Restoration of the Salton Sea

Volume 2: Embankment Designs and Optimization Study

Appendix 2B: Seepage and Stability Analysis

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Table of Contents

List of Tables

List of Figures

1.0 Introduction

This seepage and slope stability report presents the results of geotechnical analyses of seepage and stability of various embankment configuration options currently being considered for the Salton Sea restoration project in southern California. This report forms Appendix 2B in Kleinfelder's complete report for the Salton Sea restoration project. The requirements for this work were outlined by Reclamation under Tasks 4 and 5 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. The following chapters provide a brief project description, a summary of the scope of work requirements and the key personnel that performed the work and prepared this report.

1.1 Project Description

The Salton Sea (Sea), a hypersaline lake that is the largest inland water body in California, is located in a low-lying basin known as the Salton Sink. According to geologic studies reported by others (URS, 2004a), the Salton Sink over geologic time has been occupied periodically by inland seas of varying size. Over the 100-year lifetime of the current Salton Sea, the size of the lake has varied, and declining water quality has prompted current studies by various government agencies and jurisdictions to "restore" the Salton Sea. According to URS (2004a), the current lake is about 35 miles long and 15 miles wide with a water surface elevation of about -227 feet Mean Sea Level (MSL).

The restoration alternatives for the Salton Sea that Reclamation and the Salton Sea Authority are currently considering and the corresponding possible embankment components, as discussed in the Scope of Work (SOW) and during a June 2, 2006 meeting with Reclamation, are summarized below.

1) <u>Mid-Sea Dam/North Marine Lake</u> – This alternative (Alternative No. 1) would include an approximately 9-mile long impervious mid-Sea dam across the existing lake to create an upstream pool of saltwater and a downstream brine pool. The mid-Sea dam, shown on Figure B.1, would be up to 45 feet high (above the existing mudline) and would maintain a water-level differential (difference between reservoir pool and tailwater) of up to 40 feet. Perimeter dikes around the east and west downstream shorelines and a south-Sea dam would supplement water circulation into and out of the upstream saltwater lake. In addition, 12,000 acres of shallow habitat ponds would be constructed in the eastern most portion of the existing Seabed adjacent to the brine pool. This alternative is

identified as "Alternative 4" under Sub-section 1.2.7 of the SOW and is the preferred alternative of the Salton Sea Authority.

2) Mid-Sea Barrier/South Marine Lake – This alternative (Alternative No. 2) would include an approximately 7-mile long mid-Sea barrier with a SCB slurry wall and abutment outlet works to provide for water balance and salinity control. The mid-Sea barrier would be up to 21 feet high and would maintain a water-level differential of up to about 5 feet. A total of 21,700 acres of shallow habitat ponds would be constructed in the eastern-most portion of the existing seabed adjacent to the marine lake and in the northern most portion of the Sea near the entrance of the Whitewater River. The mid-Sea barrier, together with the habitat ponds, is collectively called Alternative 7 in Sub-section 1.2.7 of the SOW.

3) Concentric Lakes Dikes – This alternative (Alternative No. 3) would include a series of 4 concentric lakes and dikes at different elevations around the existing perimeter to create four cascading pools within the existing Sea. These impervious embankments would be up to 20 feet high and would be designed to retain water and provide salinity control. Embankment configurations that meet Reclamation criteria for static loading conditions only are being evaluated.

4) North Sea Dam/Marine Lake – This alternative (Alternative No. 4) would include an impervious dam along approximate existing mudline contour on El –260 feet (MSL) at the northern-most end of the existing Sea. This concept would create a northern saltwater lake and a downstream hypersaline lake. The north-Sea dam cross-sections would be similar to the mid-Sea dam cross-sections. This alternative includes 37,200 acres of shallow habitat ponds located around the perimeter of the southern-most portion of the existing Sea.

5) Habitat Enhancement Without Marine Lake – This alternative (Alternative No. 5) would include 42,200 acres of shallow habitat ponds in the southern-most portion of the existing (same as the north-Sea dam alternative), and in the northern-most portion of the existing Sea.

All the alternatives above would need to include appropriate water conveyance and management infrastructure to provide for operations meeting the overall project objectives that have been established.

With regard to the shallow habitat ponds, they would be created by constructing embankments up to 9 feet high above the existing mudline with compacted earthfill.

Foundation improvement, such as excavation and removal of existing "soft" soils, jet grouting, and stone column installations, would be contemplated for various embankment alternatives and is described in Chapter 4.0.

1.2 Purpose and Scope

The purpose of Tasks 4 and 5 was to develop and analyze seepage and stability models for the various embankment configuration options, to assist with the selection of a preferred configuration, and to assist with the optimization of the preferred configuration. The scope of work completed as part of this study is summarized below.

- Review reports of previous geotechnical investigations and analyses prepared by others
- Develop engineering parameters for use in seepage and stability models of the embankment alternatives
- Develop and analyze design options for a mid-Sea dam, perimeter dikes, a mid-Sea barrier, and habitat pond embankments.
- Identify alternative configurations that meet seepage and stability criteria of Reclamation
- Present the results of the analyses in this report

This interim report addresses embankment performance issues related to seepage and stability for a planning-level evaluation. Performance issues not addressed in this interim report, but that should be addressed as part of future phases of project study including final design, include the following:

- Settlement analyses
- Freeboard
- Erosion
- Operation and maintenance
- Risk analyses
- Efficacy of proposed ground improvement methods (e.g., jet grouting and stone columns)

Detailed discussions of site geology and seismicity are outside the scope of this report. Where appropriate, discussions related to site geology, seismicity, and engineering data measurement and acquisition are referenced to reports prepared by others (see Chapter 2.0). Detailed discussions of geologic hazards and design criteria are also outside the scope of this interim report.

1.3 Project Personnel

The following personnel from Kleinfelder performed the work described in this report:

Project Manager:	Keith A. Ferguson, P.E.
Principal Investigator/Team Leader:	Scott Shewbridge, PhD, P.E., P.G.
Project Engineer:	Elena Sossenkina, E.I.T.
Staff Engineers:	Jie Yu, P.E.
	Mark Furman, P.E.
	Jorge Meneses, PhD, P.E.

Richard Wiltshire and Paul Weghorst of Reclamation directed, coordinated and reviewed the work for this project. Perry Hensley and Robert Dewey provided technical support and input. The input from, and support of Reclamation is gratefully acknowledged.

2.0 Previous Studies and Reports

Reclamation provided several reports and technical memoranda associated with the Salton Sea restoration project. Principal sources of background information and geotechnical data used in this study are listed chronologically and are described below. These documents are referenced in Chapter 7.0, and some are cited periodically in this report.

1. *URS Corporation (2004a), "Preliminary In-Sea Geotechnical Investigation, Salton Sea Restoration Project, Riverside and Imperial Counties, California." Report to Tetra Tech, Inc., URS Project No. 27663042, dated February 27, 2004*

   The above-referenced 2004 URS report presents the results of a preliminary-phase in-Sea geotechnical investigation conducted in 2003. A total of 11 borings and 17 cone penetration tests (CPTs) were completed for this study to depths ranging from about 30 feet to 150 feet below the mudline. Many of the explorations conducted for this study were focused on a mid-Sea transect from one shore to the other. Additional explorations were conducted at other locations throughout the Sea.

   Laboratory testing completed as part of this study included classification and index testing, pinhole dispersion, unconsolidated-undrained (UU) triaxial compression, isotropically-consolidated undrained (ICU) triaxial compression, and one-dimensional incremental consolidation tests. As discussed later in Sub-section 6.3 of this report, the quantity of consolidation and strength testing in the foundation soils is considered insufficient for refinement of a complete feasibility study or final design. URS will be completing a supplementary geotechnical study of the site in the near future to gather additional field and laboratory data to help refine the geotechnical site characterization.

2. *URS Corporation (2004b), "Conceptual Design Memorandum – Mid-Sea Dam and Barrier Concepts, Salton Sea Study, Riverside and Imperial Counties, California." Report to Tetra Tech, Inc., dated September 21, 2004*

   The 2004 URS conceptual design memorandum report for the mid-Sea dam and mid-Sea barrier concepts includes the results of preliminary engineering analyses and preliminary cost estimates for several alternative mid-Sea dam and barrier concepts.

3. *California Department of Water Resources (2005), "Salton Sea Ecosystem Restoration Study, Conceptual Design for In-sea Rock Barriers, Interim Report, Working Draft No. 3." State of California, The Resources Agency, Department of Water Resources, March 2005*

   The 2005 California Department of Water Resources (DWR) interim report presents the results of DWR's analyses and conceptual design approach for In-Sea Rockfill Barriers. DWR based their analyses on the field and laboratory data provided in the preliminary-phase geotechnical study by URS (URS, 2004a).

4. *U.S. Bureau of Reclamation, "Fiscal Year 2005 Appraisal Level Study Results"*

   Studies were conducted including assessments of available geotechnical data, application of risk evaluation criteria to various restoration alternatives and considerations of conceptual design features for a mid-Sea dam, a mid-Sea barrier, ring dikes, and habitat pond embankments.

In addition to the four references cited above, project information and direction were obtained during meetings and informal discussions with Reclamation, including conceptual sketches, presentation handouts, and environmental analysis reports.

3.0 General Site Conditions

This chapter includes brief descriptions of the geologic and seismic setting of the project site. The following paragraphs borrow heavily from the previous reports cited in Chapter 2.0. Detailed discussions of the site geology and seismicity are outside the scope of this report.

3.1 Location

The Salton Sea (usually called either the "Sea" or the "lake" in this report) is the largest inland water body in California and spans portions of Riverside and Imperial Counties. The Salton Sea is a terminal hypersaline lake with a current salinity of about 48,000 mg/L (Reclamation, 2005b). The Sea 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 Sea 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).

3.2 Geologic and Seismic Setting

The geologic and seismic setting of the Salton Trough is described in detail by URS (2004a). The following paragraphs are based principally on this previous report.

Briefly, 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. According to URS (2004a), sediments in the trough may be more than 18,000 feet deep. The most recent deposits within the Salton Sea include lacustrine, deltaic, and fluvial deposition associated with periodic inundation by the Colorado River. The Salton Sea occupies two local depressions of the main trough, called the northern basin and the southern basin.

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. Detailed descriptions of the regional faulting and historical seismicity are provided by URS (2004a).

3.3 Stratigraphy

Detailed discussions of the stratigraphic profile below the seafloor are provided by URS (2004a). In their preliminary-phase geotechnical study of the site, URS drilled 11 soil borings and conducted 17 CPTs. A plan of the URS explorations and interpreted stratigraphic cross-sections from URS (2004a) are included in this report in Attachment A. The URS report identifies the following principal strata, their general descriptions and thicknesses:

Table 2B.1
Salton Sea Generalized Stratigraphy

Stratum	Description	Thickness
Seafloor Deposits	Generally highly-plastic, very soft clay and loose, silty fine sands. These deposits generally are unweathered.	Ranges from about 0 to 21 feet below the mudline. Thickest near the eastern part of the mid-Sea alignment and near the central portion of the southern basin of the sea.
Soft Lacustrine Deposits	Mostly oxidized, highly plastic clays. Predominantly soft to very soft. Limited consolidation test data reported by URS suggest this stratum is generally normally consolidated, with no clearly distinguishable, desiccated "crust" layers. Based on limited information available to date, portions of this stratum are assumed to be liquefiable.	Ranges from 0 to 26 feet. Thickest in the eastern part of the mid-Sea alignment and at the north end of the lake.
Upper Alluvium	Typically silty fine sand with interbedded silt and sand layers. Density ranges from loose to dense. Locally cemented. Based on information available to date, this stratum is considered to be potentially liquefiable.	Usually encountered along the west end of the mid-Sea alignment and at the west perimeter. Thickness ranges from about 0 to 26 feet.
Upper Stiff Lacustrine Deposits	Typically highly plastic, mostly stiff to very stiff clay, although locally firm. Based on limited consolidation test data, this stratum appears to be normally consolidated.	Encountered generally throughout the sea. Thickness ranges from about 4 to 31 feet.

Most of the URS preliminary-phase explorations terminated within or just below the upper stiff lacustrine deposits above roughly El -335 feet. Below the upper stiff lacustrine deposits, a few of the URS explorations encountered additional strata identified by URS as lower alluvial deposits (medium dense to very dense silty fine sand) and lower stiff lacustrine deposits (very stiff to hard plastic clay with sand layers). Note: the words “alluvial” and “alluvium” are used interchangeably in this report and refer to the same deposit.

3.4 Variations in Subsurface Conditions

The URS stratigraphic interpretations contained in URS (2004a) and reproduced in part above and in Attachment A of this report are based on widely spaced (about 1-mile spacing along the mid-Sea alignment) explorations with limited laboratory test data. The conclusions and engineering analyses that follow are based on these limited data. The subsurface conditions between exploration locations are expected to vary from the generalized profiles illustrated in Attachment A and described above. The stratigraphic interpretations described above will likely require refinement after more field exploration and laboratory test data become available.

4.0 Embankment Configurations

This report chapter describes the embankment configurations specified in the SOW and other configurations that Reclamation asked Kleinfelder to consider during the course of this study. The analyses for many embankment configurations include scenarios for different assumptions regarding occurrence of liquefaction and ground improvement, as noted below in Table 2B.2. Table 2B.2 also includes references to the illustrations showing the embankment configurations that were analyzed. The analyses described in this report were based on the basic embankment configurations presented in the Reclamation report (2005b). Dimensions such as assumed marine lake or pond elevations, or embankment crest elevations may change as a result of ongoing evaluations by Reclamation.

Table 2B.2
Summary of Embankment Options and Analyses

SOW Designation	Project Component	Embankment Features	Foundation Improvement of Soft Lacustrine / Alluvial Deposits	Liquefaction or No Liquefaction	Figure
4,01	Mid-Sea Dam	Rockfill	Jet grouting at upstream and downstream toes	Liquefaction	Figure B.2
4,02	Mid-Sea Dam	Rockfill	Jet grouting at centerline	Liquefaction	Figure B.2
4,03	Mid-Sea Dam	Rockfill with "notches"	None	Liquefaction	Figure B.3
4,04	Mid-Sea Dam	Sand dam with stone columns and sacrificial shells	Stone Columns	Liquefaction	Figure B.3
4,05	Mid-Sea Dam	Rockfill	None	Liquefaction	Figure B.4

Table 2B.2 (continued)

SOW Designation	Project Component	Embankment Features	Foundation Improvement of Soft Lacustrine / Alluvial Deposits	Liquefaction or No Liquefaction	Figure
4,06	Mid-Sea Dam	Rockfill with "flat" (15H:1V) side slopes	None	Liquefaction	Figure B.4
4,07	Mid-Sea Dam	Rockfill with "modified notches"	None	Liquefaction	Figure B.5
4,08	Perimeter Dike - West	Rockfill	Jet grouting at upstream and downstream toes	Liquefaction	Figure B.9
4,09	Perimeter Dike - East	Rockfill	Jet grouting at upstream and downstream toes	Liquefaction	Figure B.9
4,10	Mid-Sea Dam	Rockfill	None	No Liquefaction	Figure B.5
4,11	Mid-Sea Dam	Rockfill with "notches"	None	No Liquefaction	Figure B.6
4,12	Mid-Sea Dam	Sand dam with stone columns and sacrificial shells	Stone Columns	No Liquefaction	Figure B.6
4,13	Mid-Sea Dam	Rockfill with "Deep Notches"	None	No Liquefaction	Figure B.7
4,14	Perimeter Dike - West	Rockfill	Jet grouting at upstream and downstream toes	No Liquefaction	Figure B.10

Table 2B.2 (continued)

SOW Designation	Project Component	Embankment Features	Foundation Improvement of Soft Lacustrine / Alluvial Deposits	Liquefaction or No Liquefaction	Figure
4,15	Perimeter Dike - East	Rockfill	Jet grouting at upstream and downstream toes	No Liquefaction	Figure B.10
5,01	Mid-Sea Barrier	Phase 1 Cross-Section	None	No Liquefaction	Figure B.12
5,02	*Not Evaluated*	--	--	--	--
5,03	Habitat Pond Embankments	Earthfill with Piezometric Control	None	No Liquefaction	Figure B.14

General features of the embankment configurations that were considered in these analyses, as presented in the SOW and during meetings with Reclamation, are described below.

4.1 Mid-Sea Dam

The mid-Sea dam is an approximately 9-mile-long impervious embankment that would divide the existing Sea to create an upstream saltwater lake and a downstream brine pool. The mid-Sea dam would be up to 45 feet high (above the existing mudline) and would maintain a water-level differential (difference between reservoir pool and tailwater) of up to 40 feet. The schematic "baseline" mid-Sea dam configuration, as provided by Reclamation, is illustrated on Figure B. 1. For these analyses, a "maximum" dam section was selected near the middle of the mid-Sea alignment, illustrated by Section A-A' in Attachment A. As shown on Figure B.1, the following "baseline" embankment cross-section characteristics, as described in the SOW and as interpreted from the available subsurface information reported by URS (2004a) were considered.

- Zoned rockfill embankment to control water elevations
- 30-foot wide crest with 5-horizontal on 1-vertical (5H:1V) upstream and 7H:1V downstream slopes
- Over-water fill placement

- Central core incorporates a soil-cement-bentonite (SCB) slurry wall cutoff SCB wall extends 30 feet into foundation (5 feet into upper stiff lacustrine deposits)
- Multi-stage filter blanket downstream of core
- Pre-dredge mudline at about El-268 feet
- Dam crest at El-223 feet
- Maximum water elevation differential of 40 feet; Upstream pool at El-228 feet, and downstream pool at El-268 feet

Table 2B.2 above identifies the following variations of mid-Sea dam concepts (e.g., Cases 4,01 through 4,07 and 4,10 through 4,13), as illustrated on Figures B.2 through B.7. These mid-Sea dam alternatives incorporate various combinations of jet grouting, stone column inclusions and, for several cases, no ground improvement.

- Rockfill with jet grouting under portions of the embankment (e.g., Cases 4,01 and 4,02)
- In lieu of ground improvement or notched keyways, use of flat side slopes (e.g., Case 4,06)
- In lieu of ground improvement, use of rockfill keyways or "notches" into the foundation soils under the embankment toes (e.g., Cases 4,03; 4,07; 4,11 and 4,13)
- Homogeneous sand embankment with stone column reinforcement through a central sand zone in the embankment. This central zone would be constructed to the base of the soft lacustrine foundation soils by dredging the Seafloor and soft lacustrine materials (e.g., Cases 4,04 and 4,12). For these cases, sacrificial "shells" of sand and gravel material would be placed on the upstream and downstream faces of the stone-column-improved sand embankment.

Before construction of the mid-Sea dam, it was assumed that all Seafloor Deposits from beneath the dam footprint would be dredged, as described in the SOW. The assumed post-dredge elevation for the mid-Sea dam section analyzed is about El.-280 feet.

Material properties for the mid-Sea dam analyses are described in Sub-section 6.4.

4.2 Perimeter Dikes

Perimeter dikes would be used in conjunction with a mid-Sea dam for the mid-Sea dam/north marine lake alternative. Note that for purposes of these analyses, Reclamation has deemed the analyses of the embankment for the perimeter dikes to be generally representative of the embankments required for the concentric lakes dikes alternative.

The schematic perimeter dike / ring dike section, as provided by Reclamation, is illustrated on Figure B.8. For these analyses, embankment sections with the following "baseline" characteristics were considered, as described in the SOW and as interpreted from the available subsurface information reported by URS (2004a).

- Rockfill embankments provide water elevation control
- 20-foot wide crest with 4H:1V upstream and downstream side slopes
- Underwater fill placement
- Central core incorporates a vinyl sheetpile cutoff wall. Cutoff wall extends 20 feet into foundation
- Pre-dredge mudline at about El-260 feet
- Embankment crest at El-240 feet
- Maximum water elevation differential of about 15 feet. Upstream pool at El-240.5 feet (1/2 foot freeboard), and downstream pool at El-255 feet

As noted in Table 2B.2, ground improvement for all perimeter dike scenarios evaluated include jet grouting at the upstream and downstream toes, as illustrated on Figures B.9 and B.10.

Before construction of perimeter dikes, it was assumed that all Seafloor Deposits from beneath the embankment footprint would be dredged. For the west side, the assumed post-dredge elevation was about El-265 feet. For the east side, the assumed post-dredge elevation was about El-270 feet.

Along the west side, the perimeter dikes were assumed to be founded on Upper Alluvium, which was assumed to extend from about El-265 feet (the west-side dredge line) to El-275 feet. Along the east side, the perimeter dikes were assumed to be founded on upper soft lacustrine deposits, which extends from about El-270 feet (the east-side dredge line) to El-280 feet.

Material properties for the perimeter dike analyses are described in Sub-section 6.4.

4.3 Mid-Sea Barrier

The mid-Sea barrier is an approximately 8-mile-long, semi-pervious embankment to provide salinity control. The mid-Sea barrier would be up to about 21 feet high above the existing mudline and would support a water-level differential of up to about 5 feet. The schematic mid-Sea barrier configuration, as provided by Reclamation, is illustrated on Figure B.11. For these analyses, a "maximum" barrier section near the middle of the mid-Sea alignment was selected, illustrated by Section A-A' in Attachment A. The model geometry used in the mid-Sea barrier analyses is illustrated on Figure B.12. The following "baseline" barrier cross-section characteristics were considered, as described in the SOW and as interpreted from the available subsurface information reported by URS (2004a).

- Zoned rockfill embankment to control salinity
- 30-foot wide crest with 5H:1V upstream and downstream side slopes
- Underwater fill placement
- Semi-pervious core allows seepage through the embankment
- Pre-dredge mudline at about El-268 feet
- Barrier crest at El-247 feet
- Maximum water elevation differential of 5 feet; Upstream pool at El-252 feet, and downstream pool at El-257 feet

Before construction of the mid-Sea barrier, it was assumed that all Seafloor deposits from beneath the barrier footprint would be dredged. The assumed post-dredge elevation for the section that was analyzed is about El-280 feet. At the analysis section, the mid-Sea barrier would then be built on untreated soft lacustrine deposits, which extend to about El-305 feet.

Note that contrary to what is shown on Figure B.11, ground improvement schemes were not considered for the current set of analyses.

Material properties for the mid-Sea barrier analyses are described in Sub-section 6.4.

4.4 Habitat Pond Embankments

For various alternatives, habitat ponds would be created at various locations around the perimeter of the lakebed by constructing a network of containment dikes up to 9-feet high above the existing mudline. The schematic habitat pond embankment dike section, as provided by Reclamation, is illustrated on Figure B.13. A "maximum" embankment dike section near the northern extent of the proposed pond locations was selected for analysis, in the southern portion of the existing Sea. The following "baseline" embankment dike cross-section characteristics were considered, as described in the SOW and as interpreted from the available subsurface information reported by URS (2004a).

- Compacted homogeneous earthfill (primarily clay and silt) embankment, placed and compacted "in the dry"
- 15-foot wide crest with 3H:1V side slopes
- Excavate 10 feet of existing foundation soils (soft lacustrine)
- Pre-dredge mudline at about El-250 feet
- Dike crest at El-241.5 feet
- Upstream pool at El-241.0 feet (6 inches freeboard), and downstream pool at El-250 feet; Maximum water elevation differential of roughly 9 feet

The baseline habitat pond embankment is represented by Case 5,03 and is illustrated on Figure B.14. As described in the SOW, analyses were conducted assuming that all Seafloor deposit/soft lacustrine soils will be removed from the embankment footprint to a depth of at least 10 feet below the existing mudline prior to placement of new embankment fill. At the section analyzed, the post-dredge elevation is about El-260, which corresponds roughly to the top of a 5-foot thick upper alluvium layer, which is underlain by 5 feet of stiff lacustrine deposits and then by additional alluvium. Material properties for the compacted earthfill embankment analyses are described in Sub-section 6.4.

5.0 Seepage Analyses

The following paragraphs describe the methods and soil parameters used in seepage analyses of the various embankment alternatives listed in Table 2B.2 (see Chapter 4.0), followed by a discussion of the seepage analysis results.

5.1 Seepage Analysis Method

Seepage analyses were conducted using steady-state analysis procedures of the finite element program Seep/W version 6.17. This software was developed by Geo-Slope International, Ltd. and can analyze two-dimensional planar or axi-symmetrical problems with isoparametric and higher-order finite elements. The program is able to work with multiple soil types having anisotropic hydraulic conductivity characteristics. Boundary conditions in steady-state analyses can be modeled as constant head, no-flow, constant flow, or variable based on head condition. Infinite elements can also be included in the profile to model an infinite half-space at the edge of the model.

Fixed-head boundary conditions set to the water surface elevations described in Chapter 4.0 were used along the vertical edges of the models, the boundary nodes of the Sea bottom and submerged embankment slopes. The nodes along the bottom of the model were modeled with a no-flow boundary condition. The elements between the upstream and downstream water surfaces are modeled as a potential seepage surface. These nodes are assigned a total flux boundary condition that is automatically adjusted by the computer program to a constant head boundary based on the iterative results of successive finite element runs. After each successive iteration, the calculated pressure head at each node is compared to the elevation head. If the pressure head is positive at the node, the node becomes a constant head node with head equal to the ground surface elevation, thus, allowing groundwater to seep from the surface.

5.2 Soil Parameters Used in Seepage Analyses

Permeability (sometimes referred to as hydraulic conductivity) values for the various soils in the analysis cross-sections were selected using published empirical relationships between the soil type and permeability such as those presented by Terzaghi and Peck (1967), Freeze and Cherry (1979), and Cedergren (1967). Correlation relationships based on grain size distribution as described in EM-1110-2-1913 (USACE, 2000) and in NAVFAC DM-7.01 (NAVFAC, 1986) were also utilized. Adjustments were made to the permeability of the sandy materials based on the percentage of fines (material passing No. 200 sieve) utilizing the Kozeny-Carman equation (Carrier, 2003). Values from the ranges provided by those referenced above were assigned to the various zones as shown in Table 2B.3 below. Seepage analyses were performed using a soil anisotropy ratio (Kv/Kh) of 0.1 or 0.25 for all naturally deposited layers and 0.25 or 1.0 for all engineered/fill layers.

Table 2B.3
Assumed Permeability Values

Material	Kh cm/sec	Kh ft/sec	Kv/Kh	Resulting Kv ft/sec	Color
Rockfill	1.0E+00	3.28E-02	0.25	8.2E-03	
Fine Rockfill	1.0E+00	3.28E-02	0.25	8.2E-03	
Filter Blanket	5.4E-03	1.77E-04	1	1.8E-04	
Sand Gravel Core	1.0E-02	3.28E-04	0.25	8.2E-05	
Alluvium	4.0E-04	1.32E-05	0.25	3.3E-06	
Habitat Pond Embankment Fill	1.0E-04	3.28E-06	1	3.3E-06	
Seafloor Deposits	1.0E-05	3.28E-07	0.1	3.3E-08	
Soft Lacustrine	1.0E-05	3.28E-07	0.1	3.3E-08	
Stiff Lacustrine	1.0E-05	3.28E-07	0.1	3.3E-08	
Jet-grouted Lacustrine	1.5E-06	4.92E-08	1	4.9E-08	
SCB Slurry Wall	1.0E-06	3.28E-08	1	3.3E-08	
Membrane SCB Wall	Impervious				

5.3 Interpretation of Seepage Results

Illustrated results of the seepage analyses are provided in Appendix B and are summarized in Table 2B.4 below.

Table 2B.4
Summary of Seepage Results

SOW Designation	Figure No.	Maximum Seepage Gradient	Calculated Seepage (cfs/lineal foot)
4,01	B.B-1	7.6	1.39×10^{-5}
4,02	B.B-3	7.8	1.38×10^{-5}
4,03	B.B-5	8.0	1.48×10^{-5}
4,04	B.B-7	0.7	2.05×10^{-6}
4,05	B.B- 9	7.6	1.38×10^{-5}

SOW Designation	Figure No.	Maximum Seepage Gradient	Calculated Seepage (cfs/lineal foot)
4,06	B.B-11	7.6	1.39×10^{-5}
4,07	B.B-13	8.0	1.10×10^{-5}
4,08	B.B-15	0.9	1.33×10^{-6}
4,09	B.B-17	0.8	1.03×10^{-6}
4,10	B.B-19	7.6	1.39×10^{-5}
4,11	B.B-21	8.0	1.48×10^{-5}
4,12	B.B-23	0.7	2.05×10^{-6}
4,13	*Not Evaluated*	*Not Evaluated*	*Not Evaluated*
4,14	B.B-26	0.9	1.33×10^{-6}
4,15	B.B-28	0.8	1.03×10^{-6}
5,01	B.B-30	0.2	1.45×10^{-3}
5,02	*Not Evaluated*	--	--
5,03	B.B-32	0.3	8.54×10^{-6}

The seepage analysis figures in Attachment B each contain three figures: 1) an overall view of the section geometry, showing material properties and the boundary conditions; 2) seepage gradient contours, and; 3) total head contours. Keys to the color-coding used to represent material properties, the gradient, and head contours are shown on the Attachment B illustrations. In general, warmer colors (i.e., red) represent contours of greatest permeability, seepage gradient, and total head.

Review of the seepage analysis illustrations in Attachment B shows that for all of the rockfill mid-Sea dam alternatives evaluated, 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. On the other hand, the presence and integrity of the SCB wall, plays a major role, as will be discussed below. Figure B.15 illustrates the computed seepage gradient contours and flow vectors for Case 4,02. As can be seen on this figure, the maximum seepage gradient (i_{xy}) is 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 slurry 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, the seepage for Case 4,02 was evaluated with a 5-foot-high "defect", or hole, in the SCB slurry wall, as shown on Figure B.16. This figure shows that 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 SCB wall or twice as far as in the case of an intact SCB wall. Thus, the integrity of the SCB wall is essential for control of the dam performance with respect to piping resistance.

Computed seepage gradients for the stone-column-reinforced sand embankment dam with a soil-cement-bentonite (SCB) slurry wall with membrane, extending 60 feet into lacustrine deposits below the embankment/foundation interface are generally less than 0.3 except in the immediate vicinity of the bottom of the SCB wall. As illustrated on Figure B.17, a maximum gradient equal to 0.7 occurs at this location. Incorporation of an impervious membrane reduces the computed gradients by as much as ten times compared to those reported for other mid-Sea dam alternatives with only a SCB slurry wall cutoff.

6.0 Slope Stability Analyses

The following paragraphs describe the methods and soil parameters used in slope stability analyses of the various embankment alternatives listed in Table 2B.2 (see Chapter 4.0).

6.1 Stability Analysis Method

Slope stability analyses were conducted using the computer program Slope/W version 6.17, developed by Geo-Slope International, Ltd. This program was used to perform automatic searches of different potential failure surfaces and to compute the lowest safety factor corresponding to a critical failure surface for a particular analysis condition. The factor of safety is defined as the ratio of the available resisting forces to the driving forces. Yield acceleration is defined as the maximum horizontal acceleration required to result in a "just-stable" equilibrium condition equivalent to a factor of safety of 1. In most cases, both downstream and upstream slopes of an embankment condition were evaluated using both circular and wedge-shaped failure surfaces. For a given embankment configuration and loading scenario, the program Slope/W is able to analyze several thousand potential slip surfaces.

Slope stability analyses were conducted to evaluate the global stability of the embankment alternatives noted in Table 2B.2 (see Chapter 4.0). Parameter input into the slope stability models included the embankment geometry and the approximate unit weight and shear strength properties of the native and embankment fill soils. General information on the embankment geometry and water surface assumptions is discussed in Chapter 4.0 of this Appendix.

Failure surfaces were analyzed using Spencer’s method. Spencer’s method is a two-dimensional limit-equilibrium method that satisfies force equilibrium of slices and overall moment equilibrium of the potential sliding mass. The inclination of side forces between vertical slices is assumed to be the same for all slices and is calculated along with the factor of safety. This method utilizes the embankment slope configuration, unit weight and shear strength properties of embankment and foundation materials, and boundary and internal distribution of forces due to water pressures. After a potential failure surface has been assumed, the soil mass located above the failure surface is divided into a series of vertical slices. Forces acting on each slice include the slice weight, the pore pressure, the effective normal force on the base, the mobilized shear force (including both cohesion and friction), and the horizontal side forces due to earth pressures.

Searches for critical circular failure surfaces were performed by two different methods:

1) In some cases tangent lines of circular arcs and a grid of points representing the circle centers were specified. To help locate the failure surfaces with the lowest computed safety factors, the location and density of the circle center grid were adjusted along with the locations of the circle tangent lines.
2) In other cases, zones along the embankment face to indicate "entry" and "exit" limits for circular arcs were specified. To help locate the failure surfaces with the lowest computed safety factors, the location and width of the "entry" and "exit" limits along the slope face were adjusted.

Searches for wedge-type failure surfaces were conducted by specifying two "boxes" along the failure surface through which potential failure surfaces must pass. To help locate the failure surfaces with the lowest computed safety factors using the wedge analyses, the box positions and the density of points within the boxes were appropriately adjusted.

Further discussion of how the software was used to find the critical slip surfaces is provided below in Sub-section 6.5.1.

6.2 Analysis Cases

In general, the embankment alternatives were analyzed for two slope stability conditions:

1. End of Construction (EOC) - Post-seismic soil strengths and, where specified in the SOW, liquefied "residual" soil strengths were used for appropriate portions of the embankment and foundation. EOC strengths were used for the soft lacustrine and seafloor foundation soils (where they are not dredged) assuming these soils have not consolidated as a result of the dissipation of construction-induced pore water pressures.
2. End of Primary Consolidation (EPC) - Post-seismic soil strengths and, where specified in the SOW, liquefied "residual" soil strengths were used for appropriate portions of the embankment and foundation. EPC strengths were used for the soft lacustrine and seafloor foundation soils (where they are not dredged) assuming these soils have consolidated such that the excess pore pressures induced during construction have dissipated.

Each embankment configuration was evaluated to estimate a post-seismic static factor of safety under both EOC and EPC conditions. In addition the horizontal yield acceleration (horizontal seismic coefficient) corresponding to the acceleration required to reduce the computed safety factor to about 1.0 was also estimated.

Additional discussions of how the Slope/W software was used to help evaluate the factor of safety and yield acceleration are provided in Sub-section 6.5.

6.3 Shear Strengths Used in Stability Analyses

The following paragraphs describe how shear strength properties were selected for use in slope stability models, including normalized strengths used in the relatively weak Seafloor and soft lacustrine deposits, shear strength parameters for the other materials, post-liquefaction residual strengths, and strength anisotropy in cohesive soils.

6.3.1 Normalized Shear Strengths of Seafloor and Soft Lacustrine Deposits

For static, EOC and EPC analyses, a normalized strength approach was used to evaluate undrained shear strengths for the relatively soft Seafloor and soft lacustrine deposits. The normalized strength approach is based on the Stress History and Normalized Soil Engineering Properties (SHANSEP) technique, as presented by Ladd and Foott (1974). The SHANSEP technique is based on the observation that the shear strength (s_u) of many soils (particularly soft, normally-consolidated soils) can be normalized with respect to vertical consolidation pressure (σ'_v). Discussions of the development of static EOC and EPC shear strengths for the Seafloor and soft lacustrine deposits are presented below in Sub-sections 6.3.1.1 and 6.3.1.2. Development of shear strength parameters for these soils under seismic and post-seismic conditions is discussed in Sub-section 6.3.1.3. A summary of all soil parameters used in stability analyses, including the "static" EOC and EPC strengths for all materials, is provided in Sub-section 6.4.

6.3.1.1 End of Construction Strengths

EOC strengths for unexcavated cohesive Seafloor and soft lacustrine deposits under an embankment are appropriate for stability evaluations during the period before dissipation of construction-induced pore water pressures. Under this condition, the strengths in these relatively weak soils were defined as linearly-increasing cohesion, starting at a value of nearly zero at the preconstruction mudline and increasing by the ratio (s_u/σ'_v) multiplied by depth below the mudline and the buoyant unit weight of the soil layer being evaluated. For relatively soft soils in environments similar to the Salton Sea, and in the absence of more definitive field and laboratory test data, a value of (s_u/σ'_v), the "normalized strength ratio", equal to 0.22 for the Seafloor and soft lacustrine deposits was selected. With the limited field and laboratory test data available, it was assumed that the value s_u/σ'_v of 0.22 is conservative enough to account for long-term strength reduction effects due to soil creep, as discussed by Duncan and Buchignani (1973).

6.3.1.2 End of Primary Consolidation Strengths

EPC strengths for the unexcavated Seafloor and soft lacustrine deposits under an embankment are appropriate for use in stability evaluations after the embankments and their foundation soils have consolidated such that the excess pore water pressures induced during construction have dissipated. EPC effective stresses were based on the s_u/σ'_v ratio described above. To simplify the computation of normalized strength, the Slope/W software was allowed to calculate the effective consolidation pressure and strength by setting the cohesion (c) equal to zero and the effective friction angle, σ', equal to 12.5 degrees. As in the EOC case described above, an effective friction angle of 12.5 degrees is believed to be conservative enough to account for the increased strength due to consolidation and an appropriate strength reduction due to creep.

6.3.1.3 Seismic Strengths

A review of the limited laboratory and field test data available to date indicates that most of the cohesive foundation soils are normally consolidated (with over consolidation ratios usually less than about 2). There is no pervasive evidence of distinct, over-consolidated "crust" layers within the soft lacustrine deposits. Therefore, both undrained EOC and EPC strengths were used for evaluation of the seismic loading condition (yield acceleration determination) and the “post-seismic” loading condition. Seismic and post-seismic strengths in the Seafloor and soft lacustrine deposits were based on the "static" EOC and EPC strengths with about a 30 percent increase to remove the "creep reduction" penalties mentioned above.

A summary of all soil parameters used in the stability analyses, including the "seismic" EOC and EPC strengths, is provided in Sub-section 6.4.

6.3.2 Shear Strength Parameters of Other Materials

For purposes of these analyses, it was assumed that similar drained, frictional shear strength is appropriate for the granular Upper Alluvium under EOC and EPC conditions. Similarly, it was assumed that the upper stiff lacustrine deposits will be sufficiently stiff to not be affected substantially by new embankment loads. Therefore, for both EOC and EPC conditions the upper stiff lacustrine deposits were modeled with the same frictional and cohesive shear strength components. In the absence of more extensive laboratory data, shear strength parameters have been selected for the upper alluvium and upper stiff lacustrine deposits based on the parameters used by Reclamation in their preliminary designs (Reclamation, 2005b). The strength parameters for both the upper alluvium and the upper stiff lacustrine deposits should be further investigated and, if necessary, refined for subsequent analyses.

Assumed shear strengths of the embankment materials and shear strength parameters for jet-grouted foundation soils are also based on Reclamation (2005b). Strength parameters for stone column-improved embankment materials are based on experience and judgment, assuming an "improved" equivalent clean sand SPT blow count of at least 20 would be achieved following installation of the stone columns.

A summary of all soil parameters used in these stability analyses is provided in Sub-section 6.4.

6.3.3 Post-Liquefaction Residual Strengths

Post-liquefaction stability will be controlled by post-liquefaction "residual" strengths of liquefied soils. For these analyses, soils considered to be potentially liquefiable include: 1) portions of the soft lacustrine deposits; 2) the Upper Alluvium; 3) the downstream filter blanket (applicable to mid-Sea dam alternative); and 4) all non- stone column-reinforced sand embankment material (applies to SOW Designation 4,04 – see Table 2B.2 in Chapter 4.0 of this appendix).

At present, the best "state-of-practice" estimates of post-liquefaction residual strengths are based on correlations between in-situ index tests (such as the Standard Penetration Test) and post-liquefaction strengths back calculated from field case histories (Seed et al., 2003). Engineering judgment and the relationships presented by Seed and Harder (1990) were used in the estimates of post-liquefaction residual strengths. A summary of all soil parameters used in these stability analyses, including the post-liquefaction residual strengths, is provided in Sub-section 6.4.

6.3.4 Strength Anisotropy

The undrained strength of clayey soils such as the Seafloor and soft and upper stiff lacustrine deposits tends to vary with the orientation of the failure plane. For analytical purposes, it is convenient to express strength anisotropy in terms of a ratio of the "horizontal"-oriented undrained shear strength to the "vertical" undrained shear strength, or ($s_{u,h}$ / $s_{u,v}$). To date there are insufficient data to evaluate precisely this anisotropy ratio for the cohesive soils at the site. For the current phase of analyses, an anisotropy function was adopted based on a ratio $s_{u,h}$ / $s_{u,v}$ equal to 0.9, which is supported by research presented by Duncan and Seed (1966).

6.4 Summary of Material Properties Used in Stability Analyses

A summary of all material properties used in the stability analyses is provided below in Table 2B.5.

Table 2B.5
Material Properties for Stability Analyses

Material	Total Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)	Anisotropy Function ($s_{u,h}$ / $s_{u,v}$)
SCB Slurry Wall	120	100	30	1
Vinyl Sheetpile Wall	120	100	30	1
Core – clean sand and gravel	120	0	30	1
Rockfill	115	0	45	1
Fine Rockfill	118	0	42	1
Filter Blanket	125	0	35	1
Filter Blanket – Liquefied	125	150	0	1
Compacted Clay/Silt Embankment – EOC/EPC	120	0	25	1
Homogeneous Sand Embankment with Stone Columns – Static/Seismic	120	0	38	1
Homogeneous Sand Embankment with Stone Columns – Liquefied	120	1000	0	1
Seafloor Deposit – EOC Static	98	"c" increase based on (s_u/σ'_v) = 0.22 from preconstruction dredge line	0	0.9
Seafloor Deposit – EOC Seismic	98	"c" increase based on (s_u/σ'_v) = 0.30 from preconstruction dredge line	0	0.9

Table 2B.5 (continued)

Material	Total Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)	Anisotropy Function ($s_{u,h}$ / $s_{u,v}$)
Seafloor Deposit – EPC Static	98	0	12.5	0.9
Seafloor Deposit – EPC Seismic	98	0	17	0.9
Untreated Soft Lacustrine – EOC Static	115	"c" increase based on (s_u/σ'_v) = 0.22 from preconstruction dredge line	0	0.9
Untreated Soft Lacustrine – EOC Seismic	115	"c" increase based on (s_u/σ'_v) = 0.30 from preconstruction dredge line	0	0.9
Untreated Soft Lacustrine – EPC Static	115	0	12.5	0.9
Untreated Soft Lacustrine – EPC Seismic	115	0	17	0.9
Untreated Soft Lacustrine – Liquefied	115	250	0	1
Jet-Grouted Soft Lacustrine	120	7,200	0	1
Stone Columns in Clean Sand Fill – Liquefied	120	1000	0	1
Upper Stiff Lacustrine	118	200	33	0.9
Untreated Upper Alluvium – Static/Seismic	128	0	32	1
Untreated Upper Alluvium – Liquefied	128	400	0	1
Jet-Grouted Upper Alluvium	130	9,360	0	1
Saltwater	64	--	--	--

6.5 Stability Analysis Results

The following sub-sections describe how slope stability factors of safety for "post-seismic", EOC and EPC loading conditions were estimated. In addition to computing the "static, and post-seismic" EOC and EPC safety factors, each section was evaluated to estimate the horizontal yield accelerations under EOC and EPC conditions.

Graphical summaries of the stability analysis results for the configurations and loading conditions identified on Table 2B.2 (see Chapter 4.0) are presented in Attachment B and generally are grouped to follow the corresponding seepage analysis illustrations. Note that for cases in which the embankment is founded on the Upper Alluvium or on "treated" soft lacustrine deposits, the EOC and EPC results are similar and presented as a single result for these cases.
In the presentation of stability analysis results, "untreated" foundation material colors were used that are similar to the colors used to represent the stratigraphy reported by URS (2004a; see Attachment A).

6.5.1 Safety Factor Computations

As described above in Sub-section 6.1, the computer program Slope/W was used to compute safety factors of thousands of potential slip surfaces for a given slope configuration and loading condition. For most specified embankment configurations and loading conditions, both circular and non-circular (wedge-shaped) slip surfaces were evaluated for both the upstream and downstream slopes. The search for critical slip failure surfaces includes those that involved the embankment crest or at least a substantial portion of a slope and foundation material. Initially the search for critical failure surfaces looked at a fairly wide range of failure surfaces generated from widely-spaced grids of circle centers and wedge surface boundaries. The range of computed safety factors was then narrowed by adjusting the applicable grid boundaries and resolutions to arrive at a final computed minimum safety factor.

The results for circular failure surfaces shown in Attachment B show the locus of circle centers that were evaluated and corresponding computed safety factor contours. Typically, several thousand potential failure surfaces were analyzed for each run. The stability analysis results illustrated in Attachment B show for each section the failure surfaces with the lowest ten computed safety factors. These "lowest 10" surfaces normally have computed safety factors within about a tenth of the lowest safety factor.

6.5.2 Horizontal Yield Acceleration Evaluation

Factors of safety for a given embankment configuration were estimated using “seismic” strength parameters, and a range of horizontal seismic coefficients (e.g., 0.00g, 0.05g, 0.10g, 0.15g, etc.) in order to estimate the yield acceleration of both the upstream and downstream slopes of the dam. In most cases both circular and wedge-shape critical failure surfaces were evaluated. The summary illustrations in Attachment B include a graph of computed safety factor versus horizontal seismic coefficient (expressed as a fraction of acceleration due to gravity, or "g"). The acceleration that corresponds to a computed (or interpolated) safety factor of about 1.0 defines the yield acceleration.

6.5.3 Summary and Discussion of Stability Analysis Results

The results of stability analyses under "post-seismic" static conditions and the computed horizontal yield accelerations under both EOC and EPC conditions are summarized in Table 2B.6 below, followed by discussion of the stability analysis results.

Table 2B.6
Summary of Post-Seismic Stability and Seepage Analysis Results

SOW Designation	Design Component	Embankment Features	Foundation Improvement of Soft Lacustrine / Alluvial Deposits	Liquefaction or No Liquefaction	Calculated Post-Seismic Safety Factor	Yield Acceleration (g)	Maximum Seepage Gradient	Seepage Flow (cfs/lineal foot)
4,01	Mid-Sea Dam	Rockfill	Jet grouting at upstream and downstream toes	Liquefaction	EOC = 2.8 EPC = 2.8	EOC = 0.17 EPC = 0.17	7.6	1.39×10^{-5}
4,02	Mid-Sea Dam	Rockfill	Jet grouting at centerline	Liquefaction	EOC = 0.87 EPC = 0.87	EOC = 0.00 EPC = 0.00	7.8	1.38×10^{-5}
4,03	Mid-Sea Dam	Rockfill with "notches"	None	Liquefaction	EOC = 2.3 EPC = 2.3	EOC = 0.17 EPC = 0.17	8.0	1.48×10^{-5}
4,04	Mid-Sea Dam	Sand with Stone Columns	Stone Columns	Liquefaction	EOC = 1.9 EPC = 1.9	EOC = 0.11 EOC = 0.11	0.7	2.05×10^{-6}
4,05	Mid-Sea Dam	Rockfill	None	Liquefaction	EOC = 1.2 EPC = 1.2	EOC = 0.01 EPC = 0.01	7.6	1.38×10^{-5}

Table 2B.6 (continued)

SOW Designation	Design Component	Embankment Features	Foundation Improvement of Soft Lacustrine / Alluvial Deposits	Liquefaction or No Liquefaction	Calculated Post-Seismic Safety Factor	Yield Acceleration (g)	Maximum Seepage Gradient	Seepage Flow (cfs/lineal foot)
4,06	Mid-Sea Dam	Rockfill with "flat" (15H:1V) side slopes	None	Liquefaction	EOC = 2.3 EPC = 2.3	EOC = 0.07 EPC = 0.07	7.6	1.39×10^{-5}
4,07	Mid-Sea Dam	Rockfill with "modified notches"	None	Liquefaction	EOC = 0.7 EPC = 0.7	EOC = 0.00 EPC = 0.00	8.0	1.10×10^{-5}
4,08	Perimeter Dike – West	Rockfill	Jet grouting at upstream and downstream toes	Liquefaction	EOC = 2.1 EPC = 2.1	EOC = 0.14 EPC = 0.14	0.9	1.33×10^{-6}
4,09	Perimeter Dike – East	Rockfill	Jet grouting at upstream and downstream toes	Liquefaction	EOC = 1.7 EPC = 1.7	EOC = 0.11 EPC = 0.11	0.8	1.03×10^{-6}
4,10	Mid-Sea Dam	Rockfill	None	No Liquefaction	EOC = 1.2 EPC = 2.0	EOC = 0.01 EPC = 0.08	7.6	1.39×10^{-5}
4,11	Mid-Sea Dam	Rockfill with "notches"	None	No Liquefaction	EOC = 2.3 EPC = 2.3	EOC = 0.17 EPC = 0.17	8.0	1.48×10^{-5}

Table 2B.6 (continued)

SOW Designation	Design Component	Embankment Features	Foundation Improvement of Soft Lacustrine / Alluvial Deposits	Liquefaction or No Liquefaction	Calculated Post-Seismic Safety Factor	Yield Acceleration (g)	Maximum Seepage Gradient	Seepage Flow (cfs/lineal foot)
4,12	Mid-Sea Dam	Sand with Stone Columns	Stone Columns	No Liquefaction	EOC = 2.2 EPC = 2.7	EOC = 0.16 EPC = 0.16	0.7	2.05×10^{-6}
4,13	Mid-Sea Dam	Rockfill with "Deep Notches"	None	No Liquefaction	EOC = 3.3 EPC = 3.3	EOC = 0.17 EPC = 0.19	*Not Evaluated*	*Not Evaluated*
4,14	Perimeter Dike – West	Rockfill	Jet grouting at upstream and downstream toes	No Liquefaction	EOC = 2.7 EPC = 2.7	EOC = 0.24 EPC = 0.24	0.9	1.33×10^{-6}
4,15	Perimeter Dike – East	Rockfill	Jet grouting at upstream and downstream toes	No Liquefaction	EOC = 1.7 EPC = 2.0	EOC = 0.08 EPC = 0.10	0.8	1.03×10^{-6}
5,01	Mid-Sea Barrier	Phase 1 Cross-Section	None	No Liquefaction	EOC = 1.5 EPC = 1.7	EOC = 0.04 EPC = 0.07	0.2	1.45×10^{-3}
5,02	*Not Evaluated*	--	--	--	--	--	--	--

Table 2B.6 (continued)

SOW Designation	Design Component	Embankment Features	Foundation Improvement of Soft Lacustrine / Alluvial Deposits	Liquefaction or No Liquefaction	Calculated Post-Seismic Safety Factor	Yield Acceleration (g)	Maximum Seepage Gradient	Seepage Flow (cfs/lineal foot)
5,03	Habitat Pond Embankments	Earthfill with Piezometric Control	None	No Liquefaction	EOC = 1.0 EPC = 1.0	EOC = 0.00 EPC = 0.00	0.3	8.54×10^{-6}

The following sub-sections describe the section geometries and discuss the stability analysis results for the project alternatives considered for this study.

6.5.3.1 Rockfill Mid-Sea dam Alternatives

The schematic "baseline" mid-Sea dam, as provided by Reclamation, is illustrated on Figure B.1, and all mid-Sea dam model geometries that were considered are described in Table 2B.2 and in Sub-section 4.1.

As shown on Table 2B.6, the computed "post-seismic" safety factors for the mid-Sea rockfill dam alternatives range from about 0.7 ("modified notches" Case 4,07 with a liquefied foundation) to about 3.3 (EPC condition for the "deep notches" Case 4,13; no liquefaction).

The computed yield accelerations for the rockfill dam alternatives range from zero (e.g., liquefaction conditions for jet grouting at rockfill dam centerline, Case 4,02 and the liquefied "modified notches" concept, Case 4,07) to 0.19g for the non-liquefied rockfill with "deep notches" Case 4,13.

6.5.3.2 Stone-Column-Reinforced Mid-Sea dam Alternative

The stone-column-reinforced mid-Sea dam embankment alternative is illustrated on Figure B.3 (Case 4,04 – no liquefaction) and Figure B.6 (Case 4,12 – with liquefaction). The upstream and downstream "shells" were modeled as surface pressures rather than as materials contributing to the embankment strength or forces that drive slope failure. As shown in Table 2B.6, the computed post-seismic safety factors for the stone-column-reinforced embankment range from about 1.9 (liquefaction condition) to about 2.7 (no liquefaction).

The corresponding yield accelerations range from about 0.11g (liquefaction) to 0.16g (no liquefaction). These results suggest that the "sand dam with stone columns" concepts, as evaluated here, provide mid-Sea dam configurations that are generally more stable than configurations with jet-grouted foundations, "notched" foundation keyways, and flattened side slopes.

To investigate the effect of the downstream "shell" deformation on the overall stability under non-liquefied conditions, two series of analyses were performed considering both "minimal" deformation of the downstream shell and "large-scale" shell deformation. As shown on the Yield Acceleration vs. Displacement graph on Figure B.B-24b (Attachment B), the computed yield acceleration is about 0.16g for no displacement of the shell (see discussion above and Figure B.B-24a). With "minimal" deformation of the shell (amounting to about 10 vertical feet; see upper figure on Figure B.B-24b), the yield acceleration increases to about 0.21g. With "large-scale" deformation of the shell (see lower figure on Figure B.B-24a), the yield acceleration drops to about 0.10g.
As shown in Table 2B.5, the shear strength of the sand embankment with stone columns was assumed to be 1,000 psf. A cohesion of 1,000 psf leads to a computed yield acceleration of about 0.10g for large-scale deformation of the

downstream shell (see lower illustration on Figure B.B-24b). As shown on the lower graph on Figure B.B-24b, the computed yield acceleration of the embankment with the large-scale shell deformation is sensitive to the assumed embankment strength. With an embankment shear strength of 800 psf, the computed yield acceleration drops to about 0.07g. With an embankment shear strength of 1,200 psf, the computed yield acceleration increases to about 0.13g.

6.5.3.3 Rockfill Ring Dike / Perimeter Dike Alternative

The rockfill ring dike / perimeter dike concept, as provided by Reclamation, is illustrated schematically on Figure B.8 of this report. The perimeter dike model geometries that were considered are illustrated on Figures B.9 and B.10, as described in Table 2B.2 and in Sub-section 4.2.

As shown in Table 2B.6, the west-side perimeter dikes, which are assumed to be founded largely on Upper Alluvium, have computed post-seismic safety factors of 2.1 (Case 4,08 – liquefaction condition with partially jet-grouted foundation) to 2.7 (Case 4,14 – partially jet-grouted foundation with no liquefaction). The east-side perimeter dikes, which are founded largely on soft lacustrine deposits, have post-seismic safety factors of 1.7 (Cases 4,09 and 4,15 – liquefaction and no liquefaction, respectively). Ranges of computed yield accelerations range from about 0.08 (no liquefaction, east side) to 0.24 (no liquefaction, west side).

6.5.3.4 Rockfill Mid-Sea barrier Alternative

The rockfill mid-Sea barrier concept, as provided by Reclamation, is illustrated schematically on Figure B.11. The mid-Sea barrier model geometry is illustrated on Figure B.12, as described on Table B.2 and in Sub-section 4.3.

As shown in Table B.6, the rockfill mid-Sea barrier (Case 5,01) has computed post-seismic safety factors of about 1.5 (EOC condition with no liquefaction) to about 1.7 (EPC, no liquefaction). Computed yield accelerations range from about 0.04g to 0.07g.

6.5.3.5 Habitat Pond Embankment Alternative

The habitat pond embankment concept, as provided by Reclamation, is illustrated schematically on Figure B.13. The habitat pond embankment model geometry is illustrated by the upper illustration on Figure B.14. Further explanation of the habitat pond embankment geometry is provided in Table 2B.2 and in Sub-section 4.4.

These analyses indicate that the homogeneous compacted earthfill embankment is marginally stable (see Table 2B.6, Case 5,03). The illustrated stability results on Figure B.B-33 show that for failure surfaces on the downstream face, the computed safety factor for static, post-seismic conditions is about 1.0, with a corresponding yield acceleration of 0.00g. For comparison, at the upstream face the static, post-seismic safety factor is about 1.7, with a corresponding yield acceleration of 0.09g.

To achieve a stable downstream slope, the habitat pond embankment must be zoned in some way to control seepage pressures in the downstream face, or the downstream slope must be flattened.

7.0 Assessment of Relative Constructability and Relative Cost

A qualitative cost and constructability analysis for each of the three mid-Sea dam options, the mid-Sea barrier, concentric lakes/perimeter dikes, and habitat pond embankments was performed to assist with the comparison of the options and selection of the preferred configuration. For purposes of this evaluation, it was assumed that the borrow source materials would be the same for any of the proposed options. The initial comparative assessment is described below. Following this initial assessment, Reclamation requested refinement of cost estimates used in the comparative evaluation. The comparison model was updated based on these costs. The updated comparative analyses and the corresponding decision by Reclamation on the preferred configuration option are described in the main project report.

<u>Discussion of Constructability Issues by Configuration Option:</u>

- ***Mid-Sea dam- "Rockfill Dam with Rock Notches":*** This option would be constructed using end dumping for the core materials and the inner section of the coarse rockfill. The outer rockfill will be placed using overwater techniques. Constructability and therefore cost is complicated by the need to place three separate soil types, the graded filter blanket, and the soil-cement-bentonite (SCB) slurry wall.

- ***Mid-Sea dam-"Sand Dam with Stone Columns ":*** This option would be constructed using a poorly graded sand fill placed by overwater construction and conveyor or end dumping from the shore. An outer, sacrificial shell would be placed using a combination of end dumping and overwater placement. The poorly graded sand would be densified using stone columns working both along the completed core and on adjacent barges. The constructability is enhanced by the need to place only two material types. Costs are impacted negatively because the poorly graded sand core material may be difficult to locate and part of the work will be done over water.

- ***Mid-Sea dam- "Rockfill Dam with Jet-grouted Foundation":*** 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 overwater construction methods. Jet grouting of the soft lacustrine/upper alluvial zones would be performed prior to dredging and placement of the embankment materials. Constructability is complex due to the number of zones of different materials and the need to closely control and montor the placement of materials. In addition, the jet grouting work will generate waste on the order of 60% of the volume of the improved soil that will require special handling and disposal.

- ***Mid-Sea barrier*:** This option would be constructed using two material types placed both by end dumping and over water techniques. Constructability is positively impacted by the simplified cross-section and limited number of material types. Cost is also positively impacted by the same characteristics.

- ***Concentric Ring/Perimeter Dikse*:** This option involves placing three material types, a vinyl sheetpile wall as well as jet grouting improvements in the soft lacustrine materials. The same complications apply to this section as the mid-Sea dam- rock notches alternative. The only positive impacts come from the fact that the Concentric Ring/Perimeter Dikes will be built in shallower water. However, over-water placement of rockfill and jet grouting will be necessary and this could be complicated if the water is too shallow, and barges cannot be used.

- ***Habitat Ponds*:** This option envisions constructing short, homogeneous dikes using onsite materials. To enhance constructability and equipment mobility, it may be best to excavate the sea floor materials by dredging in the wet so these materials can be distributed over a broader area. Dry native materials could then be placed in the resulting excavation and compacted. Constructability and cost is improved by the simplified cross-section and the uniform material requirements.

Relative rankings of each of the above alternatives are presented in the summary table in the following Chapter 8.0.

8.0 Comparison of Alternative Configuration Options

Based on the results of analyses described in this appendix, an initial "qualitative" ranking model was developed. The results of this ranking are summarized in Table 2B.7 below. Relative rankings for each criterion of the model range from scores of –2 to 2 (e.g., -2, -1, 0, 1, and 2), with higher values representing more desirable characteristics.

Table 2B.7
Relative Rankings of Expected Performance, Constructability and Costs

Alternative Configuration	Relative Expected Seepage Performance	Relative Expected Seismic Performance	Relative Expected Constructability	Relative Expected Cost of Construction
Mid-Sea Dam – Modified Rock Notches	1	2	0	0
Mid-Sea Dam – Sand Dam with Stone Columns	2	2	0	0
Mid-Sea Dam - Rockfill Dam with Jet-grouted Foundation	2	2	-2	-2
Mid-Sea Barrier	0 (due to seepage losses)	0	1	0
Concentric Lakes/Perimeter Dikes	0	0	-1	-1
Habitat Ponds Embankments	2 (with cutoff)	0 (with cutoff)	1	1

The evaluation criteria include relative expected seismic stability, relative expected seepage performance, relative expected constructability and relative expected cost of construction. Based on the results of this initial assessment, two of the mid-Sea dam configuration options were identified for further evaluation: the "Modified Rock Notches" (Case 4-3, Attachment B Figure B.B-6, Case 4-11, Attachment B Figure B.B-22,) and the "Sand Dam with Stone Columns" (Case 4-4, Attachment B Figure B.B-8, Case 4-12, Attachment B Figure B.B-24A, B.B-24b).

9.0 Limitations

This appendix presents the results of analyses and conclusions in support of a comparison-level study for embankment alternatives that are currently being considered for the Salton Sea restoration project. In developing the conclusions presented in this appendix, information gathered previously by others was used, as discussed in Chapter 2.0 of this appendix. In addition, there was regular interaction with Reclamation on the general approach to this study. Engineering experience and judgment were applied during the development of the study conclusions. The results of these evaluations should be reevaluated as additional field and laboratory test data become available.

These analyses have been conducted, and this appendix prepared in general accordance with geotechnical engineering practice, as it exists in the site vicinity at the time of this study. No warranty, expressed or implied, is made.
This appendix may be used only by the client for the purposes of a feasibility-level evaluation of the project alternatives. Kleinfelder will not be held liable for any misuse of the information contained in this appendix.

10.0 References

The following references are cited in this appendix.

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Restoration of the Salton Sea

Volume 2: Embankment Designs and Optimization Study

Appendix 2B: Seepage and Stability Analysis

Figures

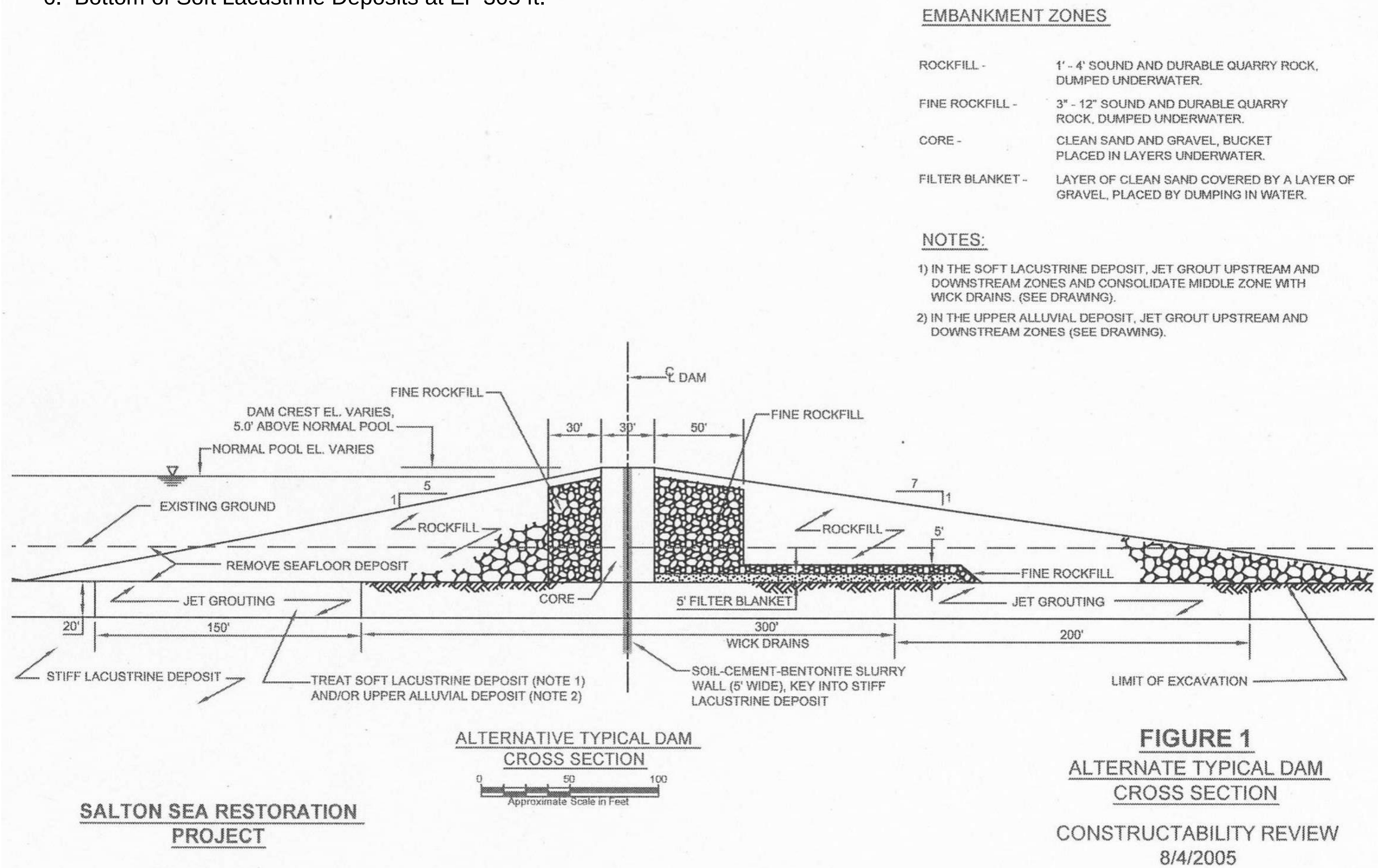
Prepared for:
U.S. Department of the Interior
Bureau of Reclamation
Lower Colorado Region
Boulder City, Nevada

Prepared by:
Kleinfelder, Inc.
Golden, CO 80401
Project No. 71100

May 2007

Note: Base drawing taken from Reclamation's Statement of Work, dated April 11, 2006.

"Baseline" Mid-Sea Dam Characteristics and Assumptions:
1. Dam crest at El -223 ft.
2. Pre-dredge mudline at El -268 ft.
3. Upstream pool at El -228 ft.
4. Downstream pool at El -268 ft.
5. Dredge exsiting Seafloor Deposits to El -280 ft.
6. Bottom of Soft Lacustrine Deposits at El -305 ft.



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SALTON SEA RESTORATION PROJECT
EMBANKMENT DESIGNS AND OPTIMIZATION STUDY
Appendix 2B – Seepage and Stability Analyses

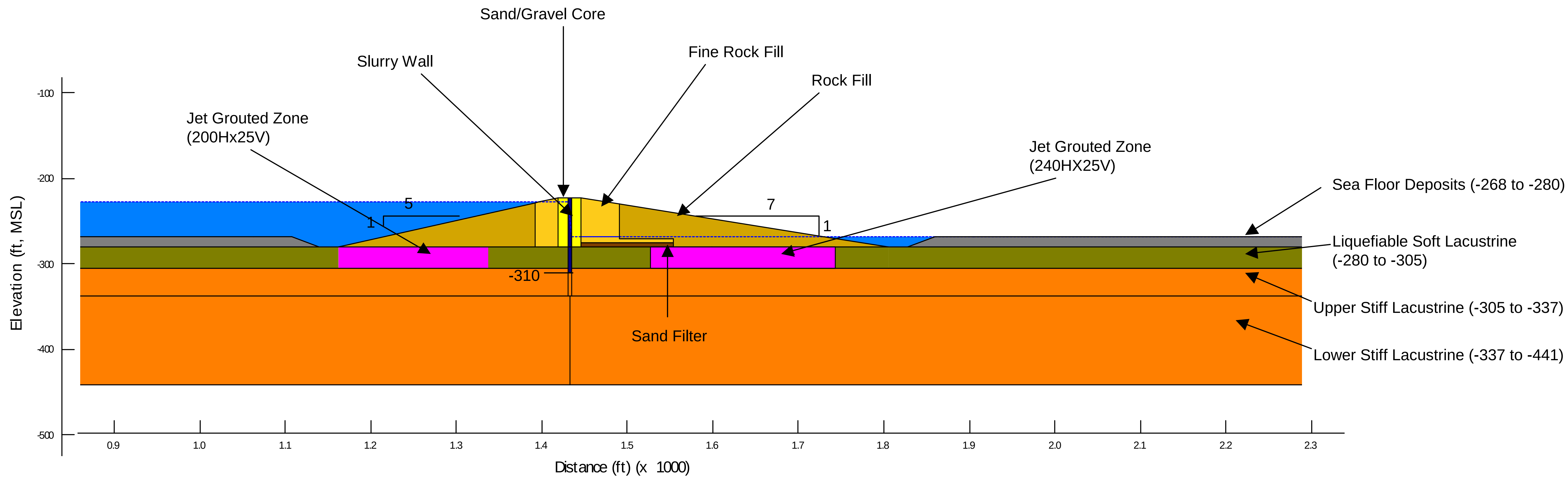
Project 71100 | By E. Sossenkina | August 2006

Mid-Sea Dam Schematic Cross Section

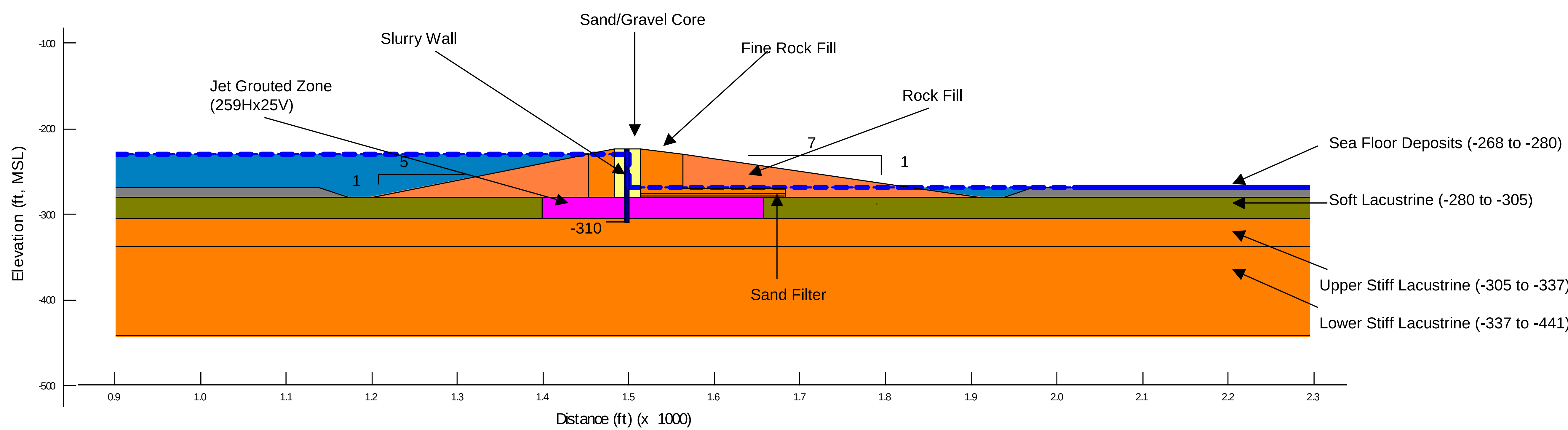
FIGURE B.1

71100/DEN6R101

Case 4,01



Case 4,02



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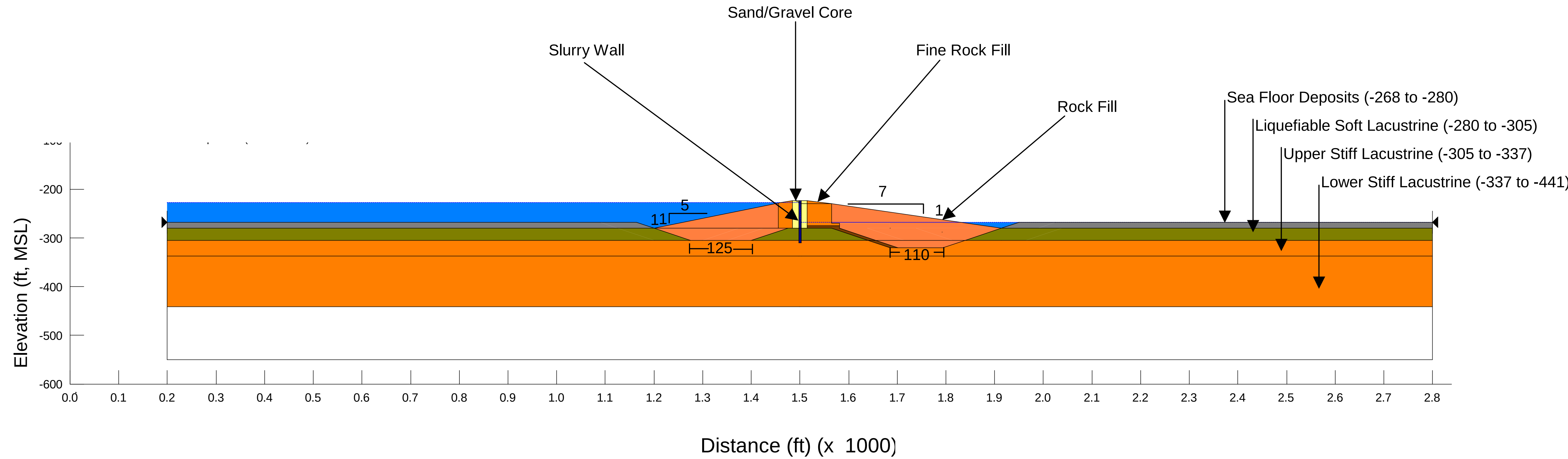
SALTON SEA RESTORATION PROJECT
EMBANKMENT DESIGNS AND
OPTIMIZATION STUDY
Appendix 2B – Seepage and Stability Analyses

Mid-Sea Dam Model Geometry
Case 4,01 And Case 4,02

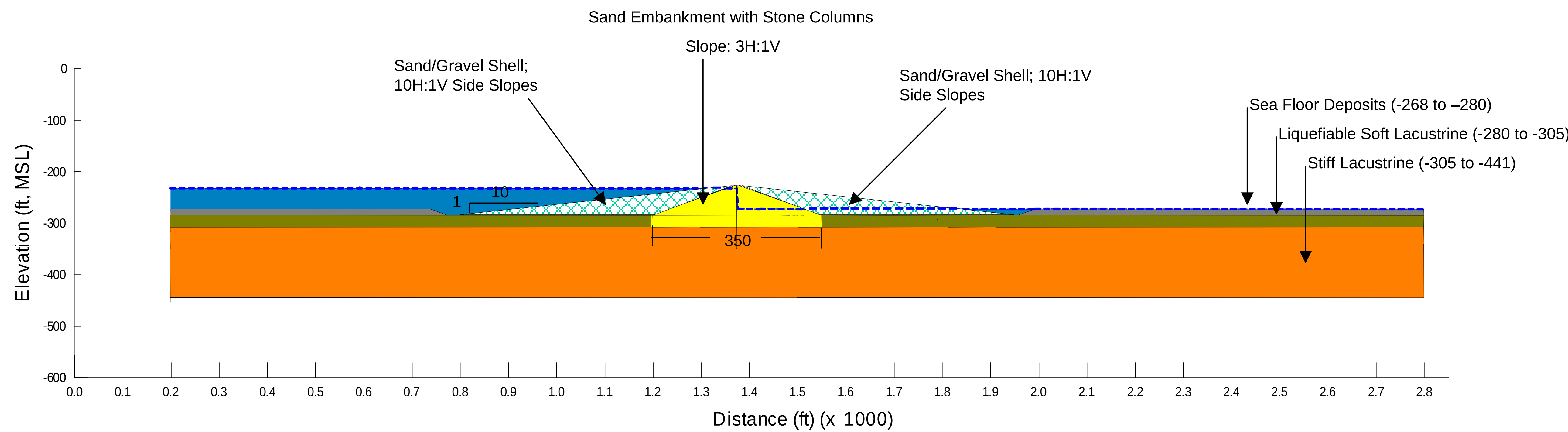
Project 71100 | By E. Sossenkina | August 2006

FIGURE B.2

Case 4,03



Case 4,03



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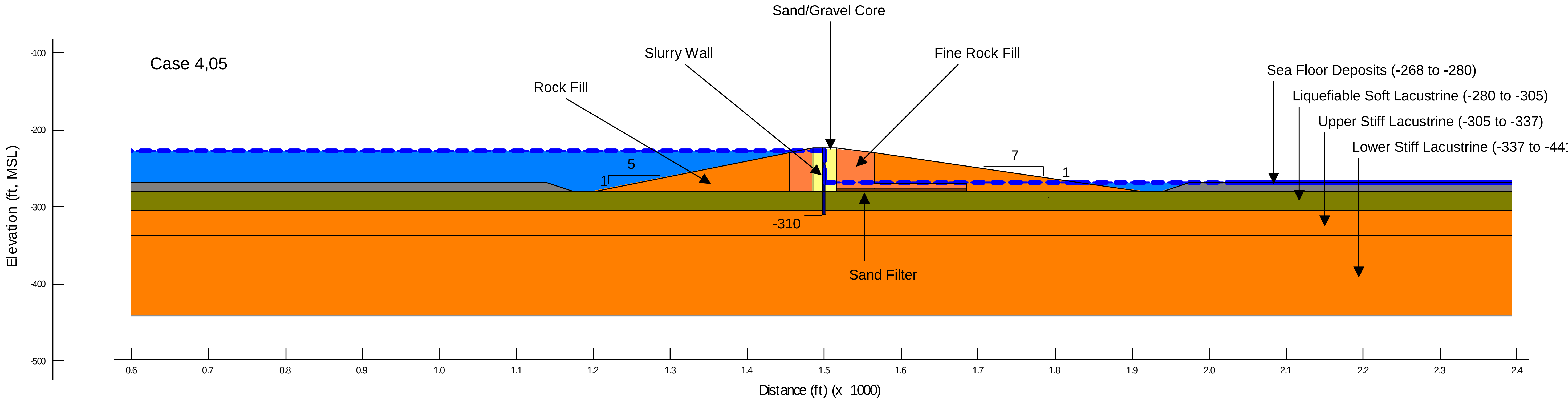
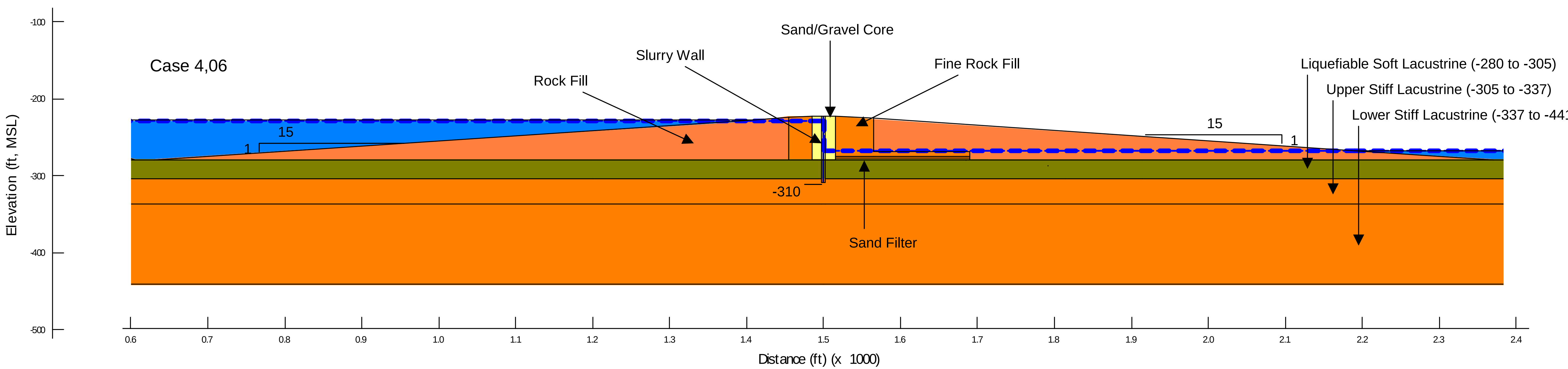
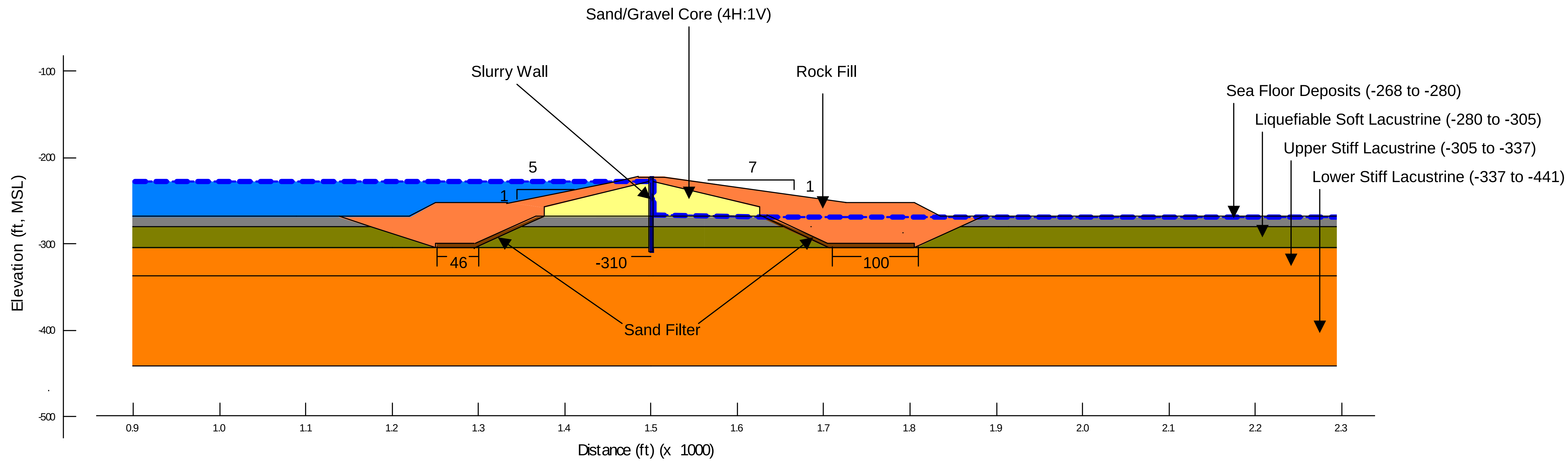
KLEINFELDER	Project 71100	By E. Sossenkina	August 2006	**FIGURE B.3**



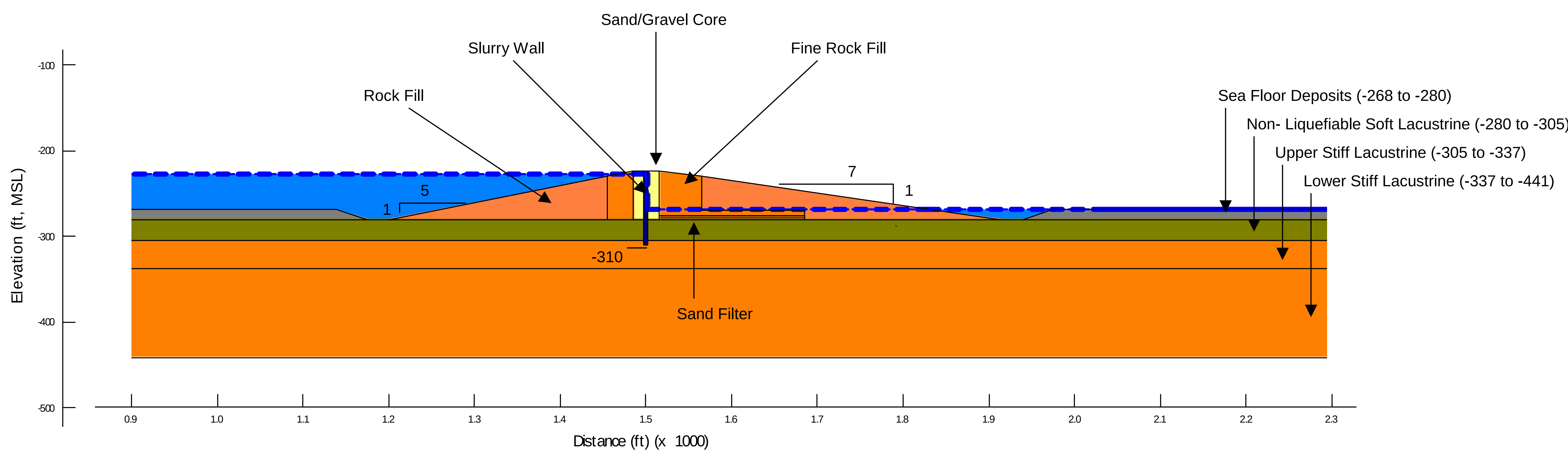


United States Department of the Interior Bureau of Reclamation	**SALTON SEA RESTORATION PROJECT** EMBANKMENT DESIGNS AND OPTIMIZATION STUDY Appendix 2B – Seepage and Stability Analyses	Mid-Sea Dam Model Geometry Case 4,05 And Case 4,06
KLEINFELDER	Project 71100 | By E. Sossenkina | August 2006	**FIGURE B.4**

Case 4,07



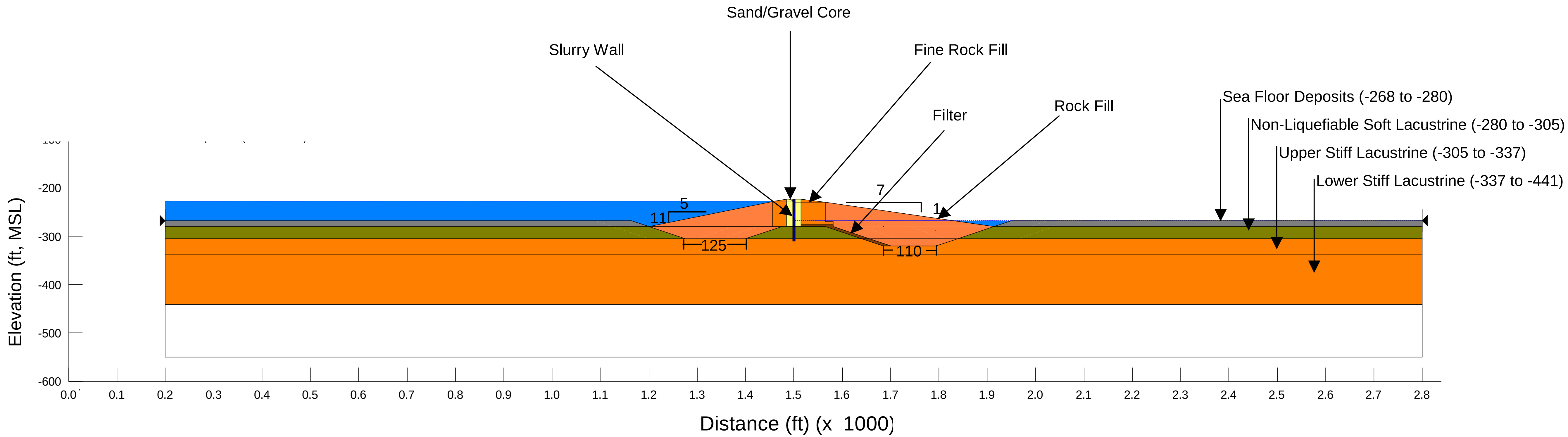
Case 4,10



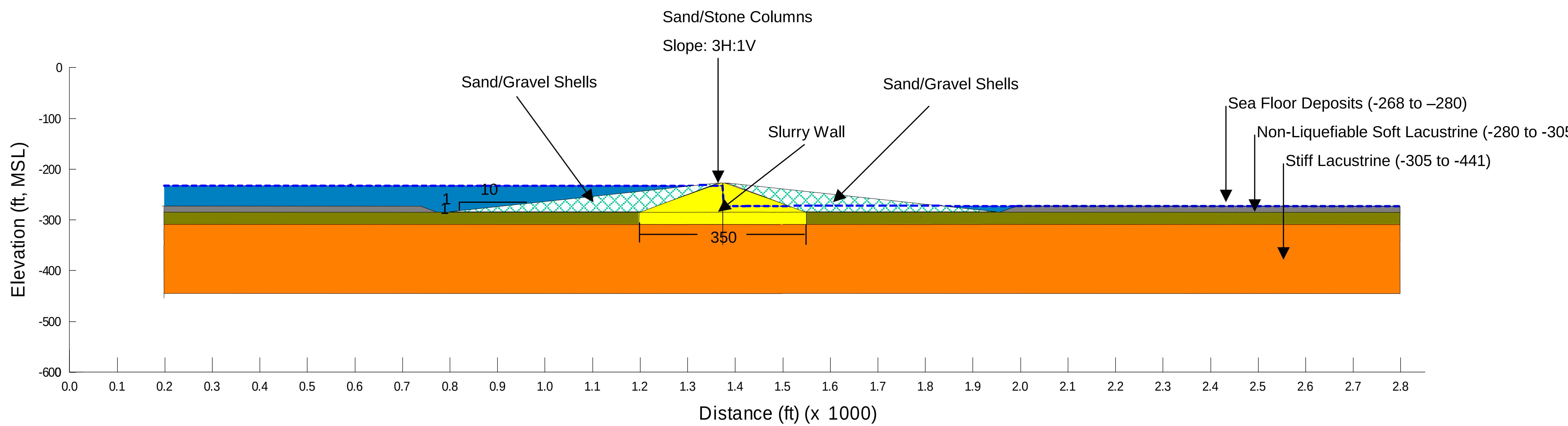
United States Department of the Interior Bureau of Reclamation	**SALTON SEA RESTORATION PROJECT** EMBANKMENT DESIGNS AND OPTIMIZATION STUDY Appendix 2B – Seepage and Stability Analyses			Mid-Sea Dam Model Geometry Case 4,07 And Case 4,10
KLEINFELDER	Project 71100	By E. Sossenkina	August 2006	**FIGURE B.5**

71100/DEN6R101

Case 4,11



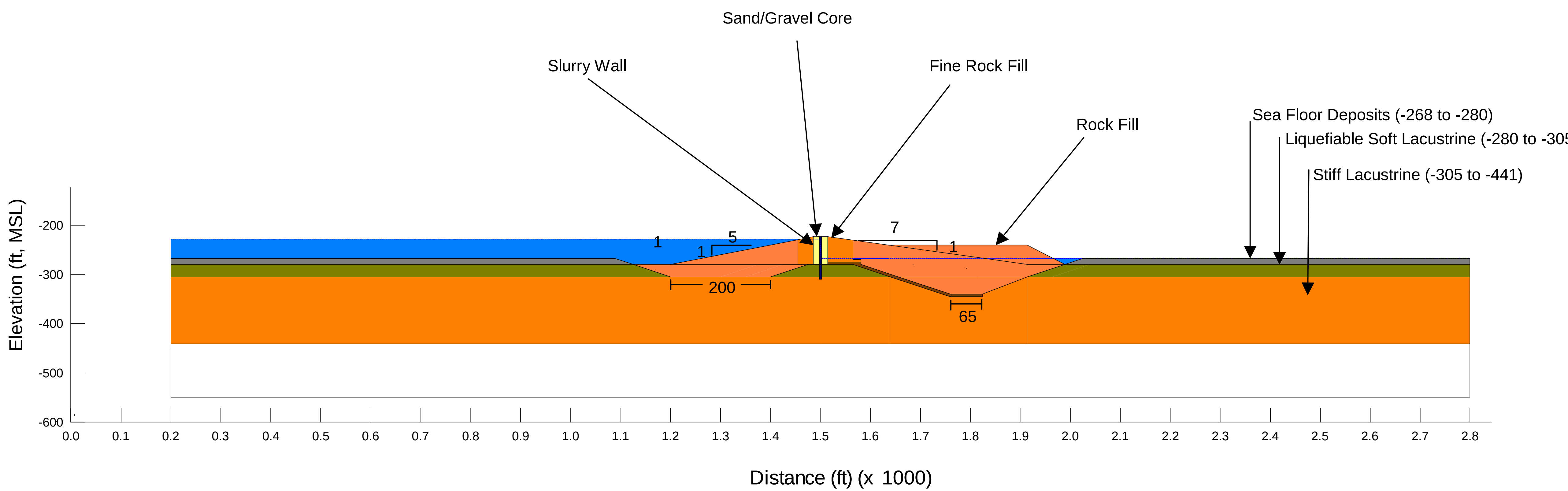
Case 4,12



United States Department of the Interior Bureau of Reclamation	**SALTON SEA RESTORATION PROJECT** EMBANKMENT DESIGNS AND OPTIMIZATION STUDY Appendix 2B – Seepage and Stability Analyses			Mid-Sea Dam Model Geometry Case 4,11 And Case 4,12
KLEINFELDER	Project 71100	By E. Sossenkina	August 2006	**FIGURE B.6**

71100/DEN6R101

Case 4,13



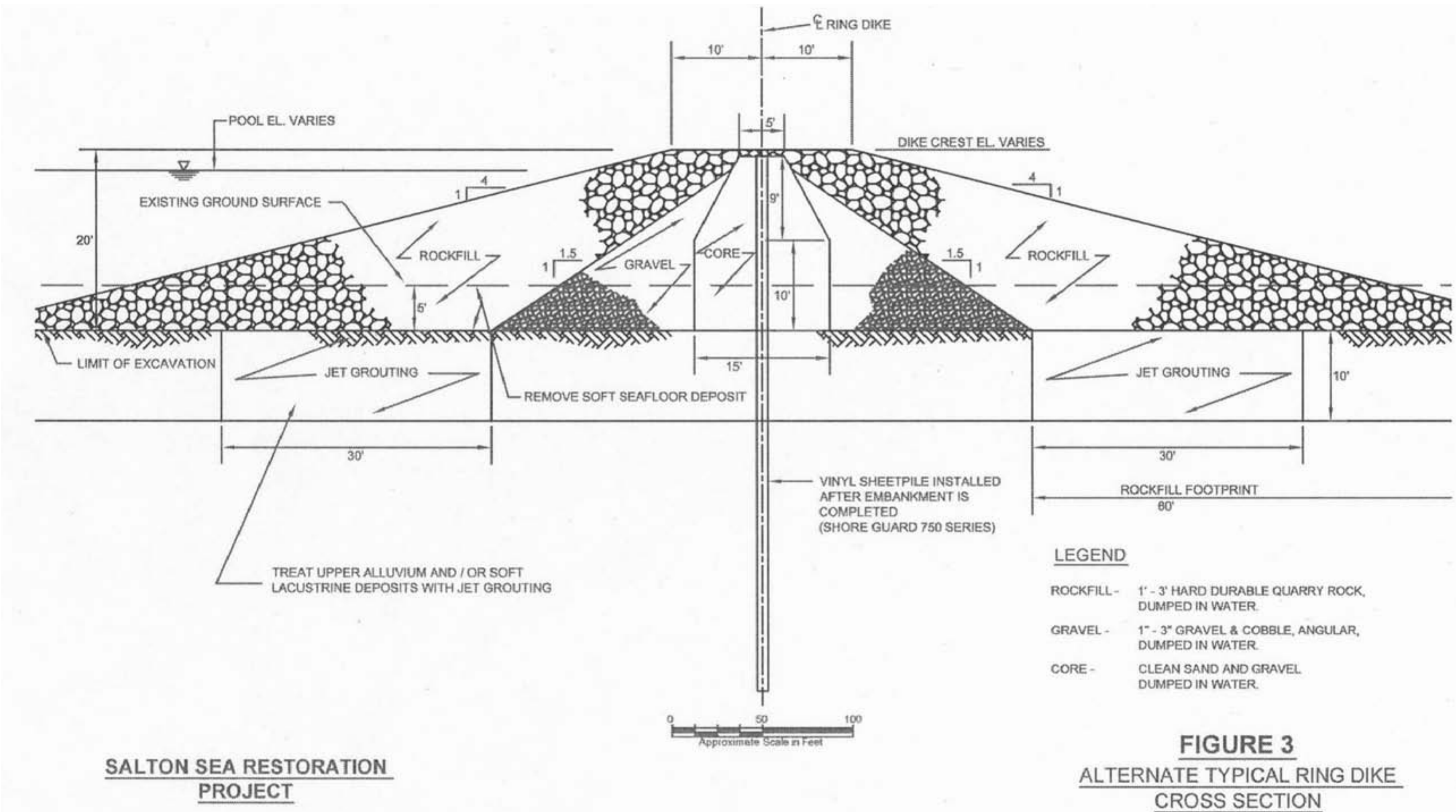
MID-SEA DAM MODEL GEOMETRY
CASE 4-13
SALTON SEA RESTORATION PROJECT, CALIFORNIA
PHASE 2 EMBANKMENT ALTERNATIVES OPTIMIZATION STUDY

United States Department of the Interior Bureau of Reclamation	**SALTON SEA RESTORATION PROJECT** EMBANKMENT DESIGNS AND OPTIMIZATION STUDY Appendix 2B – Seepage and Stability Analyses			Mid-Sea Dam Model Geometry Case 4,13
K L E I N F E L D E R	Project 71100	By E. Sossenkina	August 2006	**FIGURE B.7**

Note: Base drawing taken from Reclamation's Statement of Work, dated April 11, 2006.

"Baseline" Ring / Perimeter Dike Characteristics and Assumptions:

1. Embankment crest at El -240 ft.
2. Pre-dredge mudline at El -260 ft.
3. Upstream pool at El -240.5 ft (0.5 ft freeboard).
4. Downstream pool at El -255 ft.
5. Dredge exsiting Seafloor Deposits to El -265 ft (west side) or to El -270 ft (east side).
6. Bottom of Soft Lacustrine Deposits at El -280 ft (east side).
7. Bottom of Upper Alluvium at El -275 ft (west side).



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SALTON SEA RESTORATION PROJECT

EMBANKMENT DESIGNS AND OPTIMIZATION STUDY

Appendix 2B – Seepage and Stability Analyses

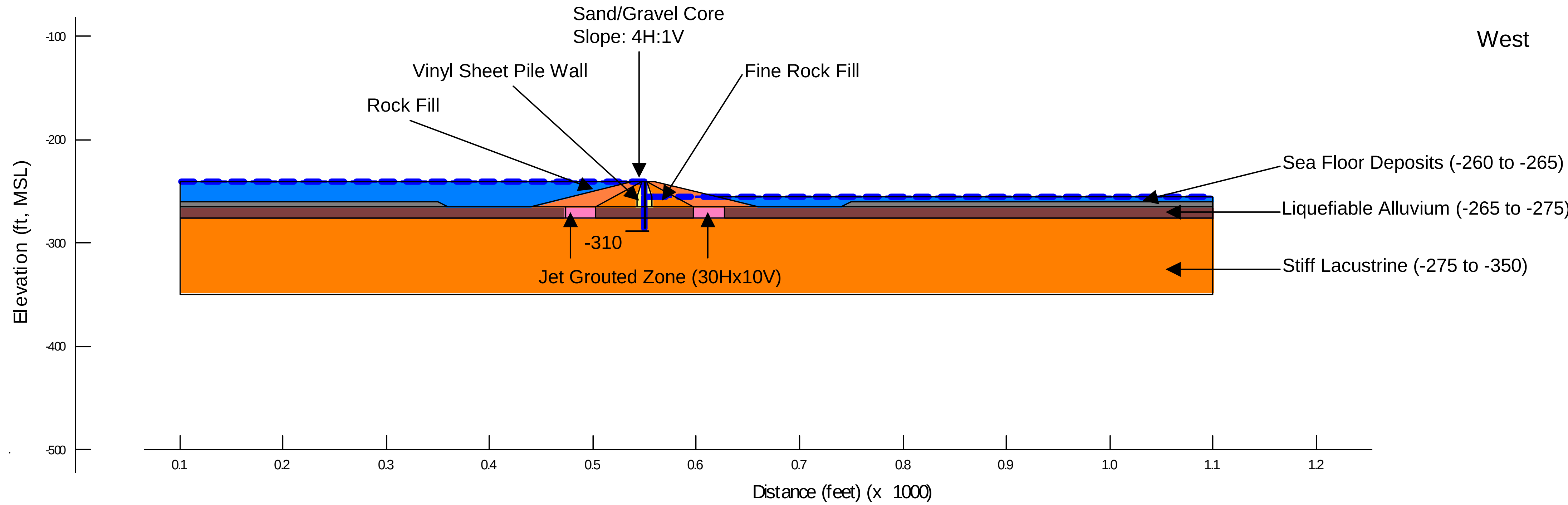
Project 71100 | By E. Sossenkina | August 2006

Ring Dike/Perimeter Dike Schematic Cross Section

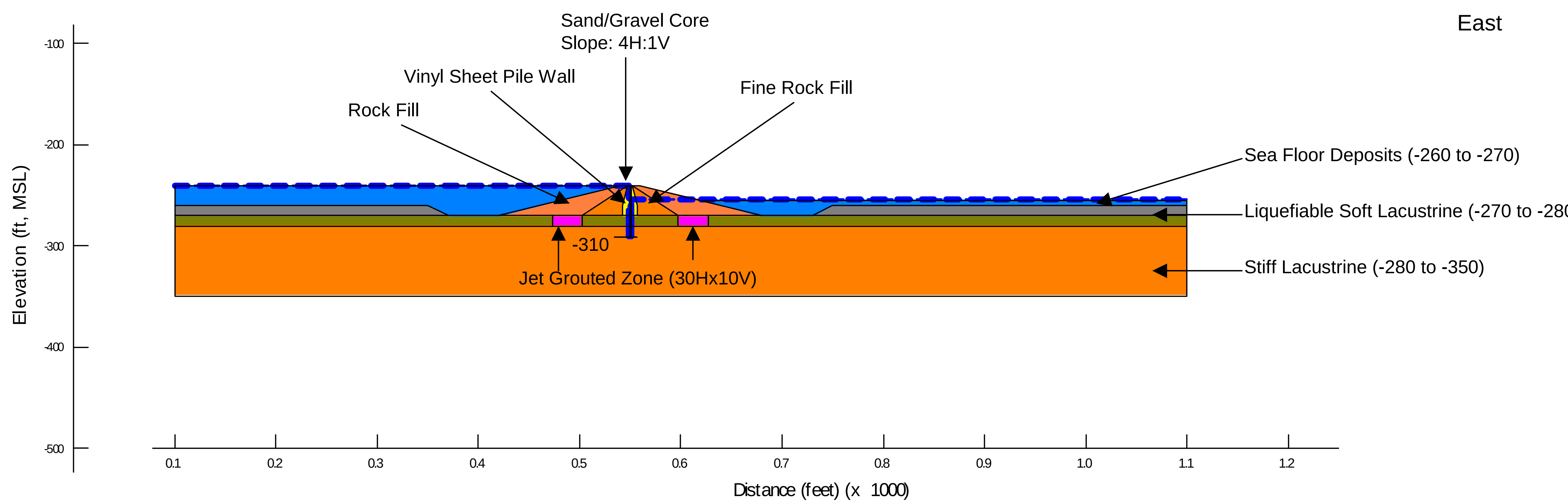
FIGURE B.8

71100/DEN6R101

Case 4,08

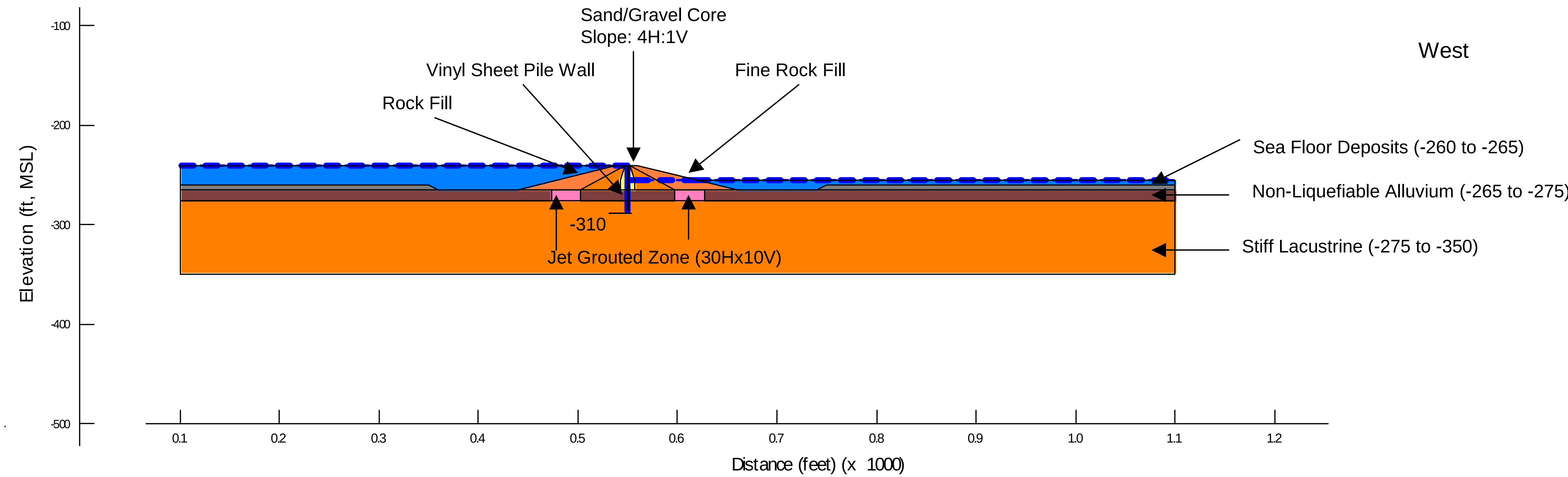


Case 4,09

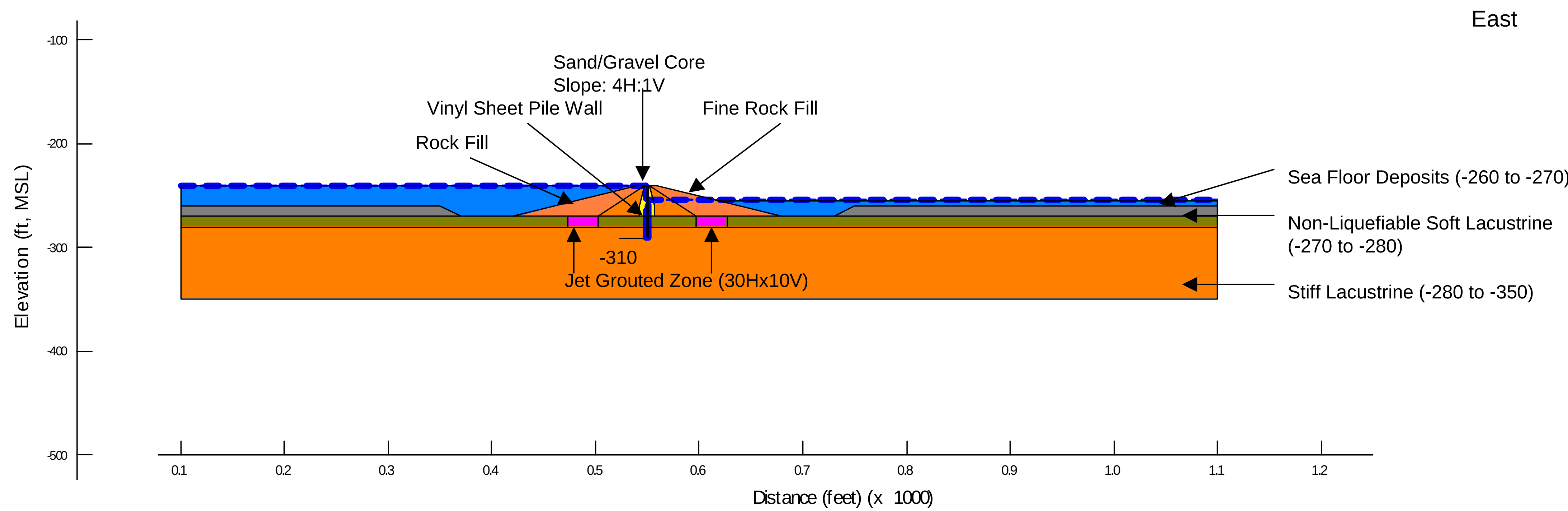


United States Department of the Interior Bureau of Reclamation	**SALTON SEA RESTORATION PROJECT** EMBANKMENT DESIGNS AND OPTIMIZATION STUDY Appendix 2B – Seepage and Stability Analyses			Ring Dike / Perimeter Dike Model Geometry Case 4,08 West And Case 4,09 East
KLEINFELDER	Project 71100	By E. Sossenkina	August 2006	**FIGURE B.9**

Case 4,14



Case 4,15

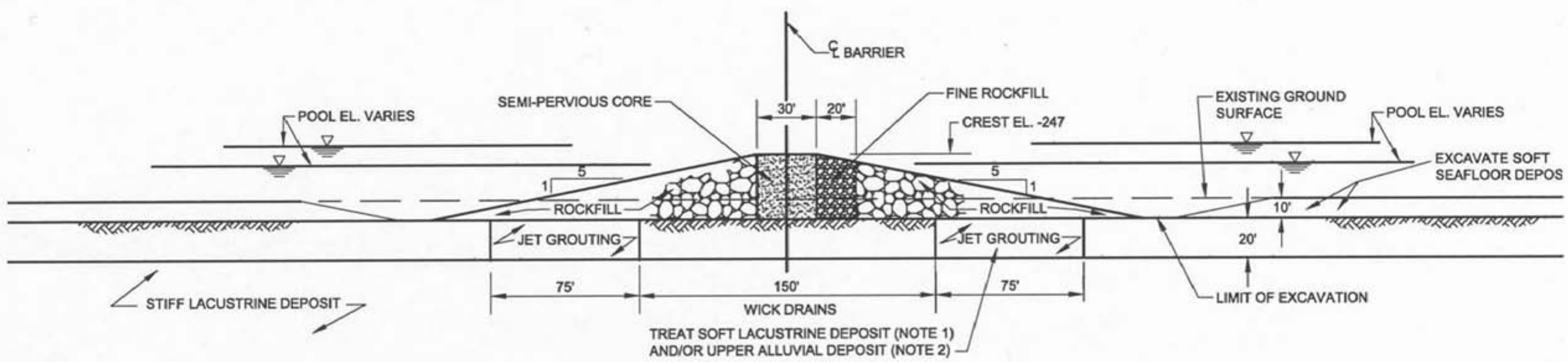


United States Department of the Interior Bureau of Reclamation	**SALTON SEA RESTORATION PROJECT** EMBANKMENT DESIGNS AND OPTIMIZATION STUDY Appendix 2B – Seepage and Stability Analyses	Ring Dike / Perimeter Dike Model Geometry Case 4,14 West And Case 4,15 East
KLEINFELDER	Project 71100 | By E. Sossenkina | August 2006	**FIGURE B.10**

Note: Base drawing taken from Reclamation's Statement of Work, dated April 11, 2006.

"Baseline" Mid-Sea Barrier Characteristics and Assumptions:

1. Embankment crest at El -247 ft.
2. Pre-dredge mudline at El -268 ft.
3. Upstream pool at El -252 ft.
4. Downstream pool at El -257 ft.
5. Dredge exsiting Seafloor Deposits to El -280 ft.
6. Bottom of Soft Lacustrine Deposits at El -305 ft.
7. No post-dredging ground improvement (contrary to what is shown on this illustration).



ALTERNATE BARRIER
TYPICAL CROSS SECTION

0 50 100
Approximate Scale in Feet

LEGEND

ROCKFILL -	1' - 4' HARD DURABLE QUARRY ROCK, DUMPED UNDERWATER.
FINE ROCKFILL	3" - 12" HARD DURABLE QUARRY ROCK, DUMPED UNDERWATER.
SEMI-PERVIOUS CORE -	SAND AND GRAVEL, BUCKET PLACED IN LAYERS UNDERWATER.

NOTES:

1) IN THE SOFT LACUSTRINE DEPOSIT, JET GROUT UPSTREAM AND DOWNSTREAM ZONES AND CONSOLIDATE MIDDLE ZONE WITH WICK DRAINS. (SEE DRAWING).

2) IN THE UPPER ALLUVIAL DEPOSIT, JET GROUT UPSTREAM AND DOWNSTREAM ZONES (SEE DRAWING).

SALTON SEA RESTORATION PROJECT

FIGURE 2
ALTERNATE BARRIER
TYPICAL CROSS SECTION

CONSTRUCTABILITY REVIEW
8/4/2005

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SALTON SEA RESTORATION PROJECT
EMBANKMENT DESIGNS AND OPTIMIZATION STUDY
Appendix 2B – Seepage and Stability Analyses

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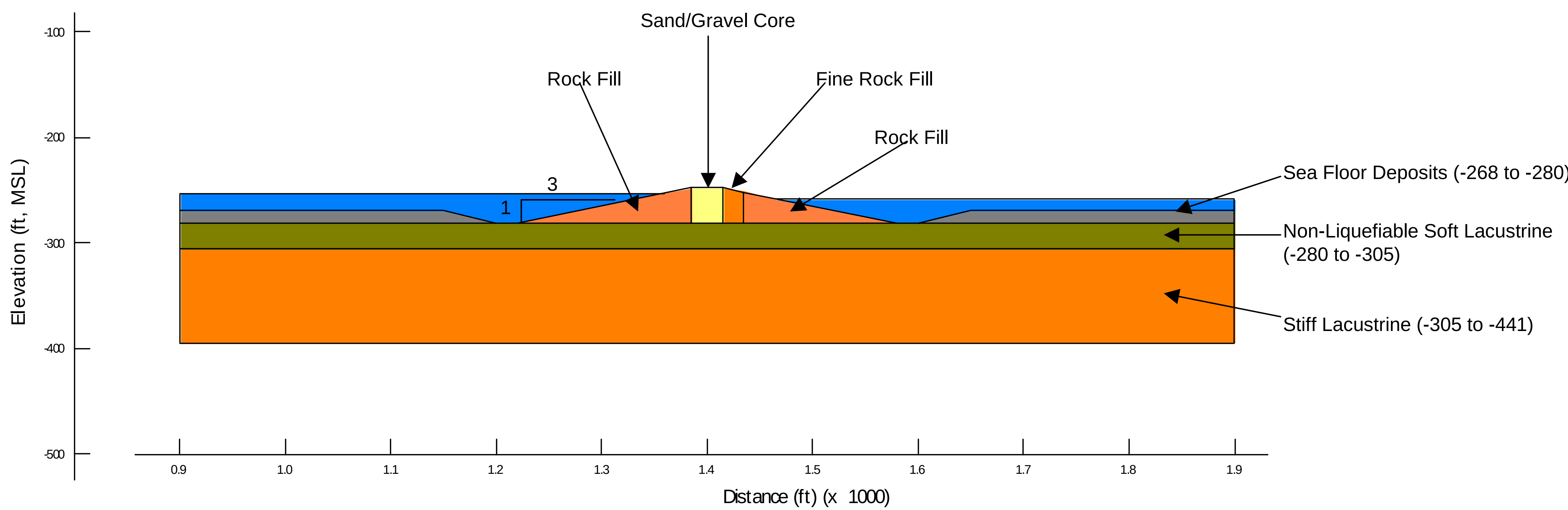
Mid-Sea Barrier Schematic Cross Section

FIGURE B.11

71100/DEN6R101

Case 5,01

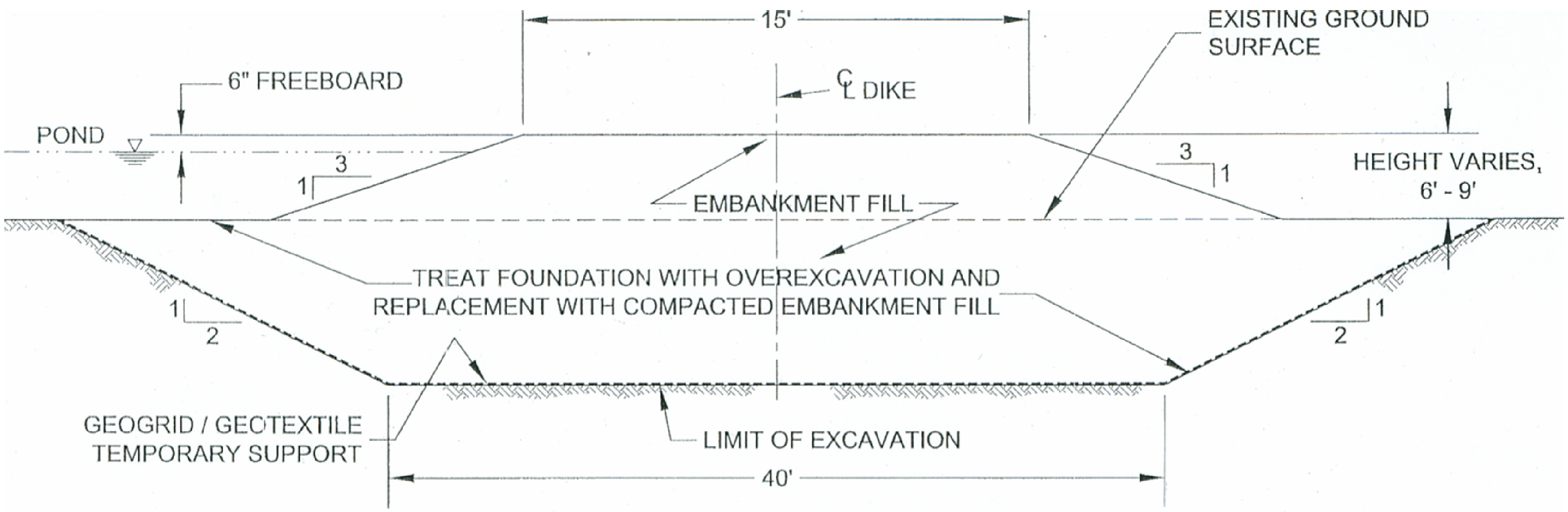


United States Department of the Interior Bureau of Reclamation	**SALTON SEA RESTORATION PROJECT** EMBANKMENT DESIGNS AND OPTIMIZATION STUDY Appendix 2B – Seepage and Stability Analyses		Mid-Sea Barrier Model Geometry Case 5,01
KLEINFELDER	Project 71100	By E. Sossenkina	August 2006
			FIGURE B.12

Note: Base drawing taken from Reclamation's Statement of Work, dated April 11, 2006.

"Baseline" Habitat Pond Earthfill Embankment Characteristics and Assumptions:

1. Embankment crest at El -241 ft.
2. Pre-dredge mudline at El -250 ft.
3. Upstream pool at El –241.5 ft.
4. Downstream pool at El -250 ft.
5. Dredge exsiting Seafloor Deposits to El -260 ft.
6. Embankment founded on 5 ft of Upper Alluvium, underlain by 5 ft of Stiff Lacustrine, underlain by additional Upper Alluvium.



TYPICAL EARTHFILL EMBANKMENT FOR HABITAT PONDS

(NOT TO SCALE)

LEGEND

EMBANKMENT FILL - COMPACTED (IN THE DRY) EARTHFILL (CLAY, SILT)

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SALTON SEA RESTORATION PROJECT

EMBANKMENT DESIGNS AND OPTIMIZATION STUDY

Appendix 2B – Seepage and Stability Analyses

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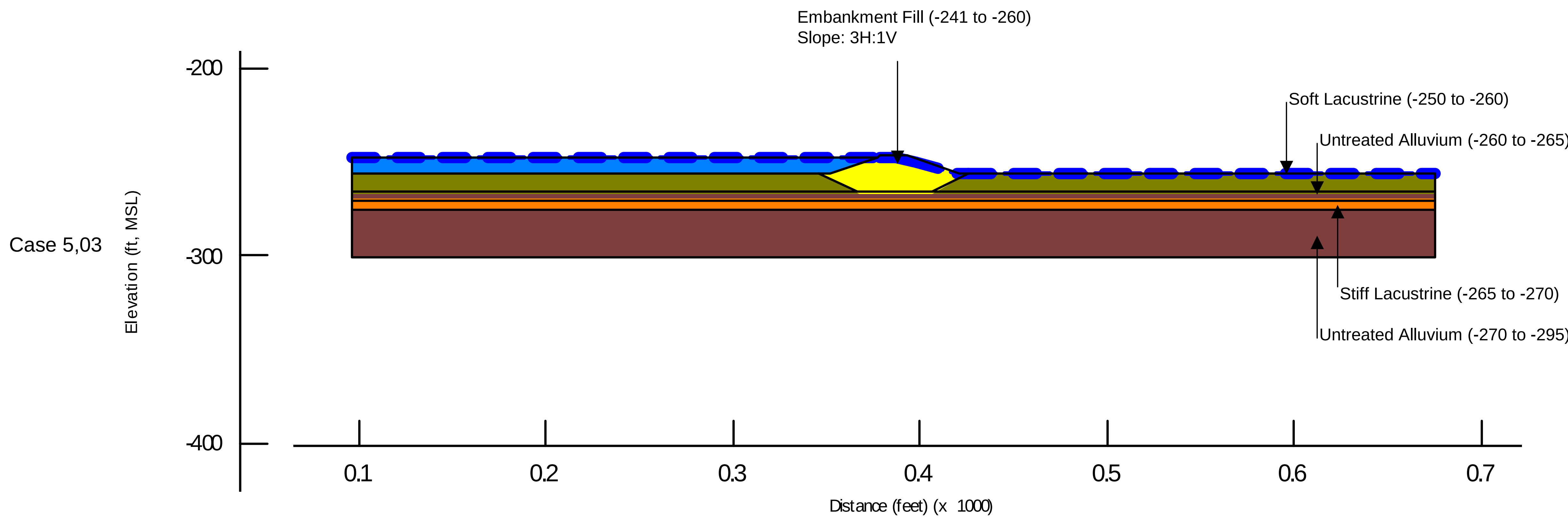
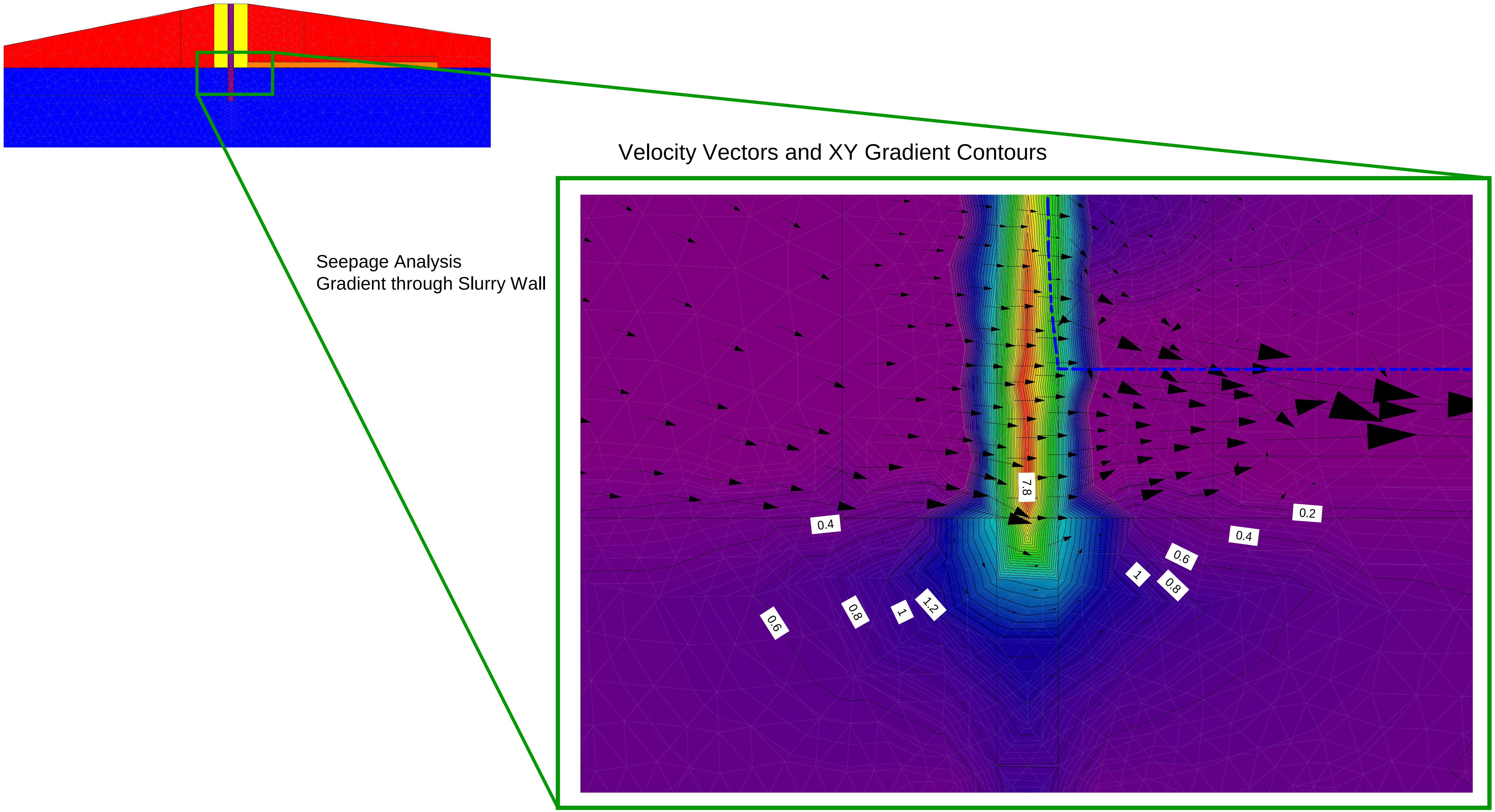
Habitat Pond Embankment Schematic Cross Section

FIGURE B.13

71100/DEN6R101



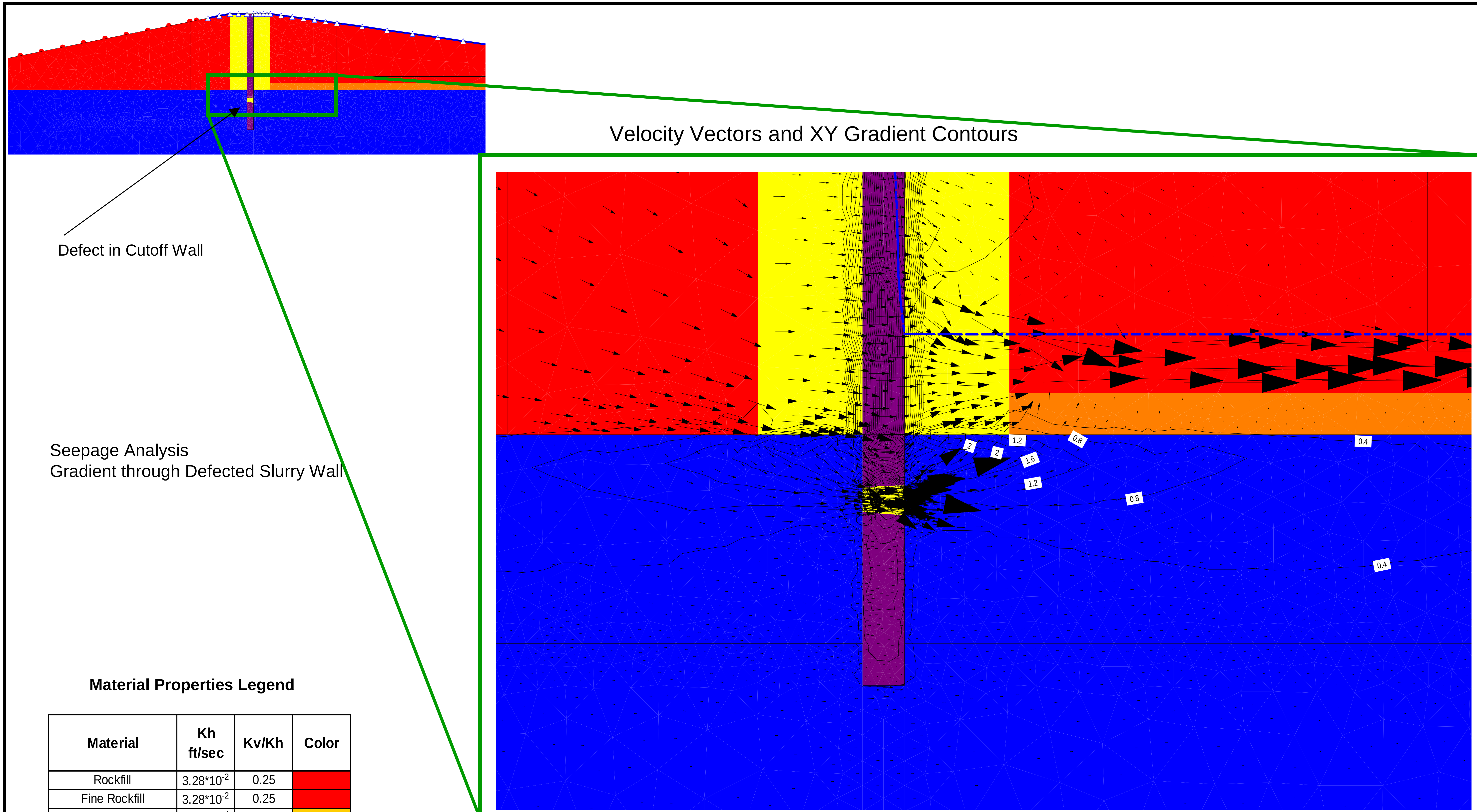
United States Department of the Interior Bureau of Reclamation	SALTON SEA RESTORATION PROJECT EMBANKMENT DESIGNS AND OPTIMIZATION STUDY Appendix 2B – Seepage and Stability Analyses			Habitat Pond Embankment Model Geometry Case 5,03
KLEINFELDER	Project 71100	By E. Sossenkina	August 2006	**FIGURE B.14**



XY Gradient i_{xy} Contours Legend

United States Department of the Interior Bureau of Reclamation	**SALTON SEA RESTORATION PROJECT** EMBANKMENT DESIGNS AND OPTIMIZATION STUDY Appendix 2B – Seepage and Stability Analyses	Seepage Analysis Gradient through Slurry Wall Case 4,02
KLEINFELDER	Project 71100 | By E. Sossenkina | August 2006	**FIGURE B.15**

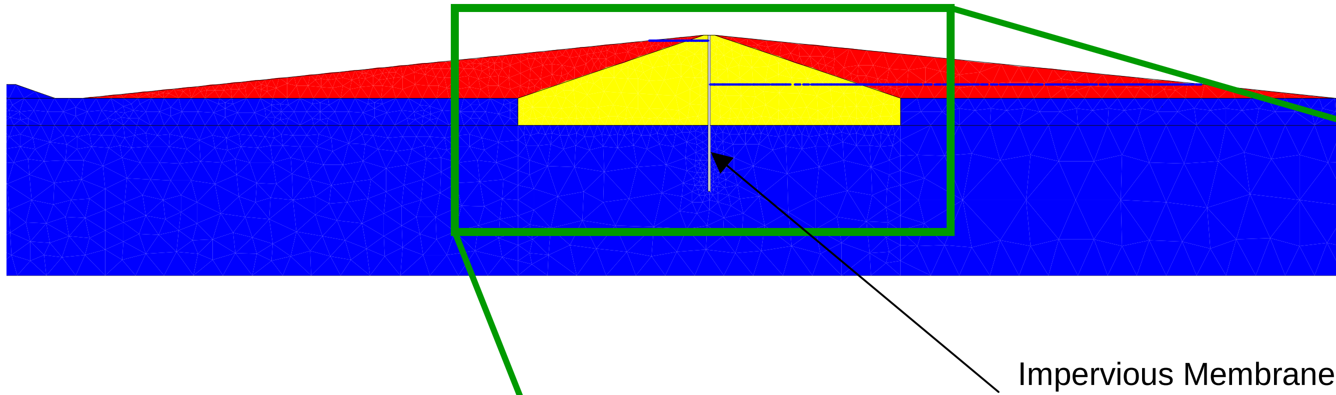
71100/DEN6R101



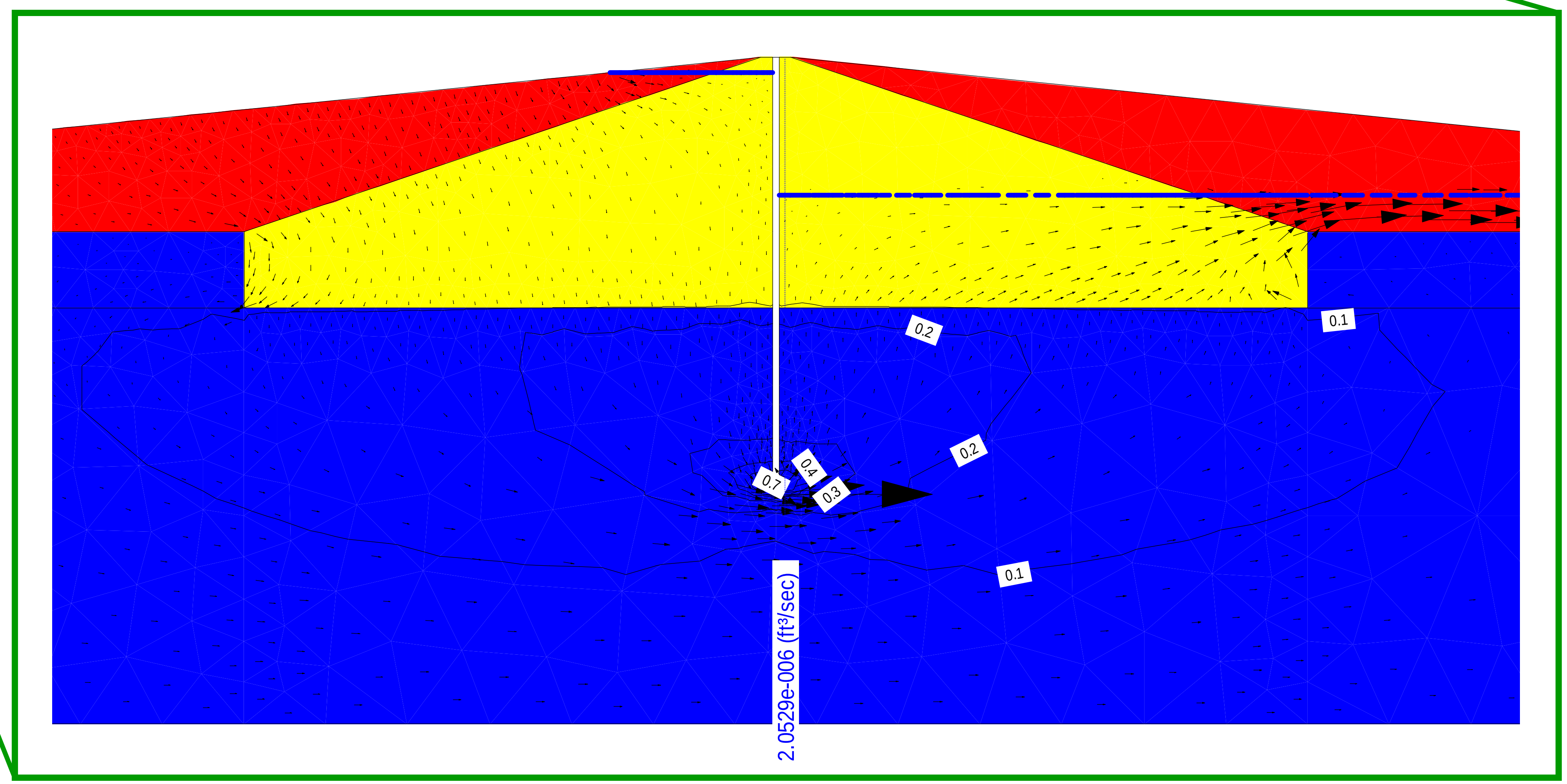
Material Properties Legend

Material	Kh ft/sec	Kv/Kh	Color
Rockfill	$3.28*10^{-2}$	0.25	
Fine Rockfill	$3.28*10^{-2}$	0.25	
Filter Blanket	$1.77*10^{-4}$	1	
Sand Gravel Core	$3.28*10^{-4}$	0.25	
Slurry Wall	$3.28*10^{-8}$	1	
Seafloor Deposits	$3.28*10^{-7}$	0.1	
Soft Lacustrine	$3.28*10^{-7}$	0.1	
Stiff Lacustrine	$3.28*10^{-7}$	0.1	

United States Department of the Interior Bureau of Reclamation	**SALTON SEA RESTORATION PROJECT** EMBANKMENT DESIGNS AND OPTIMIZATION STUDY Appendix 2B – Seepage and Stability Analyses		Seepage Analysis Gradient through Defected Slurry Wall Case 4,02
K L E I N F E L D E R	Project 71100	By E. Sossenkina	August 2006	**FIGURE B.16**



Velocity Vectors and XY Gradient Contours



Seepage Analysis
Gradient through Stone Column Embankment

Material Properties Legend

Material	Kh ft/sec	Kv/Kh	Color
Rock Fill	$3.28*10^{-2}$	0.25	Red
Seafloor Deposits	$3.28*10^{-7}$	0.1	Blue
Soft Lacustrine	$3.28*10^{-7}$	0.1	Blue
Stiff Lacustrine	$3.28*10^{-7}$	0.1	Blue
Stone Column	$3.28*10^{-2}$	0.25	Yellow
Membrane	Impervious		

United States Department of the Interior Bureau of Reclamation	**SALTON SEA RESTORATION PROJECT** EMBANKMENT DESIGNS AND OPTIMIZATION STUDY Appendix 2B – Seepage and Stability Analyses		Seepage Analysis Gradient through Stone Column Embankment
KLEINFELDER	Project 71100	By E. Sossenkina	August 2006
			FIGURE B.17