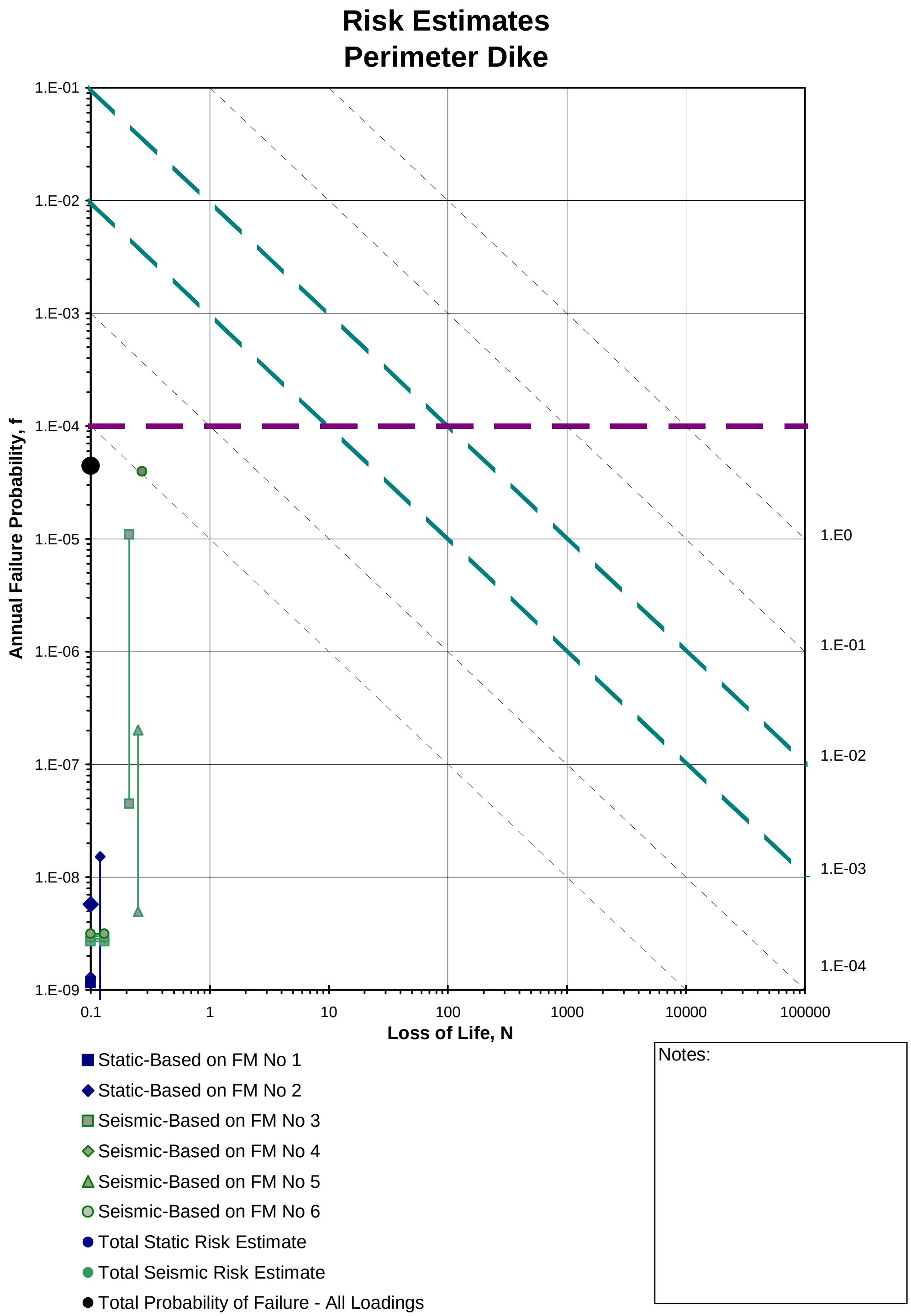


Figure D.4 f-N Chart Portraying Risks for Perimeter Dikes with Stone Columns

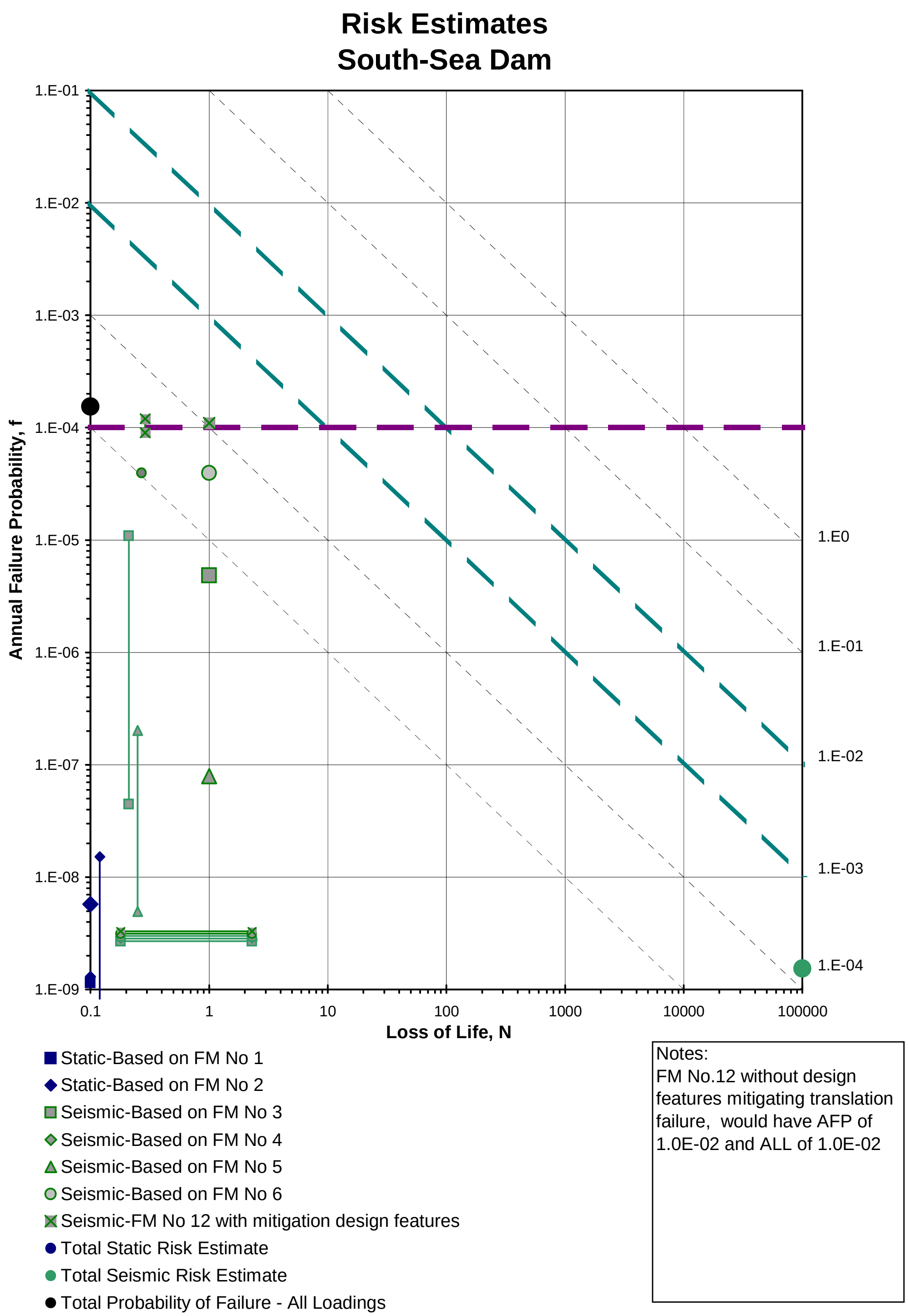


Figure D.5 f-N Chart Portraying Risks for South-Sea Dam with APF for FM Nos. 6 & 12

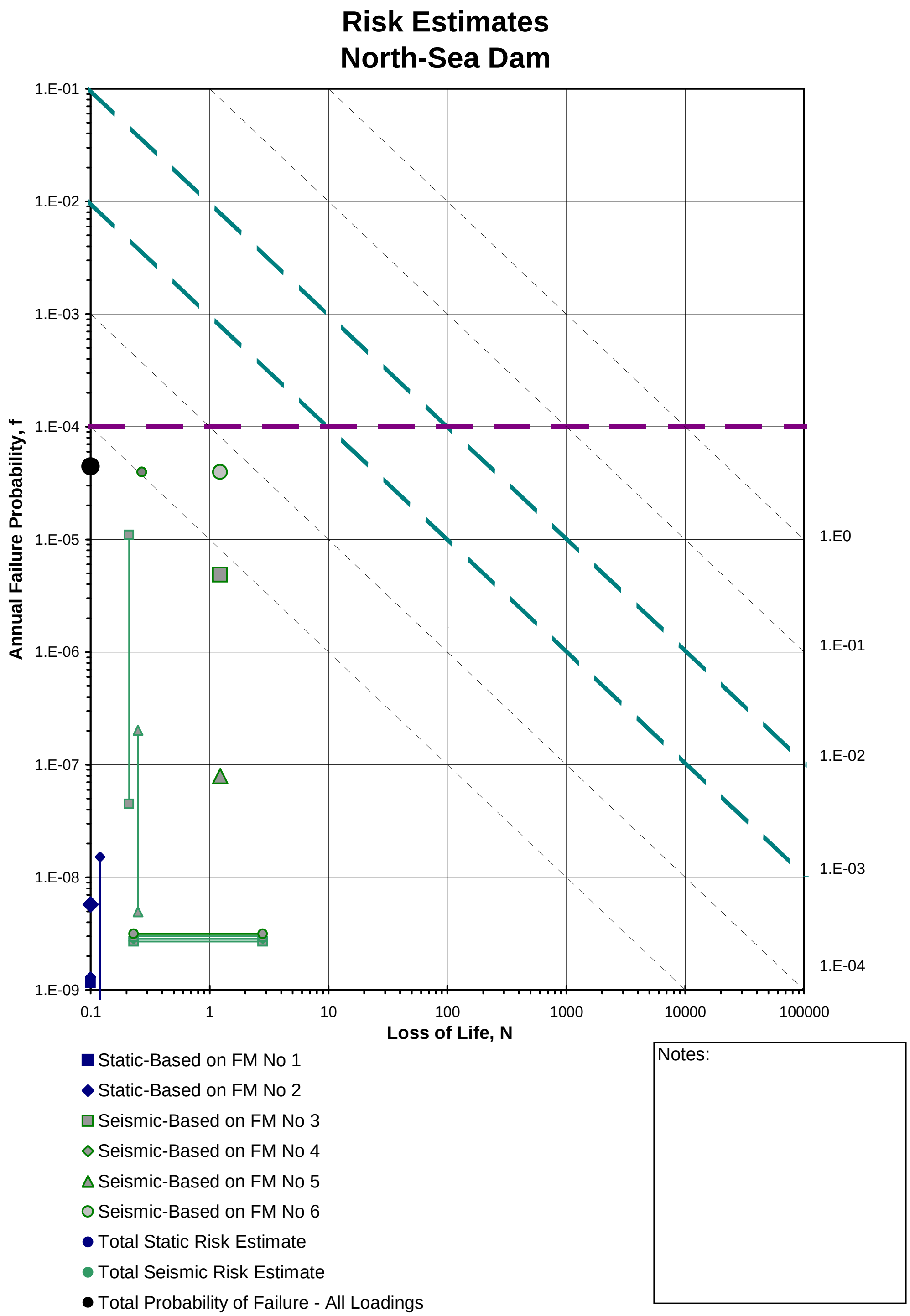


Figure D.6 f-N Chart Portraying Risks for North-Sea Dam with APF for FM No. 6

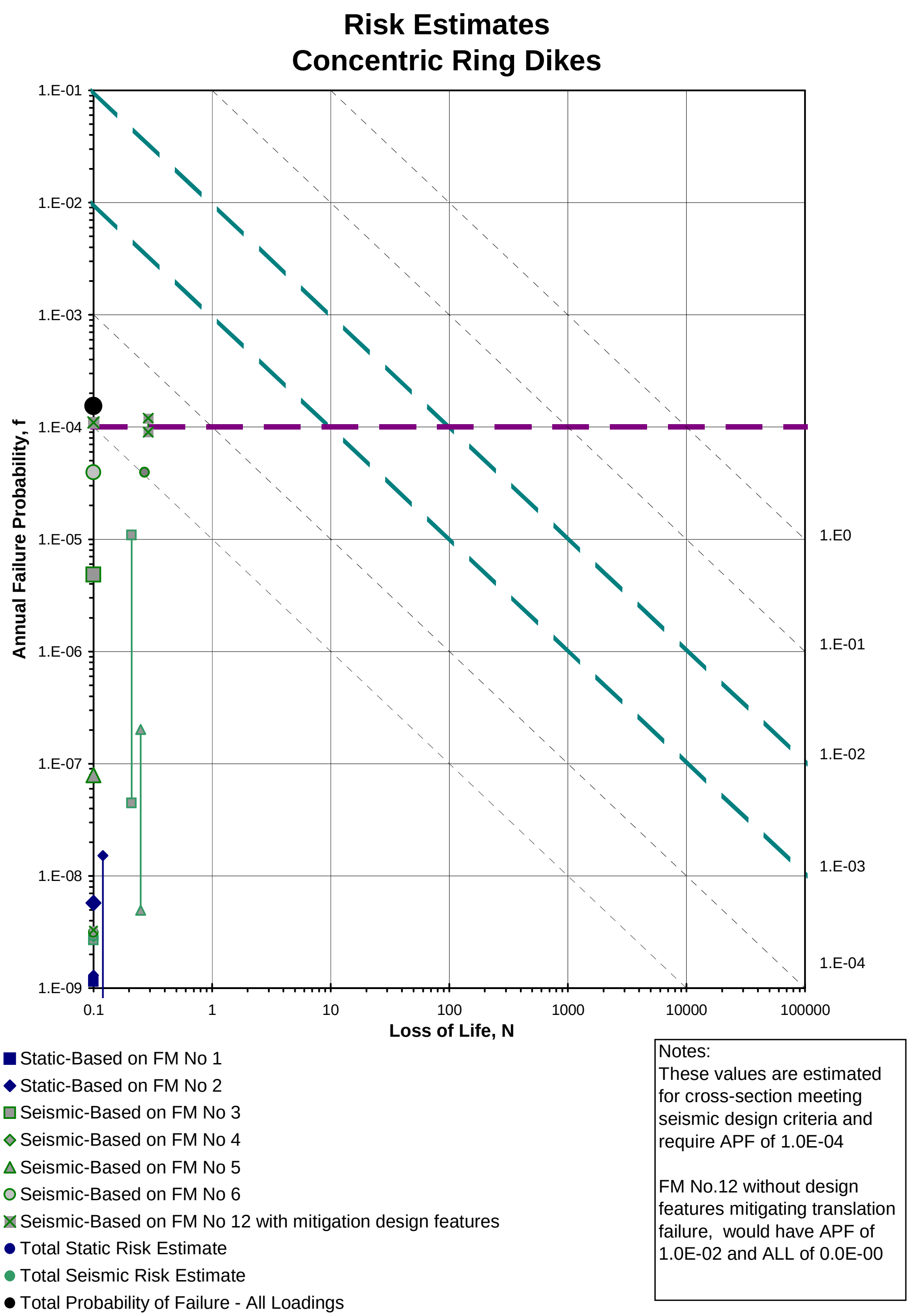


Figure D.7 f-N Chart Portraying Risks for Concentric Lakes Dikes, Outer Two Lakes with APF for FM Nos. 6 & 12

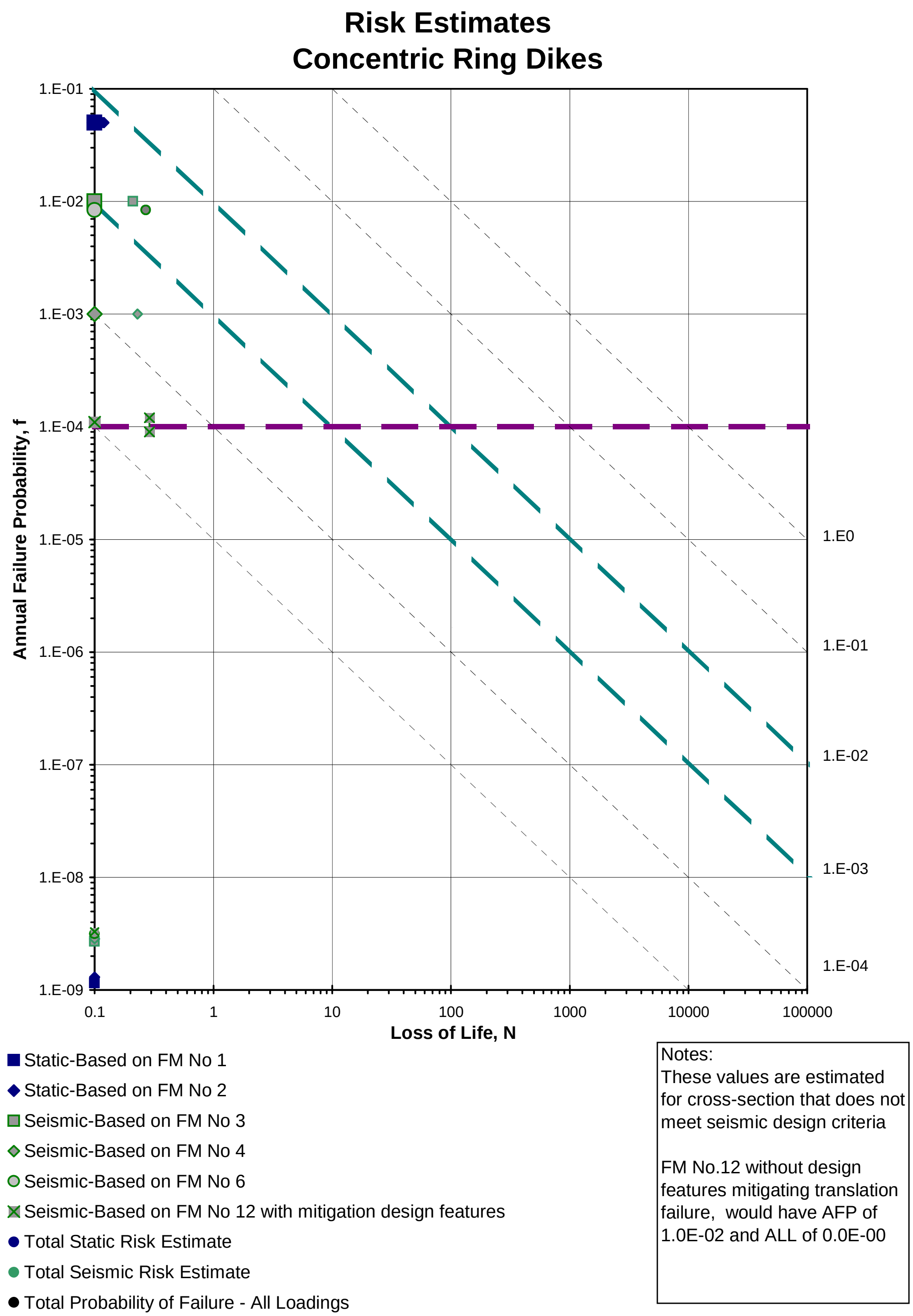


Figure D.8 f-N Chart Portraying Risks for Concentric Lakes Dikes, Inner Lakes with APF for FM Nos. 6 & 12

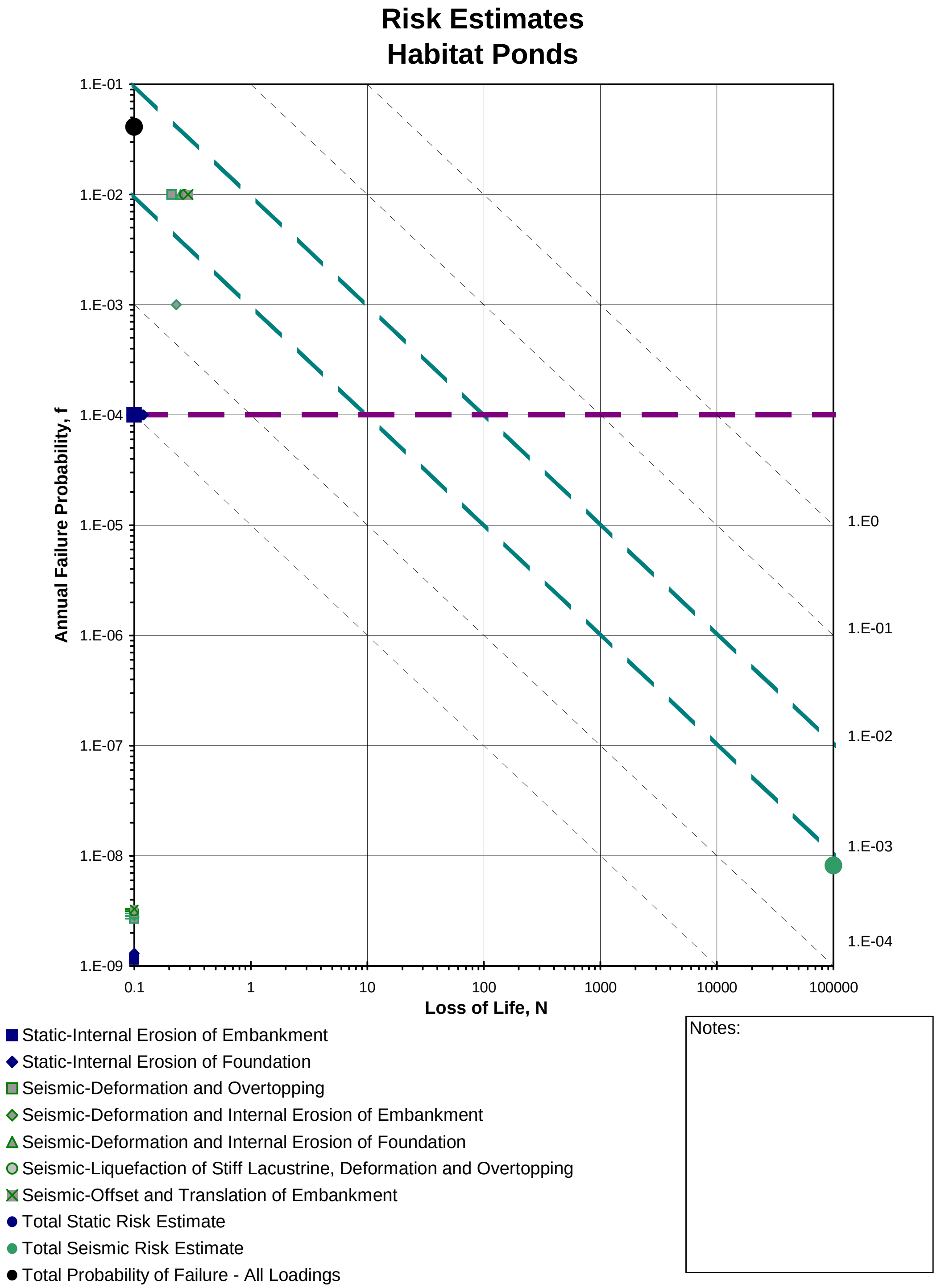


Figure D.9 f-N Chart Portraying Risks for Habitat Pond Embankments with APF for FM No. 12

After the RET had evaluated the risks for all of the failure modes for each embankment structure, the team then compiled the risks for each structure to develop a “composite” risk for each alternative. The team considered that since all static and seismic loadings for each individual structure were not independent variables (i.e., each structure would experience the loading for the same particular event at the same time, rather than as separate and independent events), the risk of failure of an alternative could be described by the risk associated with failure of the “weakest link” in the system. Therefore, annual probability of failure and annual loss of life for each alternative was considered to be the highest value for static or seismic failures for each of the structures comprising an alternative. Using the assumptions that (a) potentially liquefiable layers in the stiff lacustrine materials are identified and improved and (b) translation mitigation design features are incorporated into the designs, thecompiled results of these alternatives are summarized in Table 2D.39 below and on f-N charts on Figures D.10 through D.14 for each of the alternatives.

Table 2D.39
Summary of Alternative Risks

Embankment	**Mid-Sea Dam/North Marine Lake (1) Salton Sea Authority Alternative**			**Mid-Sea Barrier/South Marine Lake (with stone columns) (2)**			**Concentric Lakes Dikes (3)**			**North-Sea Dam/Marine Lake (4)**			**Habitat Pond Embankments (5)**		
Mid-Sea Dam (Sand Dam with stone columns)	APF 3.8 E-06	LOL 2	ALL 7.6 E-06												
Mid-Sea Barrier				APF ≤ 3.8 E-06	LOL 0	ALL 0									
Perimeter Dikes	APF ≤3.8 E-06	LOL 0	ALL 0												
South-Sea Dam	APF 1.0 E-04	LOL 1	ALL 1.0 E-04												
North-Sea Dam										APF ≤3.8 E-06	LOL 2	ALL ≤7.6 E-06			
Concentric Lakes Dikes (with translation mitigation design features)							APF 1.0 E-04	LOL 0	ALL 0						
Habitat Pond Embankments													APF 1.0 E-02	0	ALL 0
Controlling Maximums	**APF 1.0 E-04**	**LOL 2**	**ALL 1.0 E-04**	**APF ≤ 3.8 E-06**	**LOL 0**	**ALL 0**	**APF 1.0 E-04**	**LOL 0**	**ALL 0**	**APF ≤3.8 E-06**	**LOL 2**	**ALL ≤7.6 E-06**	**APF 1.0 E-02**	**LOL 0**	**ALL 0**

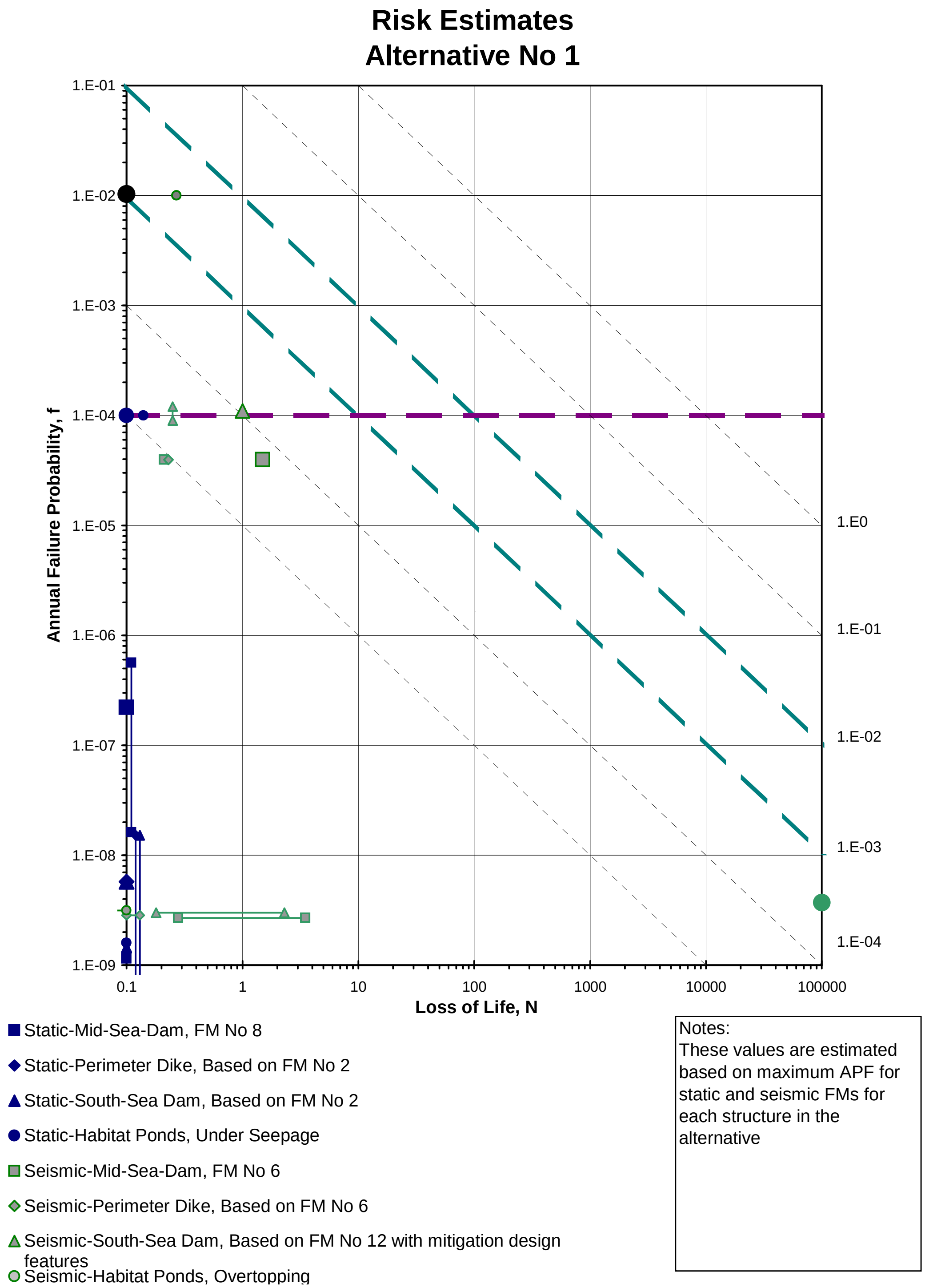


Figure D.10 f-N Chart Portraying Risks for Alternative No. 1 with APF for Hypothetical FM Nos. 6 & 12

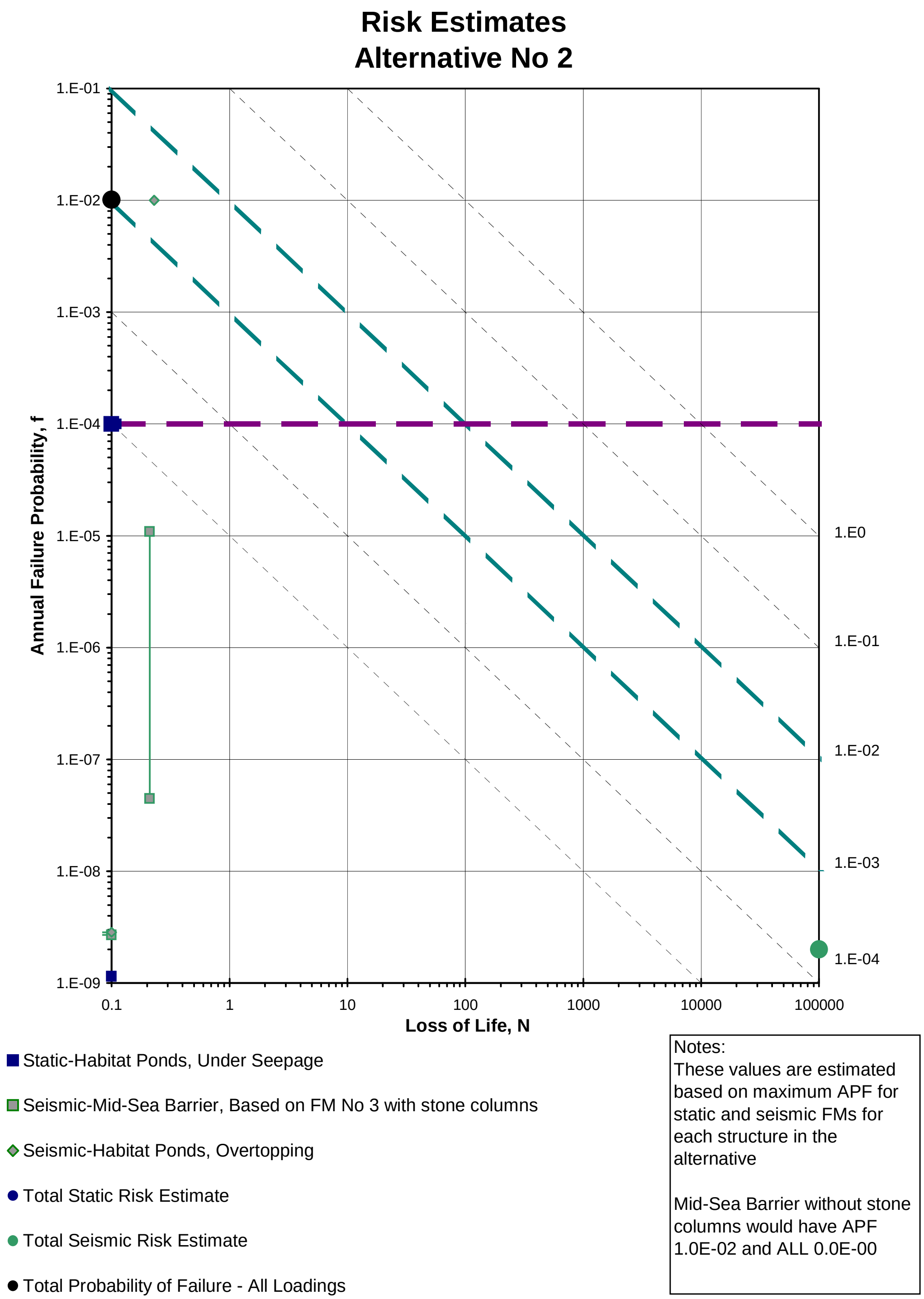


Figure D.11 f-N Chart Portraying Risks for Alternative No. 2

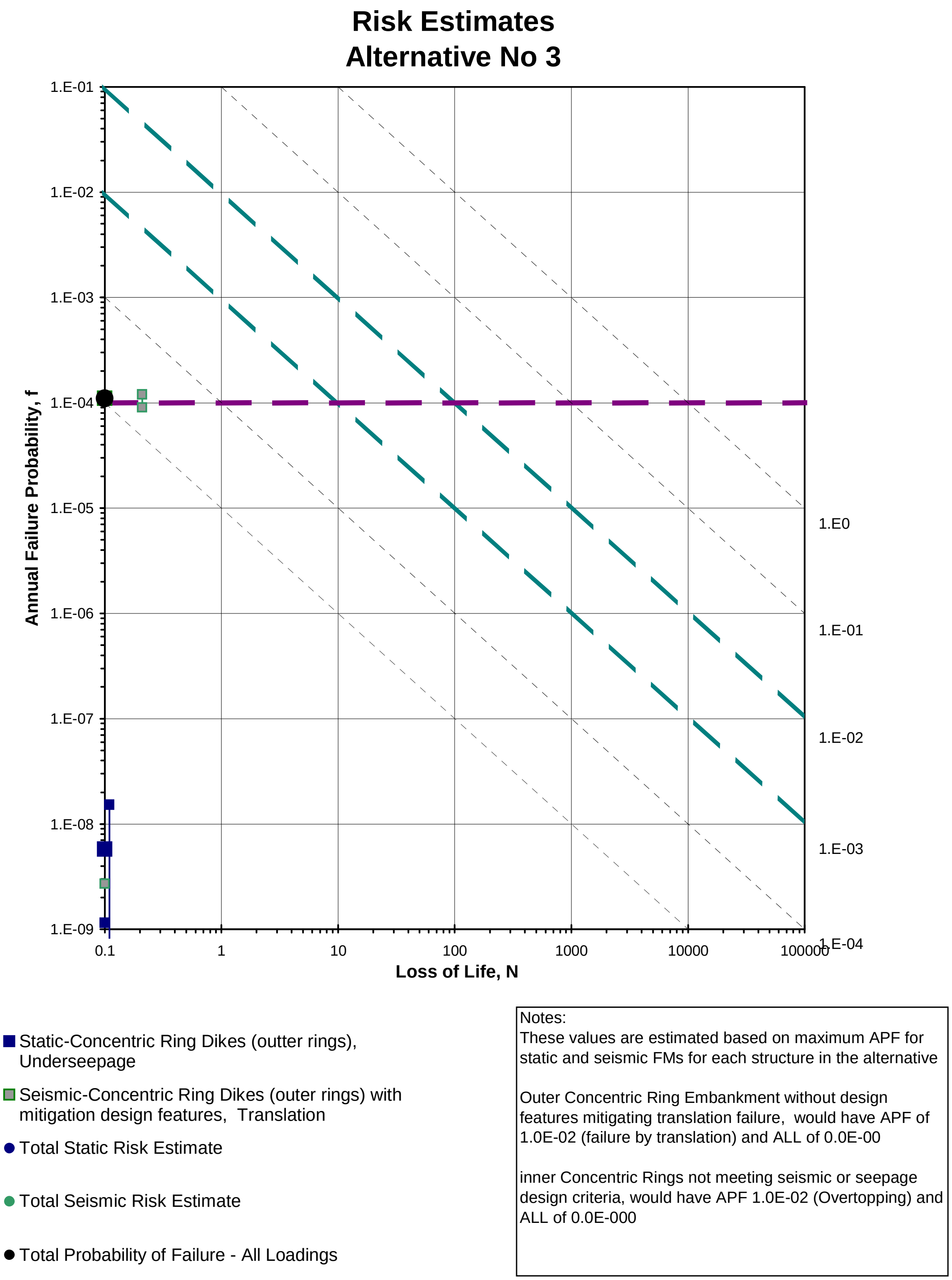
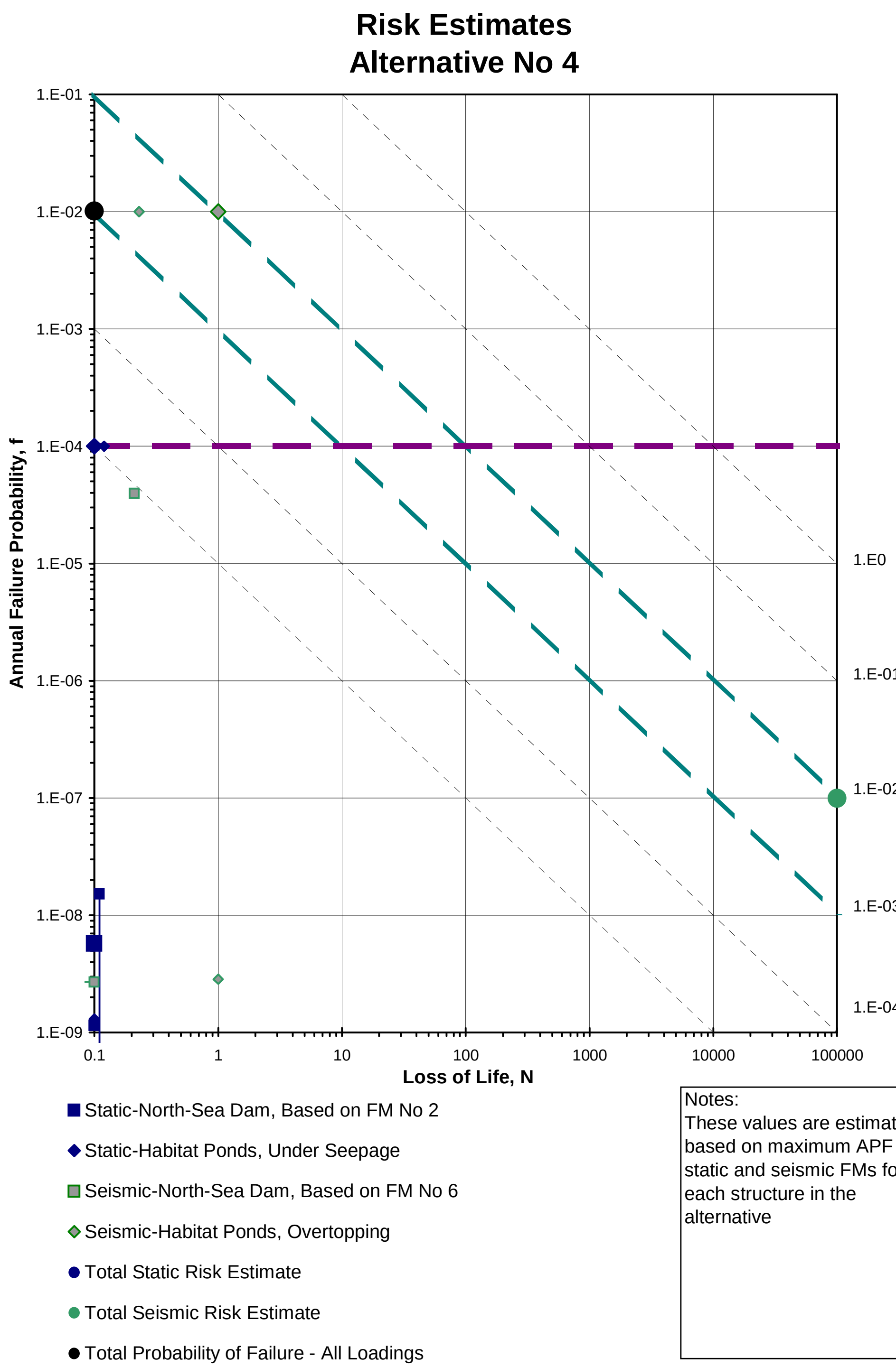


Figure D.12 f-N Chart Portraying Risks for Alternative No. 3 for cross-section meeting seismic design criteria and improved to mitigate translation failure



Notes:
These values are estimated based on maximum APF for static and seismic FMs for each structure in the alternative

Figure D.13 f-N Chart Portraying Risks for Alternative No. 4

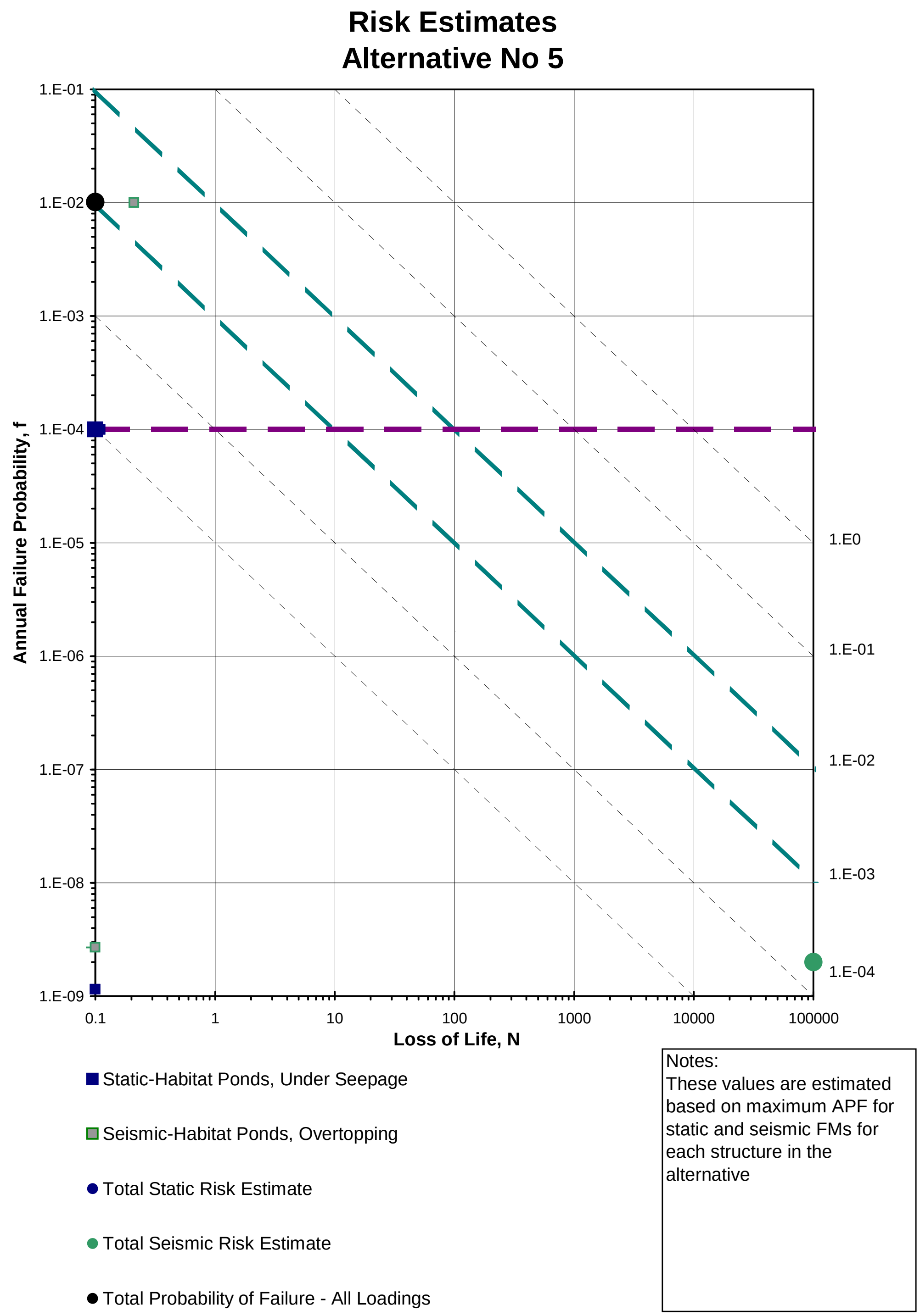


Figure D.14 f-N Chart Portraying Risks for Alternative No. 5

11.0 Recommendations and Conclusions

11.1 Uncertainties

The RET reviewed all available information regarding design and analyses of various alternatives, and data describing project site conditions. This information was used to develop probability estimates for static and seismic failure modes. The team identified the following uncertainties in the available data:

- Potential for liquefaction within the upper stiff lacustrine deposit.
- Fault offset in foundation deposits along dam alignments. For preliminary evaluations, Reclamation provided projections, which were based on limited information. Additional studies would be required to finalize concepts and designs.
- Inconsistency between the seismic hazard curve and the loading ranges and the projected fault offset curve need to be examined further and resolved if appropriate.
- The uncertainties associated with the static and seismic risk estimates warrant collection of additional data to reduce the uncertainty and decrease the risks.

11.2 Adaptations to Design Concepts Resulting from Initial Risk Analysis Results

The risk evaluation team discussed several potential revisions to the design concepts based on the initial risk analysis results. A summary of the recommendations and reasons for them are provided below:

- Extension of the SCB slurry wall to a depth of 40 feet into the upper stiff lacustrine was identified as a means to significantly reduce uncertainty about the upper stiff lacustrine foundation and was assumed as a design condition in all final risk estimates.
- Armoring the crest of the dam and reinforcing the top of the SCB wall may reduce the risks associated with overtopping.
- Inclusion of a blanket drain into the Type A material of Sand Dam may reduce the risk of piping. To minimize the risk of liquefaction in this zone under seismic loading, the drain material should be densified during construction using stone columns.
- The exploration programs for the design phase and during construction should be extensive and extend to a substantial depth into and below the upper stiff lacustrine deposit. The benefits of the exploration program are

numerous for optimizing the design, reducing costs, and in decreasing risks associated with the potential failure modes identified in this risk analysis (FM No. 6).

- The mid-Sea barrier and habitat pond embankment concepts may require cutoff walls in order to achieve the water control (balance) objectives of the project.
- The mid-Sea barrier concept may require stone columns in order to reduce the repair and replacement costs associated with the probable seismic failure of a barrier without stone columns.
- The strains predicted by the FLAC model along the centerline of the dam (SCB wall location) show the maximum shears occurring at the contact of the dam to the upper stiff lacustrine material. The model, without considering different material properties associated with the SCB wall, estimates strains of up to 15% or about 0.75 foot over the 5-foot height of the element in the model. Such strains, although large, are tolerable for a plastic (HDPE) membrane that could be installed in the SCB wall. Consequently, the analysis results suggest that a membrane in the SCB wall could offer some important redundancy and protection for large seismic events.
- The risk analysis indicated that the SCB wall membrane offers 3 to 4 orders of magnitude of reduction of the probability of failure for the seismically induced seepage failure modes.
- Thickness of internal zones in south-Sea dam and perimeter dike concepts could be increased to reduce the risk of seismic failure due to fault-offset translation. Likewise, an internal blanket of coarser material in the Type A zone may also mitigate risks to some degree.
- Segmentation of the Salton Sea Authority alternative, such as by placing cross barriers connecting the west shore to the perimeter dike, would be prudent to mitigate the consequences of failure of the south-Sea dam or perimeter dike elements due to translation (fault offset).

11.3 Intervention Activities to Reduce Risk of Failure

The team identified the following actions that could be considered to further reduce risks associated with static and seismic failure modes.

- Installation of relief wells near identified inclusions within the upper stiff lacustrine. These relief wells should reduce the likelihood of high pressure building up within the inclusion and would reduce the risk associated with internal erosion failures through embankment foundations (FM No.2, No. 5, No. 6, No. 8, and No. 11).

- Installation of instrumentation to detect developing problems. This should reduce the likelihood of unsuccessful intervention for all static failure modes.
- Compaction grouting of potentially liquefiable inclusions in upper stiff lacustrine. This measure should decrease the probability of failure under FM No. 2, No. 5, No. 6, No. 8, and No. 11.

All explorations should be carefully grouted to avoid creating additional conditions for FM No. 2, No. 5, No. 8, and No. 11.

12.0 References

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