4.0 Evaluation of Mid-Sea Dam and Barrier Options

4.1 General

A series of evaluations were completed, progress meetings held, and a decision making process developed and implemented to achieve the overall purpose and objectives of this study outlined in Sub-section 1.2. All work was initiated based on the results of previous explorations (URS, 2004) and appraisal level evaluations that had been completed (Reclamation, 2005). The sub-sections that follow summarize the general background information, or basis of the evaluations and designs that have been prepared as part of this study. Sub-section 4.2 provides a summary of the overall sequence of evaluations and decision–making steps that were followed to select a preferred configuration option for all embankment elements. Sub-sections 4.3, 4.4, and 4.5 summarize key findings from the construction materials, stability/seepage and deformation analyses pertaining to the evaluation of the configuration options. Sub-sections 4.6, through 4.10 present summaries of the “optimized” embankment configuration options A through D for the mid-Sea dam and option E for the mid-Sea barrier that were developed based on a general assessment of construction material sources, stability, seepage and deformation evaluations, risk, and consideration of construction means and methods that may be used for the various options. Sub-section 4.11 summarizes the overall process and the results of the evaluation and selection of the preferred mid-Sea dam embankment configuration option and how this configuration was used to develop optimized embankment sections for the mid-Sea barrier, perimeter dikes, south- and north-Sea embankments, and concentric lakes dike embankments meeting either static or both static and seismic stability design criteria of Reclamation (Reclamation, 2003).

4.1.1 Appraisal Level Study Results

As part of the appraisal level studies, Reclamation developed embankment configurations for the mid-Sea dam, the mid-Sea barrier, the concentric lakes/perimeter dikes, and the habitat pond embankments. These configurations are summarized on Figures 4.1 through 4.4. These configurations served as the starting point for the development, evaluation, and optimization of embankment configuration options under this study.

As noted in Sub-section 1.2, Reclamation desired to elevate design concepts to a planning level as part of this study with a thorough review of the previous work and a comprehensive reformulation and evaluation of a broad range of potential embankment configurations. This was to include the previous preferred configurations as well as options for a "rockfill dam with rock notches" concept being recommended by the Salton Sea Authority, embankments with flatter slopes, and other concepts that the evaluation team would identify. These concepts would be developed for foundations with and without liquefaction potential.

Each option would include as a minimum, adequate filter zones, drain zones, and/or toe drain systems to prevent potential internal erosion/piping due to seismic-induced cracking and to control seepage under long-term static conditions. Each option would also be evaluated to determine geometric requirements of the cross-section in order to achieve appropriate static and post-earthquake stability and deformation performance.

4.1.2 Geologic and Seismic Setting

The Sea is the largest inland water body in California and spans portions of Riverside and Imperial Counties. The Sea is a terminal hypersaline lake with a current salinity of about 48,000 mg/L (Reclamation, 2005). The lake occupies a desert basin known as the Salton Trough, a topographic low that extends from the Gulf of California northwest into southern California. Current primary sources of inflow into the lake include the New and Alamo Rivers to the south and, to lesser degrees, the Whitewater River to the north, San Felipe Creek to the west, and Salt Creek to the east. Current annual inflow into the basin is about 1.3 million-acre-feet per year (maf/yr).

The Salton Trough is a deep, closed basin bounded by mountains and deltaic deposits that prevent both drainage of the trough and inundation from the Gulf of California to the south. Sediments in the trough may be more than 18,000 feet deep (URS, 2004). The most recent deposits within the Sea include lacustrine, deltaic, and fluvial deposition associated with periodic inundation by the Colorado River. The Sea occupies two local depressions of the main trough, called the northern basin and the southern basin. A summary of the near Seafloor stratigraphy of the Sea is provided in Table C.1 in Appendix 2C.

The Salton Trough is located in a highly active tectonic region with frequent earthquakes. Tectonically, the vicinity is dominated by the San Andreas, Imperial, San Jacinto and Elsinore fault systems. Many moderate to large earthquakes have occurred on faults in the Salton Trough vicinity.

Site-specific horizontal and vertical uniform hazard spectra (UHS) at the ground surface have been developed for return periods of 10,000, 5,000, 2,500, and 500 years. These spectra were developed using probabilistic methods. The ground motions representing an event having a return period of 10,000 years were used in deformation analyses. Results of deaggregation analyses by Reclamation indicate that the dominant magnitude and distance are 7.4 and less than 10 km, respectively for the 10,000-year event. Additional information on the seismic characteristics of the study area is provided in Appendix 2C, including a description of the ground motions used in deformation evaluations.

4.1.3 Available Geotechnical Information

An exploration program was completed in 2004 to support the development of alternative restoration concepts (URS, 2004). The location of the explorations relative to the currently proposed embankment locations for Alternatives 1, 2, and 4 are shown on Figure 4.5 and for Alternative 3 on Figure 4.6.

The exploration program included borings in which standard penetration tests (SPT) and sampling were performed. Laboratory testing was conducted on boring samples. The program also included cone penetration test (CPT) "soundings". The borings and CPT soundings were spaced approximately 1-mile apart along the mid-Sea dam alignment being considered at that time and at other locations around the Sea (see Figure 4.5).

A geologic cross-section was prepared as part of these initial explorations to describe the near-surface Seafloor stratigraphy (see Sub-section 4.1.3 above and Appendix 2B) at the location of the mid-Sea dam proposed at that time (Section A-A'). This location corresponds to the location of the mid-Sea barrier being evaluated as part of this study. Reproduction of Figures 9 through 13 from the exploration report (URS, 2004) is provided on Figures 4.7a through 4.7e. Additional cross-sections will be developed around the Sea during subsequent investigation programs.

One of the significant conclusions from this investigation was as follows:

> "Rigorous analyses for the potential of soil liquefaction were not performed for this preliminary investigation due to the paucity of granular deposits that were encountered. The majority of the sediments encountered in this investigation were high plasticity clays, which would have a low potential for liquefaction."

Reclamation completed a detailed review and evaluation of the SPT and CPT data presented in the exploration program report as part of the appraisal level study risk analysis (Reclamation, 2005). Reclamation subsequently concluded that the data from the investigations indicated the potential for liquefaction of sands, silty sands and sandy silts in portions of the upper alluvial deposit and at some locations within the soft lacustrine deposit. Stability and deformation evaluations, and the risk analysis performed as part of this study have been based on Reclamation's findings.

Reclamation has been working with the Salton Sea Authority to fund and conduct addition explorations within the Sea to further characterize the upper Seafloor deposits. At the initiation of this study, it was anticipated that some, if not all of the exploration data from this supplemental exploration program would be available for evaluation and incorporation into seepage and stability evaluations, deformation evaluations and the risk analysis. However, the initiation of the program was delayed beyond the point where the new information could be utilized. Hence, the results of new explorations have not been included in the evaluations described in this report.

4.1.4 Design Criteria and Considerations

All embankments associated with the Salton Sea restoration project will be required to meet Reclamation's Public Protection Guidelines (Reclamation, 2003). These guidelines focus on the life loss and the public trust components of decision-making. Estimations of possible loss of life (LOL) are generally made following the procedures outlined in Reclamation's Dam Safety Office publication DSO-99-06 (Reclamation, 1999). The guidelines recognize the need in some circumstances for the application of risk-based analyses techniques to address economic consequences, as well as environmental and social issues. Because of the overall nature of the alternatives being considered, all three fundamental hazard classification criteria (LOL, economic, and environmental/social) will be considered before a final determination is made on the appropriate hazard classification of each embankment component of the various alternatives.

The general design criteria for the mid-Sea dam, the south- and north-Sea dams, perimeter dikes, and concentric lakes dikes, are as follows:

- ✓ Resist and control embankment seepage
- ✓ Resist and control foundation seepage
- ✓ Resist and control internal erosion
- ✓ Minimize static settlements
- ✓ Resist large offsets and slope instability during and at the end of construction and normal operation
- ✓ Resist deformations due to seismic loading
- ✓ Resist hydrologic flood loading
- ✓ Provide for constructability using proven methods
- ✓ Provide for safe construction
- ✓ Hazard classification based on consideration of LOL, economic, and environmental/social classification criteria

The general design criteria for the mid-Sea barrier and habitat pond embankment options include:

- ✓ Control embankment and foundation seepage (barrier)

- ✓ Limit and control embankment and foundation seepage (habitat pond embankments)
- ✓ Minimize static settlements
- ✓ Resist large offsets and slope instability during and at the end of construction and under normal operation
- ✓ Resist hydrologic flood loading
- ✓ Provide for constructability using proven methods
- ✓ Provide for safe construction
- ✓ Hazard classification based on consideration of LOL, economic, and environmental/social classification criteria

As the work progressed, other more specific design criteria were identified as follows:

1. Seismic loading:
   a. Provide a minimum post-earthquake slope stability factor of safety of 1.3
   b. Limit average crest deformations to less than 5-feet (or the freeboard of the structure)

2. Static loading:
   a. Provide a minimum static slope stability factor of safety of 1.5
   b. Limit vertical exit gradients at unfiltered discharge faces to maximum of 0.25
   c. Filters would be placed at all critical locations within the embankment to provide full filter protection against geologic or construction anomalies

The prior work by Reclamation also identified a number of other key design considerations as follows:

- ✓ 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.
- ✓ The high salinity of the Seawater is likely to increase construction costs. The water is corrosive to steel structures and equipment. Grout and soil-cement-bentonite 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. Suction dredges are likely required to excavate the Seafloor deposits to the required levels and in-Sea disposal may require placement at significant distances from the excavation location.

4.2 Formulation, Evaluation, and Decision Process for Section of Preferred Configuration of Required Embankments

The initial scope of work outlined for this study envisioned a sequential set of evaluations and a structured decision making process by Reclamation to select a preferred embankment cross-section configuration. However, a number of initial study outcomes required adaptation of the scope in order to achieve the desired project purpose and objective. The paragraphs below describe the sequence of study tasks as actually completed. Additional details related to several of these steps/evaluations are described in greater detail in subsequent sections.

1. Assessment of Existing Information and Construction Material Sources – Available information from previously completed site explorations, evaluation of construction material sources, and evaluation of restoration alternatives (Reclamation 2003, 2005; URS, 2004), was reviewed (Task 1). In addition, a planned supplemental exploration program was also reviewed (Task 2) and a regional construction materials assessment was performed (Task 3)

2. Seepage and Stability Evaluations - Initial cross-section options were established for a large number of potential configurations for the mid-Sea dam, perimeter dike, mid-Sea barrier and habitat pond embankments (see Table C.2 in Appendix 2B). These sections were evaluated and modified as appropriate to begin to achieve the desired static and "post-earthquake" factors of safety and seepage performance (Tasks 4 and 5). However, due to the very low strength characteristics of the foundation materials and the large and long duration seismic ground motions of the site, a concern arose with respect to seismic deformations. Subsequent evaluations showed that cross-sections meeting the required "post-earthquake" factor of safety of 1.3 had very low yield accelerations, suggesting that very large deformations would occur. (see Appendix 2B)

3. Newmark Deformation Evaluations – Once the issue of the very low yield accelerations was identified, a simplified Newmark

deformation analysis was performed. This evaluation provided an initial indication that yield accelerations on the order of 0.15 to 0.25g were going to be needed in order to limit crest deformations to the required design criteria (loss of embankment freeboard of 5 feet, maximum). This would require cross-section properties having “post-earthquake” stability factors of safety substantially higher than 1.3. This evaluation is described further in Appendix 2C.

4. Formulation and Initial Screening of Embankment Cross-Section Options – Based on the initial seepage, stability, and Newmark deformation results, a screening model was developed in conjunction with Reclamation personnel. Also at this time, the various mid-Sea dam configurations were consolidated into a total of four options:

Option A – Sand Dam with Stone Columns

Option B – Rockfill Dam with Jet Grouted Foundation

Option C – Modified Rock Notches Dam with Minimum Filters

Option D – Modified Rock Notches Dam with Maximum Seismic Filters

This effort helped clarify the important decision criteria for selection of the preferred cross section option. The additional information needs for Reclamation’s decision process were identified. At this time, factors related to cost and constructability were introduced to the decision process. Additional discussion of this effort is provided in Appendix 2B. The first draft of the Task 6 Seepage and Stability Analysis report was submitted for comment by Reclamation. Reclamation concurred with the recommendation that the “Sand Dam with Stone Columns” and the “Modified Rock Notches” configuration options required further evaluation. Option A, the “Sand Dam with Stone Columns” was selected for deformation analyses of a “representative” mid-Sea dam configuration and Reclamation’s “Rockfill Dam with Jet Grouted Foundation” was selected for deformation analyses of a “representative” perimeter dike configuration under Task 8 of the scope of work.

There are two items of note relative to Options C and D. Option C is a modified rock notches concept with “minimum filters” meaning that the dimensions of the filters in the dam were set based on the results of seepage analyses showing the location where gradients exiting from the foundation into the base of the dam would be less than or equal to 0.20 (roughly corresponding to a factor of safety (FOS) of 4 to 5 when compared to the critical gradient). Option D is a modified rock notches concept with "maximum seismic filters", meaning that the dimensions (depth etc) of the rock notches including a stabilizing berm, and internal filters were established to "maximize" the extent of the internal filters while meeting seismic design (deformation) criteria. As will be seen in subsequent sections describing these alternatives, in both cases (Options C and D), there would be an unfiltered interface between the foundation materials and rockfill material within the dam at a location that could not be subsequently accessed for repairs. Because of the uncertainties associated with foundation conditions and the potential for geologic defects or "inclusions", and Reclamation's requirements for "full filters" within the dam, Options C and D provide useful information relative to the rock notches concept but do not fully meet Reclamation's design criteria and requirements.

5. Supplemental Seepage and Stability Evaluations – Seepage and stability evaluations were refined based on initial comments by Reclamation. Some additional stability analyses of deformed sections were completed to estimate the change in yield acceleration that would occur as the sections deformed due to an earthquake.

6. FLAC Deformation Evaluations – FLAC models of the "Sand Dam with Stone Columns" option adopted for the maximum cross section of the mid-Sea dam, and Reclamation's maximum "Rockfill Dam with Jet Grouted Foundation" section of the perimeter dike were prepared and evaluated (Task 8). Results of the Newmark deformation evaluations described under step 3 above were compared to the FLAC deformation estimates and yield acceleration criteria were finalized that would limit average crest deformations to less than 5-feet for all cross section options.

7. Finalization of Decision Criteria and Cross-Section Requirements - FLAC deformation evaluations confirmed that the governing design criteria for all cross-section options was a yield acceleration of greater than or equal to 0.17g. This combined with the static seepage criteria summarized in Sub-section 4.1.4 above became the basis for the optimization and finalization of the cross-section options for the mid-Sea dam and barrier (Task 7). Further refinement of the cross-section options was made and two options were found to meet all design criteria. These options included:

   Option A – Sand Dam with Stone Columns

   Option B – Rockfill Dam with Jet Grouted Foundation

As noted under item 4 above, Option C, Modified Rock Notches Dam with Minimum Filters, and Option D, Modified Rock Notches with Maximum Seismic Filters were found to meet the crest deformation design criterion but not all of Reclamation's seepage design criteria. They were carried forward into final screening, as it was believed that these options will most closely resemble the section that will be preferred by the Salton Sea Authority.

8. <u>Final Screening of Embankment Cross-Section Options</u> – The screening model described in step 4 above was modified to include multiple technical criteria. The cost criterion was evaluated in greater detail based on the final cross-section characteristics established in step 7. The screening suggested the following ranking of the embankment configuration options:

    1) Option A – Sand Dam with Stone Columns
    2) Option D – Modified Rock Notches Dam with Maximum Seismic Filters
    3) Option C – Modified Rock Notches Dam with Minimum Filters
    4) Option B – Rockfill Dam with Jet Grouted Foundation

    The screening model also suggested that the mid-Sea barrier associated with Alternative 2 would be the highest-ranking embankment option when compared to all options for the mid-Sea dam. Additional information on the screening model and results is provided in Sub-section 4.11 below.

9. <u>Selection of Preferred Cross-Section Option</u> – Following the completion of step 8 above, a meeting was held and the updated screening model results reviewed with Reclamation personnel. Reclamation completed their decision making process following this meeting and selected Option A – Sand Dam with Stone Columns as the preferred configuration option.

10. <u>Initial Cross Section Optimization</u> – Based on Reclamation's decision, "optimized" cross-sections for the mid-Sea barrier, south- and north-Sea dams, and the perimeter and concentric lakes dikes were developed using the "Sand Dam with Stone Columns" concept as the basis (Tasks 9, 10, and 11).

11. <u>Risk Analysis</u> – A risk analysis of the "optimized" embankment cross-sections was performed (Task 13) as discussed in Appendix 2D

12. <u>Final Cross Section Optimization</u> – The risk analysis completed in step 11 identified several modifications to the various cross sections that would improve the overall safety and performance of the structures. These enhancements are included in the description of the "optimized" cross-sections described in Chapter 5.0 of this report. Appropriate details of these modifications are included in the analyses of the "optimized" embankment sections presented in Appendix 2E.

It should be noted that due to the progressive and evolutionary nature of the development of the embankment designs, information in the appendix reports do not necessarily correspond to the final optimized cross-sections.

4.3 Evaluation of Construction Material Sources

An initial evaluation of construction material sources was completed as part of Task 3 during the early stages of the project (see Appendix 2A). At that time, a broad range of construction materials was under consideration including a large amount of rockfill materials required for Reclamation's preferred embankment configuration option from the appraisal-level studies (see Figures 4.1 through 4.3). These materials included:

Rockfill:	1-foot to 4-foot diameter sound and durable quarry rock dumped under water
Fine Rockfill:	3-inch to 12-inch sound and durable quarry rock dumped under water
Sand/Gravel Core:	Clean sand and gravel, bucket placed in layers underwater
Filter Blanket:	Clean sand covered by a layer of gravel, both placed by dumping underwater

Subsequent to the selection of the "Sand Dam with Stone Columns" configuration for all of the different embankments, the focus of the evaluation of construction material sources shifted to the following major material requirements:

Type A Sand/Gravel:	Fine to coarse sand and gravel mix with maximum ¾-inch gravel size and maximum 10% fines suitable for compaction with stone columns. This material would be placed by dumping underwater from barges

	or by end dumping from trucks or conveyors.
Type B Sand/Gravel:	Fine to coarse sand and gravel mix with maximum 1¼-inch gravel size and variable fines content not intended for compaction with stone columns. This material would be placed similar to the Type A material.
Riprap Slope Protection:	1– 4-foot diameter sound and durable quarry rock dumped underwater. This material would be placed from barges or by end dumping from trucks.
Stone Column Gravel:	Sound and durable ¾- to 1-1/4 inch gravel/rock suitable for stone column construction

The initial construction materials appraisal completed as part of Task 3 as well as a subsequent site visit and evaluation indicated that sufficient quantities of rockfill, sand, gravel, and riprap materials are available from a number of sources around the Sea. These sources include the Coolidge Mountain /Aggregate Products (API) site on the west shore (located on the Torres Martinez Indian Reservation), the Eagle Mountain mine site located well northeast of the project, and relatively small borrow sites located along the east shore near the Bombay Beach area. The Bombay Beach sites are relatively thin and after further evaluation, have been eliminated from consideration as a possible source for large scale aggregate and riprap production.

The locations of the construction material sources described above, and the nature of the preferred embankment configuration provides opportunities for transport using both over water and over land methods. For example, the Torres Martinez Indian Reservation extends eastward from the material source locations out into the Sea itself. This would allow for creation of barge load-out facilities contiguous to the quarry and pit production facilities. Over land haul or conveyors could be used to transport materials from the source to the barge load-out facilities. Riprap cannot be moved by conveyor but must be transported by trucks.

Most of the embankments needed for the various project alternatives call for the creation of a broad crest width to allow for densification of the Type A sand/gravel using stone columns. The broad crest width provides the opportunity to use overland trucks to transport and place the embankment materials. It is possible that up to two-thirds of embankment materials could be placed by overland material handling methods and one-third could be placed over-water using barges. The over-water placement would be needed on the outer edges of the embankments, which cannot be reasonably reached from the edge of the broad crest areas. Temporary causeways would be required to provide access for trucks for construction of the mid-Sea dam, the south-Sea dam, the north-Sea dam, the concentric lakes dikes, and perimeter dike features. Causeways would vary from 4,000 to 7,000 feet long and would be built out from shore using end-dump techniques. Though relatively long, the shallow water depths inhibit the use of barges for constructing the temporary causeways.

As described above, all of the dam alternatives could rely heavily on overland hauling of embankment materials. However, it is possible that substantial portions if not the entire mid-Sea barrier could be built with over-water techniques allowing the barrier to emerge and become effective as the water level drops.

4.4 Stability and Seepage Analyses Results

A comprehensive series of seepage and stability analyses and evaluations have been performed to support the development of various mid-Sea dam embankment configuration options, "optimization" of the cross-section options, and selection of the preferred configuration. These analyses are summarized in a technical report presented in Appendix 2B. Supplemental stability analyses were performed to complete the optimization of the mid-Sea barrier, perimeter and concentric lakes dikes, and the north- and south-Sea dams. These analyses are discussed further in Chapter 5.0 and presented in Appendix 2E.

4.4.1 Seepage Analyses

Seepage models were developed for a variety of possible cross-sections of the mid-Sea dam, perimeter dikes, and habitat pond embankments, including study Options A through D.

Seepage analysis of all of the rockfill mid-Sea dam options show the computed seepage gradients (i_{xy}) in the foundation and through the embankment are in general less than 0.4, with the exception of areas in the immediate vicinity of the SCB slurry wall. Seepage rate per lineal foot of the embankment for various mid-Sea dam cross-sections ranges from 2.1×10^{-6} to 1.1×10^{-5} cfs/lineal foot. All alternatives produced similar results, indicating that the choice of foundation improvement (grouting, rock notches, etc) has a minimal impact on seepage analysis results. On the other hand, the presence and integrity of the cutoff wall, plays a major role. The maximum seepage gradient (i_{xy}) occurs through the SCB wall. Approximately 15 feet downstream of the wall along the embankment/foundation interface the gradient decreases to 0.4, and within 40 feet from the wall it reduces to 0.2. Permeability of the lacustrine deposits is 3 to 5 orders of magnitude lower than permeability of materials comprising the embankment and only one order of magnitude higher than that of a SCB wall. Accordingly, seepage through the foundation is minimal and flow velocities are low, compared to flows through downstream shells.

To illustrate the effect of an installation defect in the slurry wall, seepage was evaluated with a 5-foot high "defect", or hole, in the slurry wall. As expected, relatively high seepage gradients would develop around the defect. For this case the seepage gradient contour with the value of 0.2 extended approximately 100 feet downstream of the cutoff wall or twice as far as in the case of an intact slurry wall. Thus, the integrity of the seepage cutoff wall is essential for control of the dam performance with respect to piping resistance.

Seepage gradients for the stone-column-reinforced sand dam embankment with a soil-cement-bentonite (SCB) wall were evaluated assuming that a HDPE membrane was inserted to the base of the wall. The estimated seepage gradient through and around the wall changed as a result of the membrane. The estimated gradients were generally less than 0.3 except in the immediate vicinity of the bottom of the wall. The model suggests that a maximum gradient equal to 0.7 would occur at this location. Overall, incorporation of an impervious membrane reduces the computed gradients by as much as ten times compared to those estimated with only a SCB slurry wall.

4.4.2 Stability Analyses

A summary of the computed factors of safety for each of the mid-Sea embankment options, following “optimization” of the sections is summarized in Table 4.1.

Table 4.1
Summary of Computed Slope Stability Factors of Safety for Embankment Configuration Options A through D

Option	Description	Appendix B Figure No.	Strength Assumptions	Yield Acceleration, g (a_y)	Post-Earthquake Factor of Safety
A	Sand Dam with Stone Columns	B.B-24b	Parametric study completed, see Appendix B	0.17	>2.2
B	Rockfill Dam with Jet Grouted Foundation	B.B-2		0.17	>2.8
C	Modified Rock Notches Dam with Minimum Filters	B.B-6	Liquefied filter c=150 psf	0.17	>2.2
D	Modified Rock Notches Dam with Maximum Seismic Filters	B.B-25	Liquefied filter c=150 psf	0.17	>3.2

Factors of safety for the "optimized" mid-Sea barrier were estimated for two configurations meeting two design criteria: 1) static only, and 2) static and seismic. The cross-section shown on Figure 4.12 meeting the static only design criteria has a minimum static factor of 2.4 to 2.67 for drained friction angles ranging from 30 to 34 degrees for the embankment materials. These factors of safety are for the end of construction condition and are only slightly higher for the end of primary consolidation condition. The cross-section meeting the static and seismic design criteria has a minimum post-earthquake factor of safety ranging from 1.74 to 2.34 for assumed strengths of the stone column densified Type A sand/gravel core materials ranging from 1000 to 1500 psf. Likewise the yield acceleration ranges from 0.09 to 0.16g. These yield accelerations are slightly lower than the design criteria of 0.17g, but were judged acceptable for the barrier because the strength assumption used in the analysis is conservative for a target

$N_{1,60}$ blowcount of between 20 and 25 for the stone column densified embankment materials.

The factors of safety for the "optimized" perimeter and concentric lakes dikes, north- and south-sea dams, and the habitat pond embankments are discussed further in Chapter 5.0.

4.5 Deformation Analysis Results

Seismic deformation analyses of the "optimized" mid-Sea sand dam with stone columns, and Reclamation's rockfill perimeter dike option were completed using the commercial finite difference code FLAC. Model cases evaluated both liquefied and non-liquefied strengths of foundation and non-densified dam materials. The effect of a range of different material properties that would occur for various stone column improvement objectives (i.e. various target $N_{1,60}$ blow counts following densification) were also evaluated.

The deformation study is summarized in Appendix 2C. The conclusions of the study were as follows:

1. In general, the displacements estimated with the FLAC models of two different embankment configuration options fall between the displacements estimated by simplified Newmark and Makdisi-Seed methods for the surface and deconvolved ground motions. As shown on Figure 4.8, the average crest displacements computed from FLAC are more centered within these limits. Combining the FLAC results and the simplified Newmark and Makdisi-Seed results provides a sound basis to establish a planning level screening criteria for yield acceleration that can reliably and conservatively estimate adequate or marginal crest deformation performance based on the input ground motions as provided by Reclamation. For purposes of "optimizing" all cross-sections, a minimum yield acceleration criterion of 0.17g was selected.
2. The estimated crest deformations of the optimized mid-Sea sand dam would generally be less than the five feet of available freeboard included in the design. Crest deformations for a variety of strength assumptions associated with the stone column densified core, as well as the shells of the dam, are summarized on Figure 4.9. In general, to achieve the required performance, the central portion of the dam would need to be densified to an equivalent $N_{1,60}$ blowcount of 20 achieving a target undrained strength (S_{us}) of at least 1,000 psf, or a drained strength friction angle of at least 32 degrees.
3. The loss of freeboard predicted for the perimeter dikes is minimal. However, the FLAC analysis results suggest that a minimum of 2 feet of freeboard should be included in areas where foundation liquefaction would not occur, and 3 feet of freeboard should be used where liquefaction of the foundation alluvium would be expected.

4. The estimated maximum strains along the centerline axis of the dam (the location of the slurry wall cutoff) would occur at the contact between the dam and the stiff lacustrine materials. The maximum strain occurring at this location would likely range from 0.15 to 0.2 percent. A soil-cement-bentonite (SCB) wall should be capable of withstanding this level of strain without significant rupture and offset that would threaten the safety of the dam. Future FLAC modeling efforts should include an explicit SCB slurry wall to confirm the strain estimates of this study.

4.6 Option A — Sand Dam with Stone Columns

The “optimized” Sand Dam with Stone Columns embankment, Option A is shown on Figure 4.10. This option would be constructed by a combination of placing methods, including end dumping or conveyor placement for the upper portions of the central embankment materials and by dumping/placing directly into the water from barges for the lower portion of the central section, and for the outer portions of the embankment including the riprap slope protection.

The dam design includes the following features in the general order in which they would be constructed:

- ✓ Removal of all Seafloor deposits beneath the entire footprint of the dam
- ✓ Removal of soft lacustrine and/or upper alluvial deposits from beneath the central portion of the embankment. For the maximum section shown on Figure 4.10, this removal would be for a length of about 370 feet symmetrical about the centerline axis of the dam. As the section becomes lower in height near the edges of the Sea, the dimensions of the cross-section would be appropriately reduced while still meeting required static and seismic design criteria. Dredged slopes would be established to provide adequate stability for the period of the open dredged excavation.
- ✓ Treatment of the zone of upper alluvial and soft lacustrine deposits that would be left in place under the outer shells of the dam with wick drains (square pattern on 5-foot centers) to facilitate consolidation/settlement of these materials
- ✓ Placement of sand/gravel (Type A) embankment material in the dredged central section back to an elevation that is 5 feet above the soft lacustrine surface prior to dredging. The Type A sand/gravel material would extend over the soft lacustrine materials under the downstream outer shell sand/gravel (Type B) to serve as a filter blanket and would likewise extend over the soft lacustrine materials to the edge of the outer shell sand/gravel (Type B) under the upstream slope. These materials would be placed by over-water methods.

- ✓ Placement of the 10-foot-thick sand/gravel (Type B, modified to have low fines content) blanket drain in the downstream portion of the cross-section by over-water methods
- ✓ Placement of the remainder of the Type A sand/gravel by end dump or conveyor methods. The crest elevation at the end of construction would include an appropriate overbuild to accommodate consolidation/settlement of the lower stiff lacustrine deposits as well as settlement that may occur beneath the central section of the dam during stone column placement. The cross-section on Figure 4.10 shows the crest having a width of about 370 feet to create a platform for subsequent stone column placement. It also shows that the outer slopes of the Type A material would be placed at about 3 (horizontal) to 1 (vertical) (3H:1V). The actual outer slope angles of these materials will have to be adjusted to suit actual field conditions in order to prevent the possibility of a construction slope failure through the soft lacustrine foundation materials.
- ✓ Placement of the outer shells using Type B sand/gravel.
- ✓ Installation of the stone columns (triangular pattern on 10-foot centers) to form the interior densified sand/gravel section with 3H:1V upstream and downstream slopes.
- ✓ Installation of a 5-foot-wide soil-cement-bentonite (SCB) slurry wall through the central densified sand/gravel (Type A) materials, and penetrating into the upper stiff lacustrine deposit to elevation –350 msl or 40 feet below the soft/stiff lacustrine or alluvial/stiff lacustrine interface. The wall would not be required immediately and could be constructed at a later date under a separate construction contract prior to the development of more than 5 feet of differential water levels between the marine lake and brine pool side of the dam.
- ✓ Placement of the riprap slope protection materials over the upstream and downstream slopes of the dam. Details of the riprap section should be developed during final design. This detail may include a supplemental riprap bedding to serve as a filter between the coarse riprap and the Type B embankment materials.

Results of stability and deformation evaluations indicate that the required finished upstream and downstream slopes of the dam would be on the order of 5H:1V.

The gradation of the Type A sand/gravel core material that is also used for the downstream filter blanket would be designed to be filter compatible with the finer grained portions of the soft lacustrine deposits. The gradation of the Type B sand/gravel material used for the downstream blanket drain would have a low fines content compared to the regular Type B shell material.

4.7 Option B — Rockfill Dam with Jet Grouted Foundation

The "optimized" Rockfill Dam with Jet Grouted Foundation embankment, Option B is shown on Figure 4.11. This option would be constructed by placing embankment material directly into the water from barges for the lower portions of the embankment and by end dumping or conveyor placement for the upper portions once the depth of water precludes over-water construction methods. Appraisal level studies determined that the use of cofferdams for construction in the dry was too risky from a construction safety standpoint.

The dam design includes the following features in the general order in which they would be constructed:

- ✓ Removal of all Seafloor deposits beneath the entire footprint of the dam
- ✓ Jet grouting treatment of both upper alluvial and soft lacustrine deposits beneath the upstream and downstream shells of the dam. A 240-foot-wide treatment zone is required under the downstream shell and a 195-foot-wide treatment zone is required under the upstream shell
- ✓ Treatment of the zone of upper alluvial and soft lacustrine deposits that would be left in place under the central section of the dam with wick drains to facilitate consolidation/settlement of these materials
- ✓ Placement of the 5-foot-thick filter blanket of Type A sand/gravel over the upper alluvial and soft lacustrine deposits at the base of the central core, downstream of the location of the SCB slurry wall, and overlapping the downstream jet-grouted zone by 30 feet
- ✓ Placement of a 5-foot-thick fine rockfill coarse filter over the Type A sand/gravel filter blanket
- ✓ Simultaneous placement of the 30-foot-wide sand/gravel core, the 30-foot-wide upstream and 50-foot-wide downstream fine rockfill, and the outer rockfill zones to the planned crest elevation. The crest elevation at the end of construction would include an appropriate overbuild to accommodate consolidation/settlement of the upper alluvial and soft lacustrine deposits beneath the central section of the dam
- ✓ Installation of a 5-foot-wide soil-cement-bentonite (SCB) slurry wall through the sand/gravel core, the soft lacustrine and/or upper alluvial deposits and penetrating into the upper stiff lacustrine deposit to elevation –350 msl or 40 feet below the soft/stiff lacustrine interface. The wall would be constructed after the majority of the settlement of the soft lacustrine materials has occurred but prior to the development of more than 5 feet of differential water levels on the marine lake and brine pool sides of the dam

Results of stability and deformation evaluations indicate that the upstream slope of the dam would be on the order of 5H:1V. The downstream slope of the dam would be on the order of 7H:1V.

Appraisal level studies by Reclamation considered three general methods of improving soft foundation soils including: 1) excavation and replacement, 2) densification, and 3) cementation. It was determined that cementation with jet grouting would be the most practical and efficient means of foundation treatment. The jet grouting treatment approach was used for these planning studies.

The gradations of the sand/gravel core, the fine rockfill, and the outer rockfill shells would be designed for filter and drainage compatibility. Likewise, the Type A sand/gravel core material that is also used for the filter blanket would be designed to be filter compatible with the finer grained portions of the soft lacustrine deposits.

4.8 Option C — Modified Rock Notches Dam with Minimum Filters

The "optimized" Modified Rock Notches Dam with "Minimum" Filters embankment, Option C is shown on Figure 4.12. This option would be constructed by placing embankment material directly into the water from barges for the lower portions of the embankment and by end dumping or conveyor placement for the upper portions once the depth of water precludes over-water construction methods.

The dam design includes the following features in the general order in which they would be constructed:

- ✓ Removal of all Seafloor deposits beneath the entire footprint of the dam
- ✓ Treatment of both upper alluvial and soft lacustrine deposits beneath the central portion of the dam with wick drains to facilitate consolidation/settlement of these materials
- ✓ Removal of soft lacustrine or upper alluvial deposits from beneath the rockfill shell portions of the embankment for construction of the "rock notches". For the maximum section shown on Figure 4.12, the removal would be for a length of about 110 feet for the downstream rock notch and 125 feet for the upstream rock notch. The downstream notch would extend for a depth of 40 feet below the surface of the soft lacustrine or upper alluvium deposits into the upper still lacustrine materials. The upstream notch would be excavated through the soft lacustrine deposit to the top of the upper stiff lacustrine materials. As the cross-section becomes lower in height near the edges of the Sea, the dimensions of the cross section would be appropriately reduced while still meeting required static and seismic design criteria. Dredged slopes would be established at

about 3H:1V to provide adequate stability for the period of the open dredged excavation and for the desired dimensions of the notches.

- ✓ Placement of the 5-foot-thick filter blanket of Type A sand/gravel over the upper alluvial and soft lacustrine deposits at the base of the central core, downstream of the location of the SCB slurry wall, and over upstream-most slope of the dredged excavation for the downstream notch
- ✓ Placement of a 5-foot-thick fine rockfill coarse filter over the Type A sand/gravel filter blanket
- ✓ Placement of rockfill materials in the excavated notches up to the elevation where additional fine rockfill and Type A sand/gravel core materials are required
- ✓ Simultaneous placement of the 30-foot-wide sand/gravel core, the 30-foot-wide upstream and 50-foot-wide downstream fine rockfill, and the outer rockfill shell zones to the planned crest elevation. The crest elevation at the end of construction would include an appropriate overbuild to accommodate consolidation/settlement of the upper alluvial and soft lacustrine deposits, and the upper stiff lacustrine deposit beneath the central section of the dam
- ✓ Installation of a 5-foot-wide soil-cement-bentonite (SCB) slurry wall through the sand and gravel core, the soft lacustrine and/or upper alluvial deposit and penetrating into the upper stiff lacustrine deposit to elevation –350 msl or 40 feet below the soft/stiff lacustrine or alluvial/stiff lacustrine interface. The wall would be constructed after the majority of the settlement of the soft lacustrine materials has occurred but prior to the development of more than 5 feet of differential water levels on the marine lake and brine pool sides of the dam

Results of stability and deformation evaluations indicate that the upstream slope of the dam would be on the order of 5H:1V. The downstream slope of the dam would be on the order of 7H:1V.

The gradations of the sand/gravel core, the fine rockfill, and the outer rockfill shells would be designed for filter and drainage compatibility. Likewise, the Type A sand/gravel core material that is also used for the filter blanket would be designed to be filter compatible with the finer grained portions of the soft lacustrine, upper alluvium, and upper stiff lacustrine deposits.

The filter over the foundation deposits of this option is considered “minimum” because it does not extend over the bottom or downstream slope of the rock notch excavation. Terminating the filter at the upstream edge of the base of the excavation provides significant stability benefits by eliminating a liquefiable material at critical locations in the cross-section. The cross-section as shown on Figure 4.12 generally meets stability, deformation, and seepage gradient criteria discussed in section 4.1.4. For the assumption of homogenous but anisotropic (horizontal permeability > vertical permeability by a factor of 10) conditions in the upper stiff lacustrine deposit at the base of the notch excavation, estimated exit gradients from the upper stiff lacustrine deposit into the rockfill materials at the unfiltered interface at the base of the notch are less than or equal to 0.2 (see Appendix 2B). However, geologic conditions suggest some possibility for laterally persistent lenses (inclusions) of highly erodible silty sands, sandy silts and silts to exist in the upper stiff lacustrine deposits. Further, there is some likelihood that these inclusions may provide a mechanism for higher water pressures to bypass the cutoff wall and extend into the foundation under the downstream part of the dam, creating local areas with higher potential for adverse seepage gradients and erosion. Hence, this partial filters section was judged by Reclamation to not fully meet a requirement for full filter protection. The requirements for filter protection were a consideration in the evaluation and selection of a preferred cross-section configuration as discussed further in Sub-section 4.11.

4.9 Option D — Modified Rock Notches Dam with Maximum Seismic Filters

The “optimized” Modified Rock Notches Dam with “Maximum Seismic” filters embankment, Option D is shown on Figure 4.13. This option would be constructed by placing embankment material directly into the water from barges for the lower portions of the embankment and by end dumping or conveyor placement for the upper portions once the depth of water precludes over-water construction methods.

The dam design includes the following features in the general order in which they would be constructed:

- ✓ Removal of all Seafloor deposits beneath the entire footprint of the dam
- ✓ Treatment of both upper alluvial and soft lacustrine deposits beneath the central portion of the dam with wick drains to facilitate consolidation/settlement of these materials
- ✓ Removal of soft lacustrine or upper alluvial deposits from beneath the rockfill shell portions of the embankment for construction of the “rock notches.” For the maximum section shown on Figure 4.13, the removal would be for a bottom length of about 65 feet for the downstream rock notch and 125 feet for the upstream rock notch. The downstream notch would extend for a depth of 65 feet below the surface of the soft lacustrine or upper alluvium deposits into the upper stiff lacustrine materials. The upstream notch would be excavated through the soft lacustrine deposit to the top of the upper stiff lacustrine materials. As the cross-section becomes lower in height near the edges of the Sea, the dimensions of the cross section would be appropriately reduced while still meeting required static and seismic design criteria. Dredged slopes would be established at

about 3H:1V to provide adequate stability for the period of the open dredged excavation and for the desired dimensions of the notches

- ✓ Placement of the 5-foot-thick filter blanket of Type A sand/gravel over the upper alluvial and soft lacustrine deposits at the base of the central core, downstream of the location of the SCB slurry wall, and over the upstream-most slope and base of the dredged excavation for the downstream notch
- ✓ Placement of a 5-foot-thick fine rockfill coarse filter over the Type A sand/gravel filter blanket
- ✓ Placement of rockfill materials in the excavated notches up to the elevation where additional fine rockfill and Type A sand/gravel core materials are required
- ✓ Simultaneous placement of the 30-foot-wide sand/gravel core, the 30-foot-wide upstream and 50-foot-wide downstream fine rockfill, and outer rockfill shell zones to the planned crest elevation. The crest elevation at the end of construction would include an appropriate overbuild to accommodate consolidation/settlement of the upper alluvial and soft lacustrine deposits, and the upper stiff lacustrine deposit beneath the central section of the dam
- ✓ Placement of additional rockfill material to create a 40-foot-high by 280-foot-wide stability berm over the downstream slope and rockfill notch
- ✓ Installation of a 5-foot-wide soil-cement-bentonite (SCB) slurry wall through the sand and gravel core, soft lacustrine deposit and penetrating the stiff lacustrine deposit to elevation –350 msl or 40-feet below the soft/stiff lacustrine or alluvial/stiff lacustrine interface. The wall would be constructed after the majority of the settlement of the soft lacustrine materials has occurred, but prior to the development of more than 5 feet of differential water levels on the marine lake and brine pool sides of the dam. Note that the SCB wall extends to the same elevation as the base of the downstream rock notch

Results of stability and deformation evaluations indicate that the upstream slope of the dam would be on the order of 5H:1V. The downstream slope of the dam would be on the order of 7H:1V to the top of the stability berm and then about 2H:1V from the top of the berm to the dredged soft lacustrine deposit. The 2H:1V slope may need to be locally modified to provide adequate static stability depending on the strength of the foundation materials.

The gradations of the sand/gravel core, the fine rockfill, and the outer rockfill shells would be designed for filter and drainage compatibility. Likewise, the Type A sand/gravel core material that is also used for the filter blanket would be designed to be filter compatible with the finer grained portions of the soft lacustrine, upper alluvium, and upper stiff lacustrine deposits.

As noted on Figure 4.13, various combinations of notch depth and berm heights were considered to “optimize” this cross section. The dashed outline on this figure (on the downstream side) is an example of a deeper excavation with a lower berm that would also meet stability and deformation criteria. The dashed section would however require more dredging, and rockfill materials, and would present additional risk of dredged excavation instability during construction. While the primary section shown on this figure is considered “optimized” for the purpose of this study, additional refinements should be considered during final design should this configuration be selected for implementation.

4.10 Mid-Sea Barrier

The “optimized” mid-Sea barrier embankment built using the fundamental concepts of the sand dam with stone columns described in Sub-section 4.6 is shown on Figure 4.14. This is referred to as the Mid-Sea Barrier option in the subsequent comparative analysis described in Sub-section 4.11. The barrier would be constructed under a variety of operating scenarios. For example, the entire barrier could be constructed under water prior to the Sea reaching a water level at or below the planned crest elevation of –245 msl. Alternatively, the barrier could be constructed after the Sea reaches a water level that is below the planned crest elevation of –245 msl by a combination of placing methods including end dumping or conveyor placement of the upper portions of the central embankment materials and by dumping/placing directly into the water from barges for the lower portion of the central section, and outer portions of the embankment including the riprap slope protection.

Unlike the dam, which can support differing water elevations on each side, the barrier would not provide water level elevation control and is only expected to experience a maximum of approximately 5 feet of differential water levels on the opposite sides of the structure. During appraisal level studies, Reclamation described the barrier as having a semi-pervious core to allow seepage flows to pass through the structure in anticipation that the water pool on the side with fresh water inflows would become less saline over time while the pool on the side that received only seepage through the barrier would become more saline. However, the current designs include an impermeable barrier in the central portion of the dam and an outlet works between the pools to provide the control necessary to effectively achieve the desired water levels.

It should also be noted that a range of design criteria is under consideration for the mid-Sea barrier concept. For purposes of this study, a cross-section that meets only static design criteria outlined in Sub-section 4.1.4 as well as a section meeting both static and seismic design criteria have been developed and will be considered as part of a risk-based cost comparison. Both of these cross-sections are illustrated on Figure 4.14.

The barrier design includes the following features in the general order in which they would be constructed:

- ✓ Removal of all Seafloor deposits beneath the entire footprint of the barrier
- ✓ Removal of soft lacustrine and/or upper alluvial deposits from beneath the central portion of the embankment. For the maximum sections shown on Figure 4.14, the removal would be for a length of about 80 feet symmetrical about the centerline axis of the barrier (static criteria only) or for a length of about 112 feet (both static and seismic design criteria). As the section becomes lower in height near the edges of the Sea, the dimensions of the cross-sections would be appropriately reduced while still meeting required design criteria. Dredged slopes would be established to provide adequate stability for the period of the open dredged excavation
- ✓ Treatment of the zone of upper alluvial and soft lacustrine deposits that would be left in place under the outer shells of the barrier with wick drains (square pattern on 5-foot centers) to facilitate consolidation/settlement of these materials
- ✓ Placement of sand/gravel (Type A) embankment material in the dredged central section back to the elevation that is the same as the soft lacustrine surface prior to dredging. These materials would be placed by over-water methods
- ✓ Placement of the remainder of the Type A sand/gravel by end dump or conveyor methods. The crest elevation at the end of construction would include an appropriate overbuild to accommodate consolidation/settlement of the upper stiff lacustrine deposits as well as settlement that may occur beneath the central section of the dam during stone column placement. The cross-sections on Figure 4.14 show the crest having a width of 30 feet for the barrier meeting only static design criteria, and about 150 feet to create a platform for subsequent stone column placement for the cross-section meeting both static and seismic design criteria. It also shows that the outer slopes of the Type A material would be placed at about 1.5H:1V for the dam meeting static only design criteria. For the cross-section meeting both static and seismic criteria, Type A sand/gravel would predominately be used for the embankment with a minor amount of Type B material used for the outer shells. The actual width of the bottom of the dredged excavation, the corresponding width of the stone column platform, and the outer slope angles of these materials will have to be adjusted to suit actual field conditions and the densification objective of the stone columns
- ✓ Placement of the outer shells using Type B sand/gravel (static design criteria section only)

- ✓ Installation of the stone columns (triangular pattern on 10-foot centers) to form the interior densified section with 2H:1V upstream and downstream slopes. This step is required if the embankment must meet both static and seismic design criteria
- ✓ Installation of a 5-foot-wide soil-cement-bentonite (SCB) slurry wall through the central sand/gravel materials, and penetrating the upper stiff lacustrine deposit to elevation –350 msl or 40 feet below the soft/stiff lacustrine or alluvial/stiff lacustrine interface. The SCB wall would not be required immediately and could be constructed at a later date under a separate construction contract prior to the development of more than 2 feet of differential water levels on the marine lake and brine pool sides of the barrier
- ✓ Placement of the riprap slope protection materials over the upstream and downstream slopes of the barrier and the crest

Results of stability and deformation evaluations indicate that the required finished upstream and downstream slopes of the barrier meeting static design criteria only would be on the order of 5H:1V. The outer slopes of the barrier cross-section meeting both static and seismic criteria would be about 3H:1V.

The gradations of the Type A sand/gravel core material that is also used for the filter blanket would be designed to be filter compatible with the finer grained portions of the soft lacustrine deposits.

4.11 Evaluation and Selection of Preferred Dam and Dike Configurations

A decision model was developed during the course of this study to assist Reclamation with the identification and selection of a preferred embankment configuration option for the various embankments described under the five overall restoration alternatives (see Chapter 3). The decision model was initially conceived and used in step 4 of the overall evaluation process as described in Sub-section 4.2. The initial model, described further in Appendix 2B, was sufficient to determine that Option A – Sand Dam with Stone Columns, and Options D/C warranted further evaluation.

Subsequently, the initial (step 4) screening model was modified to include multiple technical criteria including:

1. Post-earthquake factor of safety of greater than or equal to 1.3
2. Estimated average crest deformation of less than 5 feet
3. Meet Reclamation's filter criteria and "full filter" requirements
4. Constructability issues rating

5. Cost estimate rating

Additional stability and seepage evaluations were performed to further optimize the embankment configurations. Cost estimates for the “optimized” embankment cross-section options were prepared and the cost estimate rating was modified in the model.

As previously noted, a combination of stability and deformation analyses help to identify a yield acceleration criteria (0.17g) that provided a conservative basis to predict when the average crest deformations would be less than 5 feet. Two embankment dam configurations, Options A, and B described in the preceding Sub-sections and shown on Figures 4.10, and 4.11 meet the required technical criteria outlined above including each having estimated yield accelerations in the range of 0.17g or higher. Options C, Modified Rock Notches Dam with Minimum Filters, and D, Modified Rock Notches Dam with Maximum Seismic Filters met all Reclamation criteria except the provision of “full filters” within the dam. Hence, for Options A, and B, the primary discriminators in the model became the constructability issues rating and the cost estimate rating.

Each criterion was scored from a minus two (-2) to a plus two (+2) depending on the relative outcome for each option being evaluated. The –2 was assigned to a very poor outcome, or the lowest relative ranking. A +2 was assigned if the outcome was very favorable or if an option had the highest relative ranking.

Weighting factors were applied to the various criteria as follows:

Technical Issues (Criteria 1, 2, and 3):	50%
Constructability (Criterion 4):	30%
Cost Estimate:	20%
Total	100%

The cost estimate rating was set based on a cost comparison of the maximum cross-sections shown on Figures 4.10, 4.11, 4.12, and 4.13. Specifically, the estimated quantities of various materials required for each lineal yard of the maximum sections were estimated. The estimated quantities were then multiplied by unit prices for the various materials as presented in Reclamations appraisal level studies to determine a total estimated price ($) per yard of embankment. The results of these cost estimates are summarized in Tables 4.2 through 4.5 for Options A through D, respectively. A summary of the estimated costs for each cross section and the corresponding ranking used in the model is as follows:

Option	Total Estimated Price ($/yd. Emb.)	Score
A	$ 113,157	1
B	$ 349,598	-2
C	$ 83,904	2
D	$ 112,074	1

A summary of the model results is provided in Table 4.6. The screening suggested the following ranking of the embankment configuration options:

Option A – Sand Dam with Stone Columns
Option C – Modified Rock Notches Dam with Minimum Filters
Option B – Rockfill Dam with Jet Grouted Foundation
Option D – Modified Rock Notches Dam with Maximum Seismic Filters

To put these screening results into a fuller context, the mid-Sea barrier cross-section was also rated according the criteria and weighting factors of the decision model. The ranking of the barrier is also presented in Table 4.6. As can be seen, the screening model results suggested that the "optimized" mid-Sea barrier associated with Alternative 2 would be the highest-ranking embankment option when compared to all options for the mid-Sea dam.

Based on the results of various analyses and the ranking described above, Reclamation selected Option A – Sand Dam with Stone Columns as the preferred embankment configuration. Using this configuration and the results of the various stability and seepage analyses of the "optimized" cross-section shown on Figure 4.10, cross-sections for the south- and north-Sea dams, and the perimeter and concentric lakes dikes were developed. Supplemental stability analyses were performed for each of the cross-sections to verify that static and, as appropriate, seismic criteria would be met. The "optimized" cross sections for the south- and north-Sea dams, the perimeter dikes, and the concentric lakes dikes are described further in Chapter 5.0.

Table 4.2
Summary of Cross-Section Costs
Option A Sand Dam with Stone Columns

No.	Item	Unit	Estimated Quantity (per yd. of emb.)	Unit Price ($)	Total Estimated Price ($/yd.emb.)
1	Dredge Sea Floor Deposits	C.Y.	1333.0	5.30	7,065
2	Dredge Soft Lacustrine Deposits	C.Y.	1111.0	5.30	5,888
3	Sand/Gravel Fill Type A**	C.Y.	2451.0	14.50	35,540
4	Sand/Gravel Shell Type B	C.Y.	2888.0	14.50	41,876
5	Stone Columns/per yard of dam alignment*	C.Y.	645.0	35.33	22,788
6	Installation of SCB Wall with Membrane	S.F.	261	10.15	2,649
	Total Comparative Cost				**113,157**

Notes:
a) *It is currently planned that one row of stone columns would be constructed for 10 feet of dam alignment.
*Estimated stone column diameter is 3 feet. The volume for a single row of stone column is approximately 563 cubic yards
b) *Estimated volume of dredge of soft lacustrine deposits underneath the sand/stone columns portion includes a 1H:1V slope
c) **Quatities above assume a sand/gravel platform with a 5H:1V side slope will be constructed to facilite installation of stone columns
d) SCB wall assumed to be 5 feet wide
e) Estimate for sand /gravel and rock fill material based on supply from Coolidge Mountain source only. Actual unit costs for each will be higher in final cost estimate if Eagle Mountain source is included.
f) Unit prices are for similar means and methods including barge placement. Unit prices for some materials will reduce if conveyor hauling and placement is included in final total project cost estimates. This is not expected to alter the outcome of the comparative analysis of the main dam options considered.
g) Riprap materials included in Type B material quantity estimate
h) No wick drains included
i) Unit prices shown in this table are not the same as developed and used in the final cost estimates presented in Section 8.0

Table 4.3
Summary of Cross-Section Costs
Option B Rockfill Dam with Jet Grouted Foundation

No.	Item	Unit	Estimated Quantity (per yd. of emb.)	Unit Price ($)	Total Estimated Price ($/yd.emb.)
1	Dredge Sea Floor Deposits	C.Y.	1000	5.30	5,300
2	Placement of Filter Drain	C.Y.	67	18.50	1,233
3	Placement of Fine Rockfill	C.Y.	487	16.80	8,183
4	Placement of Type A Sand/Gravel Core	C.Y.	158	14.50	2,296
5	Placement of Rockfill	C.Y.	1612	16.80	27,085
6	Jet Grouting of Soft Lacustrine Deposits	C.Y.	1208	250.00	302,083
7	Wick Drains @ 5' Spacing	L.F.	645	2.00	1,290
8	Installation of SCB Wall w/o Membrane	S.F.	261	8.15	2,127
	Total Comparative Cost				**349,598**

Notes:
1) SCB wall assumed to be 5 feet wide
2) Estimate for sand/gravel and rockfill material based on supply from Coolidge Mountain source only. Actual unit costs for each wil be higher in final cost estimate if Eagle Mountain Source is included.
3) Unit prices are for similar means and methods including barge placement. Unit prices for some materials will reduce if conveyor hauling and placement is included in final total project cost estimates. This is not expected to alter the outcome of the comparative analysis of the main dam options.
4) Unit prices shown in this table are not the same as developed and used in the final cost estimates presented in Section 8.0

Table 4.4
Summary of Cross-Section Costs
Option C Modified Rock Notches Dam with Minimum Filters

No.	Item	Unit	Estimated Quantity (per yd. of emb.)	Unit Price ($)	Total Estimated Price ($/yd.emb.)
1	Dredge Sea Floor Deposits	C. Y.	1000.0	5.30	5,300
2	Dredge Soft Lacustrine Deposits	C. Y.	1316.7	5.30	6,978
3	Excavation of Stiff Lacustrine	C. Y.	259.2	5.95	1,542
4	Placement of Filter Drain	C. Y.	107.8	19.50	2,102
5	Placement of Fine Rockfill	C. Y.	458.1	16.80	7,696
6	Placement of Type A Sand/Gravel Core	C. Y.	158.3	14.50	2,296
7	Placement of Rockfill	C. Y.	3179.0	16.80	53,407
8	Wick Drains @ 5' Spacing	L.F.	510.0	2.00	1,020
9	Installation of SCB Wall with Membrane	S.F.	351.0	10.15	3,563
	Total Comparative Cost				**83,904**

Notes:
1) SCB wall assumed to be 5 feet wide
2) Estimate for sand /gravel and rock fill material based on supply from Coolidge Mountain source only. Actual unit costs for each will be higher in final cost estimate if Eagle Mountain source is included.
3) Unit prices are for similar means and methods including barge placement. Unit prices for some materials will reduce if conveyor hauling and placement is included in final total project cost estimates. This is not expected to alter the outcome of the comparative analysis of the main dam options considered.
4) Unit prices shown in this table are not the same as developed and used in the final cost estimates presented in Section 8.0

Table 4.5
Summary of Cross-Section Costs
Option D Modified Rock Notches Dam with Maximum Seismic Filters

No.	Item	Unit	Estimated Quantity (per yd. of emb.)	Unit Price ($)	Total Estimated Price ($/yd.emb.)
1	Dredge Sea Floor Deposits	C.Y	1100.0	5.30	5,830
2	Dredge Soft Lacustrine Deposits	C.Y	1525.7	5.30	8,086
3	Excavation of Stiff Lacustrine	C.Y	728.3	5.95	4,334
4	Placement of Filter Drain	C.Y	183.3	19.50	3,575
5	Placement of Fine Rockfill	C.Y	458.1	16.80	7,696
6	Placement of Type A Sand/Gravel Core	C.Y	158.3	14.50	2,296
7	Placement of Rockfill	C.Y	4589.9	16.80	77,110
8	Wick Drains @ 5' Spacing	L.F.	510	2.00	1,020
9	Installation of SCB Wall w/o Membrane	S.F.	261.0	8.15	2,127
	Total Comparative Cost				**112,074**

Notes:
a) SCB wall assumed to be 5 feet wide
b) Estimate for sand /gravel and rock fill material based on supply from Coolidge Mountain source only. Actual unit costs for each will be higher in final cost estimate if Eagle Mountain source is included.
c) Unit prices are for similar means and methods including barge placement. Unit prices for some materials will reduce if conveyor hauling and placement is included in final total project cost estimates. This is not expected to alter the outcome of the comparative analysis of the main dam options considered.
d) Unit prices shown in this table are not the same as developed and used in the final cost estimates presented in Section 8.0

Table 4.6
Summary of Comparative Analysis Results - Mid-Sea Dam Configuration Option

Option No.	Option Description	Stability and Seepage Report Figure No.(3)	Strength Assumptions	Yield Acceleration (a_y) (1)	Decision Criteria: Post-EQ F.S.	Estimated Crest Deformation < 5 feet	USBR Filter Criteria	Technical Criteria Rating	Constructability Issues Rating	Cost Estimate Rating	Total Weighted Score (2)	Ranking
A	Sand Dam w/ Stone Columns	B.B-24b	parametric study completed	0.17	> 2.2	Y	Y	2	2	1	1.8	2
B	Rockfill w/ Jet Grouted Foundation	B.B-2		0.17	>2.8	Y	Y	2	-1	-2	0.3	4
C	Modified Rock Notches w/ Minimum Filters	B.B-6	Liq. Filter C=150 psf	0.17	> 2.2	Y	N	0	0	2	0.4	3
D	Deep Rock Notches w/ Maximum Seismic Filters	B.B-25	Liq. Filter C=150 psf	0.17	>3.2	Y	N	0	-2	1	-0.4	5
	Mid Sea Barrier			N/A	>1.3	N/A	Y	2	2	2	2	1

Notes:
a) Yield accelerations shown are for undeformed cross section
b) Weighted score based on Technical Criteria at 50%, Constructability at 30% and Cost Estimate at 20%
c) See Appendix 2B, Attachment B
d) All options may experience some damage during a large earthquake event. Rating does not include consideration of risk based costs associated with possible earthquake damages