APPENDIX H-4

Conceptual Design for In-Sea Rock Barriers

2006

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APPENDIX H-4
CONCEPTUAL DESIGN FOR IN-SEA ROCK BARRIERS

EXECUTIVE SUMMARY

A conceptual design using rockfill for the construction of barriers within the Salton Sea has been developed for the purposes of the Programmatic Environmental Impact Report (PEIR). The conceptual design is based on subsurface sea deposit investigations and subsequent soil testing of samples retrieved at varying depths over a wide area of the Sea Bed. These investigations were carried out by URS Corporation during the fall of 2003 and completed in February 27, 2004. The design is governed by the area's high seismicity and poor foundation conditions.

A series of earthquake ground motions was developed using the guidelines recommended by the California Department of Water Resources, Division of Safety of Dams based on an evaluation of the seismicity of area. The most severe input motion simulates a Magnitude 7.8 event on the San Andreas Fault, which runs parallel and next to the Salton Sea's eastern shoreline. The severity of input motion decreases with distance away from the eastern shore. A corresponding set of input motions is developed for locations parallel to the Salton Sea's western shoreline.

The Sea Bed deposits consist predominantly of low strength clay. This unfavorable foundation condition, coupled with the high level of input ground motions, requires typical upstream and downstream slope inclinations of 10 to 1 and 15 to 1, respectively, for barriers. This conceptual cross-section is considered appropriate due to the weak foundation layers and the very limited geotechnical data at the Salton Sea. If a preferred restoration alternative involving a barrier is selected, additional investigations will be needed along any proposed barrier alignment. It is anticipated that future investigations and analyses may be able to economize on the necessary fill volumes. The design slope inclinations assume the greatest differential between water surface elevations possible.

The conceptual design will help prevent an uncontrolled release of impounded water during the applicable input motions as well as fault rupture. The design limits the seismically induced permanent deformations of the crest to approximately half foot vertically and three feet laterally for the slopes.

Different restoration alternatives may require varying degrees of flow between the bodies of water separated by barriers (for example, discharge to the Brine Sink or recirculation water). To limit seepage through the barrier, finer grained materials are placed on the upstream slope, whereas greater flows are accommodated either through the rockfill or through culverts.

To achieve these performance characteristics, a rockfill having a maximum particle size in the 4-5 foot diameter range is required for barriers in excess of 25 feet in height. Larger size rocks could be placed on the upstream slope to resist wave action. It is anticipated that some restoration alternatives may require a quantity of rock in the range of 65 to 75 million cubic yards.

A study of potential quarry sites was undertaken in 2005 to assess the availability of rock within a 50-mile radius of the Salton Sea. The evaluation considered such issues as landownership and access, environmental impacts and potential associated mitigation actions, as well as rock suitability. The study identified two sites, Eagle Mountain Mine and Coolidge Mountain, for field exploration and additional evaluation. In October 2005, a Draft report presenting the results of field exploration at Eagle Mountain Mine was completed. The results indicated that the quality of rockfill materials at the Mine to construct in-sea barriers and dikes was deemed to be durable and suitable for use in the saline environment of the Salton Sea. The results also indicated that the available rockfill material is adequate to construct barriers, perimeter dikes, and other infrastructure components under any restoration alternatives. However, the

amount of large boulders observed which is desired for construction of higher barrier sections was very limited. The size of the rockfill piles tested was primarily a mixture of sand, gravel, and cobbles up to 12 inches in diameter. The project-level design could further consider rockfill sources if needed.

In addition, the Salton Sea Authority, in coordination with the U.S. Department of the Interior, Bureau of Reclamation (Reclamation) is currently developing a reconnaissance level investigation of Coolidge Mountain, which is near the Salton Sea. The results will likely not be completed in time to be incorporated into this document; however, if found suitable, this location could provide a relatively economical source of rockfill to construct in-sea barriers because of its proximity to the Salton Sea.

These studies have completed the investigation needed at a programmatic level; no further fieldwork will be needed for the conceptual design of in-sea rock barriers.

Introduction

This conceptual design report was developed in support of the Programmatic Environmental Impact Report (PEIR). The Department of Water Resources (DWR) and the Department of Fish and Game (DFG) are conducting these efforts jointly, as result of legislation implementing the Quantification Settlement Agreement (QSA).

The QSA implementing legislation requires that a preferred alternative for restoration of the Salton Sea be identified by the end of 2006. This very short time frame within which to conduct all of the necessary studies for the PEIR required that a number of work activities be performed in parallel. The conceptual design development described herein is one example of the studies undertaken in parallel with the development of restoration alternatives. Based on prior studies and restoration evaluations, it was anticipated that several programmatic restoration alternatives would include in-sea barriers. These alternatives may include varying performance objectives of barriers such as accommodating a differential head between adjacent bodies of water as well as varying degrees of flow/seepage through the barrier.

Prior studies, starting in December 2002, were funded by the U.S. Department of the Interior, Bureau of Reclamation (Reclamation) in coordination with the Salton Sea Authority, to develop conceptual alternatives for design and construction of in-sea barriers. A subsurface investigation of the Sea Bed deposits was initiated in September 2003 with a final report issued in February 2004.

This report documents the work activities by DWR to develop a concept which employs the use of rockfill materials to construct barriers of varying heights located anywhere in the Salton Sea. Development of the conceptual barrier design utilizes the subsurface information obtained in the prior investigation as well as the laboratory strength evaluations of the Sea Bed deposits.

The level of the conceptual design is commensurate with the level required for a PEIR. Should the preferred alternative for restoration of the Salton Sea include an in-sea barrier, additional subsurface investigations and analyses will have to be performed for specific alignments and barrier performance objectives.

Background

California's use of the waters from the Colorado River has historically exceeded its apportionment by as much as 800,000 acre-feet annually. This water became available to California because of the wet hydrologic conditions and because other Colorado River Basin states were not using their full apportionments. By the early 1990s, Arizona and Nevada neared use of their full apportionments which, beginning in the mid-1990s led to discussions on a potential Imperial Irrigation District-San Diego County Water Authority transfer.

These two agencies, in addition to Coachella Valley Water District and the Metropolitan Water District of Southern California, engaged in lengthy discussions and negotiations to reach agreement on their respective water budgets so that they collectively did not exceed California's annual appropriation of 4.4 million acre-feet from the Colorado River. The State of California, led by the Governor's Office, was an active participant in the negotiations to endeavor the parties reach an agreement. An agreement was reached and the Governor signed the implementing legislation on September 29, 2003.

The QSA constitutes an agriculture-to-urban water transfer of Colorado River water. The Salton Sea is a terminal water body which receives inflows from primarily the Whitewater, New and Alamo rivers. The latter two contribute 80 percent of the inflows as a result of agricultural water return flows. Currently, the inflow to the Salton Sea is equivalent to the amount of evaporation, which is 1.2 million acre feet annually. This balance between inflow and evaporation has kept the Salton Sea's elevation relatively stable at 228 feet below sea level. A decreased use in irrigation water and subsequent decrease in inflow will cause the shoreline to recede resulting in a smaller Sea.

Prior efforts undertaken by Reclamation have considered alternatives focused on preserving/restoring the Whole Sea. These alternatives were determined to be infeasible leading to consideration of saving and/or managing a smaller Salton Sea. This then led to concepts of constructing barriers in the Salton Sea.

PRIOR STUDIES

In the past, various in-Sea restoration proposals have been offered by organizations such as the Pacific Institute, US Filter, and Black & Veatch. These proposals were generally descriptive and conceptual in nature employing an array of features such as construction of smaller, diked impoundments, reclamation of water for export, construction of barriers by dredging, use of in-sea evaporation ponds, etc. None of these proposals were developed to an extent such that conceptual designs for in-sea barriers were formulated.

Workshops

Focused efforts to develop concepts for engineering design and construction of in-sea barriers were initiated by Reclamation in conjunction with the Salton Sea Authority. A workshop was held on January 22, 2003 in La Quinta, California to brainstorm various ideas. This workshop was followed up by a second workshop on July 23, 2003 where participants concluded that a subsurface investigation of the Sea Bed deposits was needed to better develop concepts.

As a result, an investigation was initiated in September of 2003. The investigation was performed through use of a self-propelled jackup barge to explore and sample the Sea Bed deposits. A total of 11 borings and 17 cone penetration tests were completed over a wide area of the Salton Sea to depths ranging from 30 to 150 feet below the Sea Bed with a focus on a mid-sea transect aligned east to west. Soil samples were obtained from the borings and tested for moisture content, density, particle size, plasticity indices, strength and consolidation.

Preliminary results of the investigation were discussed in a third workshop, held in December 2003, in which staff from DWR participated. The results of the investigation, as well as the results of the soil testing and a limited number of analyses, were finalized in a February 27, 2004 report by URS, *Preliminary In-Sea Geotechnical Investigation Salton Sea Restoration Project* (URS, 2004b). At the third workshop, DWR staff indicated that they would perform analyses to determine the feasibility of a rockfill barrier.

A fourth workshop was convened in March 2004. The results of the four workshops, incorporating the results of the subsurface investigation, were documented in a report by URS, *Mid-Sea Dam and Barrier Concepts* (2004a). Both of these reports are available at the Salton Sea Authority's website:

www.saltonsea.ca.gov. DWR's rockfill concept, although not completely developed at the time of the March workshop, was also evaluated.

Prior Conceptual Designs

The following is a brief synopsis of the concepts contained in *Mid-Sea Dam and Barrier Concepts* report (URS, 2004a). The reader should consult the report for a full discussion and schematics of these concepts if greater detail is desired. It should be noted, however, that the concepts below were intended for a mid-sea location, allowing minimal seepage having the greatest potential head differential, provide 5 feet of freeboard, a 30-foot crest width, and a maximum barrier height in the range of 35 to 45 feet above the existing Sea Bed.

Seismic Dike

This concept consists of a conventional zoned embankment built "in-the-dry." The design includes slopes of 5 to 1 and 7 to 1 (horizontal to vertical) for the upstream and downstream slopes, respectively. Depths of overexcavation are 10 feet beneath the embankment crest increasing to 25 feet beneath the embankment toes. The report points out the difficulty associated with the use of extensive cofferdams, dewatering and staging of construction.

Deep Soil Mixing Cellular Dam

A cellular dam is created in segments whereby sheet piles are driven and filled by mixing soil and cement using large augers. The sheet piling is anticipated to corrode over time, leaving a solidified mass in place. The cellular dam would be proportioned and founded such that a stable configuration is provided.

Zoned Rockfill Dam

This concept proposes the use of rockfill for the outer shells of the embankment and soil for the core to be placed "in-the-wet." This method of construction does not allow for the compaction of materials. However, some densification of the core materials is anticipated. Depths of excavation and slope inclinations are the same as those for the Seismic Dike concept.

Blanketed Rockfill Dam

This concept eliminates the use of soil to form the core of the zoned rockfill dam. Depths of overexcavation are similar to the previous concepts. Slope inclinations are 4:1 and 7:1 up and downstream, respectively.

The report anticipates large seepage quantities which can be minimized by either or in combination of extending a slurry wall through the crest or deposition of finer grained materials on the upstream face.

Precast Concrete Caisson

Concrete caissons would be cast onshore, floated into position and filled with soil excavated either from the foundation or the surrounding area. Stability analyses indicated a required caisson diameter of 70 feet and a height of 88 feet corresponding to a Sea elevation of 230 feet below mean sea level.

Concrete Sheet Pile Dam

Parallel rows of the precast concrete sheet piles are driven into the foundation. Crossbeams would tie the pile tops together and the space between the rows would be filled with soil and densified. No over excavation would be required. The report notes that the thickness of the piles would need to be substantial.

Dumped Fill Dike

This concept utilizes soil dumped through water without densification. This concept was eliminated from further consideration due to concerns of instability during earthquake loading.

Geotube®

In 2004, a conceptual restoration concept entitled "The Cascade Alternative" by the Salton Sea Restoration Consortium (SSRC, 2004) was proposed which utilizes Geotube® as a component of in-sea Perimeter Dikes. In this concept, Perimeter Dikes are constructed in four concentric rings within the Sea Bed paralleling its perimeter along successively lower contour elevations. A total length of Perimeter Dike of approximately 320 miles is estimated for the four rings based on an average of 80 miles per ring. The Perimeter Dike height is limited to approximately 12 feet supporting a 10-foot water depth within the lakes. The tubes would be supported on the sides and covered with material either dredged or placed by clam shell.

In 2006, "The Cascade Alternative" was re-designed and re-named to the "Concentric Lakes Alternative" as proposed by the Imperial Group (Imperial Group, 2006). This alternative is one of the programmatic alternatives among the range of alternatives being evaluated in the PEIR. The principal difference from the original design was that in the Concentric Lakes Alternative the maximum water differential height would be limited to 6 feet and not fall under the California, Department of Water Resources, Division of Safety of Dams (DSOD) jurisdiction. This alternative would provide one partial ring water body and three whole ring water bodies located around the perimeter of the shoreline.

Geotube® has been used in various projects throughout the world to stabilize ocean beaches, contain toxic sediments, construct in-ocean islands, etc. Examples of their successful use as well as problems and limitations are readily found on the Internet. The manufacturer's website can be found at: www.geotube.com.

Geotube® consists of a geosynthetic fabric formed in the shape of an oval. The tubes are restrained by staking such that the desired shape is maintained as material is placed in the tube through ports spaced at regular intervals. Typically, the Geotube® is filled by materials suspended in a slurry created by dredging in-place soil. The geosynthetic fabric allows water to drain out, leaving the material confined in the tube.

Various sizes of geotubes are commercially available having circumferences of 45 and 60 feet and lengths of 100 and 200 feet. Custom sizes can be made depending on site requirements.

According to the Imperial Group, engineering feasibility studies are underway which will address significant challenges to the effectiveness and viability of Geotube® as components of in-sea barriers. These challenges were discussed in a February 2005 meeting with the industry (Miratech, 2005). An invitation was made for the industry to address the challenges listed below. Studies addressing these issues were not available at the time this report was finalized.

Construction and Performance Challenges

- **Sand Source –** The effectiveness of Geotube® depends upon a well-defined readily accessible source of sand which can be dredged. As described previously, the Sea Bed deposits are predominantly clay. Sandy materials were found in the upper alluvial layer, underlain by the soft clays, in the northeast, southwest and west-central margins of the Salton Sea. Extensive exploration will be required to define the areas and limits of sandy deposits within the Sea Bed deposits. Work stoppage during construction due to a lack of sand will result in increased costs. As an alternative, although more expensive, sand could be imported.

- **Environmental Consequences –** Selenium has been found to be concentrated in the Sea Bed sediments. As is the case for several of the alternatives, disturbing these materials will likely

re-introduce selenium into the water column and may pose an environmental risk that would have to be considered and evaluated.

- **Bearing Capacity –** The seafloor deposits are typified by a highly compressible low strength clay. The load of the Geotube® tends to be concentrated by way of its oval shape and may exceed the available bearing capacity. To increase the bearing capacity, a geogrid could be employed to strengthen the foundation. Additionally, a design would have to account for differential settlements along the tube's alignment.

- **Armouring –** The Geotube® would have to be armoured by either rock or soil. Some projects have utilized rock which poses a puncture hazard to the geosynthetic material. To mitigate this, a geosynthetic shield has been used and the drop height limited to 3 feet. Sand cover, used in other projects, has been shown to be highly susceptible to erosion by wave action if not stabilized by an adequate vegetative cover.

- **Buttressing –** Because of the area's high seismicity, buttressing the Geotube® with hydraulically dredged material will result in a highly unstable mass susceptible to liquefaction. Hydraulically dredging materials to construct a barrier was proposed in the Dumped Fill Dike concept which was eliminated from consideration as an alternative because of its instability during seismic loading. Materials placed by clamshell are similarly anticipated to result in an unstable mass. An in-sea barrier's cross-section, with or without a Geotube®, would require flat slope inclinations to ensure integrity during earthquake loading. As such, the cross-sectional area of the Geotube® would represent only a portion of the barrier's total cross-section.

Liquefaction of Foundation Soils

The liquefaction potential of the foundation for the Perimeter Dikes is a concern. This is a concern under any alternative involving construction of structures on the Sea Bed. Liquefaction mitigation measures are available such as replacement of soil and dynamic compaction.

Cost of Ownership

Construction cost for Geotube® is estimated to range from $20 to $25 per cubic yard in place. This range of costs assumes that the foregoing challenges can be overcome and that no additional geosynthetics need be employed to either shield the Geotube® or strengthen the foundation. The appropriateness of this cost range was confirmed by Miratech (2005). A total cost of ownership, which includes the maintenance and repair of barriers constructed with Geotubes® as components over the project's life, should be considered.

Foundation Investigations

The *Mid-Sea Dam and Barrier Concepts* report (URS, 2004a) cites the following past investigations aimed at characterizing the seafloor sediments:

- Reclamation conducted an exploration program to evaluate the feasibility of constructing a 37-mile long barrier located in the Southeastern end of the Sea Bed. This effort consisted of manually pushing a ¾-inch diameter rod into the sediments at 60 sites, 7 borings and geophysical studies to characterize the deposits. (Reclamation, 1974)

- Regional geophysical surveys and subsurface investigations have been conducted for development of geothermal resources at the southern end of the Sea Bed. These studies were conducted in the fall of 1985 by Lawrence Livermore and Sandia National Laboratories. Samples of the cuttings were collected at 10-foot intervals as part of the drilling program.

- Sediment characterization efforts were conducted by Levine-Fricke (LFR, 1999) and Agrarian Research (Agrarian Research, 2003). These efforts consisted of obtaining grab samples of the near-surface-sediments to determine chemical compositions and classification of soil types for air quality emissivity studies.

The subsurface investigation initiated in September 2003 used to supplement the above information. The September 2003 investigation consisted of 11 boring and 17 Cone Penetration Tests, with locations as shown in Table H4-1.

Table H4-1
URS 2004 Report Tabulation of Borings and Cone Penetration Tests

	Borings	Cone Penetration Tests
North Area	2	4
Mid-Sea	6	7
South Area	10	6
Total	**18**	**17**

The concentration of exploration locations in the mid-sea and southern areas of the Salton Sea is indicative of investigations conducted to support specific restoration proposals. A review of the subsurface stratigraphy indicates that a general description and interpretation of the Sea Bed deposits can be adequately made to support the development and evaluation of programmatic alternatives. As such, no additional subsurface investigations were deemed necessary for the purposes of the PEIR.

It is understood that once a preferred alternative is identified, additional subsurface investigations will likely need to be undertaken along alignments of any proposed barriers. A brief description of the Sea Bed's stratigraphy is provided based on the data contained in URS, 2004 report. The reader is encouraged to read the report if a more detailed description, logs of borings, and CPT results are desired.

It is noted that Reclamation is currently collaborating with the Salton Sea Authority to perform additional geotechnical investigation as part of the Reclamation's Phase II Feasibility Study to further explore subsurface conditions at proposed locations for in-sea embankments (URS, 2006). One focus of this investigation is to better assess the liquefaction potential of the Sea subsurface. The scope of the investigations may include: barrier concepts, In-Sea geotechnical investigation, quarry reconnaissance investigation (of Coolidge Mountain), and revised concepts and unit costs. The exploration is anticipated to begin in spring or summer of 2006; however, the results will not likely be completed in time to be incorporated into the State's PEIR.

The URS report (URS, 2004b) identified six dominant and distinct layers comprising the subsurface deposits:

1. The first layer, Sea Bed deposits, is comprised of recently deposited very soft to loose highly plastic clays to silty fine sands. The thickness of this layer ranged from 0 to 21 feet, with the greatest thickness occurring in the southern and mid-Sea areas. (It is noted that throughout this the PEIR and this appendix, the term "Sea Bed" is used to describe the existing bottom elevation of the Salton Sea, while the term Sea Bed deposits is used to described the first 21 feet of soils below the existing bottom elevation).

2. Soft lacustrine deposits were found to underlie the Sea Bed deposits over much of the Salton Sea's area. These materials consist of highly plastic soft to very soft clays ranging in depth from 0 to 26 feet. The thickest deposits were found in the Whitewater River delta and the mid-sea's easterly area.

3. Upper alluvial deposits are interspersed between the soft and stiff lacustrine deposits and are predominant near the Salton Sea's perimeter. These deposits are described as composed of loose to dense silty fine sands with interbedded silt and sand lenses ranging in thickness of 0 to 26 feet. The thickest deposits were found in northeast, southwest and west-central margins of the Salton Sea.
4. Upper stiff lacustrine deposits, underlying both the soft lacustrine and upper alluvial deposits, are comprised of predominantly stiff to very stiff, highly plastic clays ranging in thickness from 4 to 31 feet. The thickest deposits were found in the mid Salton Sea's eastern and southeastern areas, the latter near the Alamo River Delta.
5. Lower alluvial deposits are similar to the upper alluvial deposits except that their density is greater, ranging in consistency from medium dense to dense. These deposits were predominant in the southern area of the Sea Bed, ranging from 0 to 22 feet in thickness.
6. The report assumes the lower stiff lacustrine deposits underlie the entire Salton Sea having, with a thickness much greater than the limit of exploration of 103 feet in depth. This layer is described as a primarily hard highly plastic clay.

California Department of Water Resources, Division of Safety of Dams Peer Review of Bureau of Reclamation Appraisal Level Design

Under Public Law 108-361, the Secretary of the Interior was required to complete a feasibility study on a preferred alternative for Salton Sea restoration. In October 2005, the DWR Division of Safety of Dams (DSOD) performed a peer review of Reclamation's 2005 Salton Sea Restoration Project Feasibility Study, Phase 1 (DWR, 2005). The review was focused on the need to treat the foundation for dams and barriers constructed within the Salton Sea. The conclusions gained from this peer review included:

- Reclamation's appraisal level design exceeds DSOD's design standards and is judged to attain a high level of performance under all severe loading conditions.
- In the case of jurisdictional impoundments for the restoration alternatives, the risk of life and property downstream of a dam is determined to be minimal.
- Considering the limited amount of subsurface data compared to the size of the Salton Sea, Reclamation assessment that liquefiable materials are present everywhere within the Salton Sea's footprint is judged to be conservative.

DSOD's design criteria to meet minimum factors of safety and to have sufficient freeboard can be met by utilizing slopes that are appropriately inclined, treating the foundation, or a combination of both.

ROCKFILL BARRIER CONCEPTUAL DESIGN

Description

Construction of in-sea barriers "in-the-wet" by placement of rockfill materials is deemed to be an economical and viable alternative. Construction in-the-wet would involve the placement of rockfill materials without the use of coffer dams or other means to create dry construction conditions. For engineering design purposes, it is desirable that the rockfill materials be well-graded having a maximum particle size in the range of 4 to 5 feet for barrier heights greater than 25 feet. For lesser heights, the maximum particle size is to be decreased so that a ratio of five is maintained between barrier height and maximum particle size.

This gradation will help prevent a flocculated structure from being created when placed in-the-wet and will result in a medium dense, highly permeable fill. The high permeability will help prevent a build up of pore pressures from cyclic shear stresses as a result of seismically induced ground motions. Placement in-the-wet of the finer-grained materials by dredging the Sea Bed deposits, which consist primarily of fat clays and localized sand lenses, will likely result in an unstable flocculated structure.

Slope Inclination

The slope inclination of the rockfill is governed by the strength of the clays within the foundation. Because the strength of the clays does not materially increase in depth within the upper layers of the Sea Bed, it is more economical to flatten the slope of the rockfill than it is to excavate any of the clays. Simply put, every cubic yard of Sea Bed removed will have to be filled with a cubic yard of rockfill at a cost that results in little to no increase in stability. Additionally, it has been found that the near surface, Sea Bed deposits contain selenium, which if introduced into the food web, may lead to detrimental effects on wildlife. Hence, not disturbing these deposits by excavation or dredging will avoid the currently sequestered selenium from being reintroduced into the water column.

A primary factor in the development of a conceptual design is the determination of an appropriate slope inclination of rockfill required to limit the shearing in the foundation clays during the design earthquake load. An initial iteration is made to limit the cumulative displacement of the clays in the shear zone within the foundation to three feet. The analyses then estimate the depth over which this displacement occurs to determine if pore pressure, shear strain and displacement time histories are limited such that post earthquake stability is ensured.

The goal of limiting the cumulative displacement to three feet is based on observed performance of embankment dams during significant earthquake loading. This performance indicates that displacements greater than three feet will increasingly impair the structure's ability to prevent an uncontrolled release of impounded water. A rockfill structure is best able to accommodate this magnitude of displacements as well as displacements due to fault offsets in the foundation.

Permeability

The rockfill will constitute a highly permeable barrier. The permeability can be increased or decreased as dictated by the needs of the various conceptual restoration alternatives. A decrease in permeability is to be achieved by placement of finer-grained materials on the upstream slope. This may require the placement of two or more successively finer-grained materials such that the gradations are compatible.

Placement of the finer-grained material on the upstream slope is preferred over placement in the core. If placed in the core, the finer-grained materials' low strength will compromise, to a degree, the integrity and performance of the rockfill. This is avoided by placing these materials on the upstream slope.

The design concept can be applied to barriers having varying heights by adjusting the maximum particle size such that a ratio of barrier height to maximum particle size of five is maintained.

Seismically Induced Permanent Deformation

The conceptual design allows for a given amount of displacement during the maximum credible earthquake load. Given the severity of this earthquake load, a design that precludes permanent displacements from occurring is neither feasible nor practical from an engineering standpoint.

The design is based on limiting the displacements within the Sea Bed deposits to less than three feet at points A and B in Figure H4-1 which are governed by the slope inclinations in conjunction with the shear strength of the Sea Bed deposits. This magnitude of displacement serves as a design goal and is based on observed performance of dams and embankments during earthquake loading. This performance suggests that, as deformations become increasingly greater than three feet, the structure's integrity becomes more and more impaired such that an uncontrolled released becomes increasingly likely.

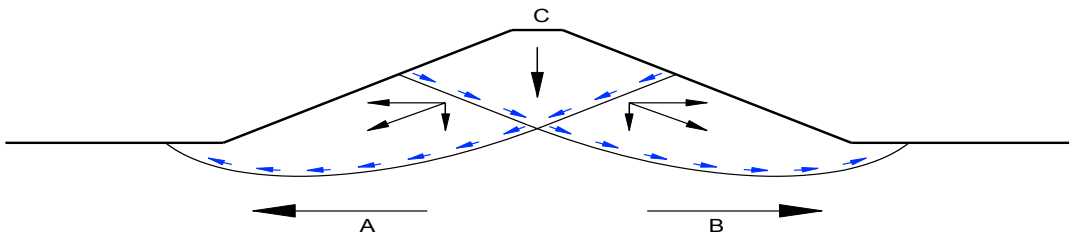

FIGURE H4-1
SEISMICALLY INDUCED DEFORMATIONS

The methodology of calculating permanent deformations is based on identification of the structure's yield acceleration. The yield acceleration is that acceleration, which if exceeded, will result in permanent irrecoverable deformations.

Accelerations less than the yield acceleration result in an elastic response - in this type of response, the accelerations at the base of the structure are amplified at the top. The resulting permanent deformations are minor and do not adversely impact the structure's integrity.

Accelerations greater than the yield acceleration cause plastic response where shearing occurs within the Sea Bed deposits. It is the plastic response of the foundation and rockfill that is considered in this report since this response governs the design. To better understand plastic response, the reader is referred to Figure H4-1 which shows the deformation of both slopes to input accelerations. Deformation of the crest at a point C is a result of vectorially adding the deformation of each slope. For example, if both the up and down stream slopes experience the same deformation, the crest's horizontal will be zero, while the vertical deformation will be double that of the slope's.

Future Investigations

It is anticipated that additional investigations will be required to develop preliminary and final designs of in-sea barriers should a preferred restoration alternative requiring barriers be selected.

A subsurface profile along the barrier's alignment would be required to identifying soil characteristics with an emphasis on shear strength. Shear strength is one of the most critical parameters to evaluate and should be correlated to the shear strength identified by field performance tests. These tests are not simple

to carry out and are probably best done as part of a construction contract. Laboratory strength tests and various finite element or other mathematical models are not a substitute for field tests.

Field tests will establish the relationship between slope inclination of the rockfill and the static shear strength of the seafloor deposits. This relationship is extrapolated to determine the slope inclination required for a given design ground motion. Hence, it is important that this relationship be accurately established.

The Great Salt Lake Causeway serves as an excellent case history in this regard. It was found that shear strength obtained from carefully tested undisturbed samples significantly underestimated the in situ strength. The existence of a salt crust, up to five feet thick in places along the alignment, was a contributing factor to difficulties experienced during construction. The design of the causeway was based on the field performance of failures during construction. The design approach was to construct the causeway such that the required strength of the Lake's deposits was slightly less than its limits. This approach was found to be more cost effective than basing a design on a conservative shear strength evaluation and applying a factor of safety for the entirety of the causeway's 12 mile length.

Once in-situ shear strengths are determined, static and dynamic mathematical modeling should be carried out to optimize the cross sectional characteristics and thereby minimize the volume of rock.

The most critical component of the seepage analysis is the selection of the estimated permeability of the proposed in-sea barrier rockfill. Field permeability tests should be conducted on test rockfill sections to verify permeability.

Preliminary geotechnical data suggest that there is a potential for liquefaction in silty/sandy soils within the foundation layers. Further data is warranted to evaluate extent of the liquefaction potential of the subsurface deposits.

GEOLOGIC SETTING

Introduction

The Salton Sea occupies a portion of the interior-draining Salton Basin. The southern end of this basin has been blocked by the deposition of deltaic sediments from the Colorado River effectively preventing drainage from the basin to the Gulf of California. The Salton Sea watershed encompasses an area of approximately 8,000 square miles from San Bernadino County in the north to the Mexicali Valley (Republic of Mexico) to the south. The several watersheds (Figure H4-2) that drain into the sea include the White Water River from the Transverse and Peninsular Ranges to the north-northwest, Salt Creek from the Orocopia and Chocolate Mountains to the east, and San Felipe Creek which drains the Peninsular Ranges to the West. The largest flow into the sea comes from the Imperial Valley to the south via the Alamo and the New Rivers. These rivers convey irrigation return flows as they pass through the extensively irrigated Imperial Valley.

Structure

The Salton basin is located in the Salton Trough, a deep north-west trending structural depression that extends from San Gorgonio Pass to the Gulf of California. The Salton Trough, located in the Colorado Desert geomorphic province, is an area of transition from the transform boundary of the North American and Pacific Plates and the echelon right stepping faults and extension of the Gulf of California and represents the structural extension of the Gulf of California into North America.

Extension occurred in the region between 22 and 14 million years ago and is marked by the Alverson Andesite and Jacumba Volcanics (Van Gilder, 2000). Down-dropped margins created during the extension became the site of alluvial and fanglomerate deposition. The rock found in the down-dropped

margins include crystalline and metamorphic basement rock, reworked sediments derived from the Colorado River, and Lower to Middle Miocene (22 to 14 million years ago) volcanic rocks.

The Salton Trough is bounded to the North by the Transverse Ranges geomorphic province, to the Northeast by the Mojave Desert geomorphic province, and to the west by the Peninsular Ranges geomorphic province. Northwest-trending faults and associated folding cross the Salton Basin, the Imperial Valley, and the mountains to the west. These faults are predominately right-lateral and can be divided into three main fault zones, the San Andreas, San Jacinto, and Elsinore. The Imperial fault was considered to be a part of the San Jacinto Fault Zone but will be included in the Imperial-Brawly Seismic Zone discussed in the faulting portion of this text.

Regional Geology

The oldest exposed rocks in the region surrounding the Salton Trough are Precambrian gneisses, anorthosites, and schists. These rocks are in turn intruded by younger Paleozoic to Cenozoic plutonic rocks.

The sediments within the Salton Trough range in age from Miocene (23.8 to 5.3 million years ago) to Holocene (within the last 10,000 years). The Salton trough is a large structural depression that has filled with about 3.7 miles of sediment since the late Cenozoic (within the last 65 million years). The oldest sediments are coarse clastic sediments (sediment consisting of broken fragments derived from preexisting rocks and transported elsewhere and redeposited before forming another rock) derived from the surrounding crystalline rocks. These deposits are overlain by essentially continuous deposits of volcanics, lacustrine, evaporites, marine, fluvial and deltaic sediments. The greatest source of sediment is from the Colorado River.

The only marine formation, the Imperial Formation, was deposited during a marine incursion that occurred not long after the initiation of the opening of the Gulf of California about five million years ago. Discontinuous outcrops of the formation are found from just south of the U.S.-Mexico Border to San Gorgonio Pass. This formation may be as old as late Miocene but is generally considered to be Pliocene (5.3 to 1.8 million years ago). The marine rocks at the northern end of the formation are thought to be Miocene and may not be correlative with the marine rocks found to the south. These rocks may predate the opening of the Gulf of California and represent a proto-Gulf (McDougall, 1999).

Faulting

The Coachella Segment of the San Andreas fault forms the northeastern boundary of the Salton Trough. The fault is evident on the ground surface from north of the Salton Sea just north of Bombay Beach located on the east shore of the Salton Sea but is not evident on the ground surface to the southeast of the sea. There has been no historic seismicity on this segment of the San Andreas fault. The latest break on this segment is likely greater than 300 years ago (Sieh and Williams, 1990). With an estimated accumulated strain of about 25 mm per year, there is a possibility that this segment could produce an earthquake with a magnitude of about M7.5 or larger with over twenty feet of offset. The San Jacinto Fault Zone is located just to the west of the Salton Sea and is comprised of a complex system of faults including the San Jacinto, San Felipe Hills, Santa Rosa, San Felipe, Superstition Hills, Superstition Mountain, Coyote Creek, and the Imperial (California Department of Conservation, 1977). The Imperial Valley, located just south of the Salton Sea, is one of the most seismically active regions in Southern California. The Imperial fault produced the magnitude 6.9 Imperial Valley earthquake in 1940. The Elsinore fault zone is located west of the San Jacinto Fault Zone and borders the southwest face of the Coyote Mountains.

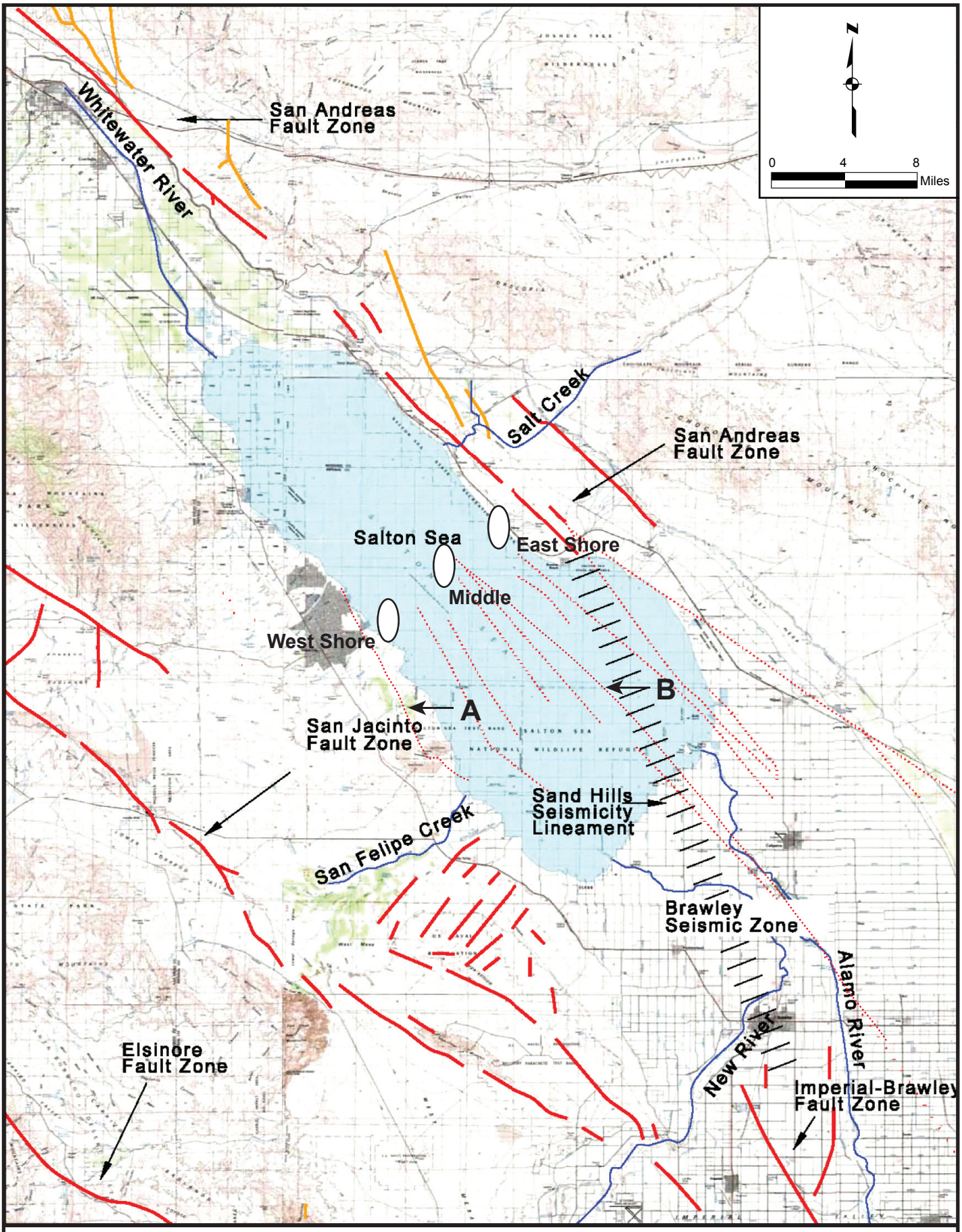

FIGURE H4-2
SALTON SEA RIVERS AND FAULT ZONES

Source: DWR, 2006

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Several faults have the potential to affect the Salton Sea and surrounding area. The closest seismic source is the Coachella segment of the San Andreas fault. This section has not ruptured during historic times and is thought to only rupture in multi segment events with the San Bernardino segment. According to DMG OFR 96-08, a slip rate of about 25 mm per year is believed to occur on the Coachella segment of the San Andreas fault. Since this segment of the fault is only expected to rupture during multi segment events, a M7.5 earthquake is expected if it were to rupture with the San Bernardino segment. A larger magnitude earthquake could possibly occur if more segments were involved.

Historically the most active seismic source affecting the Salton Sea has been the San Jacinto Fault Zone. Most notably, the M6.2 Elmore Ranch and M6.6 Superstition Hills earthquake sequence in 1987, and the M6.4 Borrego Mountain earthquake in 1968. Slip from the Borrego Mountain earthquake was observed on several of the strands of the San Jacinto Fault Zone with the greatest amount of slip occurring on the Coyote Creek fault. Sympathetic movement was also observed on a portion of the Coachella segment of the San Andreas fault (USGS, 2000).

In the Salton Trough, the most dominating structural feature is the San Andreas Fault System. Most major structural features in the region trend northwest-southeast. This trend is the result of three major northwest-southeast trending fault zones: the San Andreas, San Jacinto, and Elsinore. These fault zones, along with regional extension, account for the current structure of the region. The primary displacement along these fault zones is right-lateral; however, vertical displacement has also occurred.

San Andreas Fault

The San Andreas fault enters the Salton Trough at the northwest end of the Coachella Valley. This fault system constitutes the main structural boundary between the Pacific and North American plates.

Today, the San Andreas fault zone is traceable from the Gulf of California northward to Shelter Cove Coast in Humboldt County. Regionally, it is traceable from the town of Niland east of the Salton Sea northward through San Gorgonio pass. The fault zone continues southward into Mexico as the Sand Hills and Algodones fault. The San Andreas fault is right-lateral with an approximate offset of 200 miles. The offset in southern California is estimated to have begun in the late Miocene and early Pliocene (10 to 5 Million years ago) (Van Gilder, 2000).

San Jacinto Fault Zone

The San Jacinto Fault Zone is a major strand of the San Andreas Fault System. It extends southeastward from Cajon Pass as a series of splays into the Salton Trough.

The San Jacinto fault is an extremely active system. Right lateral displacement on the San Jacinto fault zone is approximately 19 miles. Vertical separations along the zone exceed 8,000 feet in the Santa Rosa Mountains. The San Jacinto fault is thought to be Plio-Pleistocene based on vertebrate and plant remains but may be younger than 1.0 million years as indicated by lateral offset of the late Pleistocene Ocotillo Conglomerate (Van Gilder, 2000).

Elsinore Fault Zone

The Elsinore fault zone extends from the northern Peninsular Range southward to the Gulf of California. The fault is parallel and west of the San Jacinto fault zone. Right lateral displacement along the main fault trace is approximately 30 miles. Vertical displacement and relief features along this fault reach as much as 9,000 feet. The Elsinore fault zone is considered to be older than the San Jacinto fault, between 1.8 and 2.7 Million years ago (Van Gilder, 2000).

Brawley Seismic Zone

The Brawley seismic zone is comprised of the Imperial-Brawley fault system and is a zone of high seismicity extending from the northern reach of the Imperial fault northwest into the Salton Sea. This zone is marked by en echelon northwest-trending right-lateral faults linked by conjugate left-lateral structures (Larson and Reilinger, 1991). The Sand Hills Seismicity Lineament extends southeast from the southern tip of the San Andreas fault within this seismic zone and may represent the southern extension of the San Andreas fault.

Seismicity

Introduction

It is likely that in-sea barriers, including Perimeter Dikes, will fall under the jurisdiction of the California Department of Water Resources, Division of Safety of Dams (DSOD). See Figure H4-3 for criteria to determine DSOD jurisdiction. Dams under the jurisdiction of DSOD are defined in the California Water Code. According to the California Water Code; Division 3, Dams and Reservoirs; Part 1, Supervision of Dams and Reservoirs; Chapter 1, Definitions:

- 6002. "Dam" means any artificial barrier, together with appurtenant works, which does or may impound or divert water, and which either (a) is or will be 25 feet or more in height from the natural bed of the stream or watercourse at the downstream toe of the barrier, as determined by the department, or from the lowest elevation of the outside limit of the barrier, as determined by the department, if it is not across a stream channel or watercourse, to the maximum possible water storage elevation or (b) has or will have an impounding capacity of 50 acre-feet or more.
- 6003. Any such barrier which is or will be not in excess of 6 feet in height, regardless of storage capacity, or which has or will have a storage capacity not in excess of 15 acre-feet, regardless of height, shall not be considered a dam.

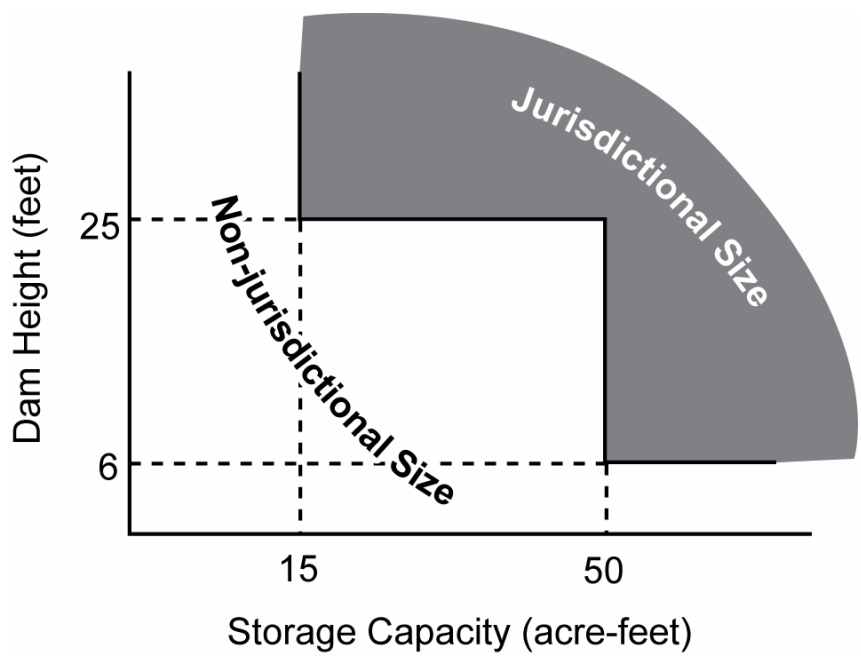

FIGURE H4-3
CALIFORNIA DEPARTMENT OF WATER RESOURCES, DIVISION OF SAFETY OF DAMS, JURISDICTIONAL DAM SIZE CRITERIA (DWR DSOD, 2006)

DSOD uses several criteria when evaluating new and existing dams. These criteria are different than those used by other departments for structures such as hospitals and bridges. The California Geological Survey, for example, will accept only a probabilistic seismic hazard analysis when evaluating ground motions for hospitals and public schools. Caltrans, on the other hand, makes use of a probabilistic or deterministic seismic hazard analyses depending upon the criticality of the structure.

A *probabilistic seismic hazard analysis* takes into consideration the uncertainties in the magnitude and location of earthquakes and the resulting ground motions that can affect a particular site. A *deterministic seismic hazard analysis* identifies nearby faults and assesses their activity. For each seismic source, an earthquake scenario consisting of the maximum magnitude a fault is capable of generating at the closest distance to the site under consideration is specified as the basis for the ground motion estimate (DWR DSOD, 2001).

A probabilistic seismic hazard analysis, expressed as a hazard curve, can be used to evaluate the conservatism of deterministically derived ground motion parameters (DWR DSOD, 2002). Deterministically derived ground motions are not always the most conservative. When appropriate, probabilistic seismic hazard analyses can be used to select the appropriate level of design.

The activity of a fault for the purpose of dams is defined as a fault that has ruptured within the last 35,000 years. DSOD chose the 35,000-year value based on the belief that "Holocene activity (the last 10,000 years) is not a sufficiently conservative criterion for elimination of a fault when estimating ground motion for dam" (DWR, 2001).

The DSOD guidelines suggest that the 50th and 84th percentile of both peak ground acceleration (PGA) and spectral accelerations (SA) be determined as a function of magnitude, distance, fault type, and site condition. Directivity affects, which typically result in the amplification of long period energy generally in directions perpendicular to the fault rupture plane, are also evaluated.

To aid in the determination of the appropriate statistical level of ground motion to use, DSOD has developed a Consequence-Hazard Matrix (Figure H4-4). This matrix considers the consequence of a dam failure and the likelihood of an earthquake. The slip rate of the causative fault is used to determine the likelihood of an earthquake, while an assessment of potential damage is used to assess the consequence of dam failure (DWR, 2002).

DSOD has indicated, in preliminary and informal discussions (Waggoner, 2004), that the Hazard Classification for jurisdictional barriers within the Salton Sea would constitute a hazard of "Moderate Consequence." This classification, in conjunction with a "High Slip Rate" for Coachella Segment of the San Andreas Fault, suggests a 50th to 84th percentile level of acceleration as shown in Figure H4-4.

Upon selection of a preferred alternative, should it employ jurisdictional sized barriers as a component of the restoration, further discussions with DSOD are recommended for an assessment of hazard classification.

Deterministic Seismic Hazard Analysis

Because of its proximity to any location within the Salton Sea, in conjunction with a potential for about a M7.5 to M7.8 earthquake, the Coachella Segment of the San Andreas fault with the assumption of a multiple segment rupture was used in the deterministic analysis.

Three attenuation relationships were used for a deterministic analysis of potential sites. The attenuation relationships were based on those from Seismological Research Letters (SRL) and include several researchers: Abrahamson and Silva; Boore, Joyner, and Fumal; and Sadigh et. al. (Seismological Society of America, 1996).

The weighted average of peak ground acceleration (PGA) from the three attenuation relationships was determined for specific areas on the eastern shore, the western shore, and the middle of the sea. These

	Very High Slip Rate 9 or greater mm/yr	High Slip Rate 8.9 to1.1 mm/yr	Moderate Slip Rate 1.0 to 0.1 mm/yr	Low Slip Rate less than 0.1 mm/yr
Extreme Consequence Total Class Weight 31-36	84th	84th	84th	50th to 84th
High Consequence Total Class Weight 19-30	84th	84th	50th to 84th	50th to 84th
Moderate Consequence Total Class Weight 7-18	84th	50th to 84th	50th to 84th	50th
Low Consequence Total Class Weight 0-6	50th	50th	50th	50th

FIGURE H4-4
DSOD CONSEQUENCE-HAZARD MATRIX. USED TO DETERMINE THE APPROPRIATE STATISTICAL LEVEL OF ACCELERATION FOR DETERMINISTIC SEISMIC HAZARD ANALYSES (DWR DSOD, 2002)

areas are labeled West, Middle, and East Shore on Figure H4-2. A magnitude 7.8 event on the San Andreas fault was used as the seismic source for all three areas.

Because this fault trends parallel to both the West and Eastern shores, the values determined for these specific areas apply along lines parallel to both shores and, by interpolation between lines, are utilized to determine the seismicity of any area within the Salton Sea.

Table H4-2 shows results for the PGA for each of the three locations. The spectral accelerations from 0.0 to 3.0 or 5.0 seconds for each area are presented in tabular format in Table H4-3 through Table H4-5.

Table H4-2
Peak Ground Accelerations for a M7.8 Earthquake on the Southern San Andreas Fault for Three Locations

Location	Source	Distance (miles)	PGA (g)		
			50th	84th	Ave
Western Shore 33.28º N, 115.89º W	San Andreas	10.7	0.29	0.45	0.37
Middle 33.33º N, 115.84º W	San Andreas	6.3	0.39	0.61	0.5
Eastern Shore 33.37º N, 115.78º W	San Andreas	2	0.57	0.9	0.74

Table H4-3
Horizontal Spectral Acceleration for the West Shore of the Salton Sea (near the middle of the sea) from a Magnitude M7.8 Earthquake on the Southern San Andreas Fault

Period (s)	AS 97		Sadigh 97		BJF 97 Mw 7.5		Weighted Average	
	50th	84th	50th	84th	50th	84th	50th	84th
0	0.26	0.40	0.31	0.46	0.30	0.50	0.29	0.45
0.03	0.26	0.40	0.38	0.57	0.34	0.56	0.33	0.51
0.05	0.28	0.44	0.43	0.65	0.37	0.61	0.36	0.56
0.075	0.33	0.53	0.49	0.74	0.41	0.66	0.41	0.64
0.1	0.39	0.62	0.59	0.89	0.44	0.71	0.47	0.74
0.12	0.44	0.71	0.62	0.95	0.48	0.77	0.51	0.81
0.15	0.50	0.82	0.68	1.05	0.52	0.85	0.57	0.91
0.17	0.55	0.90	0.72	1.11	0.55	0.90	0.60	0.97
0.2	0.60	0.98	0.77	1.20	0.58	0.96	0.65	1.05
0.24	0.64	1.05	0.78	1.23	0.62	1.03	0.68	1.10
0.3	0.66	1.11	0.80	1.26	0.65	1.10	0.70	1.16
0.4	0.66	1.11	0.77	1.24	0.66	1.13	0.70	1.16
0.5	0.64	1.09	0.71	1.16	0.63	1.10	0.66	1.12
0.75	0.57	1.00	0.61	1.02	0.51	0.92	0.56	0.98
1	0.49	0.88	0.53	0.91	0.42	0.77	0.48	0.85
1.5	0.38	0.70	0.41	0.73	0.30	0.57	0.36	0.67
2	0.29	0.54	0.33	0.58	0.23	0.45	0.28	0.53
3	0.17	0.33	0.22	0.40	0.00	0.00	0.19	0.36

Table H4-4
Horizontal Spectral Acceleration for the Middle of the Salton Sea from a Magnitude M7.8 Earthquake on the Southern San Andreas Fault

Period (s)	AS 97		Sadigh 97		BJF 97 Mw 7.5		Weighted Average	
	50th	84th	50th	84th	50th	84th	50th	84th
0	0.36	0.55	0.39	0.58	0.42	0.71	0.39	0.61
0.03	0.36	0.55	0.48	0.73	0.49	0.81	0.44	0.70
0.05	0.38	0.59	0.54	0.82	0.54	0.89	0.49	0.77

Table H4-4
Horizontal Spectral Acceleration for the Middle of the Salton Sea from a Magnitude M7.8 Earthquake on the Southern San Andreas Fault

Period (s)	AS 97		Sadigh 97		BJF 97 Mw 7.5		Weighted Average	
	50th	84th	50th	84th	50th	84th	50th	84th
0.075	0.45	0.71	0.62	0.94	0.60	0.98	0.56	0.88
0.1	0.53	0.84	0.74	1.13	0.66	1.07	0.64	1.01
0.12	0.60	0.96	0.79	1.21	0.70	1.14	0.70	1.10
0.15	0.69	1.12	0.86	1.33	0.76	1.25	0.77	1.23
0.17	0.75	1.23	0.91	1.41	0.80	1.31	0.82	1.31
0.2	0.82	1.34	0.98	1.52	0.85	1.41	0.88	1.43
0.24	0.86	1.43	0.99	1.55	0.91	1.52	0.92	1.50
0.3	0.90	1.49	1.01	1.60	0.97	1.63	0.96	1.57
0.4	0.89	1.49	0.98	1.57	0.99	1.69	0.95	1.58
0.5	0.86	1.47	0.90	1.47	0.95	1.65	0.90	1.53
0.75	0.76	1.35	0.77	1.29	0.77	1.39	0.77	1.34
1	0.65	1.18	0.67	1.15	0.62	1.15	0.65	1.16
1.5	0.50	0.93	0.52	0.92	0.44	0.84	0.49	0.90
2	0.37	0.70	0.41	0.74	0.33	0.65	0.37	0.70
3	0.22	0.42	0.28	0.50	0.00	0.00	0.25	0.46
4	0.14	0.28	0.20	0.35	0.00	0.00	0.17	0.32
5	0.10	0.21	0.00	0.00	0.00	0.00	0.10	0.21

Table H4-5
Horizontal Spectral Acceleration for the East Shore of the Salton Sea (near the middle of the sea) from a Magnitude M7.8 Earthquake on the Southern San Andreas Fault

Period (s)	AS 97		Sadigh 97		BJF 97 Mw 7.5		Weighted Average	
	50th	84th	50th	84th	50th	84th	50th	84th
0	0.54	0.83	0.51	0.76	0.66	1.11	0.57	0.90
0.03	0.54	0.83	0.63	0.95	0.79	1.30	0.65	1.03
0.05	0.56	0.88	0.71	1.07	0.87	1.43	0.71	1.13
0.075	0.67	1.05	0.81	1.23	0.97	1.58	0.82	1.29
0.1	0.78	1.25	0.97	1.47	1.08	1.74	0.94	1.49
0.12	0.89	1.44	1.03	1.58	1.10	1.78	1.01	1.60
0.15	1.04	1.69	1.12	1.73	1.16	1.90	1.11	1.78
0.17	1.15	1.87	1.18	1.83	1.22	2.00	1.18	1.90
0.2	1.26	2.07	1.27	1.99	1.31	2.17	1.28	2.07
0.24	1.33	2.19	1.29	2.03	1.43	2.39	1.35	2.20
0.3	1.38	2.29	1.32	2.08	1.58	2.66	1.42	2.35
0.4	1.38	2.32	1.27	2.05	1.74	2.97	1.46	2.45
0.5	1.34	2.30	1.17	1.92	1.77	3.08	1.43	2.43
0.75	1.22	2.16	1.01	1.68	1.57	2.82	1.26	2.22
1	1.05	1.89	0.87	1.50	1.27	2.34	1.06	1.91

Table H4-5
Horizontal Spectral Acceleration for the East Shore of the Salton Sea (near the middle of the sea) from a Magnitude M7.8 Earthquake on the Southern San Andreas Fault

Period (s)	AS 97		Sadigh 97		BJF 97 Mw 7.5		Weighted Average	
	50th	84th	50th	84th	50th	84th	50th	84th
1.5	0.78	1.46	0.68	1.20	0.81	1.54	0.76	1.40
2	0.56	1.07	0.54	0.96	0.52	1.02	0.54	1.02
3	0.33	0.64	0.36	0.66	0.00	0.00	0.35	0.65
4	0.21	0.42	0.25	0.46	0.00	0.00	0.23	0.44
5	0.15	0.31	0.00	0.00	0.00	0.00	0.15	0.31

Contours of equal median peak ground accelerations from 0.3g to 0.63g and median plus one standard deviation (84th percentile) from 0.3g to 1.0g were derived using the deterministic approach (See Figure H4-5 and Figure H4-6). Each fault zone was evaluated deterministically using the maximum magnitude listed in Appendix A of the Revised 2002 California Probabilistic Seismic Hazard Maps, a revision of CDMG Open-File report 96-08 (California Department of Conservation, 1996). Soil velocities used for the analysis were 690 ft/s within the sea and 1020 ft/s everywhere else. Dashed contours were used for areas where there was uncertainty as to the extension of the San Andreas fault south of the Coachella segment. These maps are not a site specific deterministic analysis but represent an approximation of the horizontal acceleration in and around the Salton Sea.

Figure H4-7 shows the locations of earthquakes with magnitudes greater than 2 within a 50 mile radius of a point in the middle of the Salton Sea. Figure H4-8 shows the locations of earthquakes with magnitudes greater than 4 within a 50 mile radius of a point in the middle of the Salton Sea.

National Seismic Hazard Mapping Project Probabilistic Determination

The design earthquake loading was calculated probabilistically using the USGS National Seismic Hazard Mapping Project Website (USGS, 1996). The peak accelerations are presented below in Table H4-6 for about a 475 year return period (10 percent probability of exceedance in 50 years).

Table H4-6
Ten Percent Probability of Exceedance in 50 Years

Location	Annual Exceedance Rate (USGS) (%)	Mean Return (USGS) Time (yrs)	Firm Rock (USGS) PGA (g)	Alluvium (CGS) PGA (g)
West End 33.28º N, 115.89º W	2.1	475	0.43	0.46
Middle 33.33º N, 115.84º W	1.6	475	0.61	0.53*
East End 33.37º N, 115.78º W	2.1	475	0.78	0.7*

* Sand Hills Seismicity Lineament not used in the CGS analysis resulting in a lower peak ground acceleration.

The calculations from the USGS website assume a uniform site condition of firm rock, with an average shear wave velocity of 2,500 ft/sec in the uppermost 100 ft. To account for the soft lacustrine and alluvial deposits, the California Geological Survey's Probabilistic Seismic Hazards Mapping Ground Motion Page uses soil corrections to determine the PGA for alluvial sites (California Geological Survey, 2004). The PGA for a 10 percent probability of exceedance in 50 years was determined from the USGS website, assuming firm rock, and the CGS website, using soil corrections to account for alluvium. (See Table H4-6). Apparently the CGS chose not to include the seismic hazard from the Sand Hills Seismicity Lineament which results in a less conservative PGA for areas near the middle and east shoreline of the sea

(southern half of the Salton Sea). The hazard for areas near the west shoreline did increase over the USGS firm rock site analysis from 0.43 to 0.46g.

The Estimated Probabilistic Seismic Hazard (mean return time vs. peak ground acceleration) for the west shore and the east shore are represented in Figure H4-9 and Figure H4-10 and were constructed using data obtained from the USGS Interactive Deaggregation Website. The locations used for the purpose of constructing the graphs were within the Salton Sea at 33.28 º N, 115.89º W for the West Shore, and 33.37º N, 115.78º W for the East Shore.

Design Peak Ground Acceleration

A comparison of probabilistically and deterministically determined peak ground accelerations corrected for alluvium is shown in the Table H4-7. The latter accelerations are based on DSOD guidelines which suggest an averaging of the 50th and 84th percentile. Except for the eastern shore, the deterministic values are slightly lower than the probabilistically determined accelerations and correspond to return periods ranging from approximately 300 to 500 years.

Table H4-7
Comparisons of Peak Ground Accelerations

	Probabilistic (475 year return period) PGA (g)	Deterministic (50th & 84th average) PGA (g)
Western Shore	0.46	0.37
Middle	0.53	0.50
Eastern Shore	0.7	0.74

The design peak ground accelerations for in-sea barriers considered in this report are the greatest values in Table H4-7 for lines parallel to the Salton Sea's shoreline. These meet and exceed DSOD guidelines and are generally representative of seismicity equivalent to a 10 percent probability in 50 years, or a mean return period of 475 years.

Displacement

Three different strands within the San Andreas Fault System were evaluated for the potential of horizontal displacement near the eastern shore and within the Salton Sea using an empirical relationship of potential average rupture length for displacement developed by Wells and Coppersmith, 1994. The first is the Coachella and San Bernardino Segments of the San Andreas fault and the remaining two are possible fault traces within the Salton Sea labeled 'A' and 'B' on Figure H4-2. These traces are dotted on Figure H4-2 and it is unknown if any seismic event along these traces would manifest itself on the surface. The results of average horizontal displacement are listed in Table H4-8.

Table H4-8
Estimate of Average Horizontal Fault Offset

Location	Within the Salton Sea	Length (miles)	Average Horizontal Offset (ft)
San Andreas fault (Coachella and San Bernardino Segments)	No	123.6	16.4
Strand in Salton Sea (A)*	Yes	14	1.6
Strand in Salton Sea (B)*	Yes	37+	4.6

* See Figure H5-2 for locations of 'A' & 'B'.

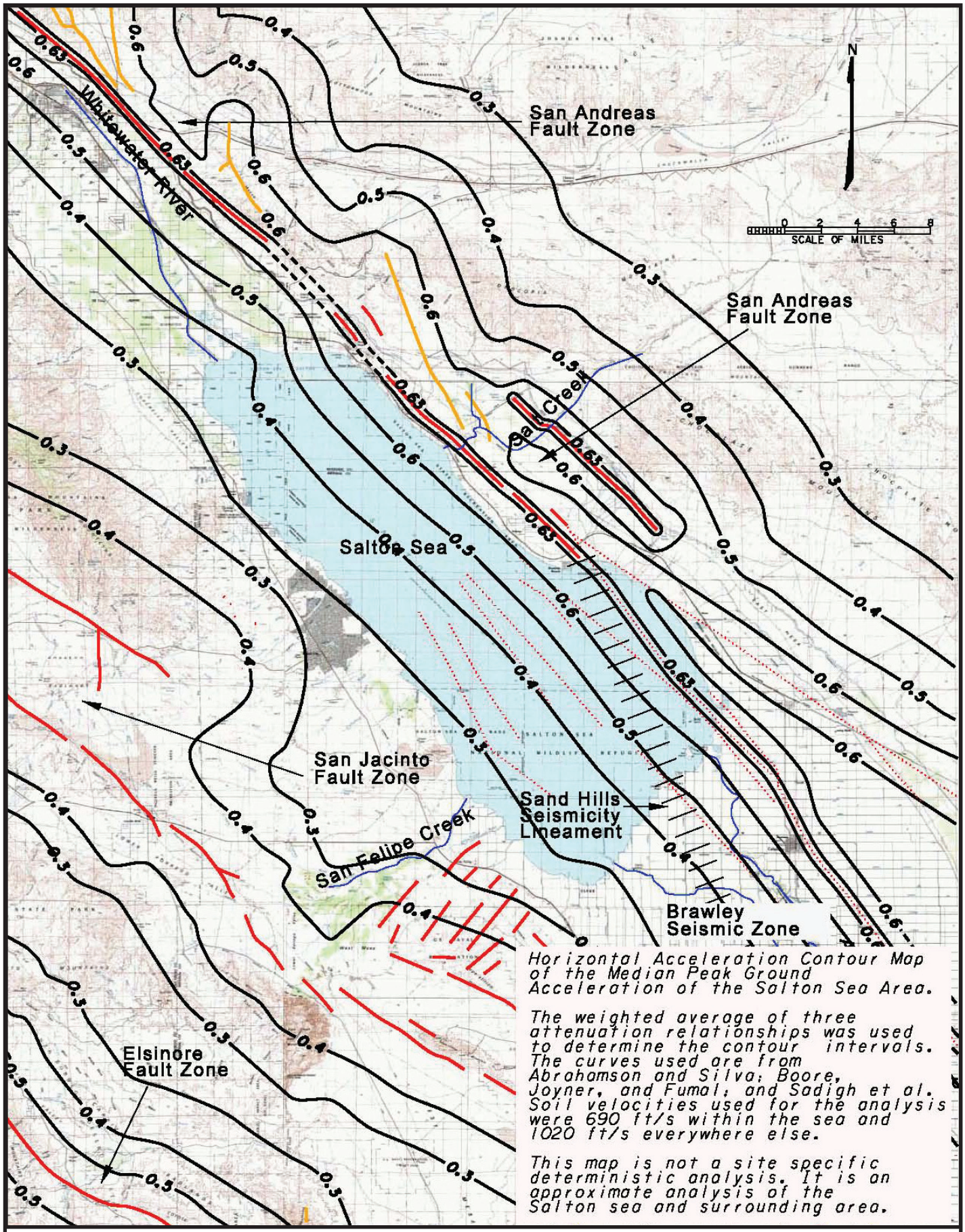

FIGURE H4-5
CONTOUR MAP OF APPROXIMATE MEDIAN HORIZONTAL PEAK GROUND ACCELERATIONS FROM 0.3G TO 0.63G

Source: DWR, 2006

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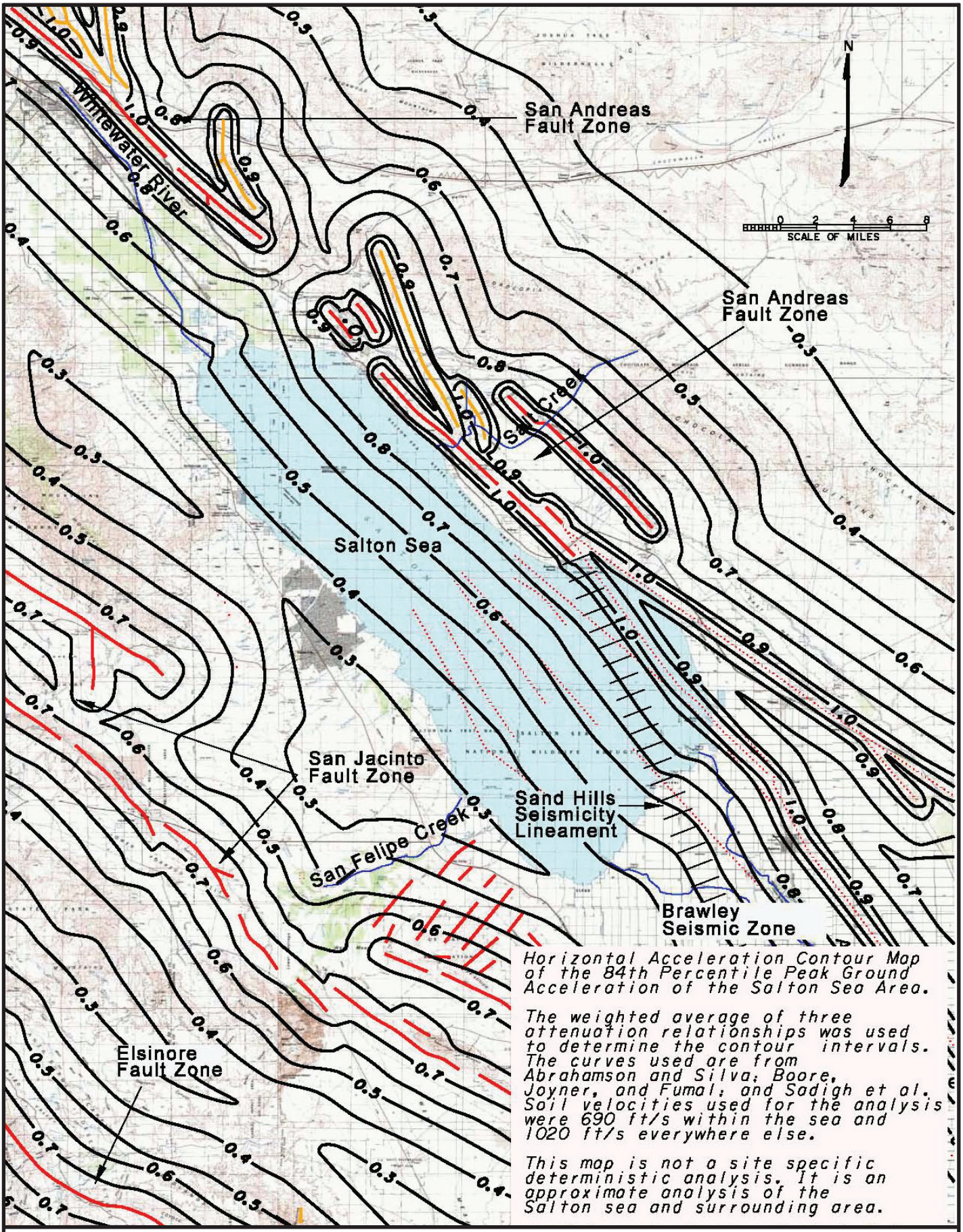

FIGURE H4-6
CONTOUR MAP OF APPROXIMATE 84TH PERCENTILE HORIZONTAL PEAK GROUND ACCELERATIONS FROM 0.3G TO 1.0G

Source: DWR, 2006

ES112005003SAC figure_H4-6.ai 07/21/06 tdaus

Earthquakes within 50 miles of Salton Sea In-Sea Barriers
1 Jan 1900 - 31 Aug 2004, M > 2

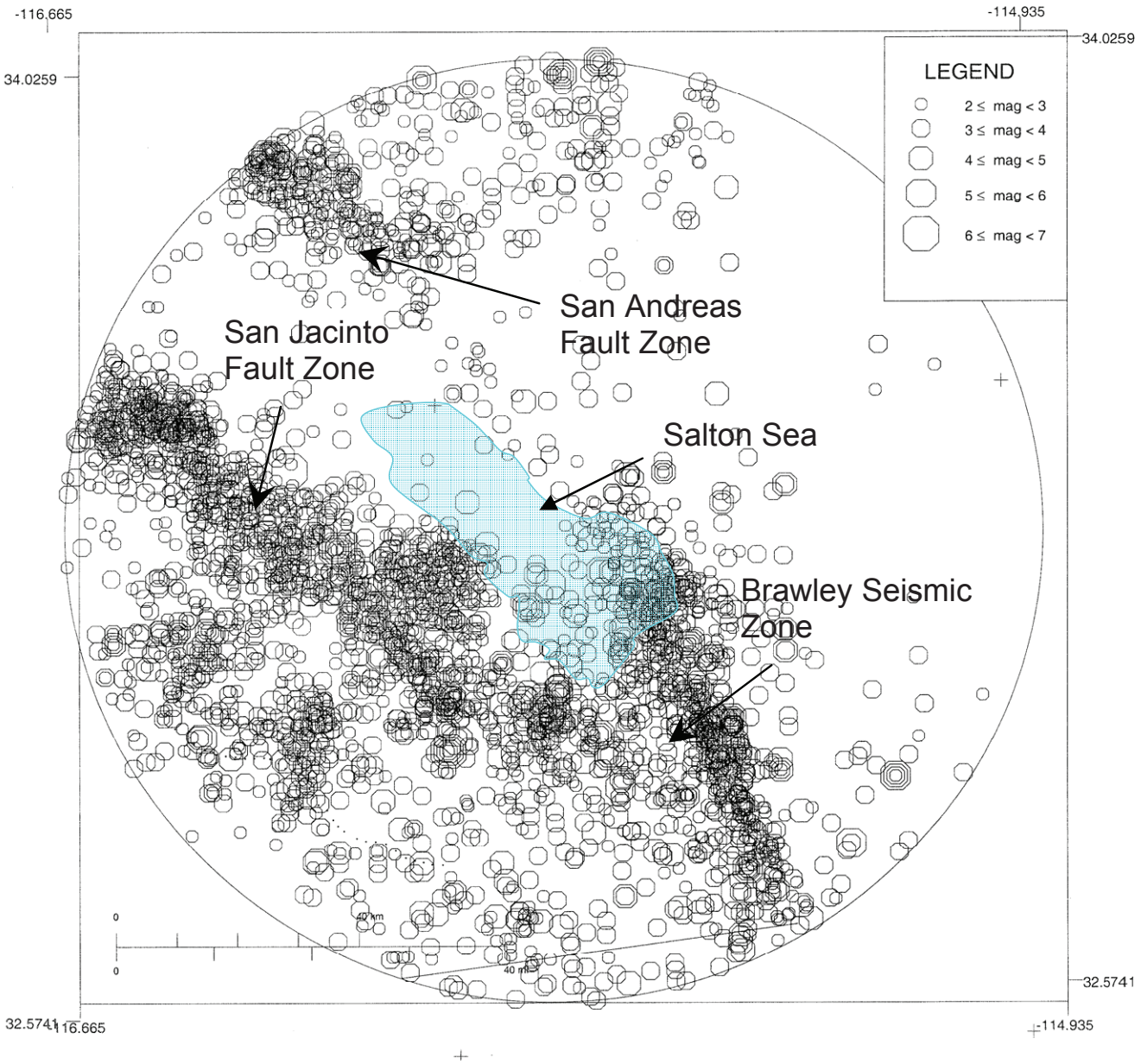

FIGURE H4-7
EARTHQUAKES GREATER THAN MAGNITUDE M2 WITHIN 50 MILES OF THE MIDDLE OF THE SALTON SEA

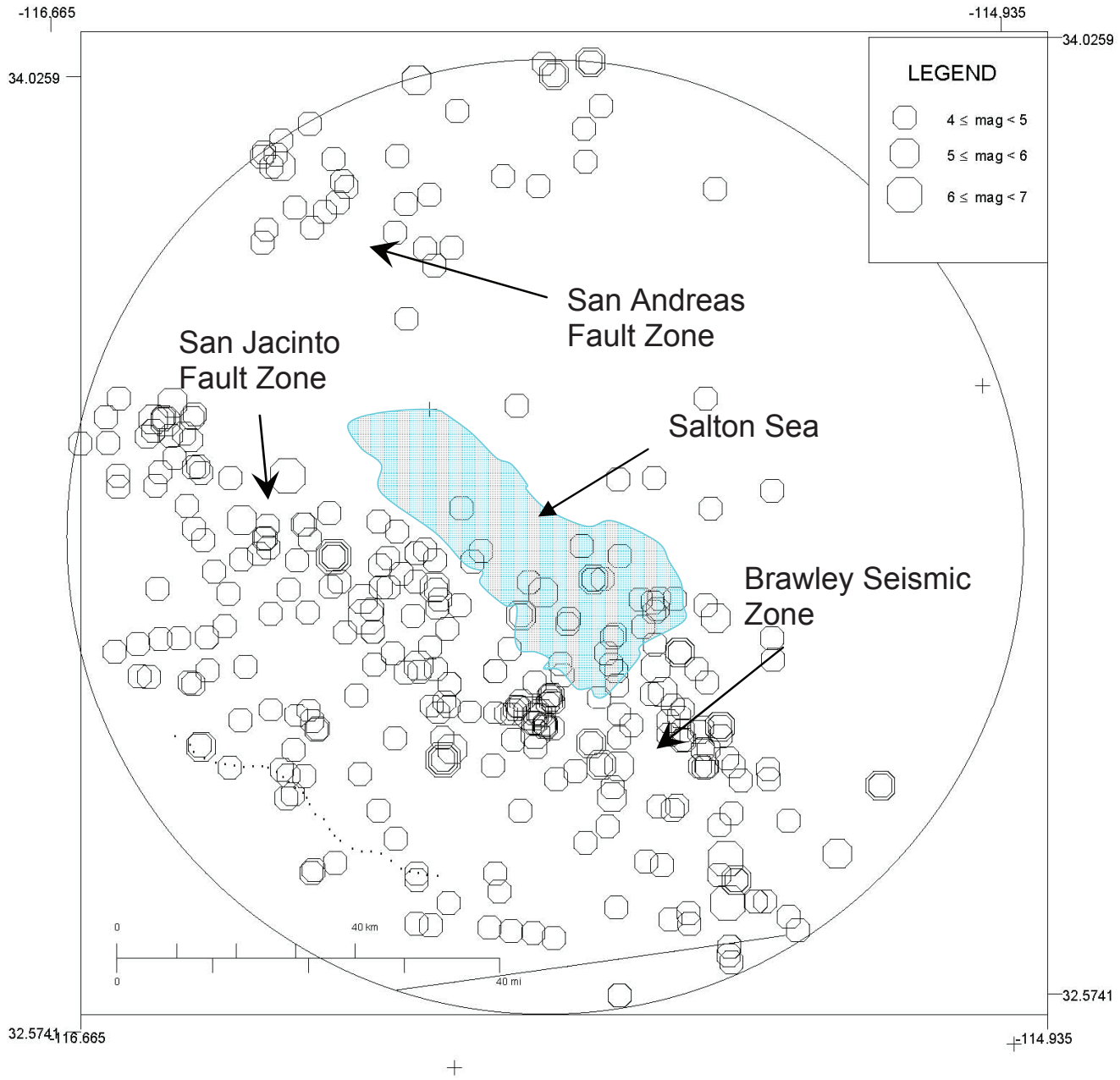

FIGURE H4-8
EARTHQUAKES GREATER THAN MAGNITUDE M4 WITHIN 50 MILES OF THE MIDDLE OF THE SALTON SEA

ES112005003SAC figure_H4-8.ai 07/21/06 tdaus

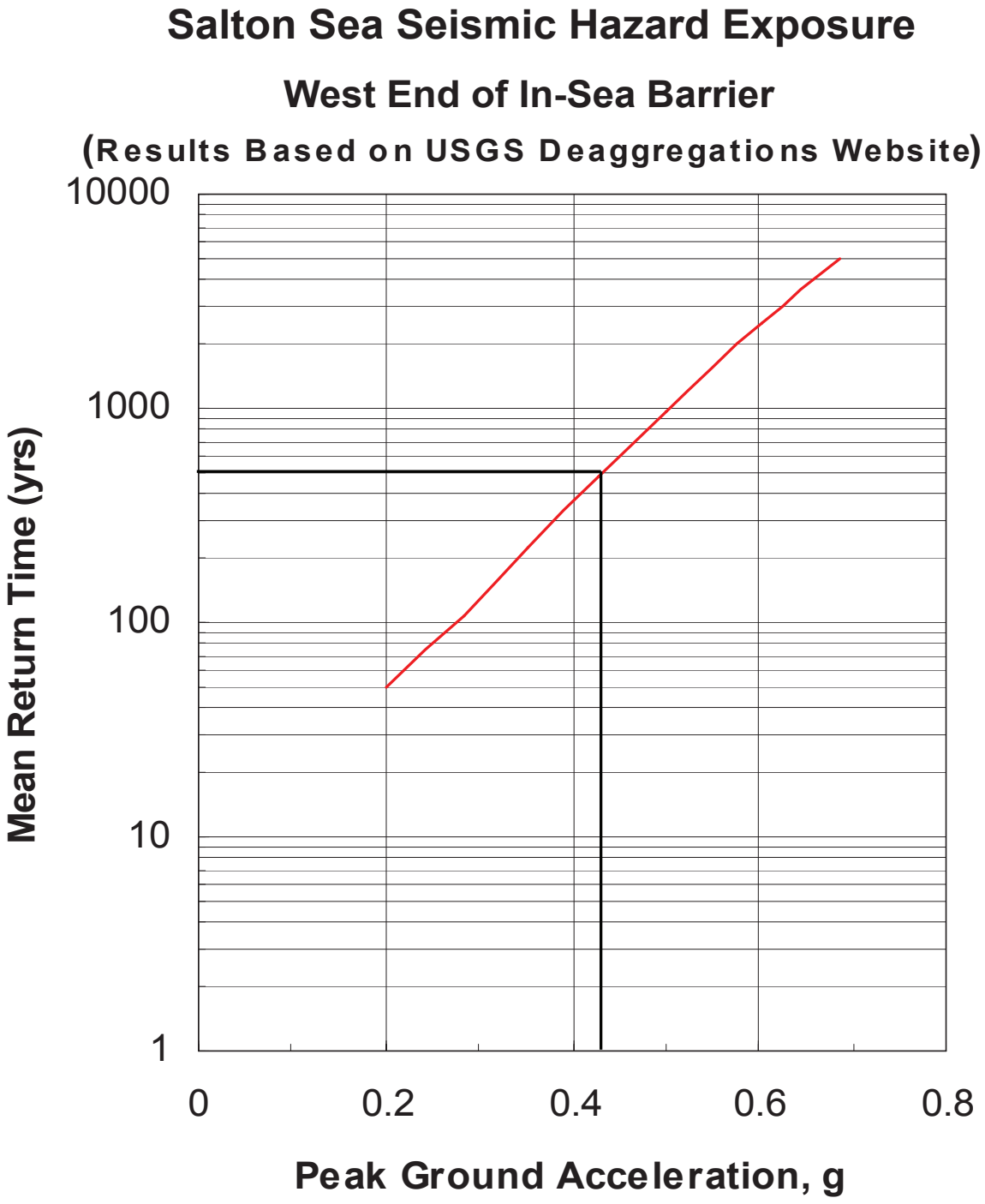

FIGURE H4-9
ESTIMATED PROBABILISTIC SEISMIC HAZARD FOR THE WEST SHORE NEAR THE MIDDLE OF THE SALTON SEA

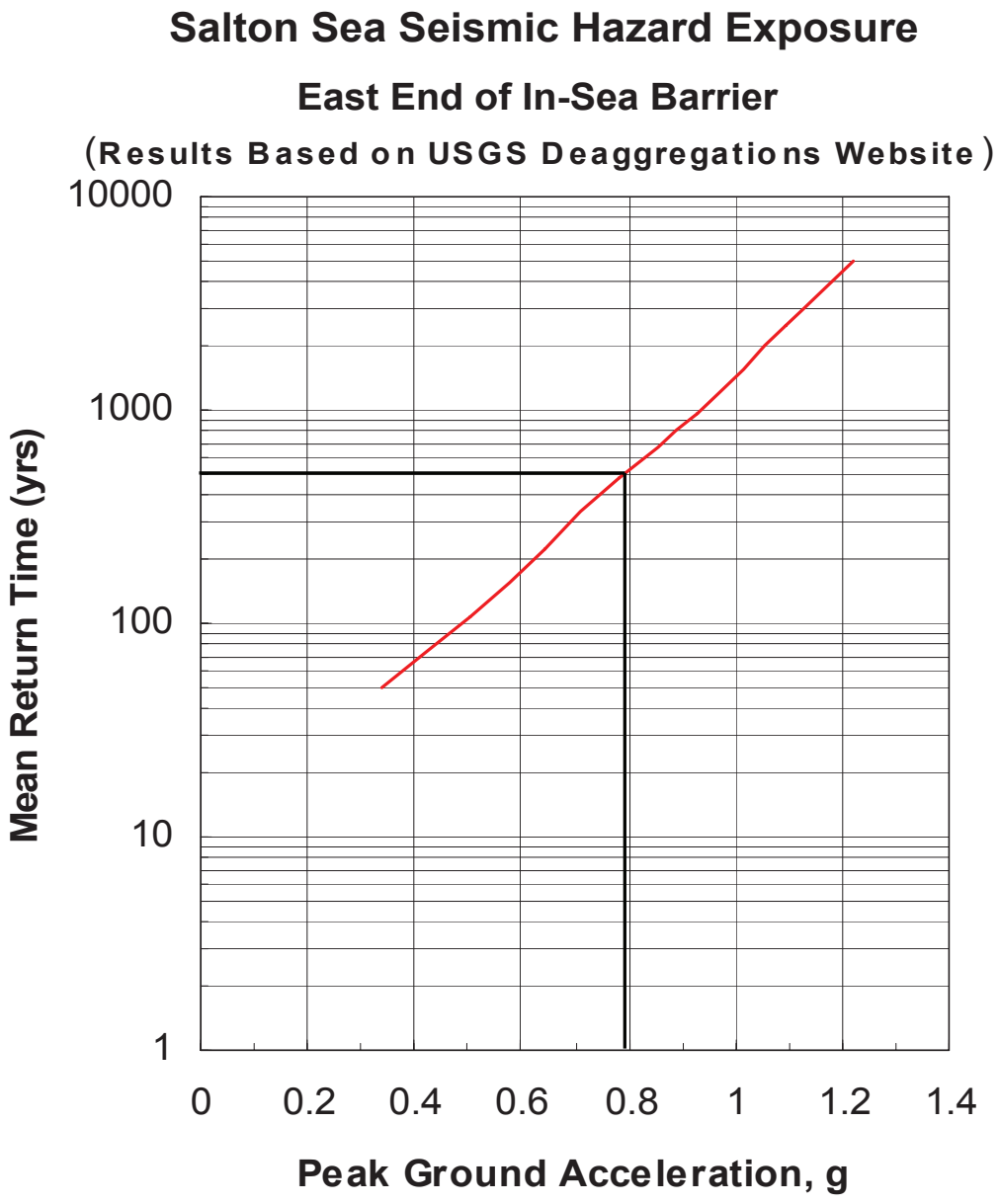

FIGURE H4-10
ESTIMATED PROBABILISTIC SEISMIC HAZARD FOR THE EAST SHORE NEAR THE MIDDLE OF THE SALTON SEA

FOUNDATION AND ROCKFILL DENSITY AND STRENGTHS

Introduction

The stability analyses conducted as part of this investigation require selection of density and strengths of the foundation and rockfill materials. These engineering properties were selected based upon a review of the foundation investigation documented in the February 2004 URS report (URS, 2004b) and published data for rockfill materials. The rationale used to select these engineering properties is discussed in detail below.

Foundation

General

Geotechnical testing was conducted on samples of the foundation materials obtained from the 11 borings drilled as part of the URS investigation. This testing included general material characterization, strength, and consolidation tests conducted on the six dominate subsurface deposits.

Density

Density testing was performed as part of the general material characterization. A summary of test results and adopted values presented in the URS report are repeated in Table H4-9. Due to the variation in deposit thicknesses underlying the Salton Sea, a uniform foundation density of 110 pcf was adopted for use in the stability analysis. This density was selected because it represents a reasonable mid-range value for the foundation deposits. In addition, a density of 110 pcf was also used for foundation materials in the URS report.

Table H4-9
Density and Water Content Tests Results Published in 2004 URS Report

Deposit	In-Place Density (pcf)						In-Place Water Content (percent)					
	No. of Tests	High	Low	Avg.	Med.	Adopted	No. of Tests	High	Low	Avg.	Med.	Adopted
Sea Bed	28	127	93	102	99	100	34	98	22	63	66	62
Soft Lacustrine	21	120	97	110	113	110	31	65	30	43	41	44
Alluvial	11	128	115	123	124	125	26	41	15	25	24	26
Stiff Lacustrine	29	128	102	119	121	120	64	52	19	32	32	32

Strength

Strength testing conducted as part of the URS investigation included Torvane tests on thin walled samples and unconsolidated undrained (UU) and consolidated undrained (CU) triaxial tests. Strength testing conducted by URS was focused on the Sea Bed and Soft Lacustrine deposits because these weaker materials will have a significant influence on the stability of in-sea barriers.

In the URS investigation, undrained foundation strengths were estimated by a combined review of the UU and CU triaxial tests presented as a function of effective overburden pressure. Whereas, effective foundation strengths were estimated by a review of the CU triaxial tests and presented using Mohr-Coulomb parameters. Undrained foundation strengths were ultimately selected using CU triaxial

tests combined with a review of published correlations as discussed in the September 2004 URS report. An undrained strength ratio of 0.25 and 0.35 was adopted for horizontal and vertical shear, respectively. This anisotropic undrained strength was used to evaluate "end-of-construction" stability of various in-sea barrier alternatives.

In this investigation, the CU triaxial tests were used to estimate effective and total foundation strengths for use in the "steady-state" and "simplified seismic" stability analyses conducted as part of this investigation. These strengths were estimated using stress path points with failure assumed to occur at 10 percent strain. Figure H4-11 through Figure H4-16 present stress path plots with corresponding trendline estimates of the Mohr-Coulomb strength parameters. Figure H4-11 through Figure H4-14 present the test data separated by Seafloor and Soft Lacustrine deposits and Figure H4-15 and Figure H4-16 present a compilation of the test data. As shown, the stress path points correlate fairly well with the linear trendlines presented on the figures with some deviation at higher confining stresses for tests conducted on the Soft Lacustrine deposits.

Since the amount of CU test data is currently limited for the individual foundation deposits and there exist some variability in the tests conducted on the Soft Lacustrine deposits, a uniform foundation strength representing a compilation of the test data was adopted for the Sea Bed and Soft Lacustrine Deposits for use in the stability analysis. The adopted Mohr-Columb strength parameters for the Sea Bed and Soft Lacustrine Deposits are: Effective: $\phi' = 29^o$, $c' = 0$ psf ; and Total; $\phi = 13.5^o$, $c = 0$ psf. It should be noted that CU tests were not conducted on the Stiff Lacustrine and Alluvial deposits. It is generally believed that these materials will be stronger than the Sea Bed and Soft Lacustrine Deposits. However, additional exploration and testing of these materials during future design investigations are required to confirm this assumption.

For comparative purposes, undrained strengths of the foundation deposits, presented as a function of effective stress, were also estimated using the CU and UU test data. Figure H4-17 presents a summary of the CU test data for the Sea Bed and Soft Lacustrine Deposits and Figure H4-18 presents a summary of the UU test data for the Sea Bed and Soft and Stiff Lacustrine Deposits.

As shown in Figure H4-17, an undrained strength ratio of 0.29 was estimated from the CU tests. The CU test data correlate fairly well with the linear trendline presented on Figure H4-17, whereas, the UU test data presented in Figure H4-18 indicates a lot of variability in foundation strength. A variety of data correlations are presented on Figure H4-18 including: median trendlines for each foundation deposit; median and linear trendline for all foundation materials; published strength correlations (Skempton's) based upon the median PI of each foundation material; and the 0.25 and 0.35 strength ratios adopted in the URS investigation. An undrained strength ratio of 0.35 and 0.6 represent reasonable lower bound and average strengths for the UU data, respectively. However, these strengths are higher than estimated by the CU data and illustrate the need for additional testing and characterization of foundation materials during future design investigations.

Figure H4-19 presents shear strength as a function of effective stress for the anisotropic undrained strength ratios adopted in URS investigation and the total and undrained strength parameters developed in this investigation. As shown, the total strength parameters adopted in this investigation represents the most conservative selection and is similar to the horizontal undrained strength ratio adopted in the URS investigation. The total strength parameters were utilized in the pseudo-dynamic stability analyses conducted as part of this investigation. Sensitivity analyses using the range of foundation strengths presented on Figure H4-19 were also performed as part of the stability analyses to examine the influence of the foundation strength on barrier stability.

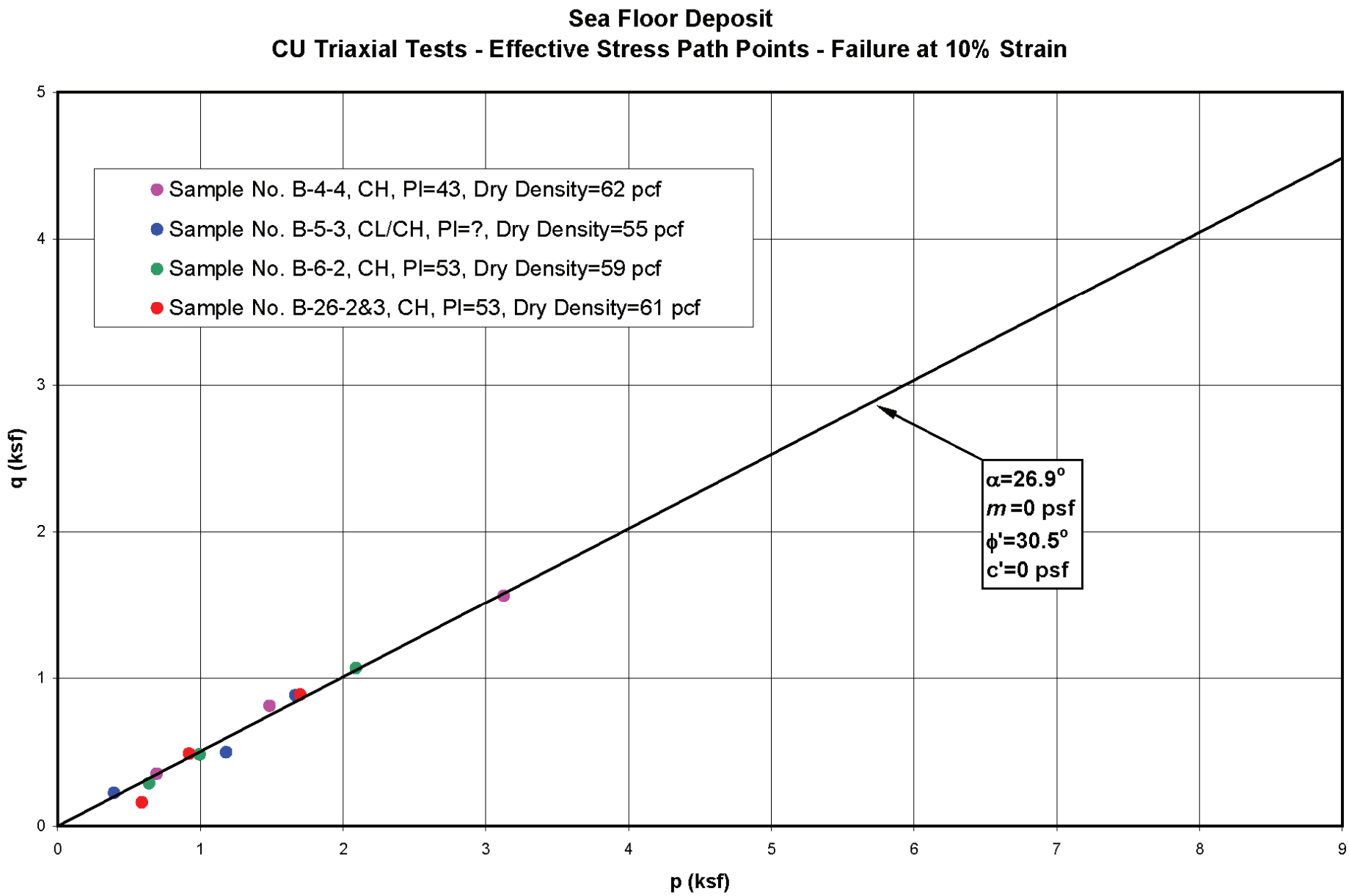

FIGURE H4-11
SEA FLOOR DEPOSITS EFFECTIVE STRESS PATH POINTS AT 10 PERCENT STRAIN

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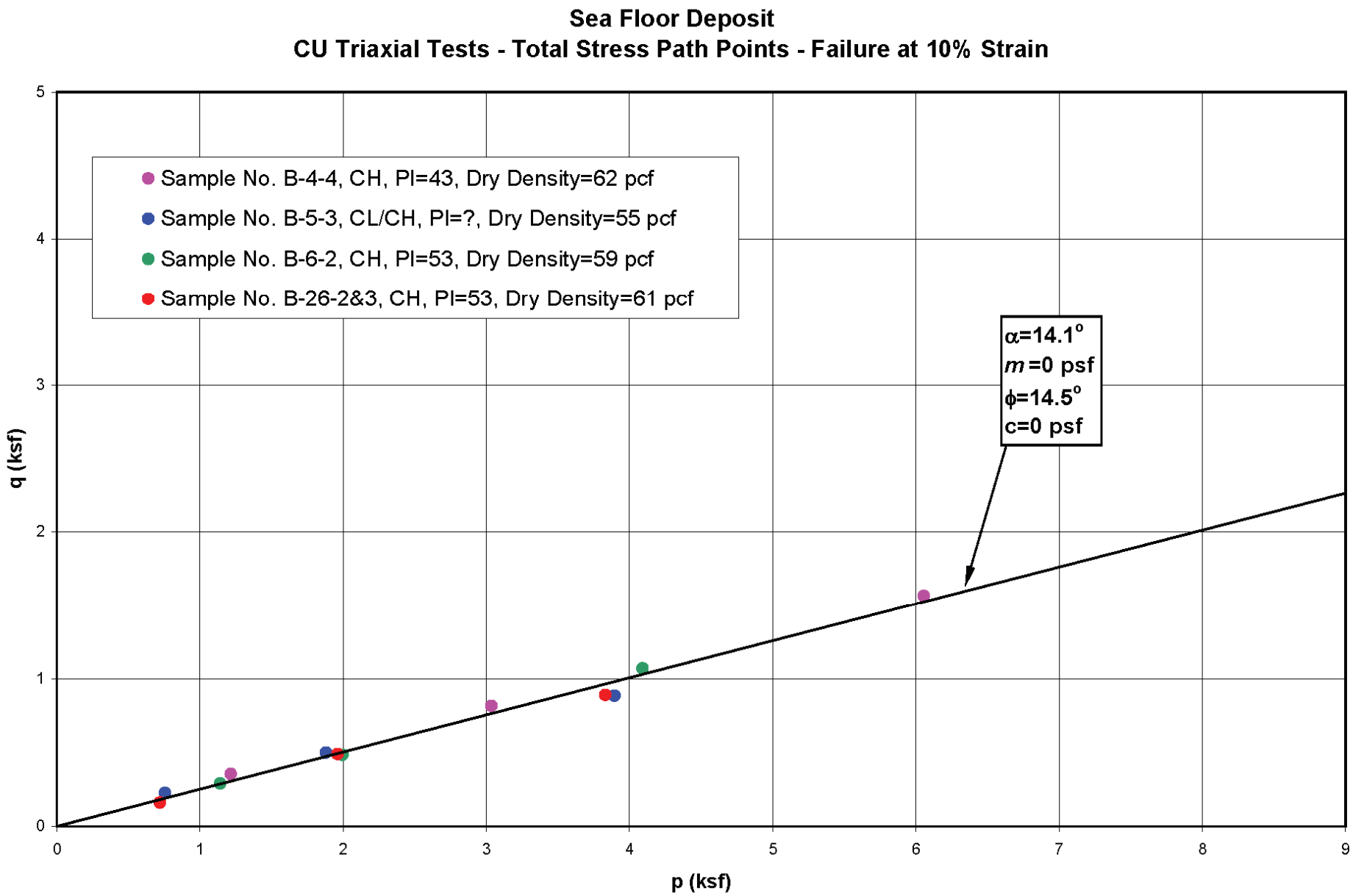

FIGURE H4-12
SEA FLOOR DEPOSITS TOTAL STRESS PATH POINTS AT 10 PERCENT STRAIN

ES112005003SAC figure_H4-12.ai 07/24/06 tdaus

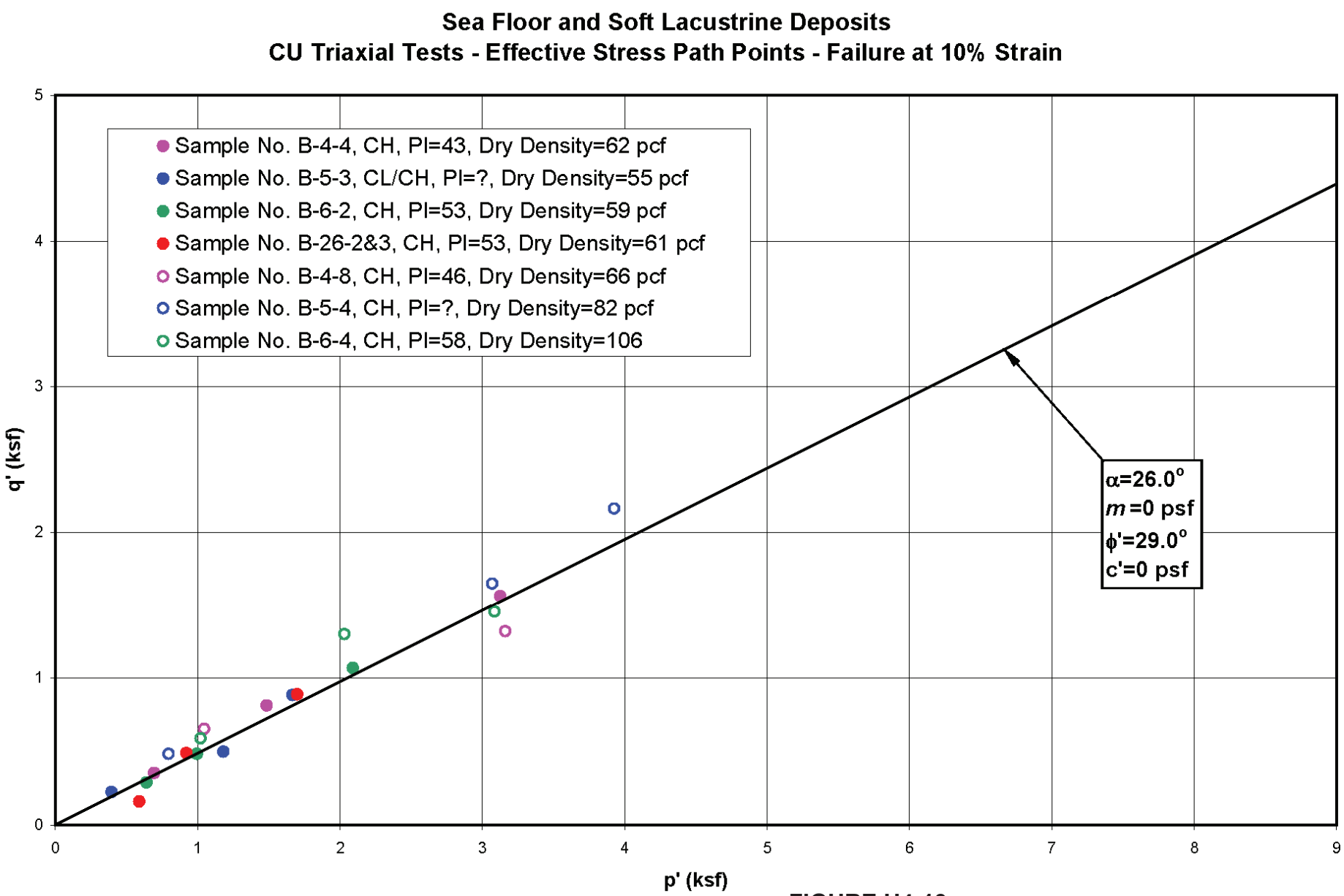

FIGURE H4-13
SOFT LACUSTRINE DEPOSITS EFFECTIVE STRESS PATH POINTS AT 10 PERCENT STRAIN

ES112005003SAC figure_H4-13.ai 07/24/06 tdaus

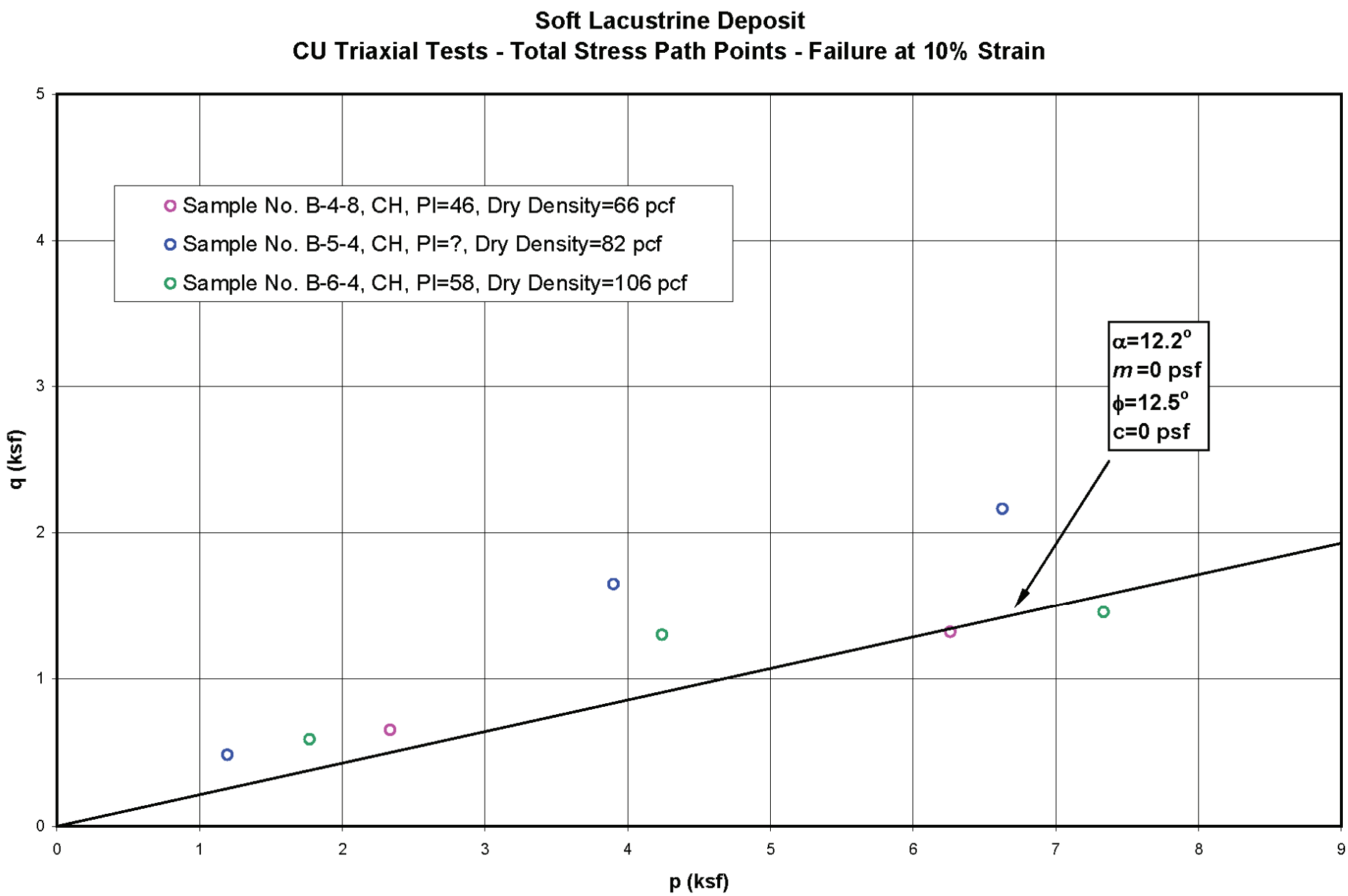

FIGURE H4-14
SOFT LACUSTRINE DEPOSITS TOTAL STRESS PATH POINTS AT 10 PERCENT STRAIN

ES112005003SAC figure_H4-14.ai 07/24/06 tdaus

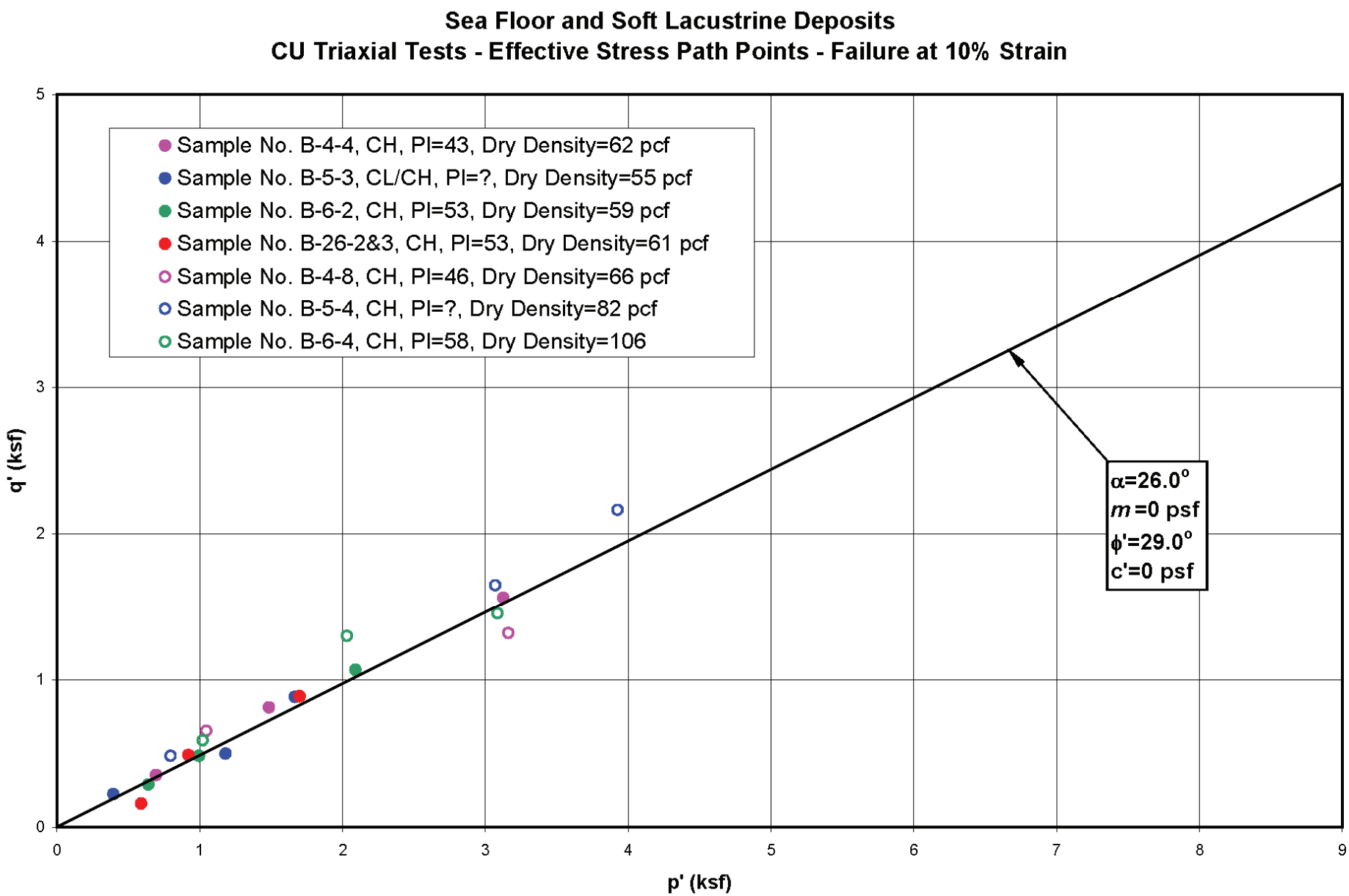

FIGURE H4-15
SEA FLOOR AND SOFT LACUSTRINE DEPOSITS
EFFECTIVE STRESS PATH POINTS

ES112005003SAC figure_H4-15.ai 07/24/06 tdaus

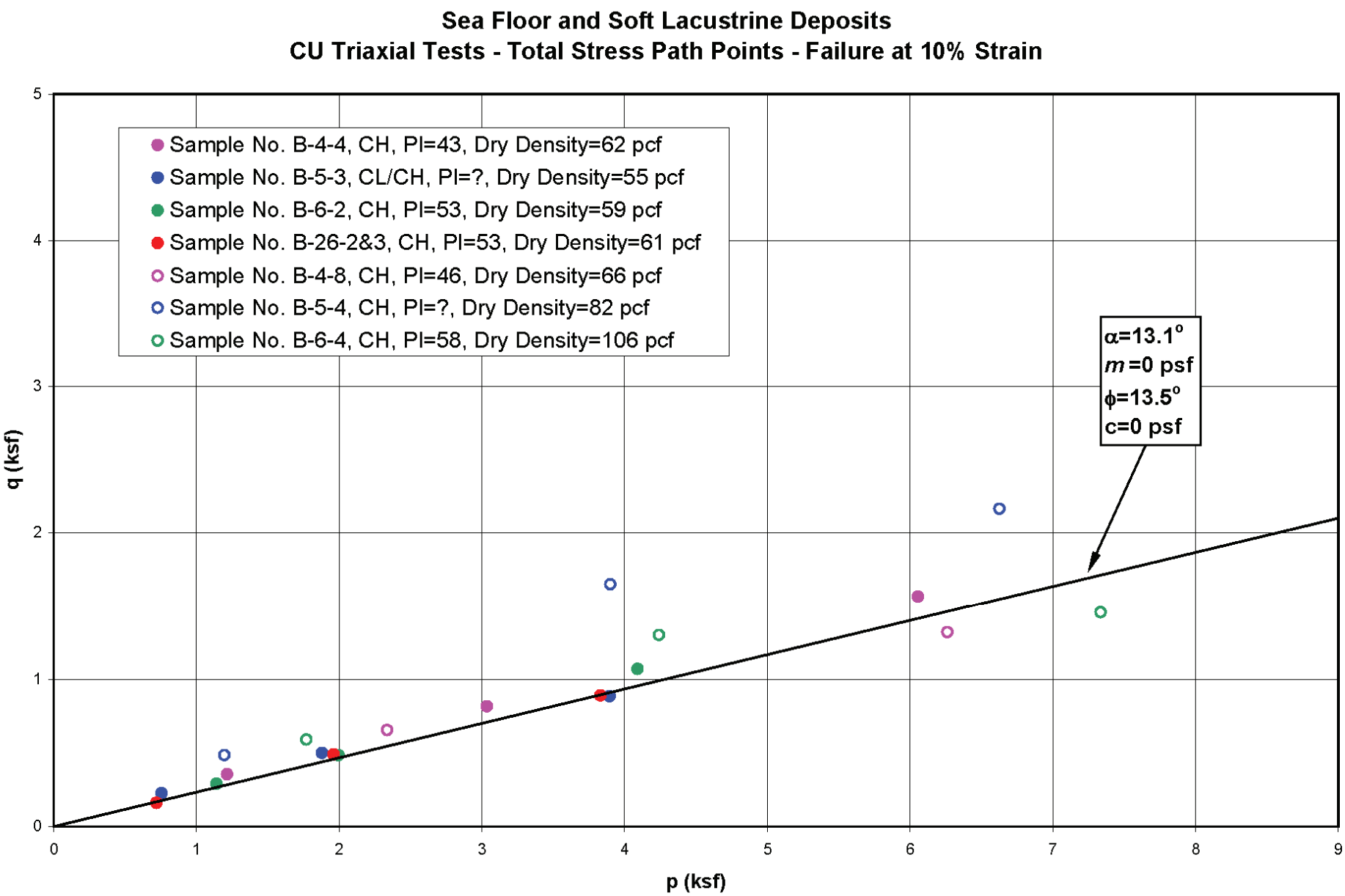

FIGURE H4-16
SEA FLOOR AND SOFT LACUSTRINE DEPOSITS
TOTAL STRESS PATH POINTS

ES112005003SAC figure_H4-16.ai 07/24/06 tdaus

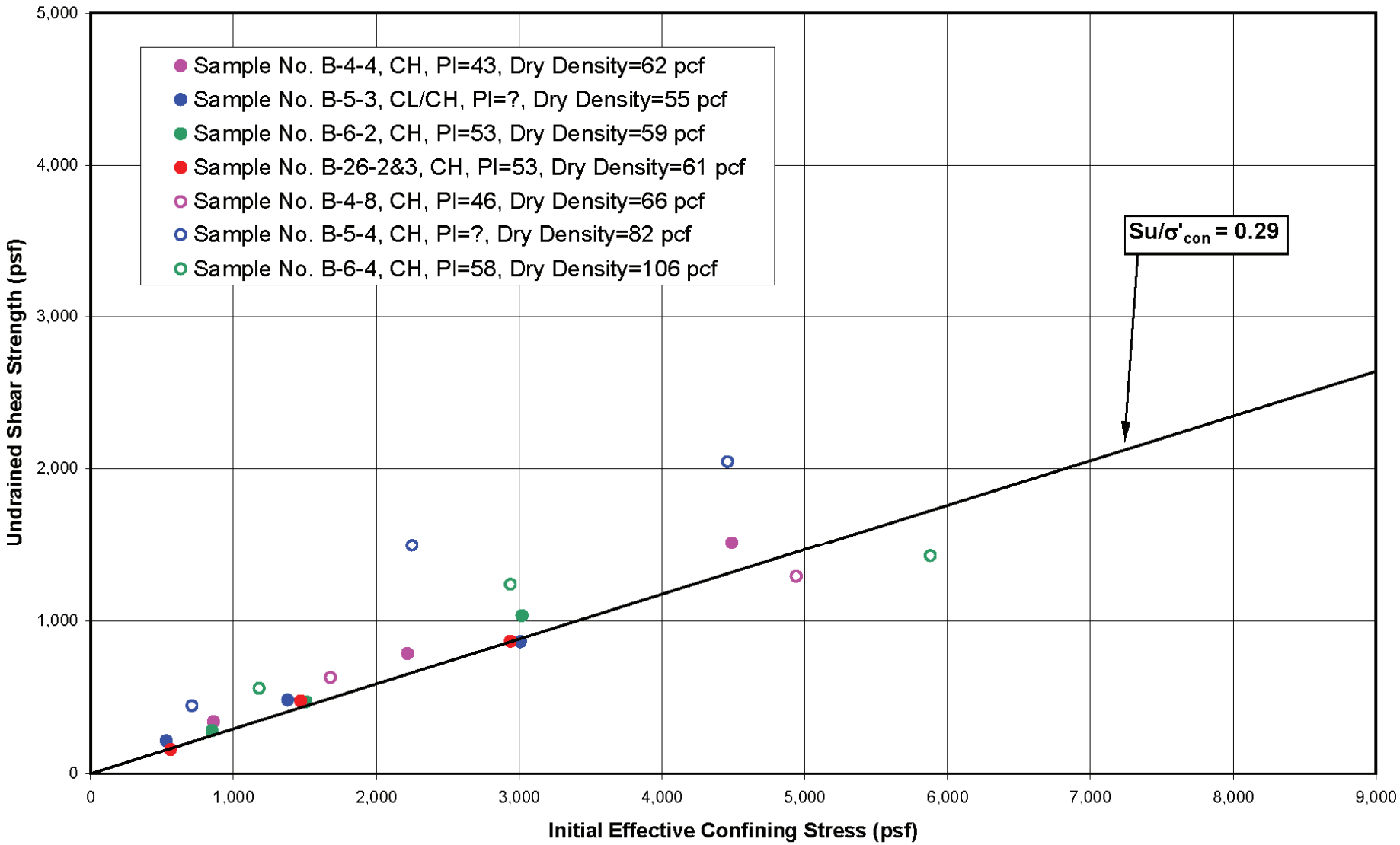

FIGURE H4-17
SEA FLOOR AND SOFT LACUSTRINE DEPOSITS
UNDRAINED STRENGTH FROM CU TESTS

ES112005003SAC figure_H4-17.ai 07/24/06 tdaus

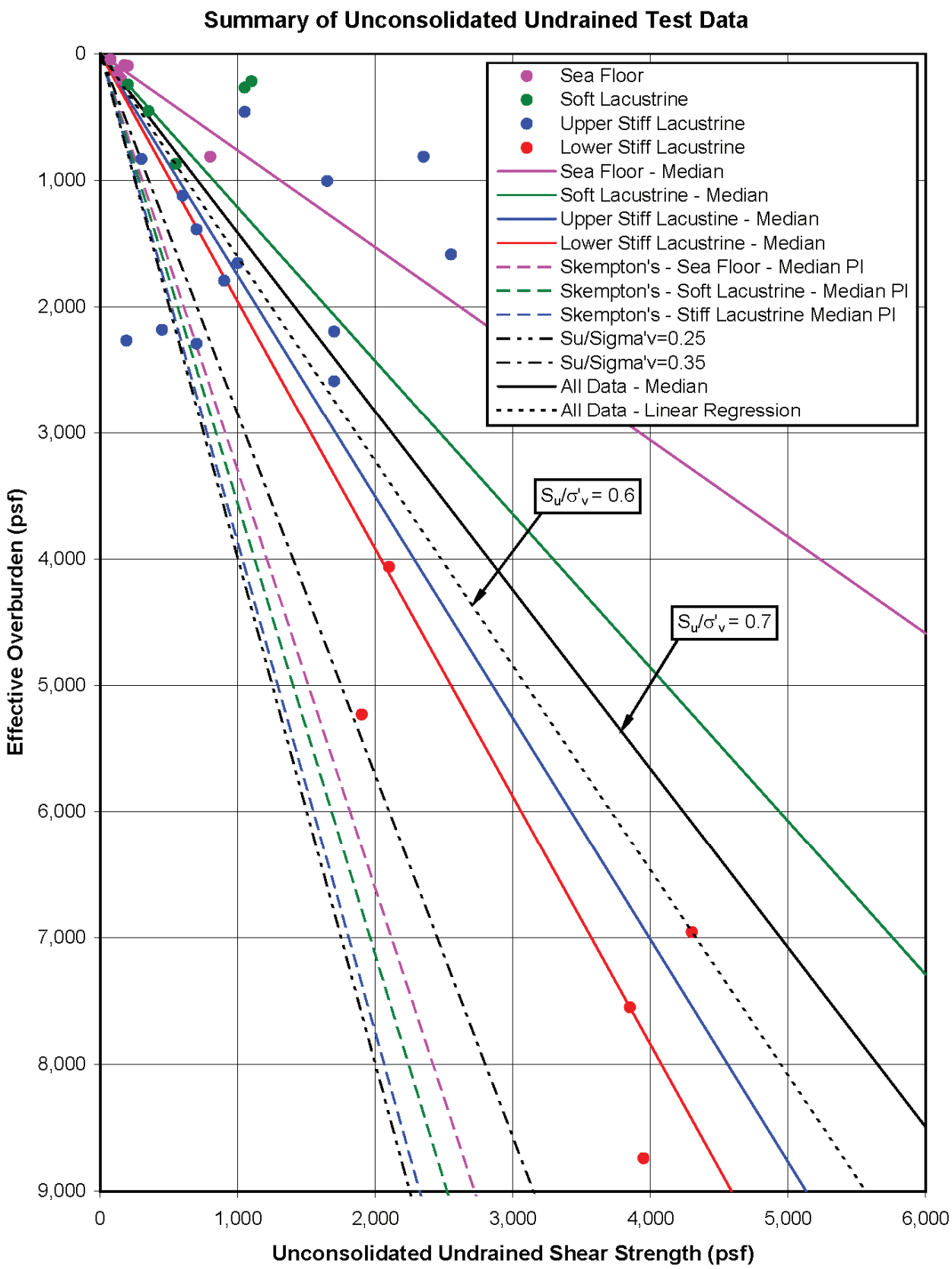

FIGURE H4-18
SEA FLOOR AND LACUSTRINE DEPOSITS
UNDRAINED STRENGTH FROM UU TESTS

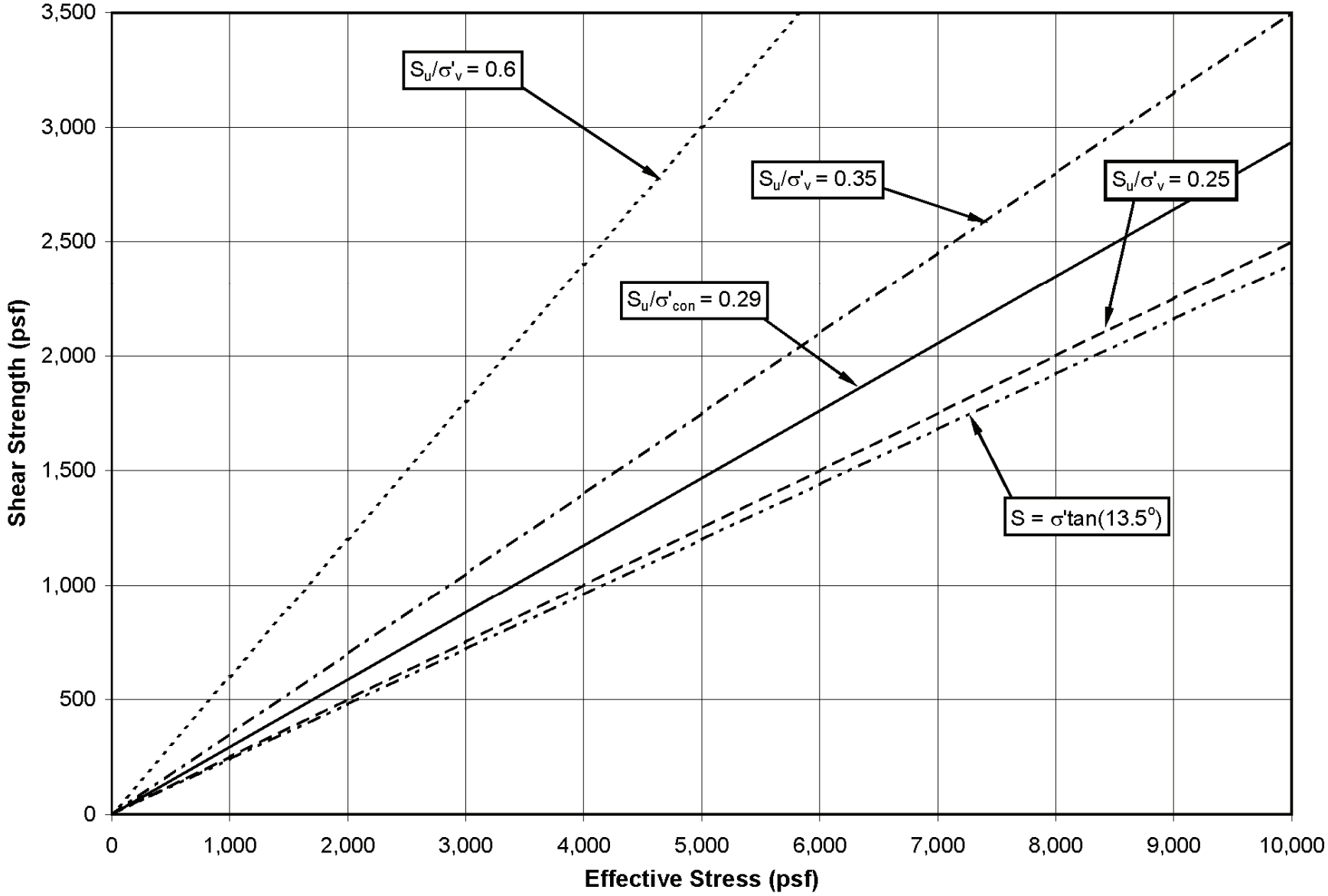

FIGURE H4-19
COMPARISON OF UNDRAINED AND TOTAL FOUNDATION STRENGTH CORRELATIONS

Rockfill

Density

The rockfill density was selected based upon a review of published data and test fill data from the recently constructed Diamond Valley Reservoir and Seven Oaks Dam projects (Metropolitan, 1994, ACOE, 1992, Becker et al., 2000). Based upon a review of the published data, a rockfill density of 140 pcf was adopted as a reasonable estimate for use in the stability analysis.

Strength

Rockfill strength was estimated based upon a review of published data (Barton and Kjaernsli, 1981, Becker et al., 2000, Leps, 1970, Marachhi et al., 1969). Figure H4-20 presents a summary of shear strength test data prepared by Leps (1970) which is commonly used to estimate strengths of rockfill materials. Considering the heights of the proposed in-sea barrier and uncertainty in rockfill source, a friction angle of 45 degrees was selected as a reasonable estimate of rockfill strength.

STABILITY ANALYSES

Introduction

Stability analyses were conducted to provide an initial estimate of in-sea barrier stability and to assist selection of reasonable slopes for use in the cost estimate model. Since the Salton Sea is located in an area of relatively high seismicity, the stability of the in-sea barrier during postulated future seismic events is one of the foremost design considerations. Accordingly, a focus of the stability analyses was to provide an estimate of yield accelerations for critical sliding surfaces to assist estimation of permanent deformations during postulated seismic events. In addition, steady-state stability analyses were also conducted as part of this investigation. Lastly, supplemental analyses were performed using residual strengths to estimate post-liquefaction stability.

The majority of the stability analyses were conducted for a 45-foot high section primarily to maintain consistency between this and earlier reports. The maximum depth of water evaluated for conceptual design is 47 feet. This depth is based on the lowest elevation of the existing Sea Bed of 275 feel below msl and a water level of 228 feet below msl. With 5 feet of freeboard, the greatest barrier height would be 52 feet for locations at the deepest portion of the Salton Sea. In addition, supplemental analyses were performed on barriers heights of 25 feet and 15 feet with varying foundation strengths to develop insight into the influence of these parameters on barrier stability.

Methodology

Stability analyses were performed using the geotechnical modeling software Slope/W (GEO-SLOPE, 2004) and using Spencer's method of analysis (Spencer, 1967). The stability of the upstream and downstream slopes were evaluated for two loading conditions:

1. long-term barrier stability with steady-state seepage conditions using effective shear strengths, and
2. pseudo-dynamic stability with application of a horizontal seismic load using total shear strengths.

Potential circular sliding surfaces extending through the full barrier height into the upper, weaker Sea Bed and Soft Lacustrine foundation deposits were evaluated for the two loading conditions. Sliding surfaces were not extended into the underlying Alluvial and Stiff Lacustrine deposits, since it is generally believed that these materials will be stronger than the overlying deposits. For simplicity, steady-state phreatic surfaces used in the stability models were assumed to be linear from the upstream to downstream water surfaces.

Model Description

Common Geometry Parameters

Stability analyses conducted as part of this investigation included many common geometric parameters as input into the Slope/W models. These common parameters are summarized below:

- Marine Sea Water Elevation: -230 feet msl
- In-Sea Barrier Crest Width: 20 feet
- In-Sea Barrier Freeboard: 5 feet
- Tailwater Depth (at downstream toe): 0 feet

Note that the downstream tailwater depth was assumed to be zero feet in the stability analyses conducted in this investigation. Barrier stability for tailwater depths greater than zero feet may be approximated by interpolating between the upstream full tailwater condition and downstream zero tailwater condition for any specific barrier section.

Barrier Sections

As discussed previously, stability analyses were conducted on barriers of varying height with the majority of stability analyses conducted on the maximum barrier section. The maximum barrier section had a height of 45 feet with upstream and downstream slopes varied from 5H:1V to 15H:1V. Supplemental analyses were also performed on barrier sections with heights of 25 and 15 feet and 10H:1V upstream and downstream slopes.

The thickness of the upper, weaker Sea Bed and Soft Lacustrine foundation deposits included in the stability models were varied to represent foundation conditions near the perimeter of the Salton Sea. As such, cross-sections representing barrier heights of 45, 25, and 15 feet, were associated with the maximum thickness of the weaker foundation deposits of 40, 10, and 10 feet, respectively. The maximum estimated thickness of these deposits was used to represent the worst known foundation condition.

Foundation and Rockfill Strengths

As discussed in the previous section regarding foundation and rockfill density and strengths, the design strengths of the weaker, upper foundation deposits were adopted based upon CU tests conducted on the Sea Bed and Soft Lacustrine deposits. The adopted Mohr-Columb strength parameters used in the stability analyses for these materials were: Effective: $\phi' = 29^o$, $c' = 0$ psf ; and Total: $\phi = 13.5^o$, $c = 0$ psf. A drained rockfill strength of $\phi' = 45^o$ was also used in the analyses.

Supplemental stability analyses were also conducted using two other foundation strength correlations discussed in section on foundations. In particular, undrained strength ratios (S_u/σ'_v) of 0.29 and 0.6 were used in supplemental analyses conducted on the maximum barrier section with 10H:1V upstream and downstream slopes. The undrained strength ratio of 0.29 was estimated from the CU tests conducted on the Sea Bed and Soft Lacustrine deposits, whereas the undrained strength ratio of 0.6 was estimated as an average strength from the UU tests conducted on the Sea Bed, Soft and Stiff Lacustrine deposits.

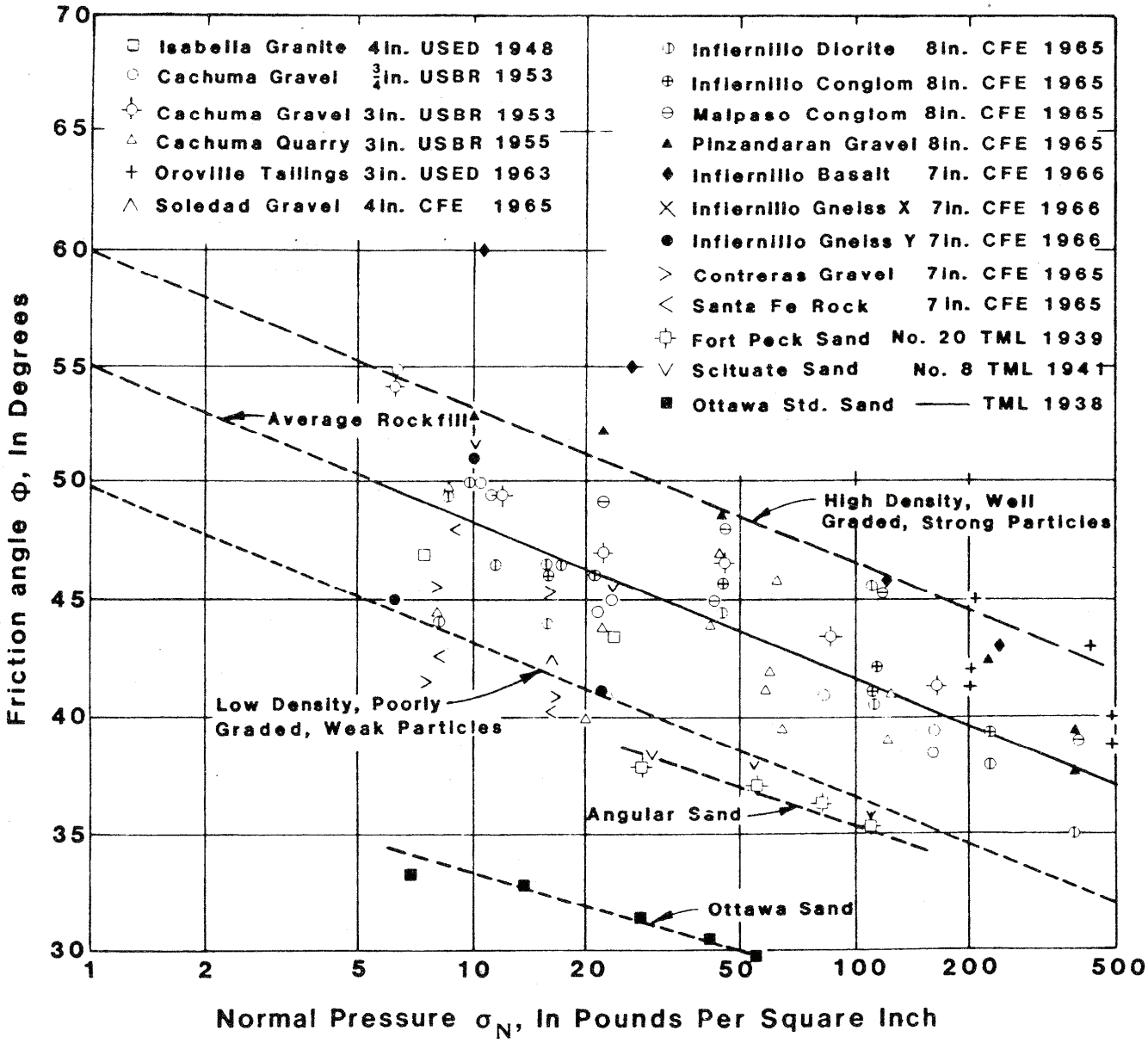

FIGURE H4-20
SHEARING RESISTANCE OF ROCKFILL FROM LARGE TRIAXIAL TESTS

Analysis Results

Forty-Five-Foot High Barrier

Figure H4-21 through Figure H4-24 show summaries of the stability analyses results for the steady-state and pseudo-dynamic loading conditions. Figure H4-21 presents the variation of Factor of Safety with barrier slope for long-term barrier stability with steady-state seepage conditions. As shown, the Factor of Safety ranged from 3.2 to 8.0 for upstream sliding surfaces and 2.0 to 5.1 for the downstream sliding surfaces for barrier slopes ranging from 5 to 1 (horizontal to vertical) to 15 to 1, respectively. The Factors of Safety outside of these slope inclinations can be linearly interpolated as shown in Figure H4-21. Figure H4-24 presents the variation of yield acceleration with barrier slope for pseudo-dynamic loading conditions. As shown, yield accelerations ranged from 0.08 to 0.14 for the upstream sliding surfaces and 0.02g to 0.12g for the downstream sliding surfaces with barrier slopes also ranging from 5 to 1 to 15 to 1, respectively. The Slope/W output results are also included for reference in Attachment H4-1.

Supplemental Analyses with Varied Barrier Heights

Table H4-10 presents a summary of the supplemental stability analysis results for barrier heights of 25 and 15 feet with steady-state and pseudo-dynamic loading conditions. Figure H4-25 and Figure H4-26 present the variation of Factor of Safety with seismic load used to estimate the yield accelerations presented in Table H4-10. The stability analysis results for the 45 foot high barrier are also included in this table and figures for comparison.

As shown in Table H4-10, the Factor of Safety and yield accelerations were higher for the 25-foot and 15-foot high barriers, when compared to the 45-foot high barrier. Since a 10-foot thickness of weaker foundation deposits was used in both the 25-foot and 15-foot barrier stability models, the Factor of Safety and yield accelerations for the 25-foot high barrier were also higher than the 15-foot high barrier due to a smaller thickness of weaker foundation materials relative to the amount of overlying stronger barrier rockfill materials included in the 25-foot barrier model. The Slope/W output results are also included for reference in Attachment H4-1.

Table H4-10
Stability Analysis Results for Barrier Heights of 45, 25, and 15 Feet

Barrier Height (feet)	Steady-State		Pseudo-Dynamic	
	Factor of Safety		Yield Acceleration (g)	
	Upstream Slope	Downstream Slope	Upstream Slope	Downstream Slope
45	5.6	3.5	0.13	0.10
25	6.4	4.9	0.20	0.20
15	5.7	4.8	0.17	0.17

Supplemental Analyses with Varied Foundation Strengths

Figure H4-27 and Figure H4-28 present a summary of the supplemental stability analysis results for a 45-foot high barrier with varied foundation strengths for pseudo-dynamic loading conditions. These figures present the variation of Factor of Safety with seismic load and were used to estimate the yield accelerations presented in Figure H4-29. As shown, yield accelerations ranged from 0.13 to 0.28 for the upstream sliding surfaces, and 0.10 to 0.28 for the downstream sliding surfaces with foundation strengths ratios (S/σ') ranging from 0.24 to 0.6. A strength ratio of 0.24 correlates to the Mohr-Columb strength parameter ($S=\sigma'\tan(13.5^{o})$) adopted from the CU tests on Sea Bed and Soft Lacustrine deposits. The Slope/W output results are also included for reference in Attachment H4-1.

Supplemental Analyses with Residual Strengths (Post-Liquefaction)

An evaluation of the liquefaction potential of the foundation soils was performed by Reclamation (2005) and URS (2005). The evaluation was based on a review of limited standard penetration tests (SPT) and cone penetration tests (CPT) data presented from the Salton Sea field investigation and laboratory testing program conducted by URS Corporation in 2003 and 2004. The field investigation included SPT and CPT spaced about 1-mile apart (with some pairing of SPT and CPT data) along the eight to nine miles of foundation length of a mid-Sea alignment.

As mentioned in the previous section, "Prior Studies, Foundation Investigations," the URS report identified six dominant and distinct layers comprising the subsurface deposits. Based on the limited field data, Reclamation concluded that there is a potential for liquefaction in some silty to fine-sand layers with low blowcounts within both the soft lacustrine layer and upper alluvial layer, given the very high seismic loadings postulated for the site (Reclamation, 2005). URS also concluded that the available data indicates that there may be some layers within the foundations soils for the mid-Sea dam that are potentially liquefiable.

As described in Prior Studies section, the Upper Alluvial layer is described as 0 to 26 feet thick, consisting of loose to dense, silty-fine sand, with interbedded silt and clay lenses. This layer is essentially non-existent in the central part of the Salton Sea and east shore, and only exists near the western shore. The Soft Lacustrine layer is similarly 0 to 26 feet thick and is described as soft to very soft, highly plastic clay. This deposit essentially dominates the entire seafloor. It is thickest in the central part and eastern shore, and thinnest near the western shore.

Slope stability analyses were performed by DWR to evaluate the post-earthquake stability of a mid-Sea barrier or perimeter dike. Generalized DWR cross-sections developed for slope stability analysis were used. For this analysis, a dam height of 45 feet with upstream and downstream slopes of 10H:1V and 15H:1V were assumed, respectively. From the 2003-2004 URS field exploration, only limited SPT blowcounts were available in the soft lacustrine and upper alluvial deposits. Most of the reported SPT values were converted from CPT data. The reported SPT $N_{1(60)}$ values in the soft lacustrine deposit ranged from 8 to 29, with an average of 14; The reported SPT $N_{1(60)}$ values in the upper alluvial deposits ranged from 6 to 24, with an average of 15. Reclamation identified several foundation layers within the Soft Lacustrine Deposit with equivalent $N_{1(60)}$ values below 15, especially values below 10, which were judged to be liquefiable under the postulated high seismic loadings. Similarly, Reclamation identified several foundation layers within the Upper Alluvial Deposit with equivalent $N_{1(60)}$ values below 15, especially values below 10, which were similarly judged to be liquefiable (Reclamation, 2005).

The residual strength for the soft lacustrine deposits selected for analysis was based on the relationship developed by Seed and Harder (URS, 2005). DWR used a residual undrained shear strength S_r of 250 psf for the soft lacustrine deposits as recommended by Reclamation and URS based on CPT data converted to SPT $N_{1(60)}$ values. An S_r value of 250 psf is judged to be conservative. Table H4-11 summarizes the post-liquefaction stability analyses. The Slope/W output results are also included for reference in Attachment H4-1.

Table H4-11
Post-Liquefaction Stability Analyses for Barrier Heights of 45 and 15 Feet

Barrier Height (feet)	Post-Liquefaction (Residual Strength)	
	Factor of Safety	
	Upstream Slope (10:1)	Downstream Slope (15:1)
45	2.0	1.8
15	3.8	4.2

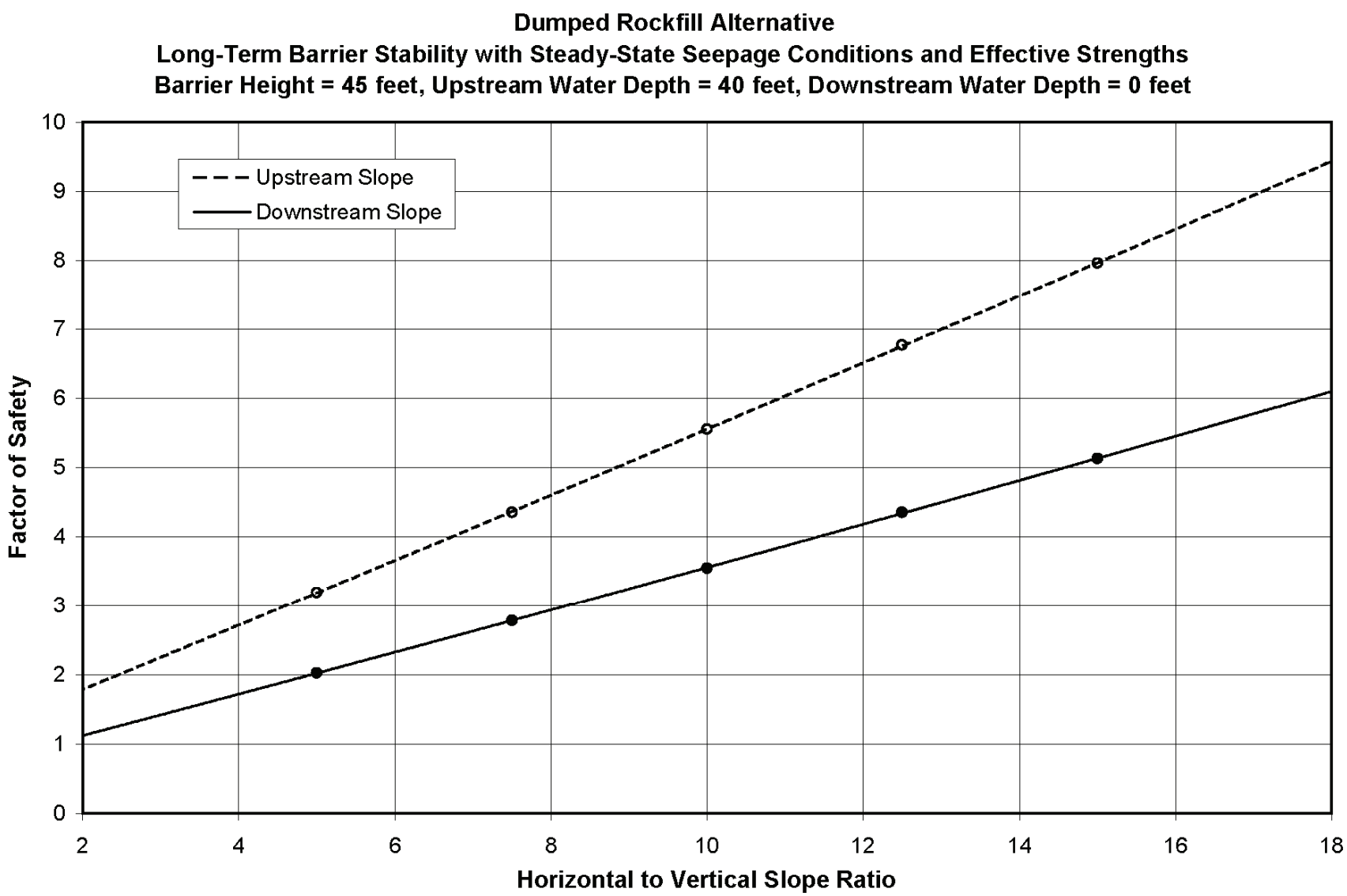

FIGURE H4-21
VARIATION OF FACTOR OF SAFETY WITH BARRIER SLOPE FOR STEADY-STATE CONDITIONS (45-FOOT HIGH BARRIER)

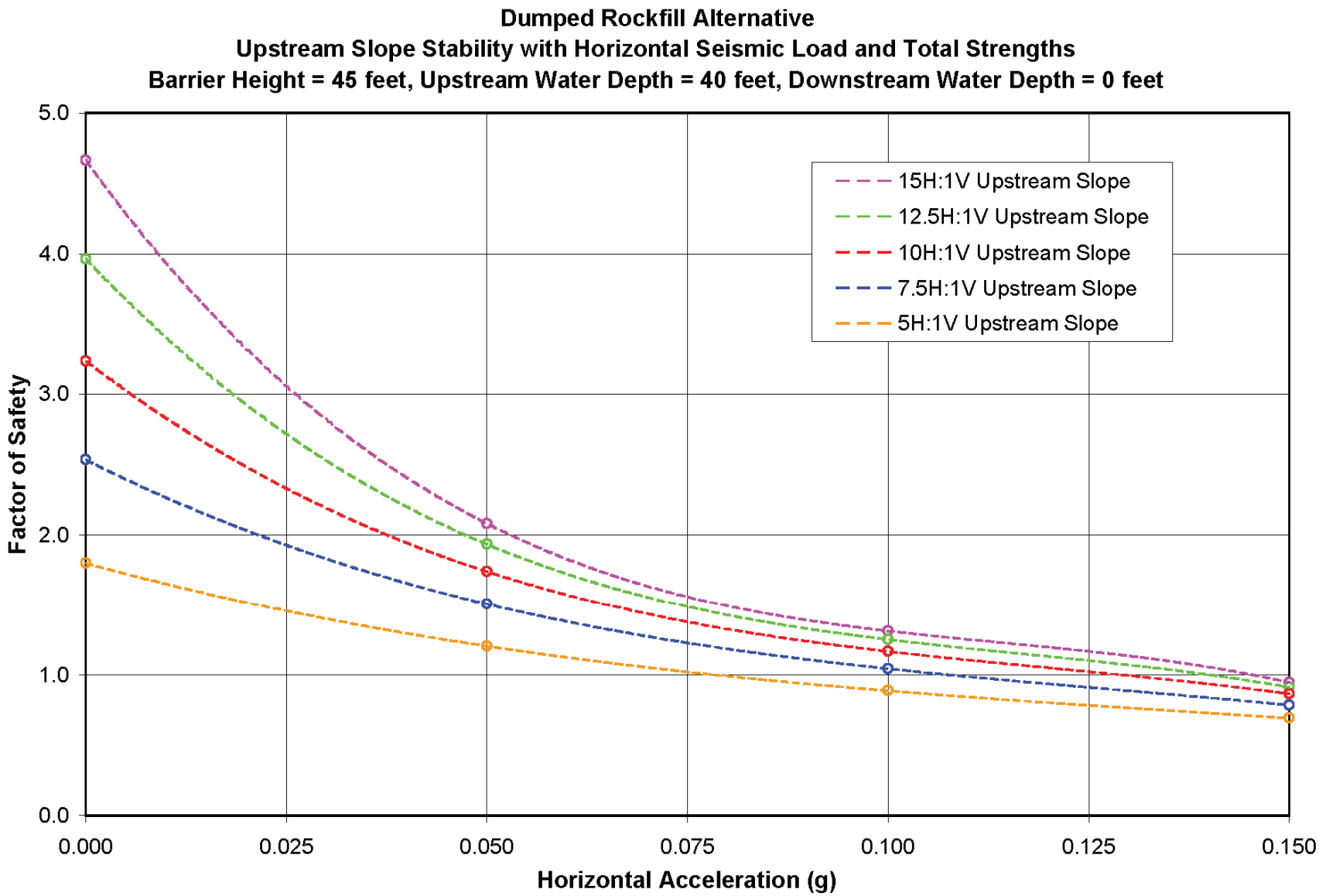

FIGURE H4-22
VARIATION OF FACTOR OF SAFETY WITH SEISMIC LOAD FOR UPSTREAM SLIDING SURFACES (45-FOOT HIGH BARRIER)

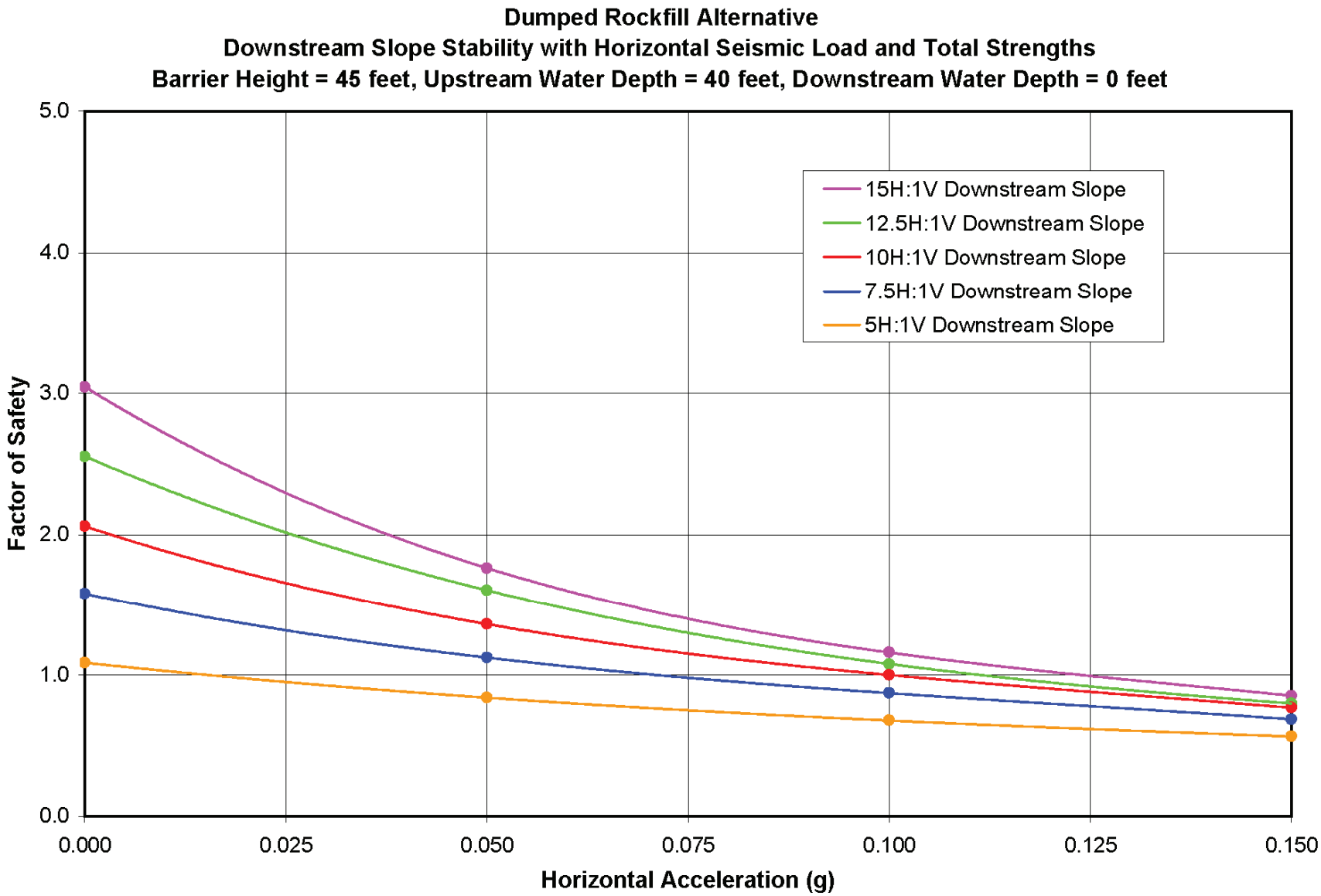

FIGURE H4-23
VARIATION OF FACTOR OF SAFETY WITH SEISMIC LOAD FOR DOWNSTREAM SLIDING SURFACES (45-FOOT HIGH BARRIER)

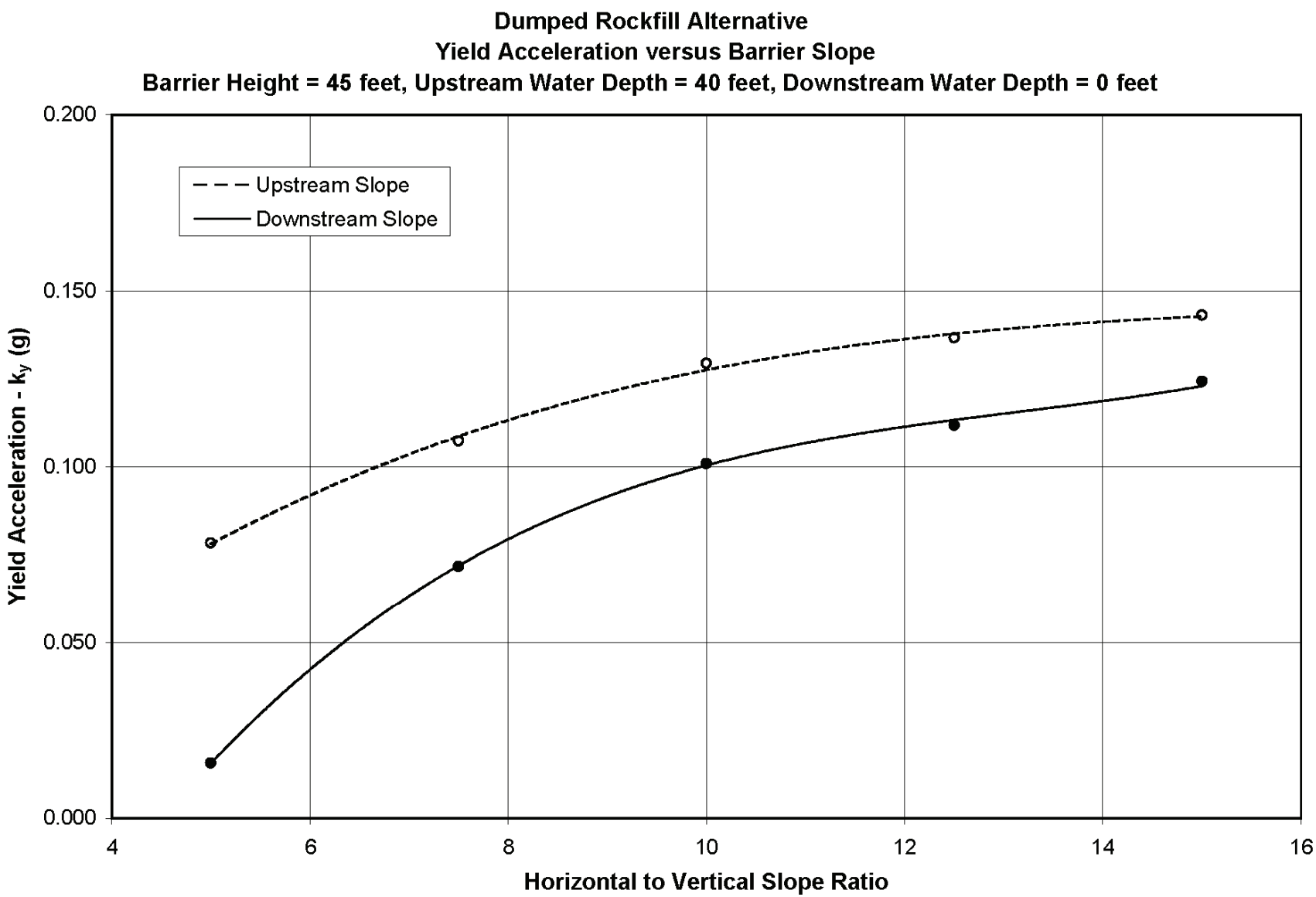

FIGURE H4-24
VARIATION OF YIELD ACCELERATION WITH BARRIER SLOPE (45-FOOT HIGH BARRIER)

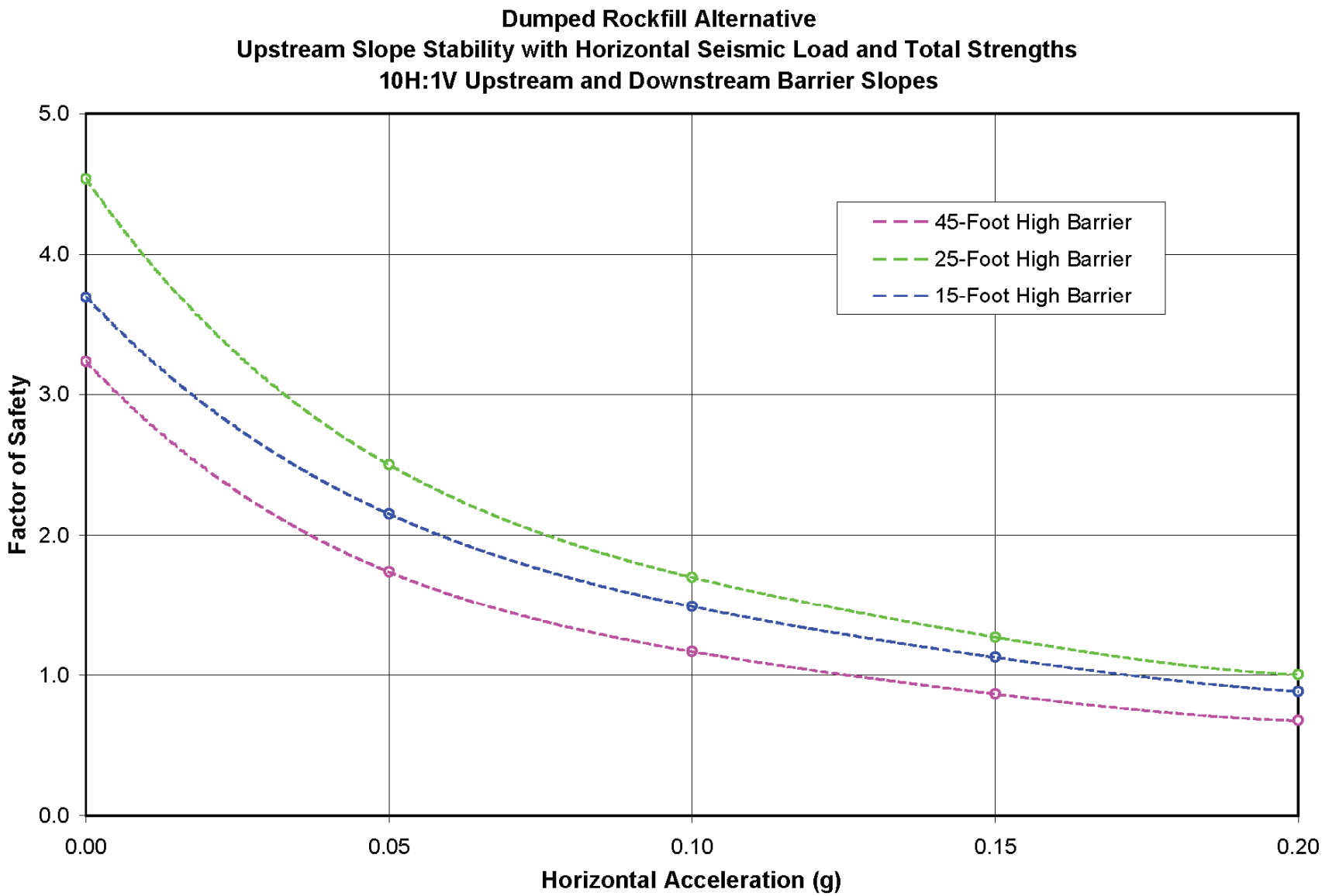

FIGURE H4-25
VARIATION OF FACTOR OF SAFETY WITH SEISMIC LOAD FOR UPSTREAM SLIDING SURFACES (45-FOOT, 25-FOOT, AND 15-FOOT HIGH BARRIERS)

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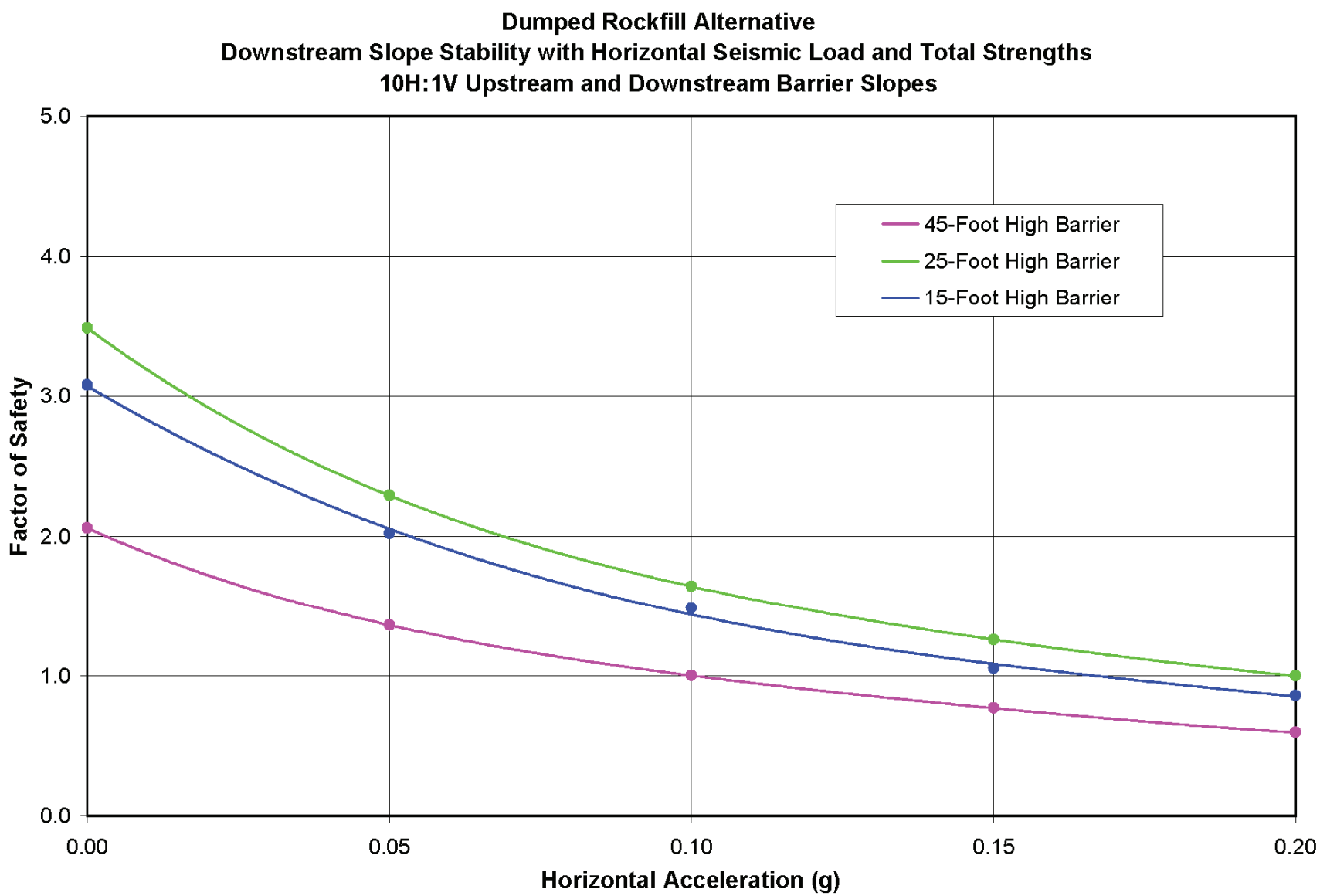

FIGURE H4-26
VARIATION OF FACTOR OF SAFETY WITH SEISMIC LOAD FOR DOWNSTREAM SLIDING SURFACES (45-FOOT, 25-FOOT, AND 15-FOOT HIGH BARRIERS)

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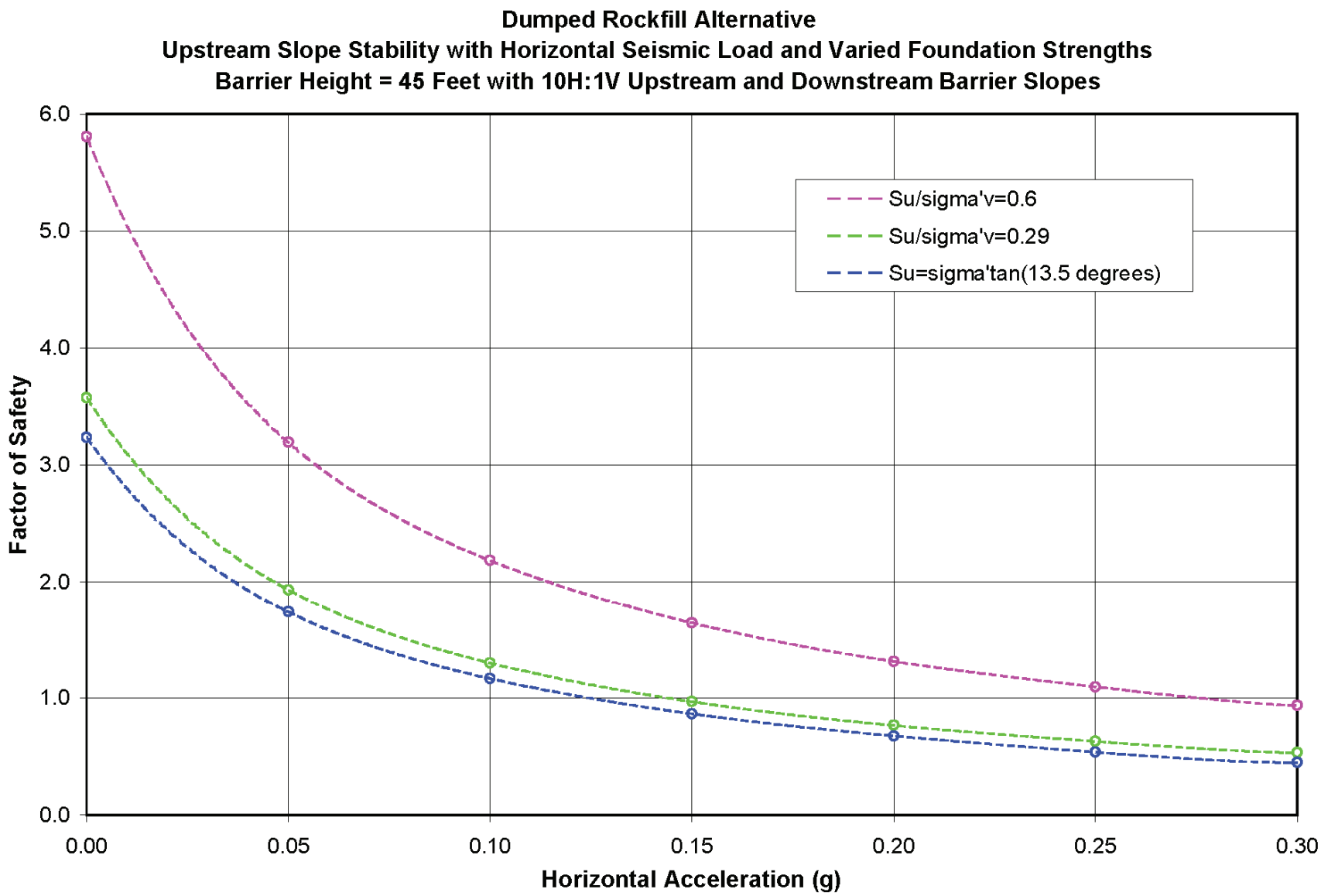

FIGURE H4-27
VARIATION OF FACTOR OF SAFETY WITH SEISMIC LOAD FOR UPSTREAM SLIDING SURFACES (45-FOOT HIGH BARRIER WITH VARIED FOUNDATION STRENGTH)

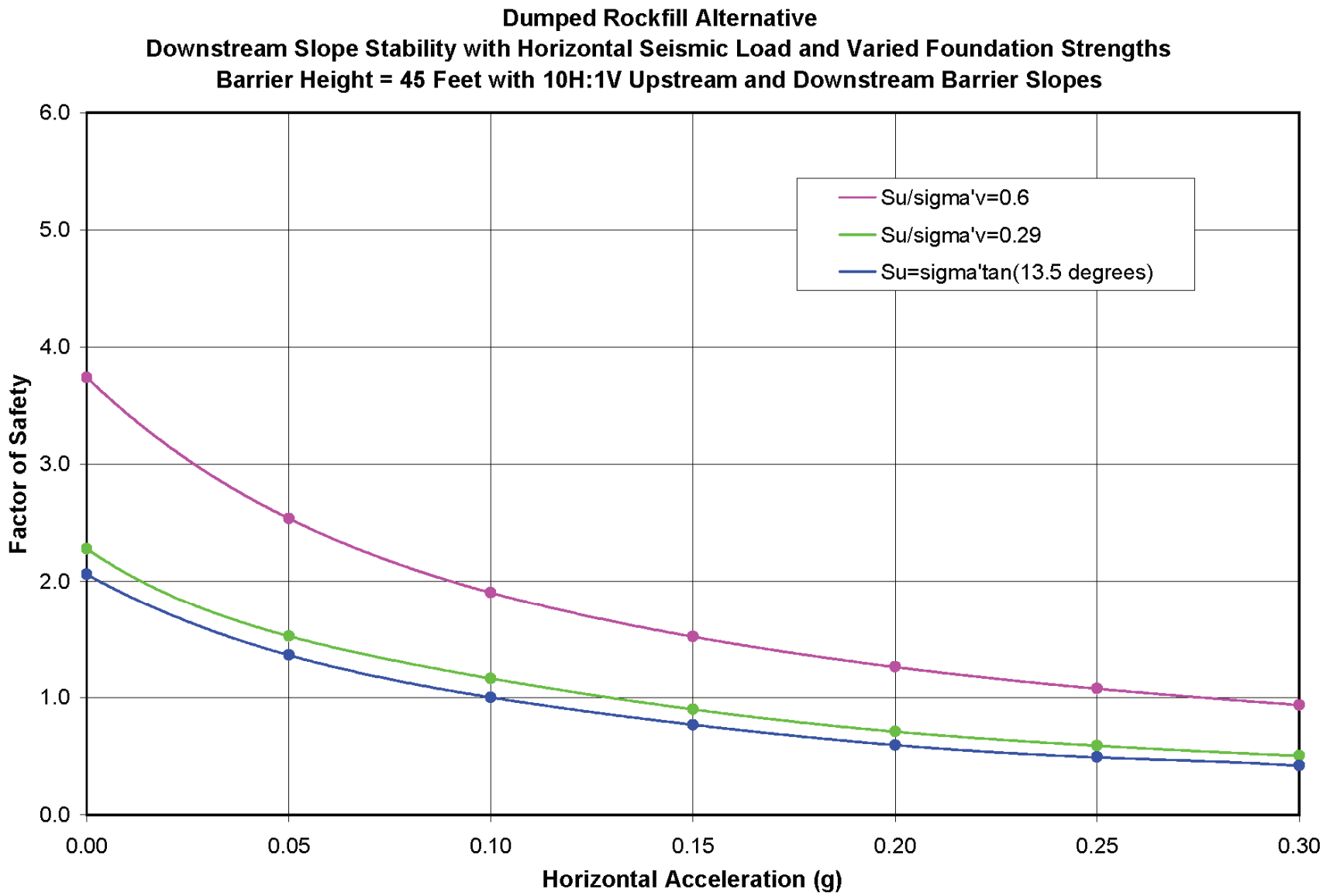

FIGURE H4-28
VARIATION OF FACTOR OF SAFETY WITH SEISMIC LOAD FOR DOWNSTREAM SLIDING SURFACES (45-FOOT HIGH BARRIER WITH VARIED FOUNDATION STRENGTH)

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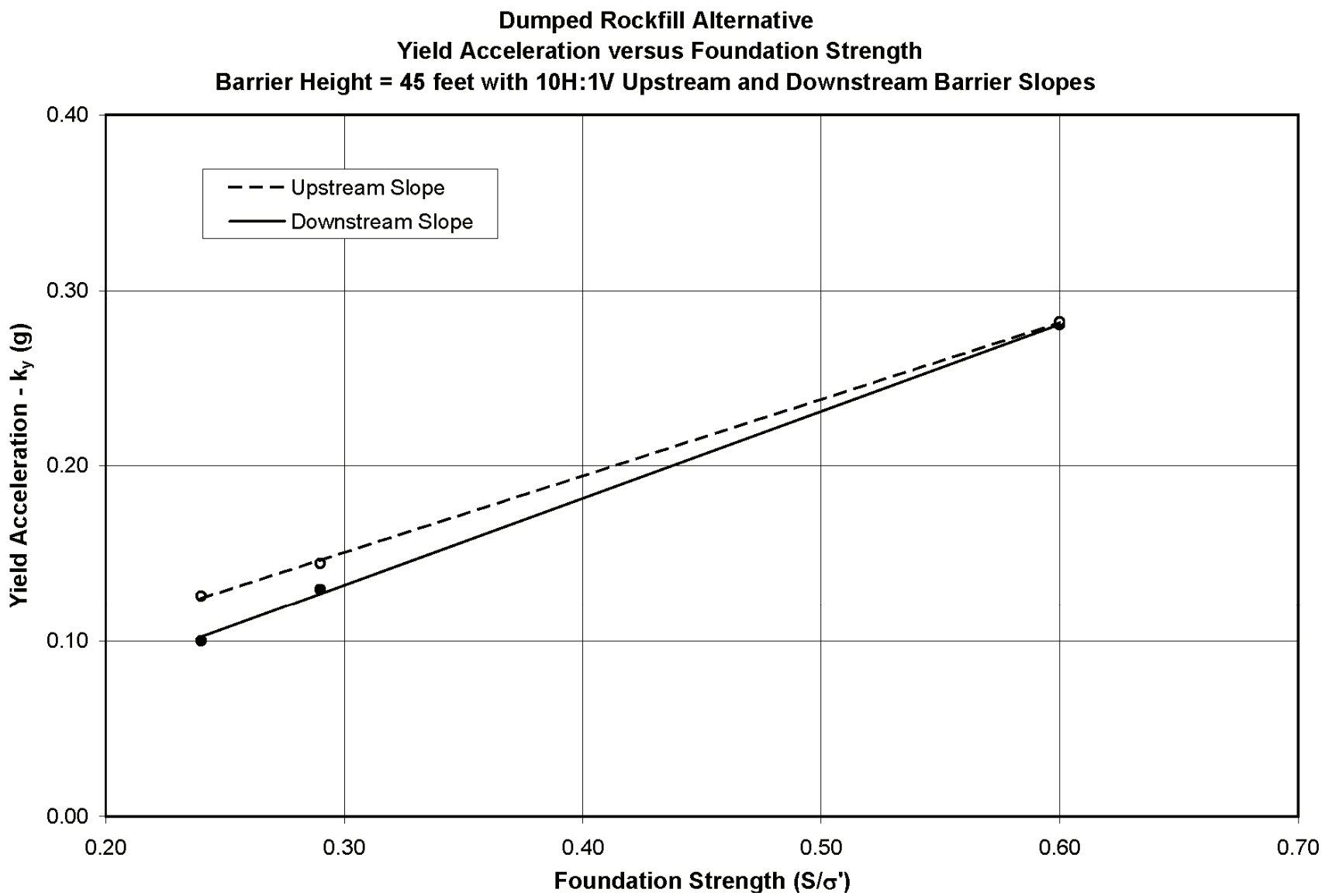


FIGURE H4-29
VARIATION OF YIELD ACCELERATION WITH FOUNDATION STRENGTH (45-FOOT HIGH BARRIER)

It is noted that liquefaction-induced settlements in the foundation soils were estimated by URS (2005). The results indicate that minor settlements of less than 6 inches would occur.

More detailed foundation investigations and analyses would be needed in order to optimize the final dam configuration and slopes. As mentioned previously, URS, in cooperation with Reclamation, is currently planning additional field exploration as part of Reclamation's Phase II Feasibility Study (URS, 2006). The focus of this investigation would be to better characterize the foundation and determine its liquefaction potential. Field work is anticipated to begin in spring or summer of 2006. However, the results will not likely be completed in time to be incorporated into the State's PEIR.

EARTHQUAKE GROUND MOTIONS

Introduction

A suite of ground motions was developed to represent the design earthquake based on the peak ground accelerations shown in Table H4-7 and the response spectra shown in Tables H4-3 through H4-5.

Selected Ground Motions

The ground motion that may occur at a site due to the design earthquake has a wide range of potential characteristics. One characteristic is the effect of directivity – this effect accounts for the phenomenon where the rupture front propagates towards a site. This effect is greatest when the direction of slip is in alignment with the site. Motions with significant forward directivity can be shorter in duration than average motions, but will typically have large velocity pulses in the fault normal direction. Motions with backward directivity may be much longer in duration but with a reduction in the long period response spectrum. In addition, the fault rupture process, path, and basin effects can have a significant effect on the time history characteristics. Developing ground motions for near field conditions is an especially uncertain process and is an active area of research.

A total of six ground motions were selected to represent the design earthquake recorded during earthquakes having magnitudes of M6.9 to M7.9. Because few ground motions have been recorded at near field locations during large magnitude strike slip events, the selected ground motions are compared to six synthetic accelerograms developed for M8 to M8.2 earthquakes at a location north of the Salton Sea also along the San Andreas Fault.

This suite of motions includes five consistent pairs of fault normal and fault parallel components (i.e., each pair are orthogonal components of the same motion that have been rotated to their fault normal and fault parallel directions). The two pairs of synthetic ground motions developed by Somerville (2003) were also intended to represent orthogonal components of the same motion, although they have the appearance of being more independent than many natural recordings.

A brief description of the selected earthquake records is given below. Additional information is provided in Table H4-12.

- PS10FN / PS10FP: Pump Station 10 record from 2002 Denali earthquake (M7.9). Record was reprocessed by USGS and rotated to fault normal (21.6°) and fault parallel (111.6°) components.
- LUC51 / LUC321: Lucerne Valley record from 1992 Landers earthquake (M7.3). Record was rotated to fault normal (51°) and fault parallel (321°) components.
- ELC53 / ELC143: El Centro Array #9 record from 1940 El Centro earthquake. Although earthquake was only M6.9, the record has a significant duration and was obtained at a location

roughly 40 miles from the Salton Sea site. Record was rotated to fault normal (53°) and fault parallel (143°) components.

- R1300 / R1390: Modified synthetic records selected from a suite developed by Paul Somerville using fault rupture modeling (hybrid Green's function method).
- R1500 / R1590: Modified synthetic records selected from a suite developed by Paul Somerville using fault rupture modeling (hybrid Green's function method).
- S1021m: Modified synthetic record selected from a suite developed by Walt Silva as reported by Wong (2004) using fault rupture modeling (stochastic finite-fault ground motion model).
- SIM8: Synthetic record for M8 originally developed by Seed and Idriss (1969). This record is included to provide consistency with previous analyses and past practice.

As shown in Table H4-12, the selected ground motions prior to modification cover a range in directivity conditions. Most of the motions appear to represent a significant length of fault rupture toward the site from 42 km to 335 km, with the two sets of synthetic records developed by Somerville modeling the greatest distance (240 km and 335 km). Each of these motions represents a different fault rupture scenario and each must, in some sense, be considered independently.

Spectral Matching of Ground Motions

None of the unmodified records listed in Table H4-12 provide a good match to the target response spectra over the entire frequency range of interest. The simplest approach to improve the spectral match is to multiply the original record by a uniform scaling factor. This scaling preserves the original character of the ground motion, although factors that are especially large (> 2) or small (< 0.5) may produce records that are no longer physically reasonable.

Although simple, uniform scaling can lead to records that significantly exceed the response spectrum or are unacceptably deficient over extensive ranges in frequency. This problem seems to worsen for records with pronounced directivity effects or for target spectra representing large magnitude earthquakes at near field locations. To address this concern, non-stationary spectral matching was used to adjust the earthquake records developed for the eastern shore using the computer program RSPMATCH (Abrahamson, 1998). This program modifies the acceleration history in the time domain by adding wavelets at appropriate times and of suitable magnitudes and frequencies. This method is a significant improvement over frequency domain approaches as it tends to preserve the character of the original motion.

Spectral matching with RSPMATCH was performed over a frequency range of 100 Hz to 0.25 Hz (i.e., response periods of 0.01 to 4 seconds). A limited amount of additional modification was performed to each record to ease its use in analysis. To simplify the analysis, the acceleration histories developed for the eastern shore were scaled by a uniform factor to produce the motions for the western shore.

Five sets of earthquake records were developed as shown below. The effects of forward directivity were considered in Cases 1 and 3 by selecting motions with appropriate time domain characteristics and then modifying the time histories to satisfy the target spectrum.

- Case 1: Eastern shore. Target spectrum modified for full forward directivity in the fault normal direction. Spectral matching by RSPMATCH. Includes PS10FN, LUC51, R1390, and R1590.
- Case 2: Eastern shore. Target spectrum not modified for directivity. Spectral matching by RSPMATCH. Includes PS10FP, LUC321, ELC53, ELC143, R1300, R1500, S1021m, and SIM8.

Table H4-12
Summary of Selected Ground Motions Prior to Modification

Description of Original Unscaled Records from Strike-Slip Events or Simulations

Name	Earthquake Name	Mw	Rupture Distance (km)	Epicentral Distance (km)	Rupture Length (km)	Length Ratio X [1]	Site Type	Orientation to Fault	Source of Record [2]	PGA (g)	PGV (m/s)	Dur. D_{5-95} (sec)
PS10FN	2002 Denali	7.9	3	88	> 340	0.25	Holocene gravel	Normal	USGS	0.29	1.15	26.3
PS10FP								Parallel		0.32	1.51	25.3
LUC51FN	1992 Landers	7.3	2	42	85	0.5	6m Dec. Granite	Normal	COSMOS	0.74	1.11	13.2
LUC321FP								Parallel		0.72	1.10	13.8
ELC53FN	1940 El Centro	6.9	8.3	--	> 40	0.15	> 300m alluvium	Normal	PEER	0.21	0.23	24.1
ELC143FP								Parallel		0.29	0.38	23.9
R1590FN	Synthetic	8.2	5	240	470	0.5	soft rock	Normal	Somerville	0.56	2.18	50.1
R1500FP								Parallel		0.35	1.05	39.6
R1390FN	Synthetic	8.2	5	335	470	0.7	soft rock	Normal	Somerville	0.96	2.13	78.1
R1300FP								Parallel		0.77	0.93	65.4
S1021M	Synthetic	8	7.5	--	576	--	soil	Normal ?	Silva	0.47	0.93	73.0
SIM8	Synthetic	8	--	--	--	--	soft rock ?	Parallel ?	Seed/Idriss	0.42	0.39	37.6

Notes:

[1] Length ratio is the fault length that ruptures towards the site divided by the total rupture length.
X=1.0 indicates full forward directivity, X=0.0 indicates full backward directivity.
X for 1940 El Centro taken from Somerville et al (1997). Otherwise, values estimated from reported distances.

[2] Source of record before rotation or other modification.

- Case 3: Western shore. Target spectrum modified for full forward directivity in fault normal direction. Spectral matching by factoring Case 1 records by 1/1.75. Includes PS10FN, LUC51, R1390, and R1590.
- Case 4: Western shore. Target spectrum not modified for directivity. Spectral matching by factoring Case 2 records by 1/1.70. Includes PS10FP, LUC321, ELC53, ELC143, R1300, R1500, S1021m, and SIM8.
- Case 5: Natural records. No spectral matching or scaling. Used for comparison to Cases 1 through 4. Includes PS10FN, PS10FP, LUC51, LUC321, ELC53, and ELC143.

Although 10 of the original records are composed of 5 pairs of fault normal and fault parallel components, these pairs are no longer consistent after spectral matching. Different modifications were applied to each component by RSPMATCH.

Several of the records were difficult to modify using RSPMATCH without significantly changing the original character of the record. These included the recordings from the Lucerne Valley site. For these records, an attempt was made to balance the requirements of reasonable earthquake character and spectral matching.

Ground Motions for Preliminary and Final Barrier Design

The ground motions developed for this study are intended to match the level commensurate with that of a Programmatic Environmental Impact Report. To better estimate applicable ground motions for future preliminary and final barrier designs, given site specific locations, the following should be considered:

- The deterministic response spectra are based on current attenuation relationships. However, the Next Generation Attenuation of Ground Motions Project (NGA) sponsored by the Pacific Earthquake Engineering Research Center (PEER) is expected to propose revised and updated attenuation relationships. Many of the participants in the NGA project are also responsible for the current attenuation relationships. Any new relationships should be reviewed and the target spectrum modified as appropriate.
- Ongoing research into directivity effects may eventually permit improvements to the estimated response spectra for forward directivity conditions (e.g., (Somerville, 2003). The current study uses an accepted broad band model for adjusting the response spectrum, which may be conservative in some situations.
- Additional efforts may identify records that are more appropriate for the Salton Sea site. Specific concerns related to this site can be addressed, such as basin effects and the potential for fling movements. Refinements to the spectral matching may be desirable.

The usefulness of the Case 5 evaluation may be improved by adding additional natural recordings.

Displacement and Yield Acceleration

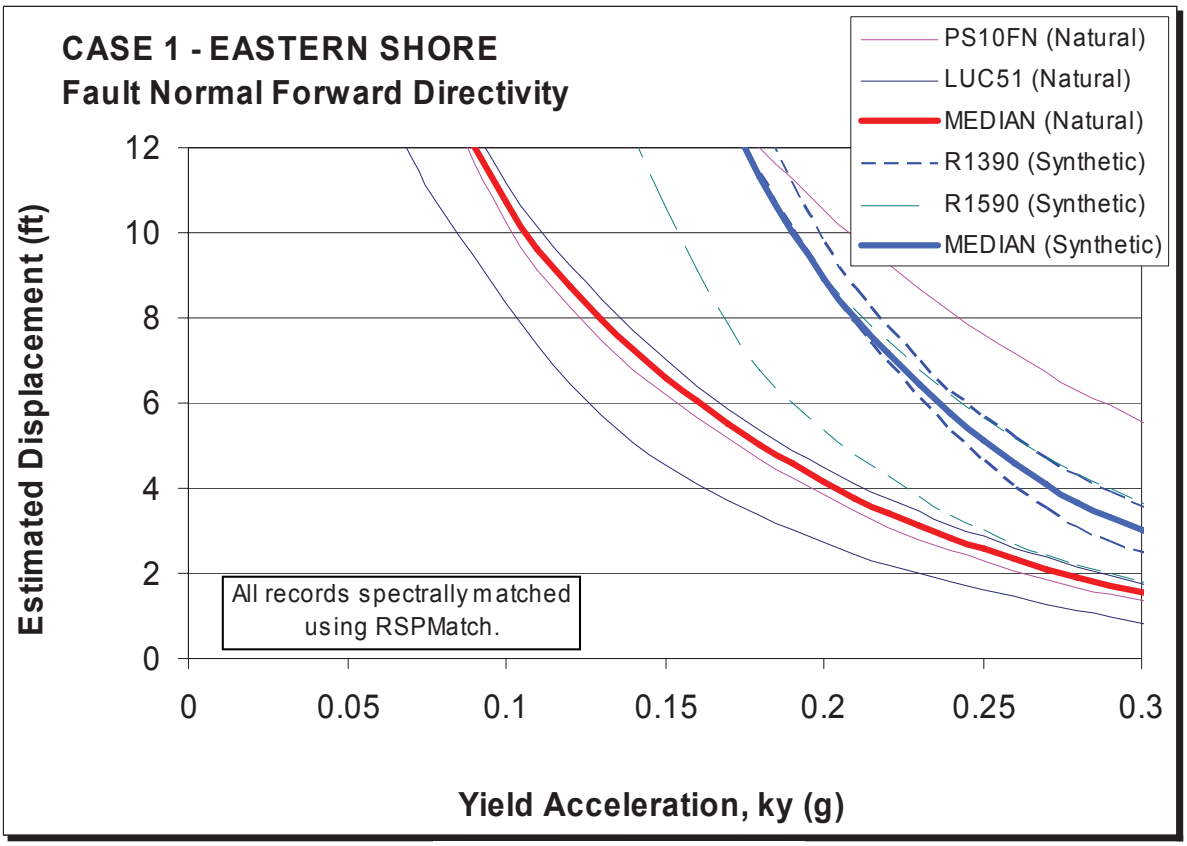
Plots of yield acceleration versus displacement were developed from the design ground motions. The displacements were estimated using an integration procedure based on the method of Newmark (Newmark, 1965). Newmark's method accounts for permanent deformations which occur whenever the acceleration from the ground motions exceed the yield acceleration as determined previously (Figure H4-22 and Figure H4-29).

Five different sets of ground motions were evaluated as discussed previously and summarized in Table H4-13**.** Displacement estimates were made for the western shore and the eastern shore to bracket the range in displacement predictions. Displacement estimates and response spectra for the design motions are summarized on Figure H4-30 through Figure H4-34.

Table H4-13
Summary of Analysis Cases for Displacement Calculations

Case	Location	Significant Forward Directivity?	Target Spectrum	Spectral Matching Technique	Results
1	Eastern Shore	Yes	Modified	RSPMATCH	Figure H4-30
2	Eastern Shore	No	Unmodified	RSPMATCH	Figure H4-31
3	Western Shore	Yes	Modified	Case 1 / 1.75	Figure H4-32
4	Western Shore	No	Unmodified	Case 2 / 1.70	Figure H4-33
5	Varies	Varies	None	None	Figure H4-34

Figure H4-30 through Figure H4-33 clearly show the severity of loading represented by the current seismic hazard evaluation. Figure H4-34 summarizes displacement estimates based on natural, unmodified recordings and is provided solely for comparison. The Lucerne Valley and Pump Station 10 recordings are actual near field motions from large strike slip earthquakes. Very few recordings of this type have been made anywhere in the world. The El Centro record is from a smaller magnitude earthquake but was made at a location roughly 40 miles from the site.



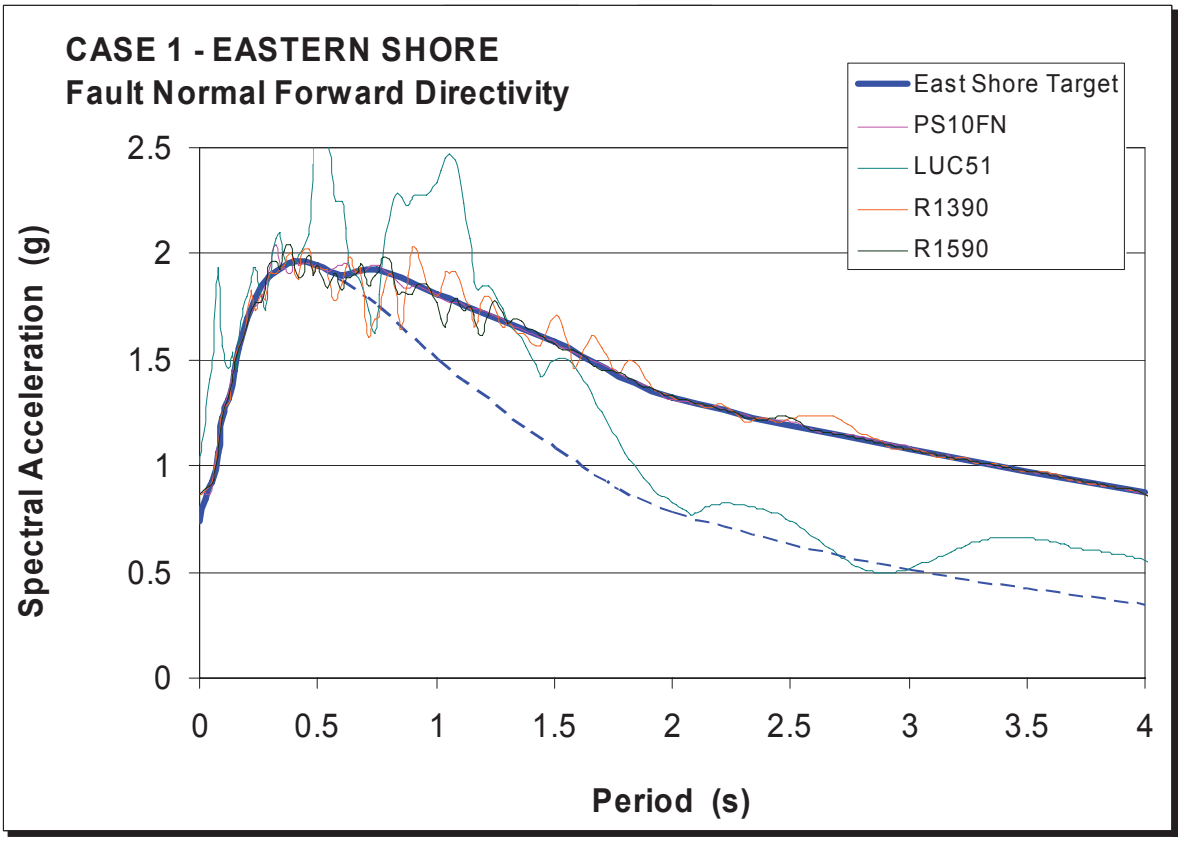
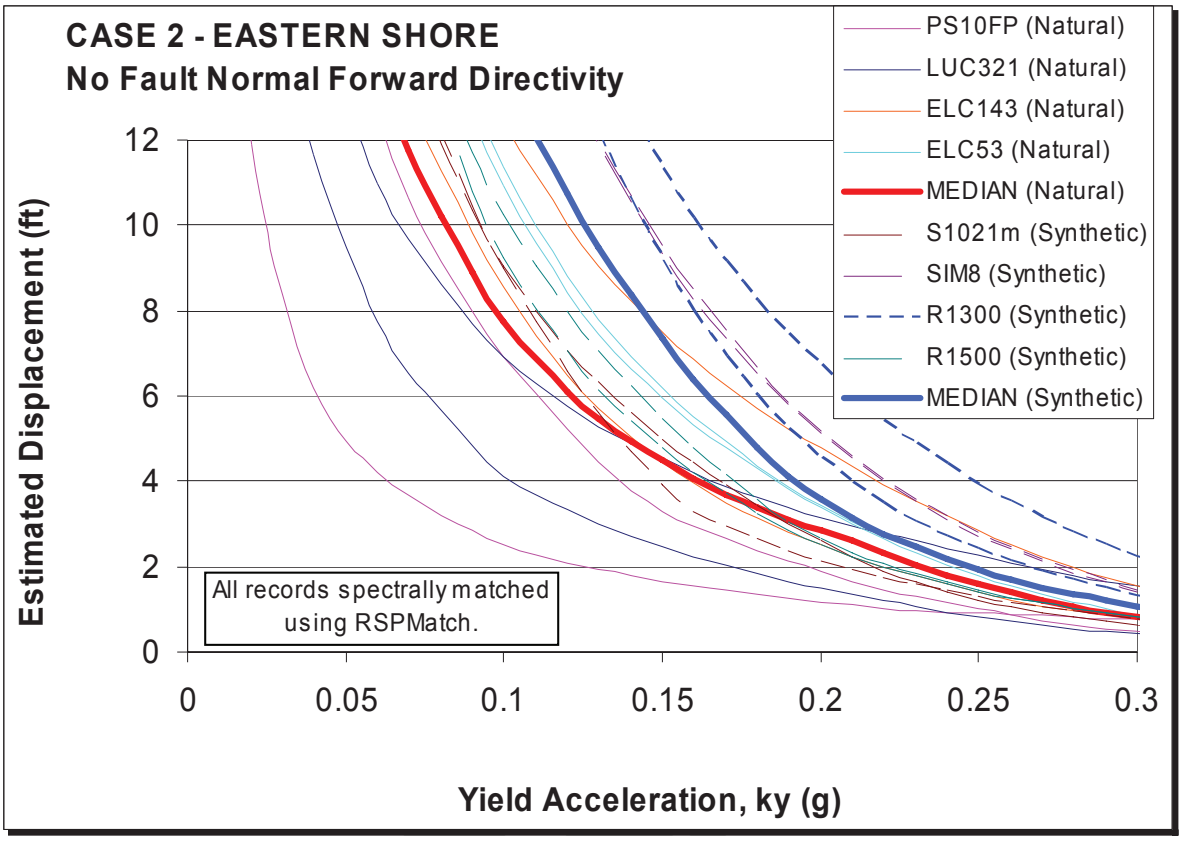


FIGURE H4-30
DISPLACEMENT AND RESPONSE SPECTRA FOR CASE 1 (EASTERN SHORE)



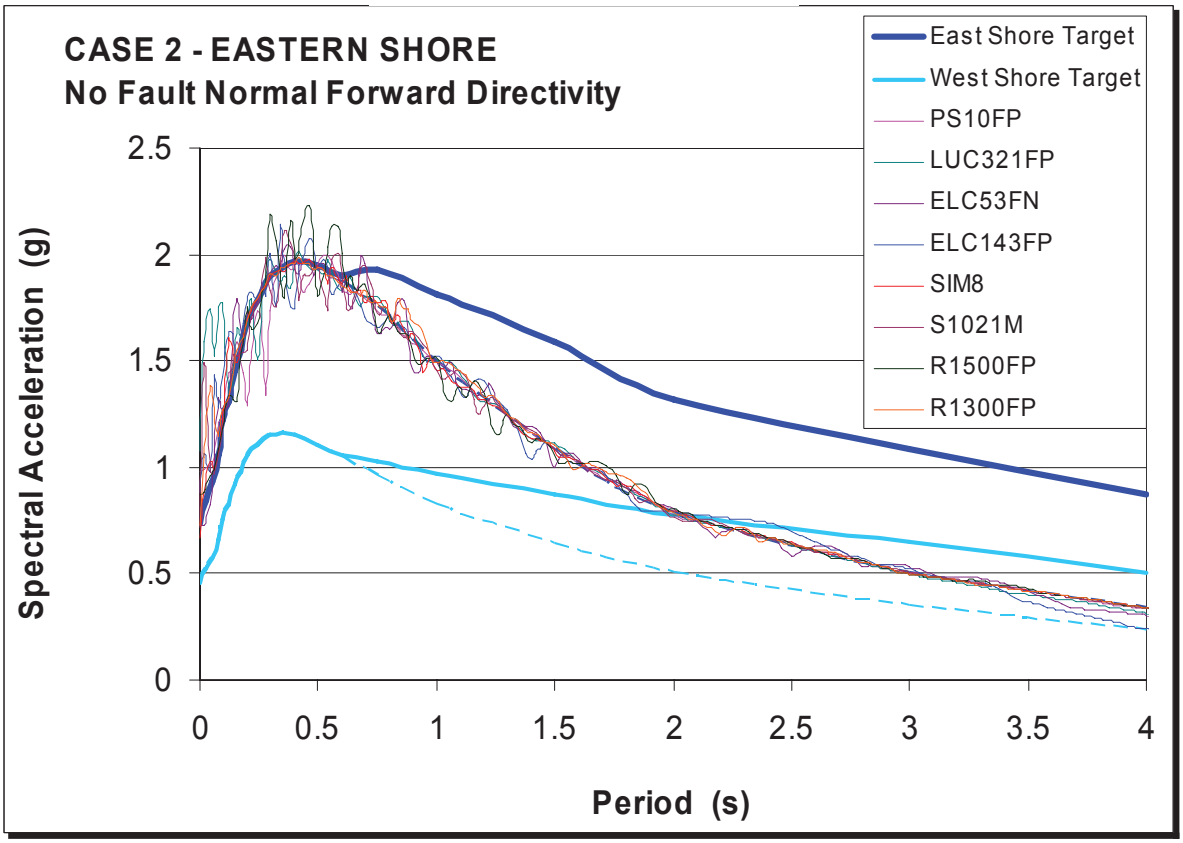
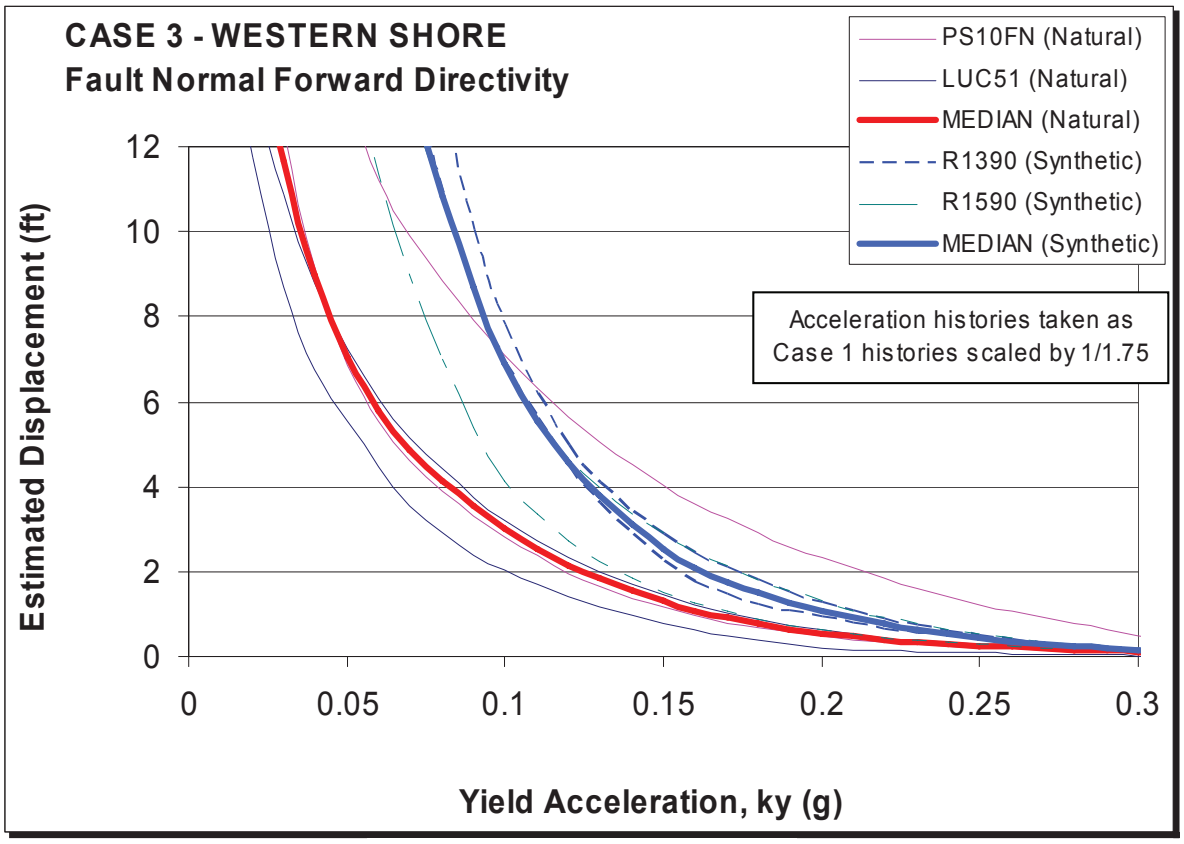


FIGURE H4-31
DISPLACEMENT AND RESPONSE SPECTRA FOR CASE 2 (EASTERN SHORE)



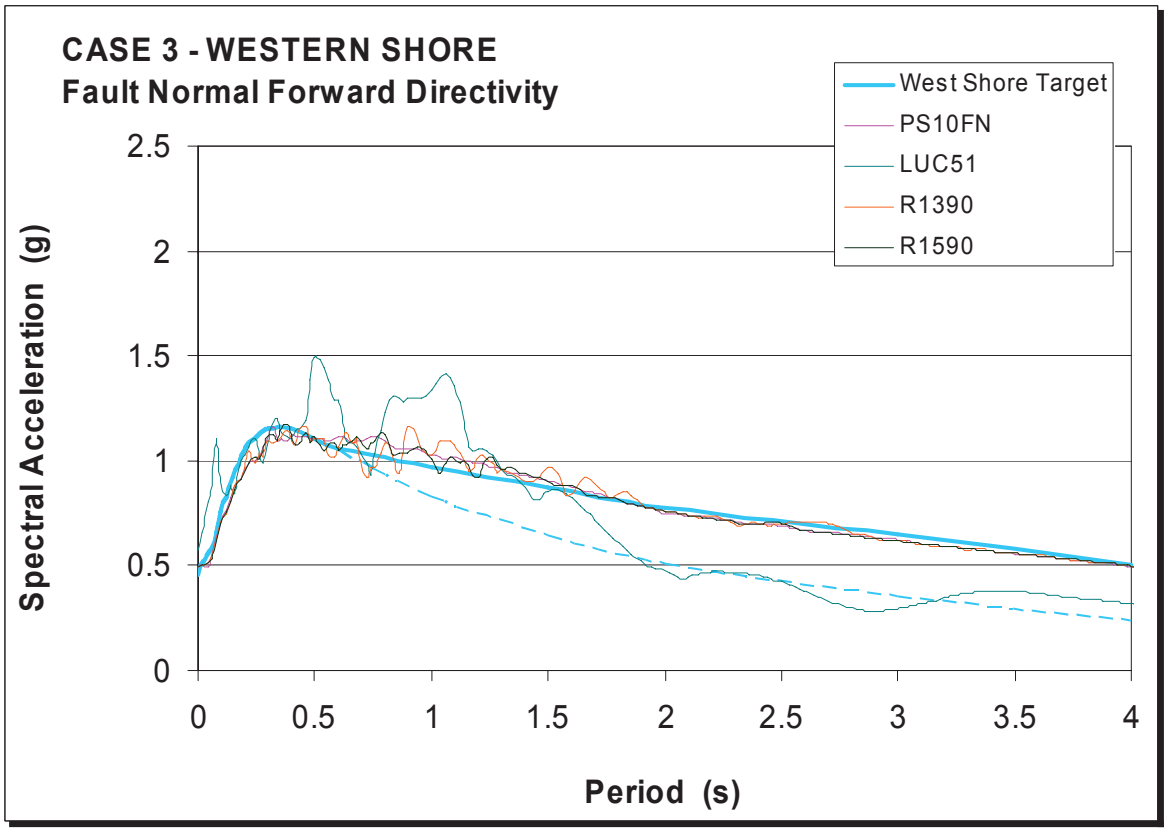
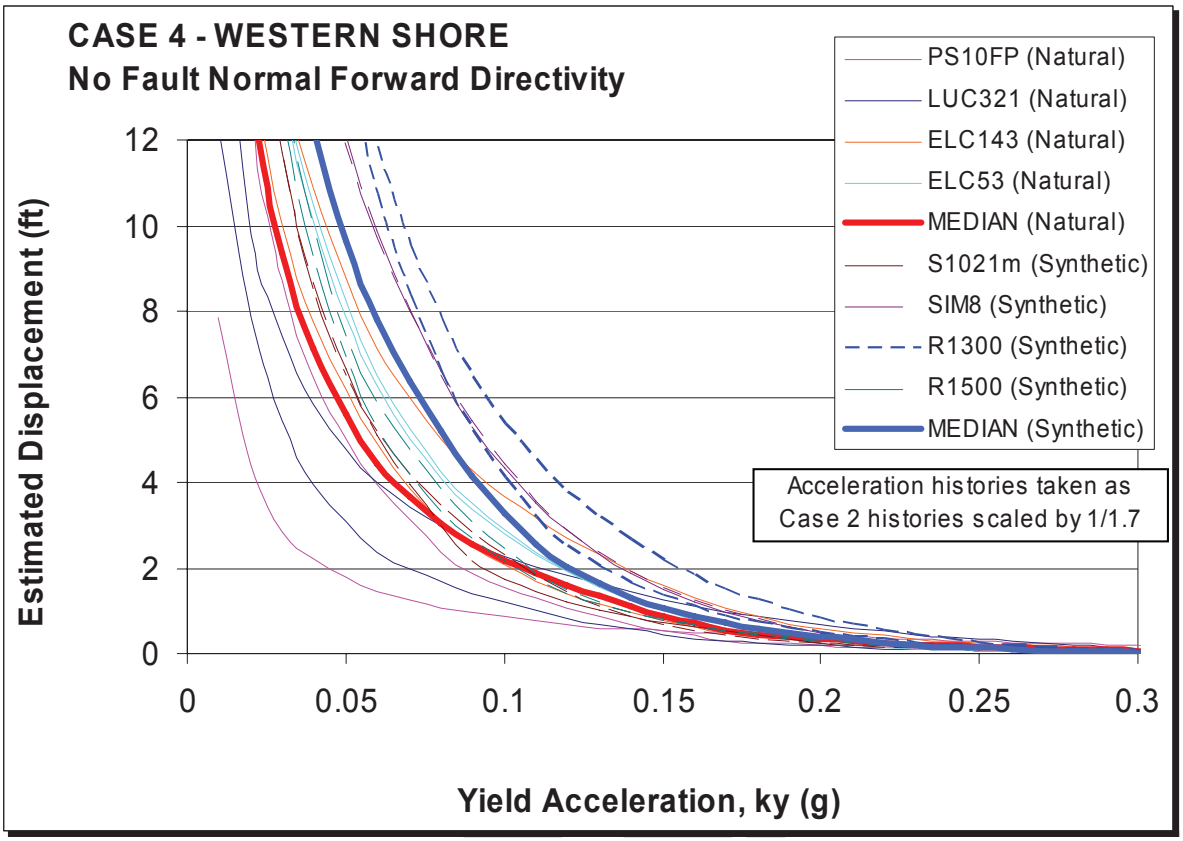


FIGURE H4-32
DISPLACEMENT AND RESPONSE SPECTRA FOR CASE 3 (WESTERN SHORE)



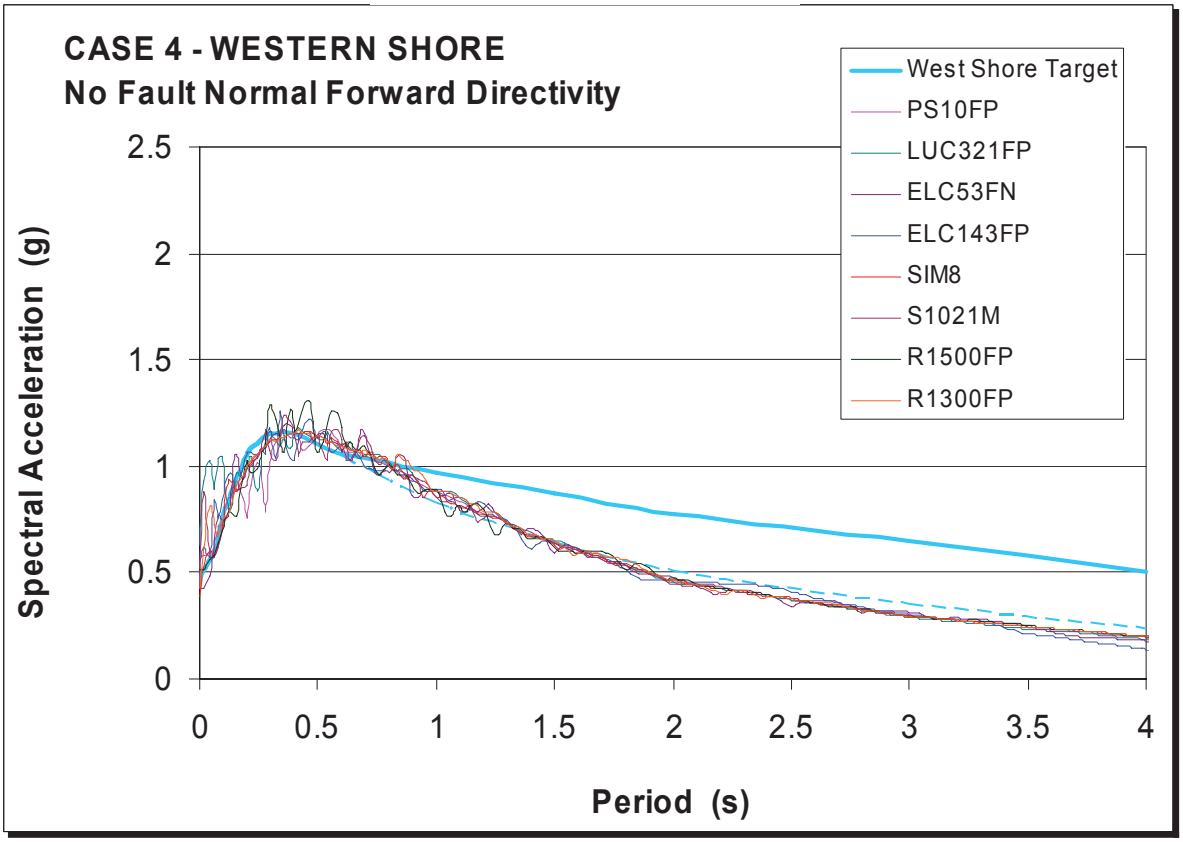
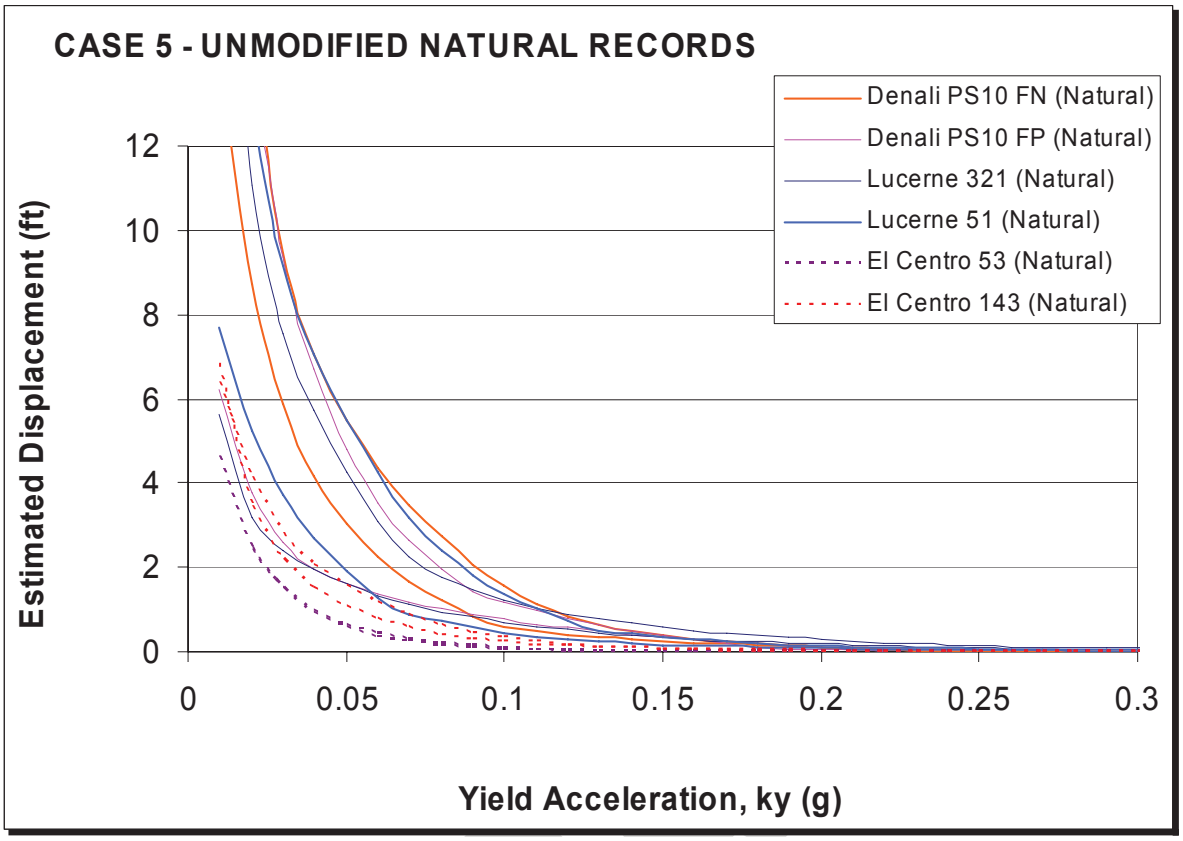


FIGURE H4-33
DISPLACEMENT AND RESPONSE SPECTRA FOR CASE 4 (WESTERN SHORE)



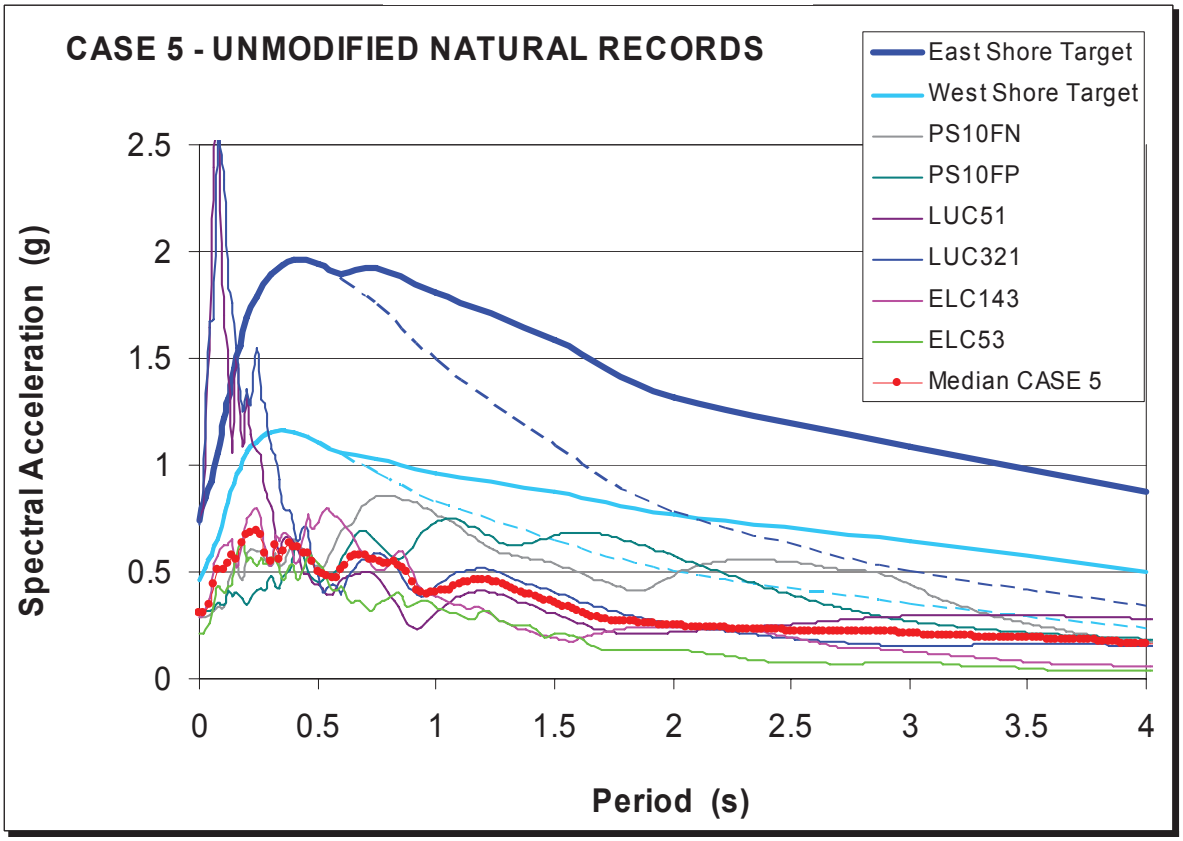


FIGURE H4-34
DISPLACEMENT AND RESPONSE SPECTRA FOR CASE 5

SETTLEMENT ANALYSIS

Introduction

Settlement analyses were performed for barrier heights of 45, 25 and 15 feet. These analyses were done to estimate the additional quantity of rockfill materials needed to accommodate the anticipated settlements of the Sea Bed deposits.

Analysis Parameters

General Parameters

The barrier sections examined in this analysis had the following general parameters:

- Barrier Crest Width: 20 feet
- Barrier Crest Elevation: -225 feet MSL
- Freeboard: 5 feet
- Marine Sea Elevation: -230 feet MSL

Barrier Sections

The settlement analyses were conducted on barriers of varying heights up to a maximum height of 45 feet. To be consistent with the stability analysis, upstream and downstream side slopes of 5H:1V, 10H:1V, and 15H:1V were examined in the settlement analysis. Additional settlement analyses were also performed on barrier heights of 15 and 25 feet with side slopes of 10H:1V.

Subfloor Deposit Thicknesses

A review of the consolidation test results presented in the February 2004 URS report (URS, 2004b) indicated the Sea Bed and Soft Lacustrine deposits would be the major contributors to foundation settlement. Similar to the stability analysis models, the thickness of the Sea Bed and Soft Lacustrine deposits were approximated after reviewing the geologic cross-sections described in the URS report. Since the barriers could be located anywhere in the Salton Sea, the following maximum deposit thicknesses were selected to estimate the settlement:

- Barrier Height – 45’; Sea Bed Deposit thickness – 15’; Solt Lacustrine Deposit Thickness – 25’
- Barrier Height – 25’; Sea Bed Deposit thickness – 5’; Solt Lacustrine Deposit Thickness – 5’
- Barrier Height – 15’; Sea Bed Deposit thickness – 5’; Solt Lacustrine Deposit Thickness – 5’

Consolidation Parameters

The material properties used for the consolidation analysis were based on the laboratory testing results presented in the URS report. Results of these tests were reviewed and used to evaluate the amount of settlement expected under the crest of the varying barrier heights. A summary of the consolidation parameters used in the analyses is presented in Table H4-14.

Table H4-14
In-Sea Rockfill Barrier Consolidation Parameters

Parameter	Symbol	Sea Bed Deposit	Soft Lacustrine Deposit
Total Unit Weight (pcf)	γ_t	100	110
Water Content (per cent)	W	62	44
Compression Index	C_c	0.65	0.35
Recompression Index	C_r	0.10	0.05
Initial Void Ratio	E_o	1.9	1.3
Overconsolidation Ratio	OCR	1.5	1.0
Coefficient of Consolidation (sq ft/yr)	C_v	15	20

Analysis Methodology and Results

General

The settlement analysis focused on the Sea Bed and Soft Lacustrine deposits because of their high compressibility characteristics. Only the primary settlement is considered for this analysis since secondary compression settlements are considered negligible. The maximum settlement will occur beneath the crest and the minimum settlement at the toe of the barrier.

The evaluation of the consolidation settlement was based on relevant parameters including preconsolidation pressure, stress increase due to the placement of the barrier, in-situ stress conditions, and compression indices for the virgin loading curve and the unload-reloading curve.

Time Rate of Consolidation

The results of the laboratory testing also provide information on the rate of consolidation, which predicts the end of primary consolidation. The coefficient of consolidation, required to conduct this study, is derived from the laboratory testing data using Taylor's square root of time method:

$$C_v = \frac{T\left(H_{dr}\right)^2}{t}$$

where H_{dr} is the length of the drainage path (assumed to be the deposit thickness for one-way drainage), T equal a time factor of 0.848 at 90 percent consolidation, and time t begins when barrier is constructed. The coefficient of consolidations (C_v) estimated for the Sea Bed and Soft Lacustrine deposits are 15 and 20 square feet/yr, respectively.

Analysis Results

Table H4-15 presents a summary of the settlement analysis results. The maximum estimated settlement under the crest of the 45-foot-high barrier with side slopes ranging from 5H:1V to 15H:1V is about 5.8 feet. As shown, the varied side slopes do not factor into the maximum settlement results. The maximum estimated settlement for the barrier section with heights of 25 and 15 feet is about 2.1 and 1.8 feet, respectively.

Table H4-15
In-Sea Rockfill Barrier Settlement Analysis Results

Barrier Height[a] (ft)	Side Slopes	Sea Bed (ft MSL)	Deposit	Deposit Thickness[b] (ft)	Consolidation Settlement[c] (ft)	Total Consolidation Settlement[d] (ft)	Consolidation (per cent)
45	5H:1V	-270	Seafloor	15	3.4	5.7	13
			Soft Lacustrine	25	2.3		
	10H:1V	-270	Seafloor	15	3.4	5.8	13
			Soft Lacustrine	25	2.4		
	15H:1V	-270	Seafloor	15	3.4	5.8	13
			Soft Lacustrine	25	2.4		
25	10H:1V	-270	Seafloor	5	1.4	2.1	8
			Soft Lacustrine	5	0.7		
15	10H:1V	-270	Seafloor	5	1.2	1.8	12
			Soft Lacustrine	5	0.6		

Notes:

[a] Barrier crest of -225 MSL assumed for Sea level of -230 MSL.

[b] Assumes no overexcavation of soft materials.

[c] Calculated at the centerline of the barrier with rockfill unit weight of 140 pcf, and C_c =0.65, C_r = 1.5 for the Sea Bed deposits and C_c = 0.35, C_r = 0.05, and OCR = 1.0 for the Soft Lacustrine deposits.

[d] Does not include post-construction settlement of barrier.

The results of the settlement rates of various deposit thicknesses are shown in Table H4-16. Depending upon the thickness of the foundation deposit, 90 per cent of the settlement would be expected to occur within one to ten years. Settlement would be complete within two years for the thinner deposits and within forty years for the thickest deposits.

Table H4-16
In-Sea Rockfill Barrier Settlement Rate Analysis Results

Drainage Path* H_{dr} (ft)	Deposit	Coefficient of Consolidation C_v (sq ft/yr)	Time to 90 Per Cent Consolidation, t_{90} (yrs)
5	Seafloor	15	1
	Soft Lacustrine	20	1
10	Seafloor	15	6
	Soft Lacustrine	20	4
15	Seafloor	15	13
	Soft Lacustrine	20	10
20	Seafloor	15	23
	Soft Lacustrine	20	17
25	Soft Lacustrine	20	27

Note:

* Assumes one-way drainage

SEEPAGE ANALYSES

Introduction

The in-sea barrier is intended to be a semi-pervious to pervious rockfill embankment that would allow flow between bodies of water with a head differential between its upstream and downstream sides. The main factor in establishing the intended head differential is the amount of seepage being expressed through the barrier.

The purpose of this evaluation is to provide an initial estimate of the range in seepage that could be expected to pass through the proposed in-sea barrier constructed to various heights. This seepage analysis contains a description of the assumptions and methods used in determining the amount of seepage, as well as providing an estimate and discussion of the actual estimated seepage on a flow per barrier-mile basis.

Assumptions Used in Analysis

In the development of the initial flow-net and the computer-developed seepage model, underlying assumptions regarding the material type and permeabilities were made. The following sections describe the basis for the assumptions used for these two items.

Rockfill Permeabilities

Rockfill permeabilities adopted for this seepage analysis are based on a review of permeability tests conducted on two recently constructed dams, research into similarly constructed embankments, a review of published data, and empirical correlations for an assumed rockfill gradation for the in-sea barriers. Similarly constructed embankments that were reviewed as part of this study include: the Lower Jones Tract Levee Repair, DWR temporary barriers, and the rockfill causeway across the Great Salt Lake in Utah. Additionally, test fill and laboratory permeabilities tests that were reviewed include tests conducted on rockfill materials from the recently constructed Seven Oaks and Diamond Valley Dams in Southern California.

Seven Oaks Dam

Seven Oaks Dam is a flood control dam constructed in the 1990s by the U.S. Army Corps of Engineers, Los Angeles District. The dam is located on the Santa Ana River in the upper Santa Ana Canyon about 8 miles northeast of the City of Redlands, in San Bernardino County, California (ACOE, 2006). Construction of the dam required placement of nearly 40 million cubic yards of earth and rock materials.

In-situ test fill percolation and laboratory permeability tests were conducted on materials selected to represent the upstream shell (Zone 5) and alluvial transition (Zone 3) materials placed in Seven Oaks Dam. Laboratory permeability tests included standard size and large scale 36-inch tests. Figure H4-35 presents a construction photograph illustrating the respective locations of the upstream shell and alluvial transition materials in the dam embankment.

Diamond Valley Reservoir

The Diamond Valley Reservoir project included construction of three earth-rockfill dams constructed in the 1990s by the Metropolitan Water District of Southern California. The project is located southwest of the town of Hemet in Riverside County, California. Construction of the East, West, and Saddle Dams required placement of more than 110 million cubic yards of earth and rock materials. Figure H4-36 presents a construction photograph of the West Dam illustrating the respective locations of the upstream and downstream shell materials.

FIGURE H4-35
CONSTRUCTION OF SEVEN OAKS DAM

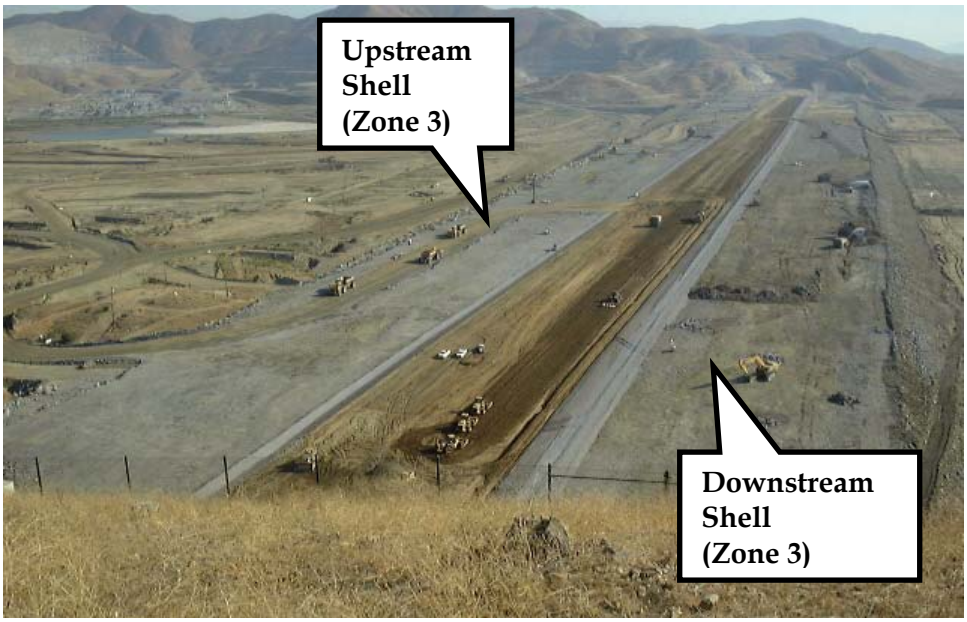


FIGURE H4-36
CONSTRUCTION OF DIAMOND VALLEY RESERVOIR WEST DAM

In-situ percolation tests were conducted in test fills constructed from materials selected to represent the shell materials (Zone 3) for the three dams. Figure H4-36 presents a summary of the tested gradations and corresponding permeability estimates.

Great Salt Lake Rockfill Causeway

The Southern Pacific Transportation Company completed a rockfill causeway across the Great Salt Lake in 1959, creating two separate but interconnected portions of the lake (Figure H4-38). In addition to culverts and an open breach, water is conveyed between the two parts of the lake through the semi-pervious rockfill barrier. The differential head between the two portions of the lake is less than 5 feet, typically ranging from 1 to 2 feet. Although the causeway has gone through many phases of modifications and repairs, the initial construction of the barrier was accomplished using skow-dump barges similar to the proposed construction method of the Salton Sea barrier.

Construction of the Salt Lake causeway began by dredging a channel 25 to 40 feet deep and 150 to 500 feet wide. The channel was backfilled with sand and gravel, and quarry rock was used as fill to raise the top of the causeway above the surface of the lake. The causeway was completed with riprap that varied in size from 1-ton capstone 15 feet below the surface to 3-ton capstone at the top. Since 1970, sand, gravel and rock have been added periodically to the causeway to maintain its height as it settles and to replace material removed by erosion from wave action.

The U.S. Geological Survey Water-Resources Investigations Report (USGS, 2000) on the Great Salt Lake causeway barrier uses a permeability range of 2.4 to 64 cm/sec in their analysis of the water and salt balance of the Great Salt Lake barrier. These estimates were based on data from dye-injection studies in 10 wells drilled into the barrier fill. The report indicates that for a head differential of 3 feet, the annual flow through the 12-mile barrier ranged from 8 to 21 million acre-feet (MAF). This equates to an average seepage rate of about 1.1 MAF/year per barrier mile. It should be noted that the Great Salt Lake barrier was intended to be relatively permeable.

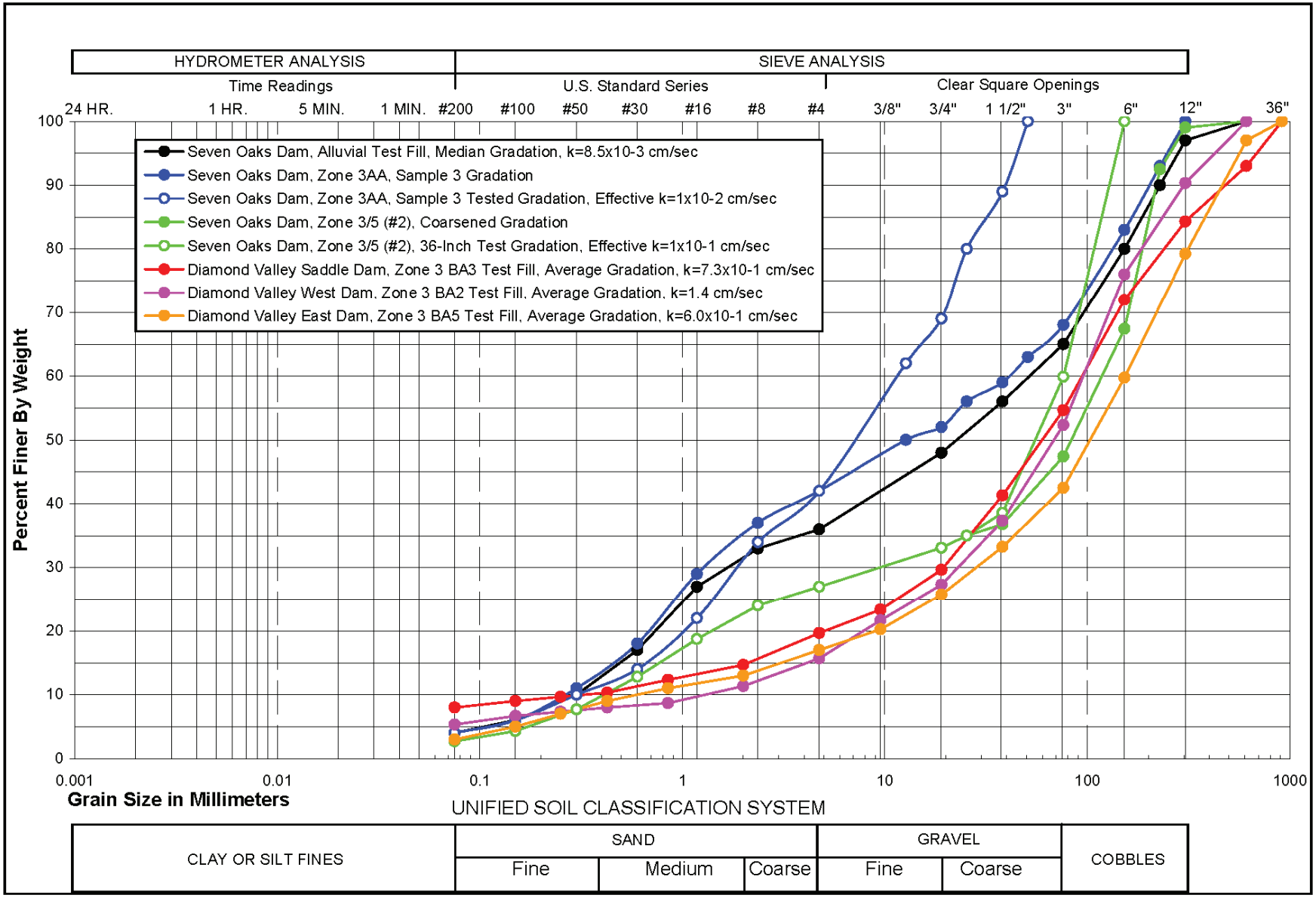


FIGURE H4-37
ROCKFILL MATERIAL GRADATIONS AND PERMEABILITY ESTIMATES

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FIGURE H4-38
GREAT SALT LAKE CAUSEWAY

Lower Jones Tract Levee Repair

On June 3, 2004, part of a levee protecting the Upper Jones Tract unexpectedly breached, inundating the 19-square mile tract 10 miles west of Stockton. The levee was repaired using barge-mounted clamshell and dragline cranes to place rockfill into the breach (Figure H4-39). Although the length of the embankment and depth of placement are quite different, construction of the Jones Tract Levee repair had some similarities to that of the proposed Salton Sea barrier.

FIGURE H4-39
BARGE-MOUNTED CRANES PLACING ROCKFILL INTO JONES TRACT BREACH

Based on conversations with DWR personnel and consultants (Peddy, 2004, Sciandrone, 2004), the rockfill plug for the Jones Tract levee repair consists of three different material gradations. Initially, in an effort to resist tidal forces, a minus-36-inch rockfill gradation was placed on the bottom 1 to 3 feet of the breach. A smaller gradation of minus-24-inch rockfill was then used for the bulk of the embankment

construction, bringing the height of the repaired levee approximately to the water surface. A minus-6-inch "choker stone" cap was placed over the outside of the repaired breach, bringing the top of the levee back to its original elevation. Figure H4-40 presents the typical rockfill and "choker stone" gradation limits used for the Jones Tract repair.

Although the removal of the floodwaters from within the island is complete, seepage is occurring through the rockfill plug. Unfortunately, the rate of seepage is not being measured at this time. Future plans for the levee repair include the installation of a seepage collection trench and flow metering device which could be used to characterize the permeability of the rockfill plug. As is the case with the original levee, the repaired breach is intended to be a relatively impermeable structure. As constructed, however, seepage is occurring through the plug. It is believed that most of the seepage is occurring through the base layer of minus-36-inch rockfill. Future placement of the minus-6-inch "choker stone" on the upstream slope is anticipated to reduce this seepage.

South Delta Temporary Barriers Project

The South Delta Temporary Barriers Project, initiated in 1991, consists of four rock barriers across South Delta channels. Of the four rock barriers, the Head of Old River barrier serves as a fish barrier, in place most years in the spring between April 15 and May 30. The remaining three barriers serve as agricultural barriers and are installed between April 15 and September 30 of each season. The objectives of the program are to increase water levels, circulation patterns, and water quality in the southern Delta area for local agricultural diversions, and to improve operational flexibility of the State Water Project to help reduce fishery impacts and improve fishery conditions (DWR Bay-Delta Office, 2006).

The barriers are created by dumping rockfill into the river channel at slack tide using end-dump trucks and bulldozers, while barrier removal is accomplished using clamshell-bucket cranes (see Figure H4-41). The barriers are intended to be semi-impermeable, with a head differential of up to 15 feet (Burns, 2004, Woodland, 2004). The material used in these temporary barriers consists of locally available rockfill, typically using the material from the previous season's barrier removal, supplemented with imported material as necessary. The maximum size of the particles varies from 24 inches to 36 inches, with a minimum particle size of ¾ inch. Due to tidal currents, a significant portion of the fines are carried downstream during placement.

Permeabilities of the temporary rock barriers are difficult to estimate. Some efforts relating to measuring flows above and below the barriers are currently being conducted, but to date have not produced any useful data. Additionally, specific gradations of the rockfill used in the barriers can vary widely from site to site, and from season to season depending on available materials and condition of the stockpiled rockfill from the previous barrier removal. It has been suggested that a pilot test be conducted during the construction of a future temporary barrier to simulate the placement methods and material type anticipated to be used in construction of the Salton Sea barrier.

Published Data

Reclamation's "Design Standards for Embankment Dams, Chapter 8 Seepage Analysis and Control" presents a range of permeability values for embankment shell materials. Published values range from about $1x10^{-2}$ to 1 cm/sec.

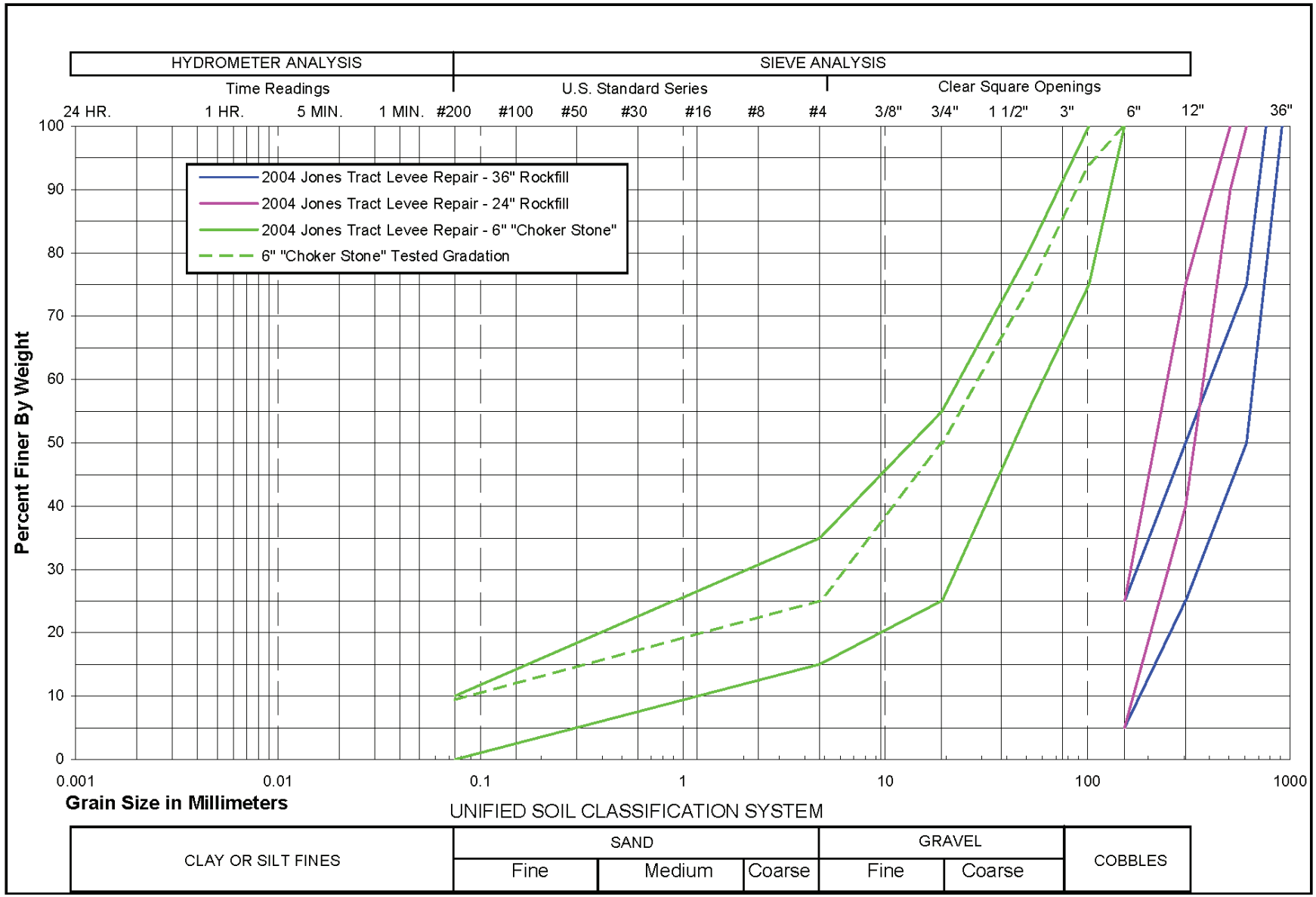

FIGURE H4-40
TYPICAL ROCKFILL AND "CHOKER STONE" GRADATION LIMITS USED AT JONES TRACT

FIGURE H4-41
TEMPORARY BARRIER LOCATED ON OLD RIVER NEAR TRACY, CALIFORNIA

Empirical Correlations and Hypothetical Rockfill Gradation

Empirical permeability correlations ($k=D_{10}^2$ and $k=0.35D_{15}^2$) were also applied to hypothetical gradation limits for the in-sea barrier rockfill to provide a rough comparison to the published data and permeability testing discussed previously. The hypothetical gradation limits for the in-sea barrier rockfill were based upon a review of the rockfill specification limits for the shell material used in the Diamond Valley Reservoir Dams and the tested gradation of the 6-inch "choker stone" used for the Jones Tract Levee Repair. The in-sea barrier rockfill was assumed to be a "dirtier" rockfill material with a greater allowable percentage of gravels, sands, and fines than the Diamond Valley shell material with the fine specification limits similar to the "choker stone" gradation. It is believed that a "dirtier" type rockfill material could be placed without substantial segregation from bottom dumped barges in the in-sea barrier. However, additional investigations on the rockfill source, material and particle sizes produced at the quarry, placement method, and in-place gradation of test fills are required to verify this. The hypothetical in-sea barrier rockfill gradation limits are presented in Figure H4-42. The Diamond Valley shell material specification limits and "choker stone" gradation are also presented on this figure for comparison.

Applying the empirical correlations to the hypothetical fine and coarse in-sea barrier rockfill gradation limits indicate an approximate permeability range from about $1x10^{-2}$ to $1x10^{1}$ cm/sec. This permeability range correlates fairly well to published values and the testing conducted for Seven Oaks Dam and Diamond Valley Reservoir. However, it should be noted that the empirical correlations are specifically applicable to sands and gravels and are not directly applicable to materials containing larger particle sizes.

Estimated Rockfill Permeabilities for the Seepage Analysis

The most critical component of this seepage analysis is the selection of the estimated permeability of the proposed in-sea barrier rockfill. The permeability range selected for use in the seepage analysis has been based on many factors. These factors include: in-situ test fill percolation and laboratory permeability tests for rockfill materials used on other projects, published permeability values, and empirical correlations for a hypothetical rockfill gradation. Based on these factors, a hydraulic conductivity range of $1x10^{-2}$ to 1 cm/sec has been selected for use in the seepage model. It is believed that 1 cm/sec represents a reasonable upper limit for the in-sea barrier rockfill. However, a less pervious rockfill can likely be

achieved with a permeability range of about $1x10^{-2}$ to $1x10^{-1}$ cm/sec. Additional investigations on the rockfill source, material and particle sizes produced at the quarry, placement method, and in-place gradation of test fills are required to accurately estimate the in-sea barrier rockfill permeability.

FEM Analysis Procedure

Once the assumptions for the proposed barrier had been established, the SEEP/W model was developed. A 2-dimensional cross section of the barrier was created, and the assumptions were input into the model. Seepage analyses were run for three selected permeability values, each at three different embankment heights/head differentials, generating the results which are presented below.

Seep/W (FEM Analysis) Model

SEEP/W is a finite element software product for analyzing groundwater seepage and excess pore-water pressure dissipation problems within porous materials such as soil and rock. The program can model both saturated and unsaturated flow, making it possible to analyze seepage as a function of time (GEO-SLOPE, 2004).

The first step in the SEEP/W analysis was to create a cross section of the embankment, after which the model assumptions were entered. Key inputs for the model are as follows:

- Embankment sideslopes: 7H:1V (upstream and downstream);
- Homogeneous isotropic embankment;
- Foundation permeability: 10^{-7} cm/s;
- Crest width: 20 feet; and
- Freeboard: 10 feet.

It is anticipated that seepage quantities are more influenced by the differential head and permeabilities of the rockfill and foundation than by the slope inclinations. Therefore, a sensitivity analysis was not performed to estimate seepage for varying embankment slopes.

As previously mentioned, results from the seepage analysis for the proposed in-sea barrier were generated using three different embankment hydraulic conductivity values ($k = 1x10^{-2}$, $1x10^{-1}$, and 1 cm/sec) each at three different head differentials (10 ft, 20 ft, and 40 ft).

Results

Using the developed cross section and the program inputs mentioned above, SEEP/W produced seepage estimates for the nine conditions selected for analyses. The seepage results are discussed in the following section, and the nine individual model outputs are found in Attachment H4-2.

Estimated Seepage through the Salton Sea Barrier

Figure H4-43 presents a summary of the results of the seepage model using SEEP/W. The values of estimated seepage range from a minimum of 220 AF per year per barrier-mile ($k = 1x10^{-2}$ cm/sec, $\Delta h = 10$ feet) to a maximum of 290,000 AF per year per barrier-mile ($k = 1$ cm/sec, $\Delta h = 40$ feet). Figure H4-43 also presents the estimated annual seepage through the Great Salt Lake Causeway for comparative purposes.

Further Study Needs

This seepage evaluation of the proposed Salton Sea in-sea barrier represents an initial evaluation based on currently available appraisal level barrier design and construction information. The seepage values generated from the analysis are based on an estimated range of embankment permeability values

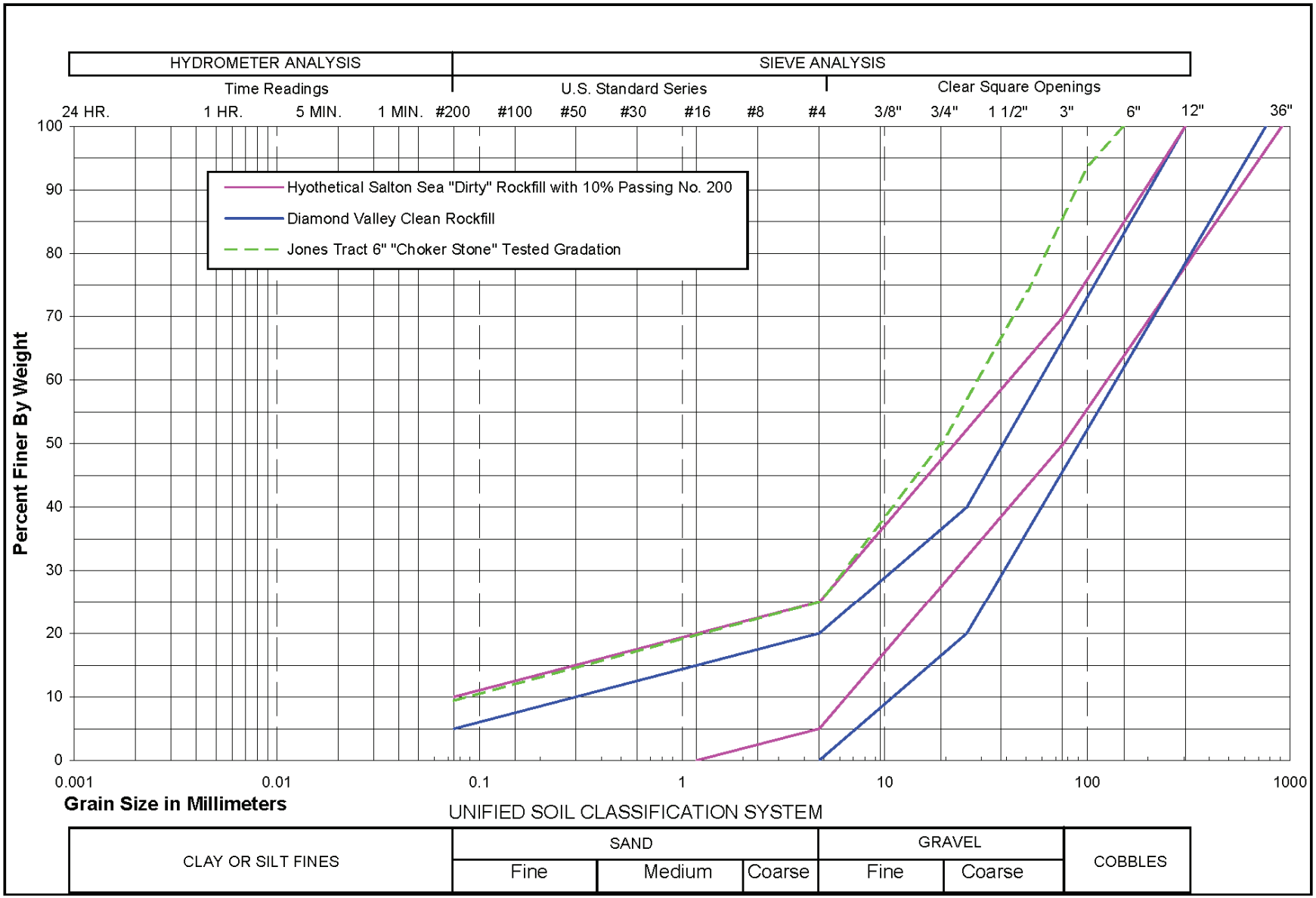

FIGURE H4-42
HYPOTHETICAL IN-SEA BARRIER ROCKFILL GRADATION LIMITS

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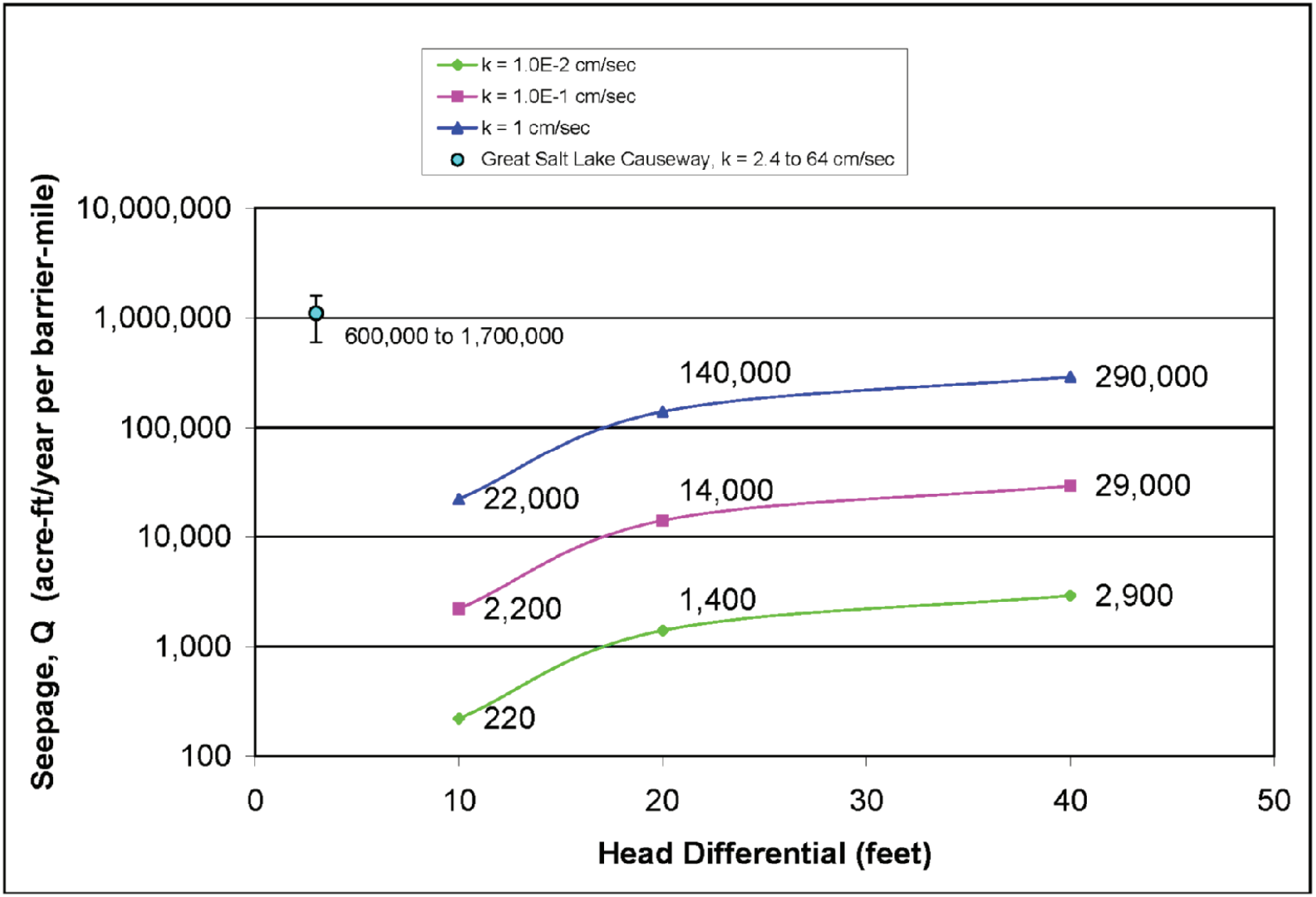

FIGURE H4-43
IN-SEA BARRIER SEEPAGE ANALYSIS RESULTS

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generated from this preliminary data. Obviously, more site, material and design data are needed to further refine the presented seepage rates for the proposed in-sea barrier.

An accurate estimate of the permeability of the proposed in-place barrier rockfill material is essential in the development of the seepage model used in this analysis. Further refinement of the estimated permeability can be attained through additional design development, geologic investigations of potential quarry areas, and test fills. Field measurements of seepage from other projects, such as the Jones Tract levee repair and the South Delta Temporary Barriers, may also assist in selection of appropriate rockfill permeabilities.

CONSTRUCTION

Method

Identification of a feasible construction method is a necessary component of the conceptual design at a programmatic level. It should be understood that various techniques are available for placement of rockfill in-the-wet. It should not be assumed and it is not suggested, that the following sequential steps and/or methods are requisite to the construction of in-sea rock barriers. More cost effective techniques may be employed by contractors when bidding on a construction contract.

A sequence of feasible steps for constructing in-sea rock barriers is as follows: establishment of one or more rock quarries, transport of materials from the quarries to the shoreline, construction of one or more harbors, and placement of material through the water column by barges.

An investigation identifying potential rock quarries in the Salton Sea's vicinity was performed by CH2M HILL in 2005. A Draft report presenting the results of the field exploration at Eagle Mountain Mine and estimates of available rock source at the site was completed in 2005. These results are discussed in Appendix H-5.

Transport of materials can be accomplished by truck, railroad or conveyor systems. Regional air quality conditions may limit the use of trucks and pose a significant adverse impact to local communities and existing highways. Use of existing railroads may be an option which will be considered in the quarry investigation. Railroads have been successfully utilized to transport large volumes of materials as demonstrated by the construction of Oroville Dam where 80 million cubic yards of embankment material was transported 12.5 miles by rail (DWR, 1974).

Conveyor systems transported 21.5 million cubic yards of fill in 800 days in the Atlanta International Airport construction project (Nissalke, 2004). The 4.5-mile-long conveyor system averaged 45,000 cubic yards per day, crossed a major interstate, and eliminated a projected 9,000 daily truck trips of 15 miles each way. The overland conveyor system was credited with minimizing or eliminating air quality, noise, water quality, traffic and public safety or community impacts.

A drawback of commercially readily available belt conveyor systems is that they are limited to transporting materials up to 2 feet in diameter. Thus, a combination of the foregoing transportation alternatives may be required to accommodate all of the materials.

The ideal location for constructing harbors is where the Salton Sea's bathymetry is steep. These locations exist in areas along the Salton Sea's entire northeastern and northwestern shorelines and an area adjacent to the Salton Sea Test Base. Utilization of these areas will reduce or minimize the need for dredging as well as other necessary harbor infrastructure. Locations can be selected to minimize the impact on nearby shoreline communities.

Use of bottom dump barges such as six-pocket-dump scows, is a feasible and cost effective method of placing material. Use of large construction equipment as utilized in oceanic environments, is precluded by

virtue of the Salton Sea's inland location. It may be necessary depending on the desired size, that barges are constructed on site.

The Great Salt Lake Causeway in Utah, as referred to previously, is an example of a large project where the foregoing construction methodology was successfully used. The transcontinental railroad originally circled the north end of the Great Salt Lake climbing a steep gradient through the Promontory and North Promontory Mountains. The Southern Pacific Railroad, seeking a way to eliminate the climb and thereby reduce the length by 42 miles, constructed a 12-mile-long wooden trestle in 1902. After 50 years, the trestle had reached the end of its useful life and it was decided to build a causeway of rock and gravel in an alignment parallel to the trestle.

A rock quarry was established at the east end of the Lake where eight power shovels loaded dump trucks that carried the rock to a conveyor loading station. The larger rock was crushed and the material transported via a 2-mile long conveyor system at a rate of 3,500 tons per hour to be stockpiled beside a barge harbor. At the harbor, twin conveyor belts, each six feet in width, loaded 6,000 tons of rock and gravel per hour in barges.

A total of 11 barges were used, and of these, 6 barges were 250 feet long and 55 feet wide, capable of carrying 2,000 cubic yards of material. The other 5 barges had a capacity of approximately 1,000 cubic yards. Tugboats, 61 feet long powered by twin 500 hp motors, towed the barges to the drop zones along the alignment. A total of 44 million cubic yards was transported and placed in this manner (Washington Group International, 2006).

Rock Quarry Investigation

An investigation was performed to determine the availability and suitability of rock at various locations by the Salton Sea. The investigation was carried out by CH2M HILL for the California DWR, Colorado River and Salton Sea Office, as part of the feasibility study and PEIR.

Work activities under the first task order considered a number of sites and recommended the following three for further evaluation (CH2M HILL, 2005a):

- Eagle Mountain Mine Area
- Mesquite Mine Area
- Coolidge Mountain

Follow up work activities, under a second task order, found the materials at the Mesquite Mine to be unsuitable in terms of both quality and quantity and recommended that field investigations be carried out at both Eagle and Coolidge mountains (CH2M HILL, 2005b).

A third task order, Quarry Field Exploration, dated April 21, 2005 was issued to carry out a field investigation of both sites. The permitting process has been initiated with the Bureau of Indian Affairs to conduct field work at Coolidge Mountain on the Tribal Lands of Torres Martinez Desert Cahuilla Indians. However, the permitting process has been stalled to date because of environmental issues. More recently, the Salton Sea Authority, in coordination with the USBR, is currently developing a reconnaissance level investigation of the Coolidge Mountain. The results will likely not be completed in time to be incorporated into this document. But, if found suitable, this location would provide a relatively economical source of rockfill to construct in-Sea embankments because of its proximity to the sea.

The field exploration program at Eagle Mountain Mine was conducted between June and July 2005. The field work consisted of drilling and geophysical surveys. The investigation considered landownership, access, suitability of materials, existing infrastructure and environmental consequences. The program included drilling, logging, and sampling core borings at selected locations to estimate the thickness of the rock piles, to characterize the gradation and lithology of the material contained within the piles, and to

collect samples for laboratory testing. A geophysical exploration program across the waste rock piles was also conducted to evaluate the nature of the rock piles in continuous profiles and, if possible, to confirm the location of the original ground topography below the waste rock piles estimated from the pre- and post-mine topography.

A laboratory testing program was also performed to evaluate the suitability of the rock for use as fill material for the embankments and barriers proposed in the sea. The laboratory testing program consisted of a suite of rock durability tests, along with index testing including gradation, percent passing the #200 sieve (percentage of fines), and Atterberg Limits.

The results indicated that the material encountered in the two primary waste rock piles at the west end of the Eagle Mountain Mine is primarily a mixture of sand, gravel, and cobbles up to 12 inches in diameter (CH2M HILL, 2005c). Up to 5 percent of the waste rock encountered is estimated to be between 12 inches and 3 feet in diameter, and approximately 1 percent of the material is estimated to be in excess of 3 feet in diameter. The material was determined to be durable and suitable for use in the saline environment of the Salton Sea. Volume estimates generated from a digital terrain model of the west end of the mine indicates that approximately 71 million cubic yards of material are available in the two primary waste rock piles in the West End of the mine (see Appendix H-2). The remaining waste rock piles (A through H) comprise of about an additional 26 million cubic yards of material. Rockfill volumes for construction of the barrier, perimeter dikes, and forebay/sedimentation basins proposed in some of the restoration alternatives are as high as 65 to 75 million cubic yards.

The results indicated that there is sufficient quantity of rockfill materials in the vicinity of the Salton Sea to construct in-sea embankments. The available material is adequate to construct barriers and perimeter dikes under any alternatives. However, the amount of large boulders observed was very limited. Therefore, it may be necessary to open a new quarry to create large boulders desired to construct higher embankments sections. The project-level design could further consider rockfill sources if required.

The method of transporting the materials may be by railroad, conveyors, trucking, or a combination thereof. Reference 10 includes the conceptual infrastructure needs and improvements required associated with the use of the Eagle Mountain site as a source of previously quarried rock material.

BARRIER DESIGN

Target Yield Accelerations

The target yield accelerations are those which limit the displacement, or shearing within the foundation, to 3 feet as discussed in the previous section “Rockfill Barrier Conceptual Design, Seismically Induced Permanent Deformation.” They are determined based on the variation of displacement with acceleration shown in Figure H4-30 through Figure H4-33. The selected accelerations are based on the medium of the recorded time histories without the effects of directivity focusing. The synthetic time histories are only shown for comparison purposes. The effects of directivity focusing on the displacements in the foundation, as well as the crest of the barrier, are quantified. The target yield accelerations to limit displacements to 3 feet for both shores are shown in Table H4-17.

Table H4-17
Target Yield Accelerations Corresponding to 3 feet of Foundation Displacement

Shore	Target Yield Acceleration
Western	0.08 g
Eastern	0.20 g

Design Slope Inclinations

Western Shore Design Slope Inclinations

The design slope inclinations are determined by finding the slope inclinations which correspond to the target yield acceleration of 0.08g as shown in Figure H4-24. Upstream and downstream slope inclinations of 5 and 8:1, respectively, are determined in this manner.

Accounting for directivity effects, the target acceleration of 0.08g corresponds to 4 feet of displacement as indicated in Figure H4-32. The design slope inclinations and resultant vertical crest settlement are shown in Table H4-18 for a 45-foot-high barrier.

Table H4-18
Western Shore – Barrier Slope Design – 45 Feet in Height

Barrier Component	No Directivity 0.08 g/3 feet		Directivity 0.08 g/4 feet	
	Slope Hor:Vert	Vertical Crest Displacement	Slope Hor:Vert	Vertical Crest Displacement
Upstream	5:1	0.6	5:1	0.8
Downstream	8:1	0.4	8:1	0.5
Total Crest		1.0		1.3

Eastern Shore Design Slope Inclinations

The yield accelerations are shown to reach upper limits of 0.15g and 0.125g for the upstream and downstream slopes, respectively, in Figure H4-24. These values fall below the target yield acceleration of 0.2g, which indicates that the design goal of limiting displacements to less than 3 feet cannot be met for barrier locations near the Eastern Shore.

Slope inclinations of 9.5 and 15:1 are shown to correspond to a yield acceleration of 0.125g in the Figure H4-24. In turn, a yield acceleration of 0.125g corresponds to a displacement in the foundation of 6 feet as indicated in Figure H4-31. This acceleration corresponds to 8 feet when accounting for directivity effects as shown in Figure H4-30.

The adverse effects of these undesirable large displacements are mitigated by the very flat slope inclinations of 9.5 and 15:1. These slope inclinations, in combination with the displacements, result in the same amount of vertical crest displacements as those for the Western Shore as shown in Table H4-19.

Table H4-19
Eastern Shore – Barrier Slope Design – 45 Feet in Height

Barrier Component	No Directivity 0.0125 g/6 feet		Directivity 0.0125 g/8 feet	
	Slope Hor:Vert	Vertical Crest Displacement	Slope Hor:Vert	Vertical Crest Displacement
Upstream	9.5:1	0.6	9.5:1	0.8
Downstream	15:1	0.4	15:1	0.5
Total Crest		1.0		1.3

As the foregoing tables show, directivity effects result in a nominal increase in total vertical crest displacements. These displacements can be accommodated by the freeboard determined in the design.

The amount of freeboard is largely dictated by potential wave height, which in turn, is dependent on an impoundment's fetch. Thus, the configuration of various restoration alternatives and the respective size of impoundments will determine the necessary freeboard. The freeboard should be sufficient to accommodate settlement of the foundation as well as seismically induced displacements.

As an example, 5 feet of freeboard is recommended for a 45-foot-high mid–sea barrier. The initial freeboard, after completion of construction, would be in excess of 5 feet to account for settlement. This amount of freeboard will safely accommodate the vertical displacement due to a 1 in 500-year earthquake event.

The design slope inclinations are irrespective of the barrier's alignment or orientation. Alignments oriented parallel to the east and west shores are more prone to directivity effects while those that are perpendicular are less so. As a rule of thumb, the slope inclinations vary by approximately a factor of 2 over a distance of roughly 10 miles between the east and west shores.

Typical Cross-Section for Appraisal Programmatic-Level Cost Estimates

The barrier volume calculations (below) were performed to estimate the volume of rockfill needed if the barrier is located at the mid-sea alignment. However, for the purpose of cost estimates at the programmatic level, it was assumed that the barrier had typical upstream and downstream slopes of 10:1 and 15:1, respectively. These slope inclinations assumed are considered appropriate due to the weak foundation layers and the very limited geotechnical data at the Salton Sea. But it is anticipated that future investigations and analyses may be able to economize on the necessary fill volumes.

Barrier Volume Calculation

Input to Spreadsheet

A simple spreadsheet was developed to facilitate the calculation of volumes for varying barrier configurations. A sample of the spreadsheet's input and output is shown in Figure H4-44. The geometry considered in the calculations is shown graphically in Attachment H4-4. The input parameters are defined as follows:

Sea Elevation

Represents the target elevation of the impoundment adjacent to the barrier's upstream slope.

Excavation

Represents the depth of excavation and is fixed to be one value over the entire footprint of the barrier.

Freeboard

Represents the height of fill above Sea Elevation.

Crest Width

Represents the width of barrier at its highest elevation. This allows for a differentiation of volumes if a roadway or vehicular access is desired. Absent the need for access, the recommended width is zero.

Percent Settlement

Represents the anticipated settlement, input as a percentage of the height of fill based on Table H4-13.

Average Seafloor Elevation

Represents the elevation of original ground based on bathymetric surveys.

Length

Represents the length in feet for a given Sea Bed elevation along of the alignment. As an example, if an alignment is to follow a given Sea Bed contour elevation, only one row of input is needed, i.e. the Sea Bed elevation and the total length.

Example for Mid-Sea Alignment

For a mid-sea alignment, the length is approximately 9 miles. This length is segmented to represent average Sea Bed elevations as shown in Figure H4-44. The input assumes typical slope inclinations of 10:1 and 15:1 for upstream and downstream, respectively, to be applied along the alignment's entire length. Cost estimates are included in Appendix H-7. The example indicates zero crest width, no excavation and 13 percent to account for settlement based on data provided in Table H4-13 and H4-15.

Sea Elevation		-230		
Downstream Slope		15		
Upstream Slope		10		
Depth of Excavation		0		
Freeboard		5		
% Settlement of Fill above O.G.		13		
Crest Width		0	Unit cost multiplier	
Unit cost for Excavation, $/CY		0	2	0
Unit cost for Rockfill, $/CY		0	2	0
Avg Seafloor Elevation (ft MSL)	Length (lineal feet)	Depth of Fill (excav + settl.) (ft)	Barrier Volume (CY)	Excavation Volume (CY)
-228.5	1,002	4.0	6,421	0
-232.5	1,218	8.5	35,842	0
-237.5	1,760	14.1	143,866	0
-242.5	2,518	19.8	403,419	0
-247.5	3,184	25.4	843,263	0
-252.5	2,383	31.1	942,788	0
-257.5	3,241	36.7	1,790,896	0
-262.5	4,129	42.4	3,037,611	0
-267.5	7,286	48.0	6,884,806	0
-271	18,941	52.0	20,967,336	0
Total	45,662		35,056,249	0
Cost for Rockfill				$0
Cost for Excavation				$0
Total				$0

FIGURE H4-44
MID-SEA VOLUME CALCULATION – TYPICAL SLOPE INCLINATIONS

The barrier fill volume for a mid-sea alignment example was estimated to be 35 million cubic yards of rockfill. Additional rockfill volumes will be needed to construct Perimeter Dikes and associated infrastructure components.

Sensitivity of Fill Volume to Slope Inclination and Barrier Height

The variation of yield acceleration with slope inclination shown on Figure H4-24 indicates that the barrier's yield acceleration only slightly increases with greater slope inclinations. The example above, for a mid-sea alignment, was re-ran to determine the effect on barrier volume by steeping the downstream slope from 15:1 to 12:1. This resulted in a relatively small decrease in fill volume.

The foregoing volume calculations do not incorporate the effects of barrier height and varying foundation conditions on the slope design.

The results, shown in Figure H4-25 and Figure H4-26, indicate that the factors of safety for these barrier heights are higher than those for the 45-foot high barrier over the entire range of horizontal accelerations. Thus, it is possible that, with additional foundation investigations and analyses, steeper slopes for lower height barriers can be shown to provide satisfactory performance. The reduction in volume of material due to these effects will not be significant for a mid-sea alignment since the majority of the alignment will require a barrier height of 25+ feet for an impoundment elevation of -235 feet msl. Additionally, barrier alignments near the Eastern Shore, regardless of height, will require slope inclinations indicated in Table H4-19 due to the high seismicity.

Sensitivity of Fill Volume to Foundation Strength

The foundation strengths were varied to determine the effect on factor of safety and corresponding yield acceleration for a 45-foot high barrier given 10:1 up and downstream slopes as shown in Figures H4-27, H4-28, and H4-29. It should be noted that the shear strength overburden ratios of 0.6 and 0.7 shown in these figures, represent the linear regression and median of all the strength data - they are considerably higher than the adopted value of 0.29.

The data clearly show the strong influence of foundation strength on yield acceleration and suggest that slopes steeper than 10:1 could provide adequate performance for alignments near the Eastern Shore based on average foundation strength values.

The foregoing demonstrates the need that future investigations carefully evaluate the foundation strengths along a selected alignment. The elements of any future investigations in this regard were discussed in the previous section "Rockfill Barrier Conceptual Design, Future Investigations."

Barrier Design – Faulting in the Foundation

Potential foundation offsets of 1.6 feet and 4.6 feet were determined for two strands within the Salton Sea as discussed in the previous section "Geologic Setting, Seismicity, Displacement." The barrier design will result in a footprint, which will more than adequately accommodate these offsets. Additionally, the recommended gradation will readily mitigate adverse effects of movements whether they are due to differential settlements or foundation offsets. Also, the recommended design is not intended to serve the purpose of a "water tight" structure whereby seepage is minimized, intercepted and collected as typically done in embankment dams.

There are a few limited number of cases where historical performance of dams has shown adequate performance when offsets in the foundation have occurred. Also, the construction of embankment dams on active faults is not unprecedented.

Based on the foregoing discussion, it was not considered necessary to undertake a field investigation to determine fault locations and offsets for the purposes of the PEIR.

Summary and Conclusion

A design for in-sea rock barriers has been formulated to the extent that available information and data permits. The design is considered adequate for the purpose of developing the necessary water infrastructure components for the various restoration alternatives being considered for the PEIR.

The following design elements were considered:

- Geologic Setting;
- Division of Safety of Dams Guidelines;
- Rock Gradation;
- Settlement;
- Seepage;
- Static Stability;
- Post-liquefaction Stability;
- Seismicity;
- Input Ground Motions;
- Up and Downstream Slopes;
- Seismic Performance;
- Fault Offset in the Foundation; and
- Rockfill quarry investigation.

Limitations and Uncertainties

A rockfill barrier constructed in-sea will face significant design and construction challenges. The existing amount of existing subsurface data is extremely limited given the large footprint of the proposed barrier. Additional site explorations and laboratory tests will be required if the preferred alternative involves a barrier component. Strength testing of the Stiff Lacustrine and Alluvial Deposits and further evaluation of the liquefaction potential of the foundation soils are required.

An accurate estimate of the permeability of the proposed in-place barrier rockfill material is essential in the development of the seepage model used in this analysis. Further refinement of the estimated permeability can be attained through additional design development, geologic investigations of potential quarry areas, and test fills.

Placement of the finer-grained material on the upstream slope is preferred over placement in the core. However, the feasibility of an upstream blanket to adequately plug the rockfill barrier should be further evaluated during project-level analysis.

The deterministic response spectra are based on current attenuation relationships. However, the Next Generation Attenuation of Ground Motions Project (NGA) sponsored by the Pacific Earthquake Engineering Research Center (PEER) is expected to propose revised and updated attenuation relationships. Any new relationships should be reviewed and the target spectrum modified as appropriate.

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ATTACHMENT H4-1

Slope/W Stability Models

LONG-TERM BARRIER STABILITY WITH
STEADY-STATE SEEPAGE CONDITIONS AND EFFECTIVE STRENGTHS
BARRIER SLOPES VARIED FROM 5H:1V TO 15H:1V
BARRIER HEIGHT = 45 FEET

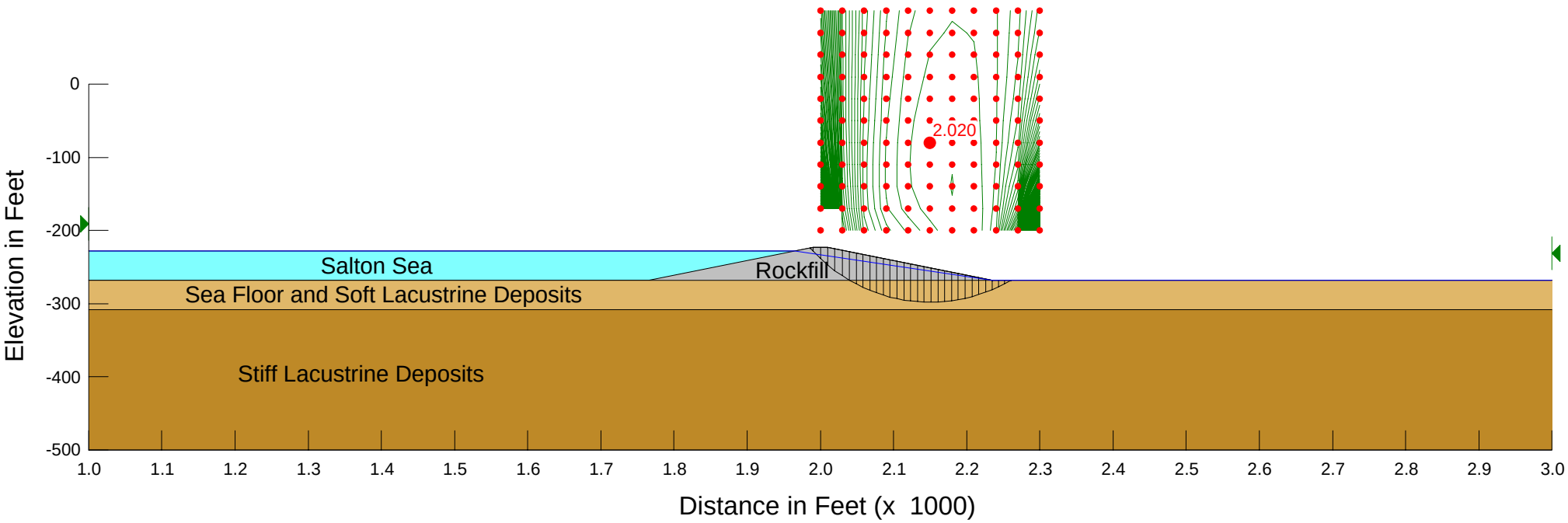
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29 degrees, C=0 psf (Effective Strength)

Horizontal Acceleration = 0g

Factor of Safety = 2.0 (Spencer's Method)



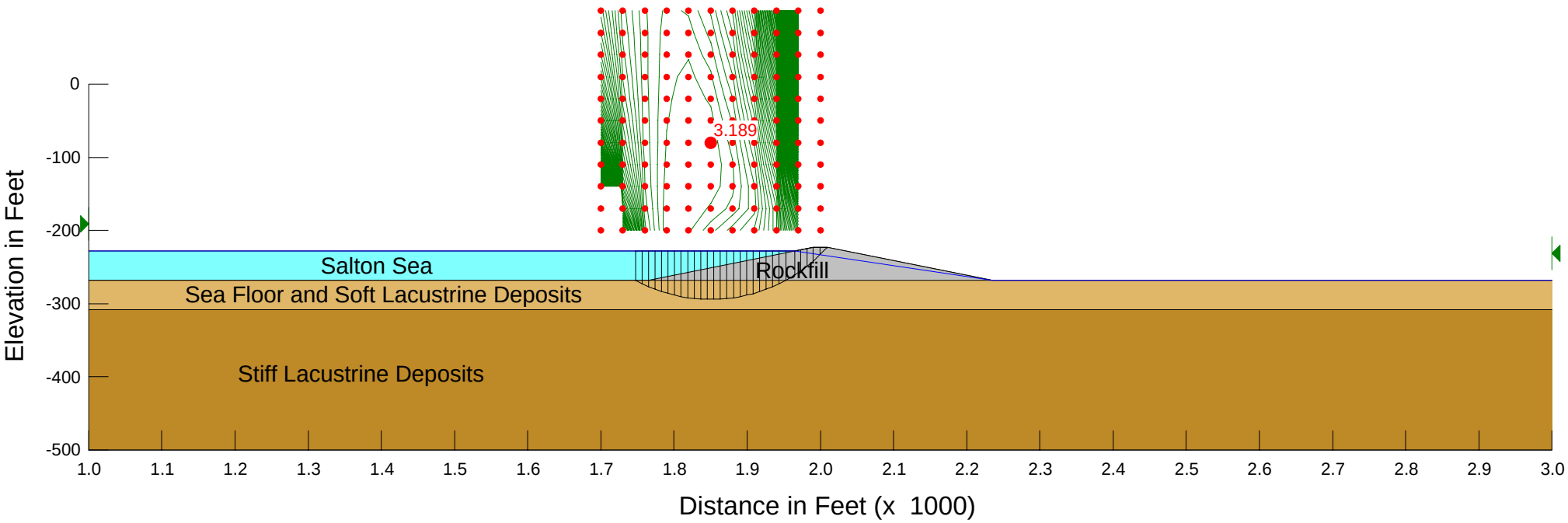
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29.0 degrees, C=0 psf (Effective Strength)

Horizontal Acceleration = 0g

Factor of Safety = 3.2 (Spencer's Method)



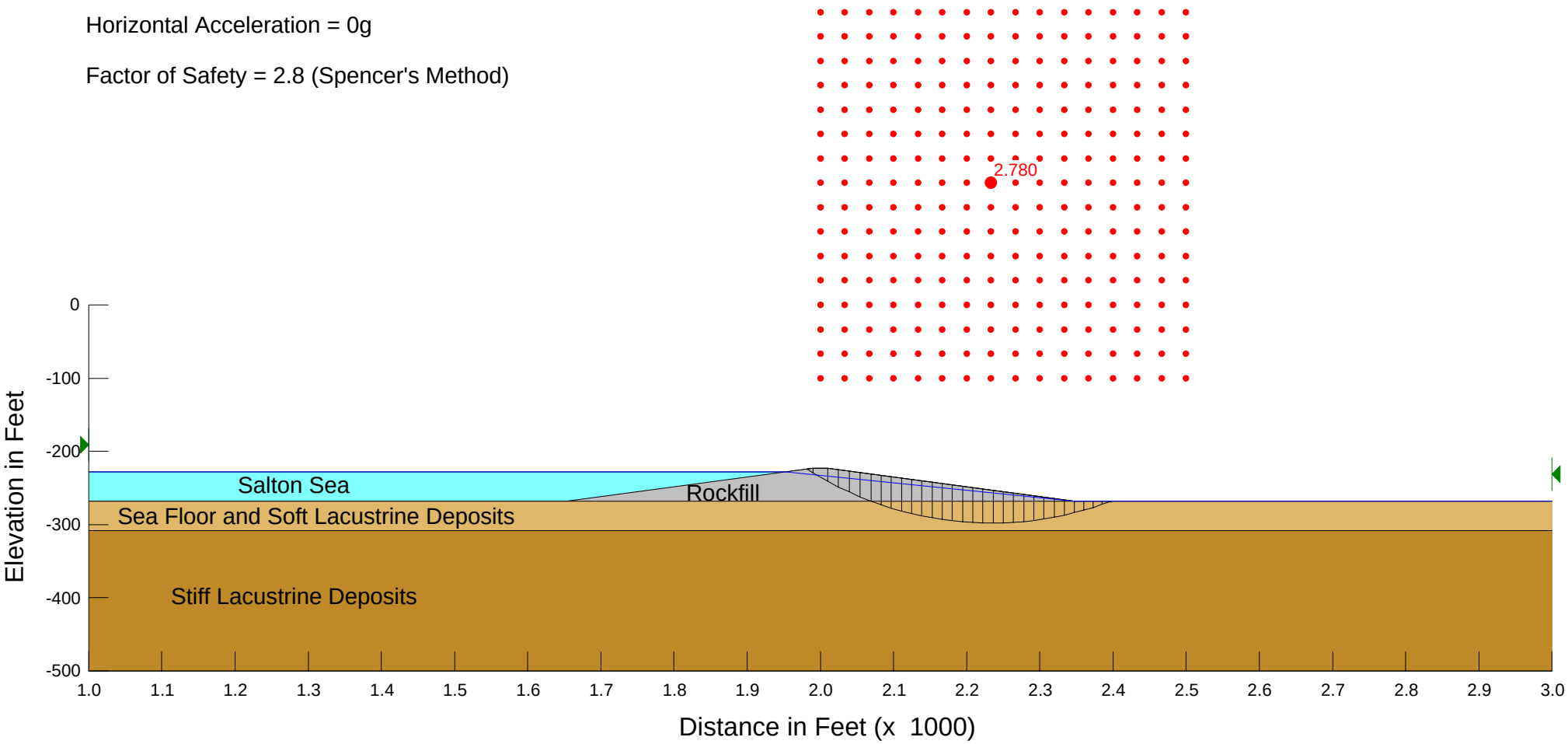
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 7.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29.0 degrees, C=0 psf (Effective Strength)

Horizontal Acceleration = 0g

Factor of Safety = 2.8 (Spencer's Method)



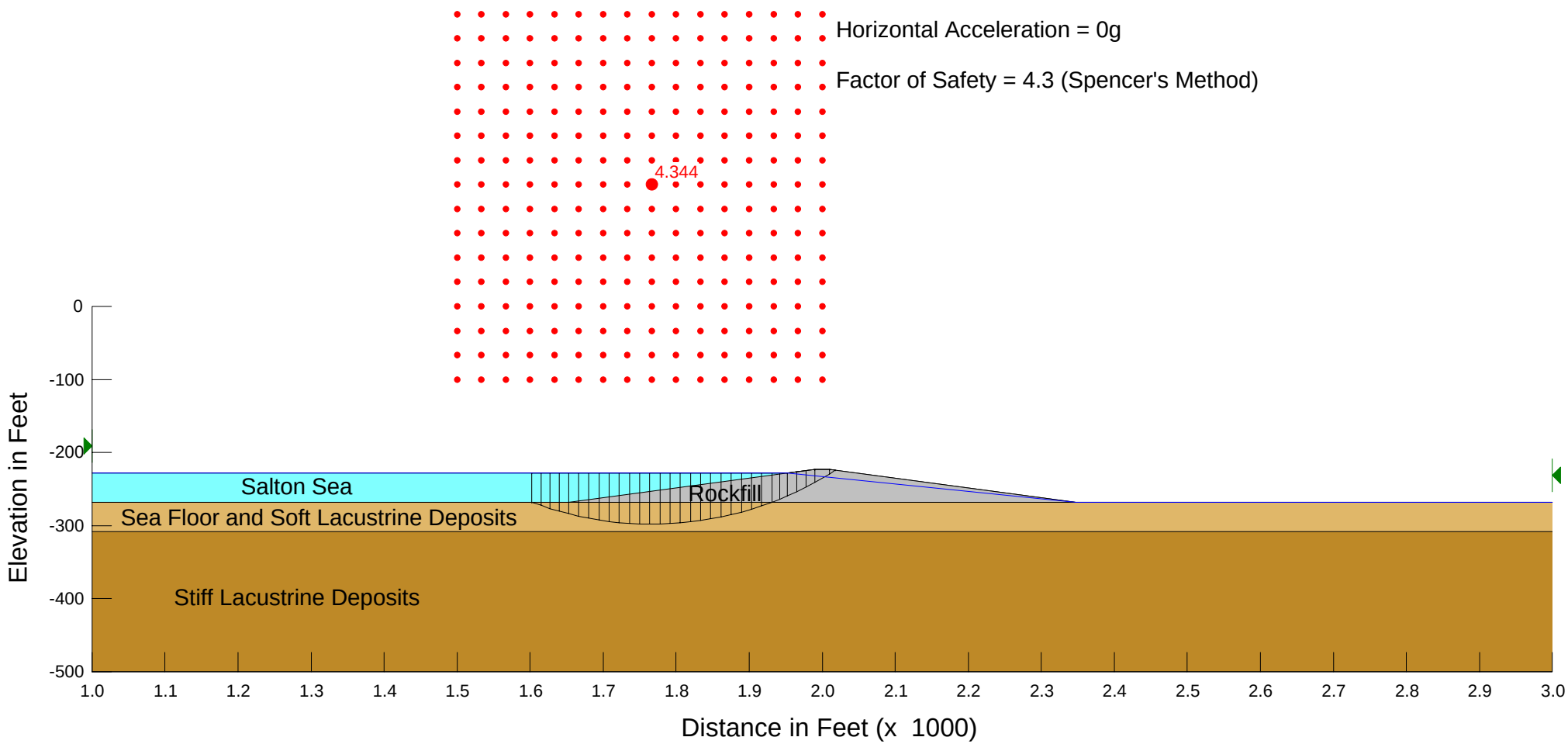
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 7.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29.0 degrees, C=0 psf (Effective Strength

Horizontal Acceleration = 0g

Factor of Safety = 4.3 (Spencer's Method)



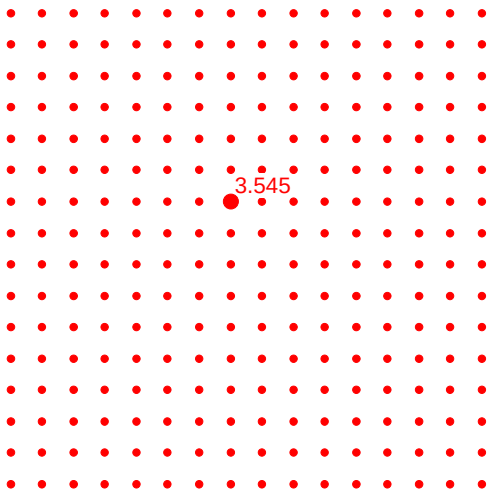
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

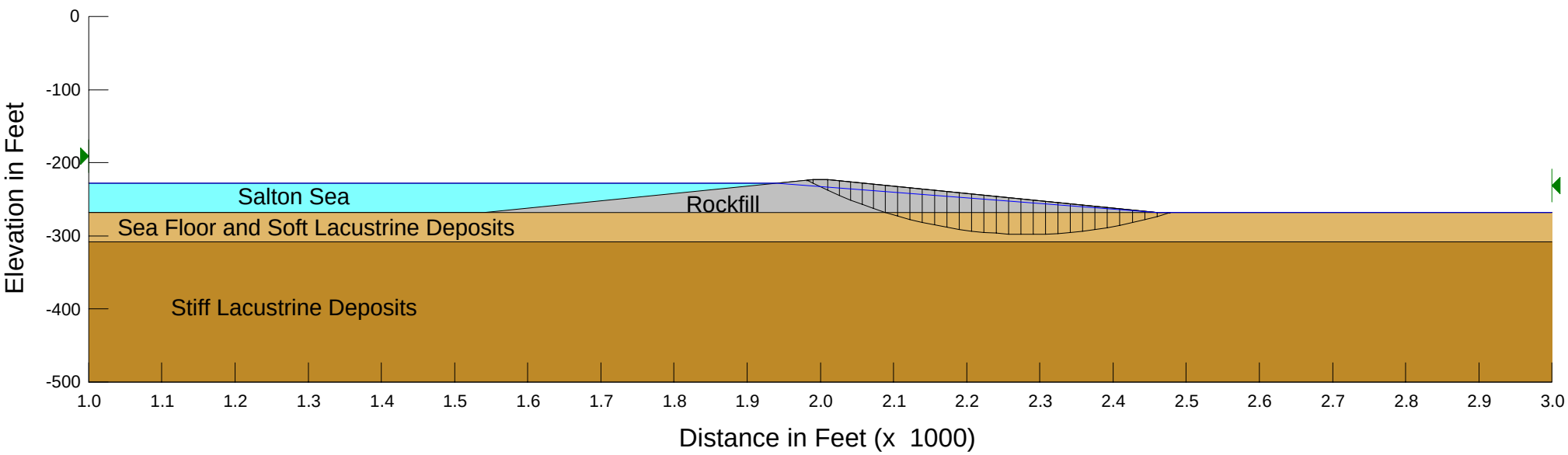
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29 degrees, C=0 psf (Effective Strength)

Horizontal Acceleration = 0g

Factor of Safety = 3.5 (Spencer's Method)





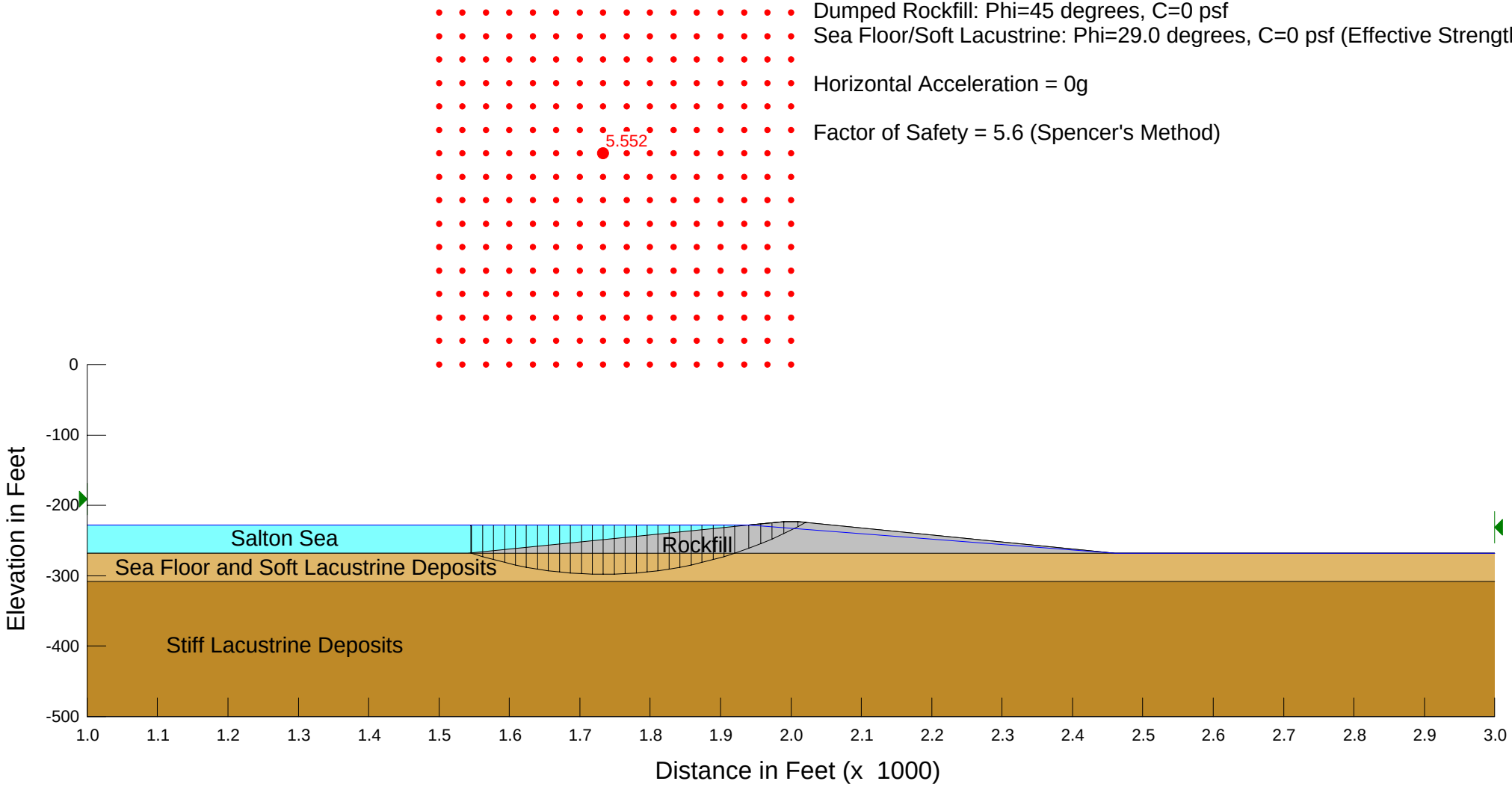
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29.0 degrees, C=0 psf (Effective Strength

Horizontal Acceleration = 0g

Factor of Safety = 5.6 (Spencer's Method)



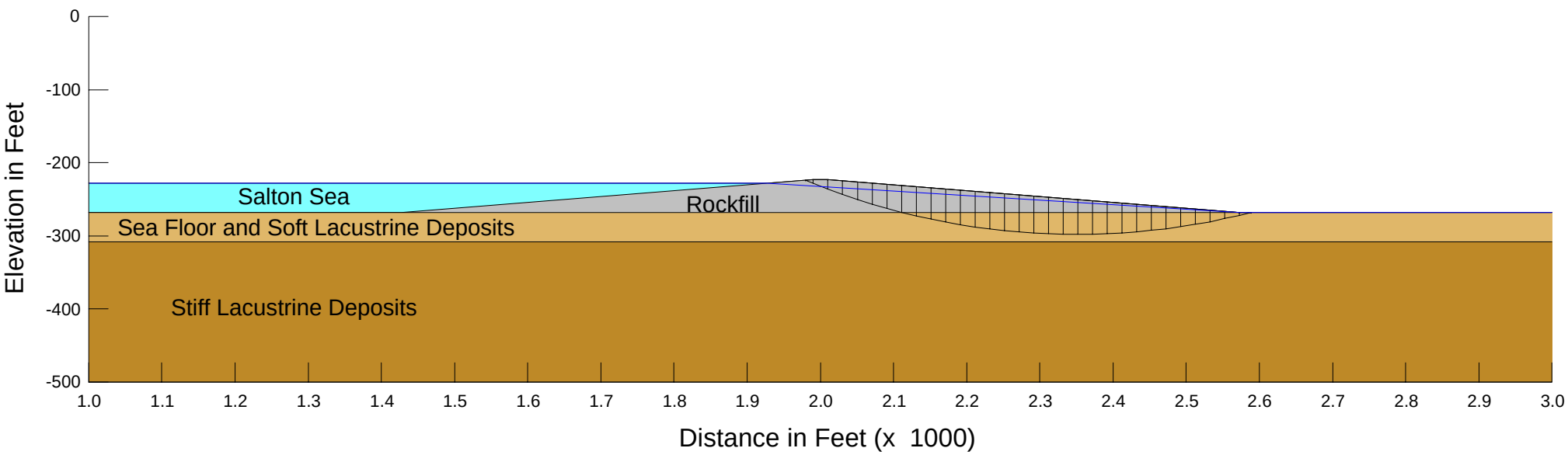
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

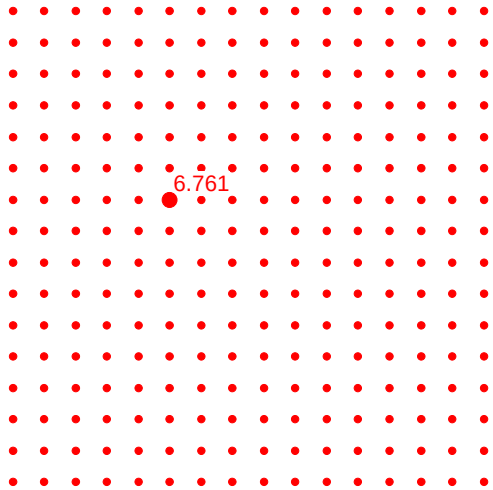
45' High Barrier with 12.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29 degrees, C=0 psf (Effective Strength)

Horizontal Acceleration = 0g

Factor of Safety = 4.344 (Spencer's Method)





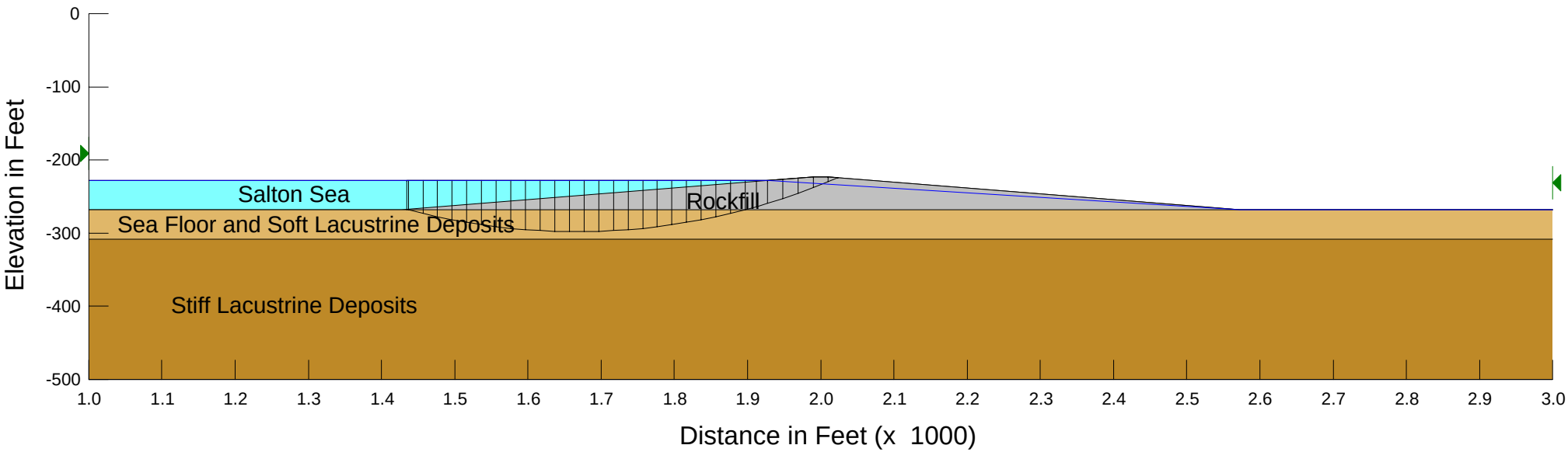
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 12.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29.0 degrees, C=0 psf (Effective Strength

Horizontal Acceleration = 0g

Factor of Safety = 6.8 (Spencer's Method)



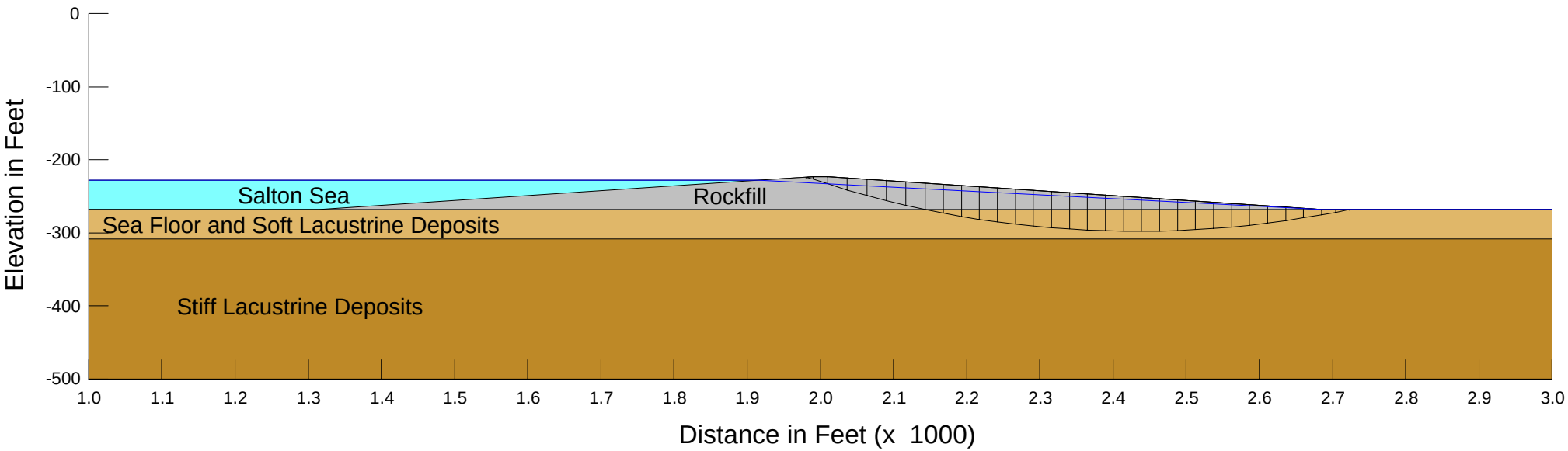
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 15H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29.0 degrees, C=0 psf (Effective Strength)

Horizontal Acceleration = 0g

Factor of Safety = 5.125 (Spencer's Method)



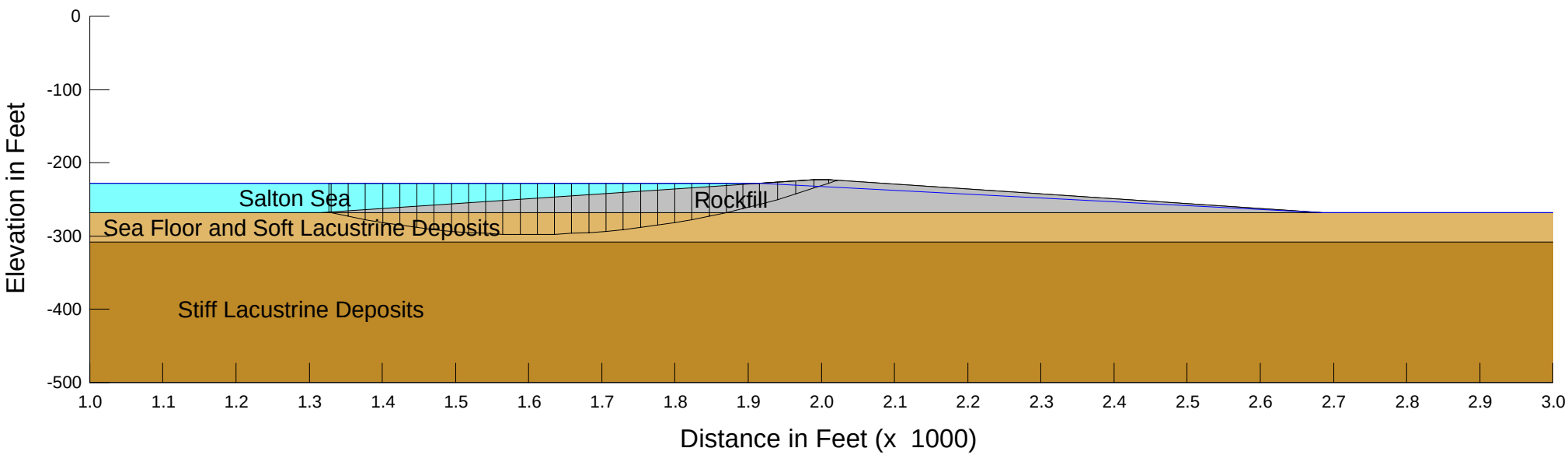
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 15H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29.0 degrees, C=0 psf (Effective Strengtl

Horizontal Acceleration = 0g

Factor of Safety = 7.955 (Spencer's Method)



PSEUDO-DYNAMIC BARRIER STABILITY WITH
HORIZONTAL SEISMIC LOADS AND TOTAL STRENGTHS
BARRIER SLOPES VARIED FROM 5H:1V TO 15H:1V
BARRIER HEIGHT = 45 FEET

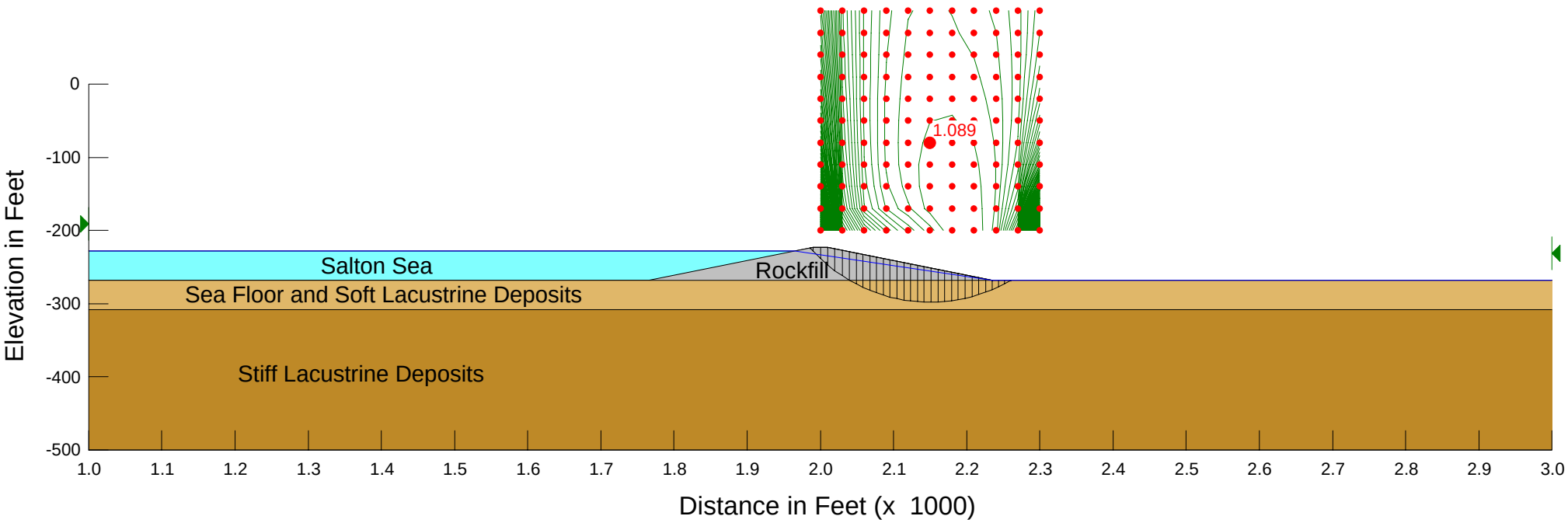
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 1.1 (Spencer's Method)



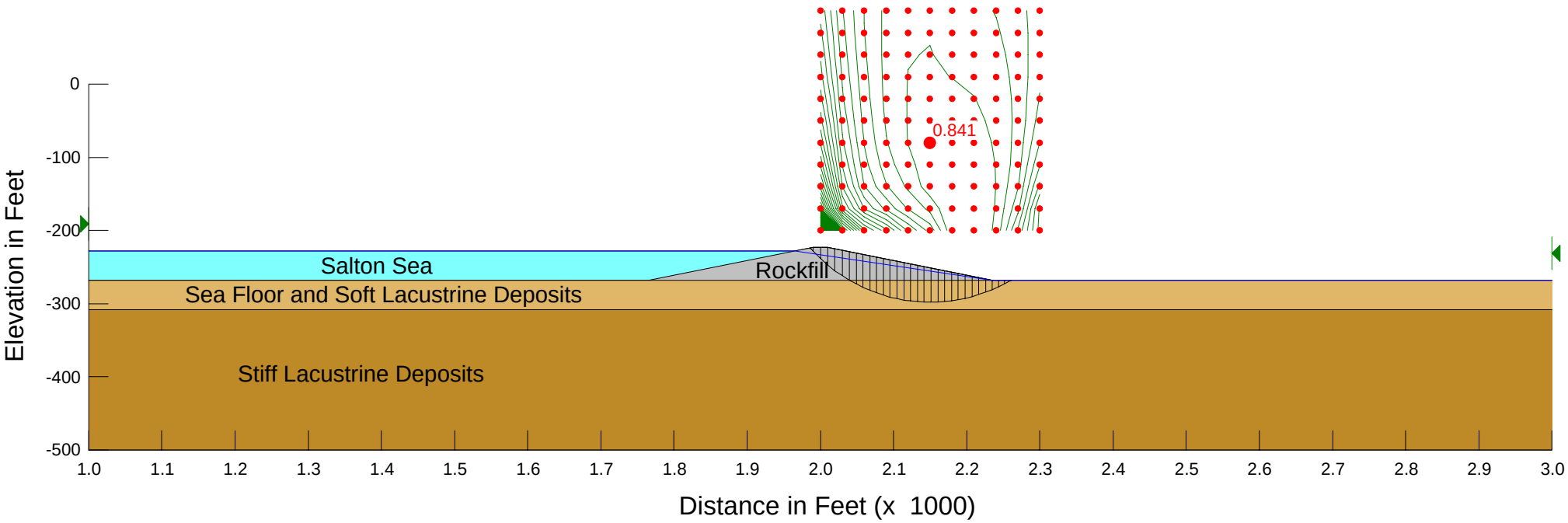
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 0.8 (Spencer's Method)



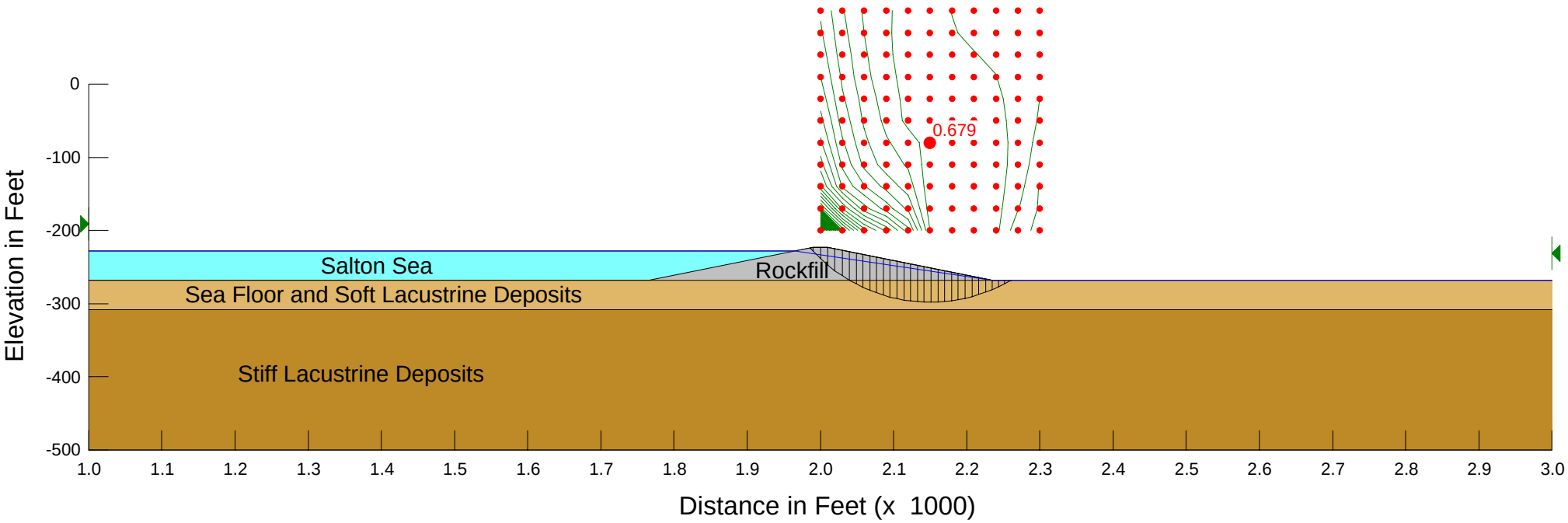
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 0.7 (Spencer's Method)



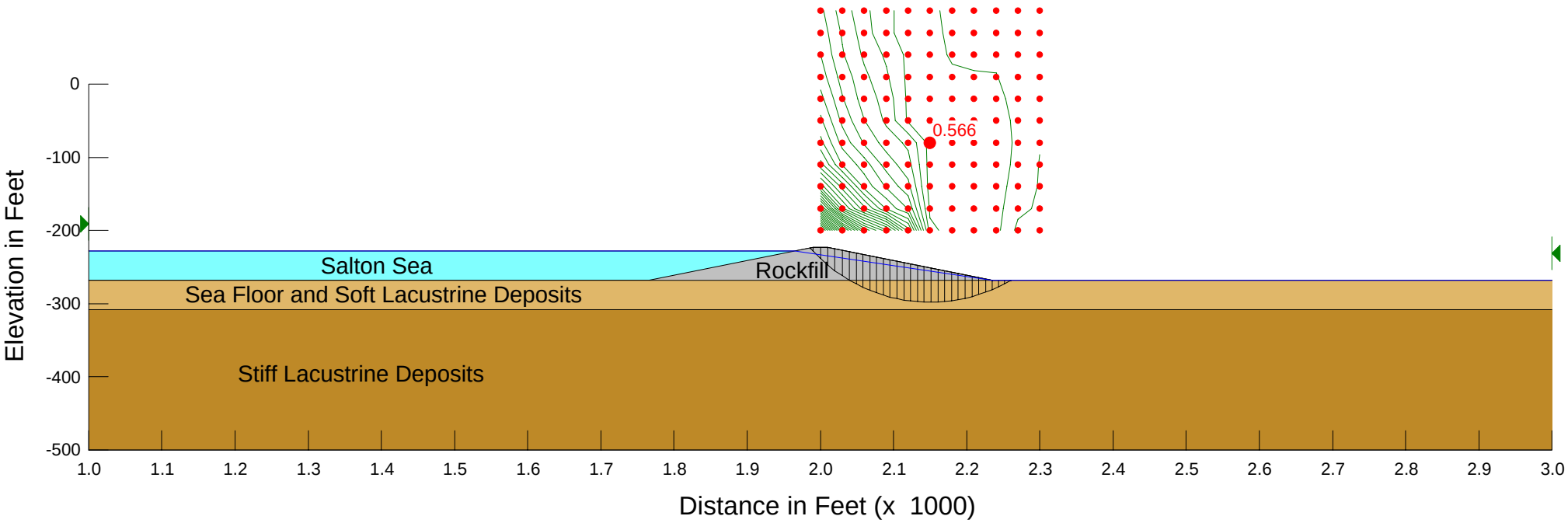
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 0.6 (Spencer's Method)



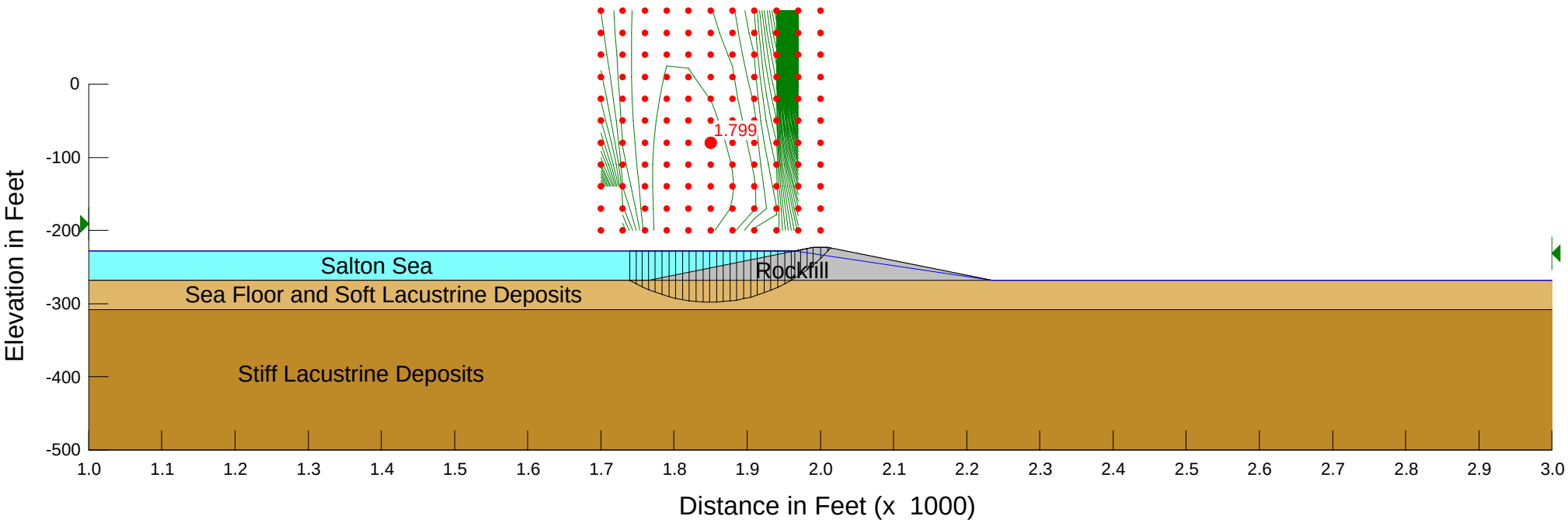
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 1.8 (Spencer's Method)



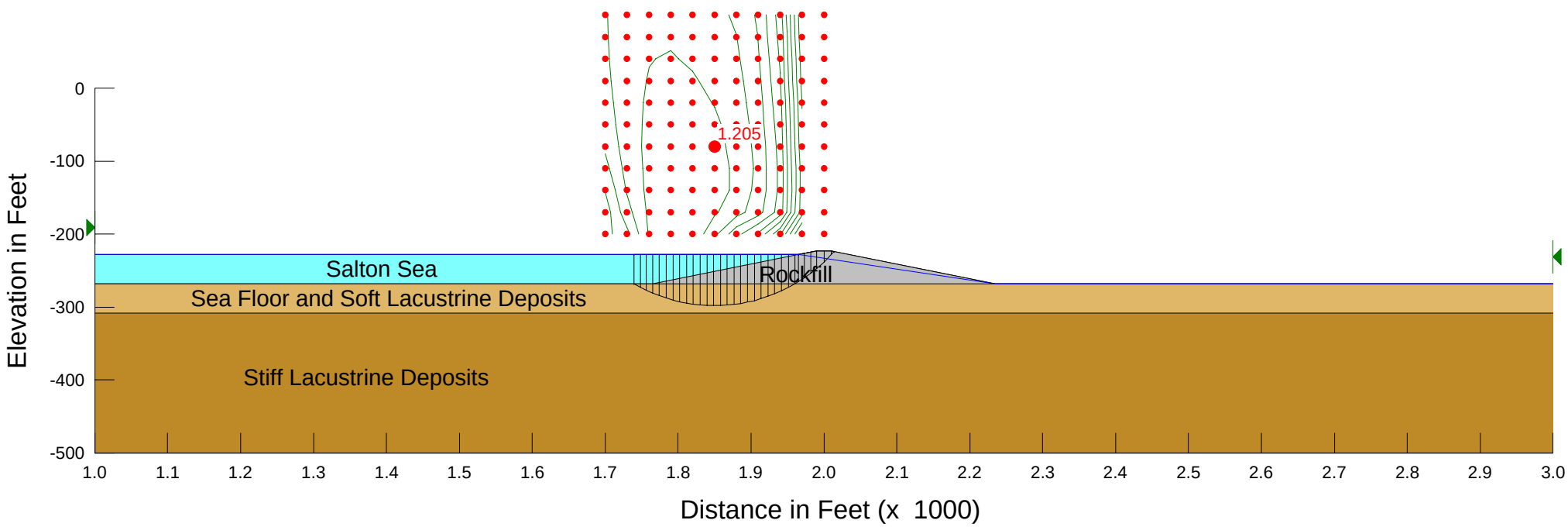
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 1.2 (Spencer's Method)



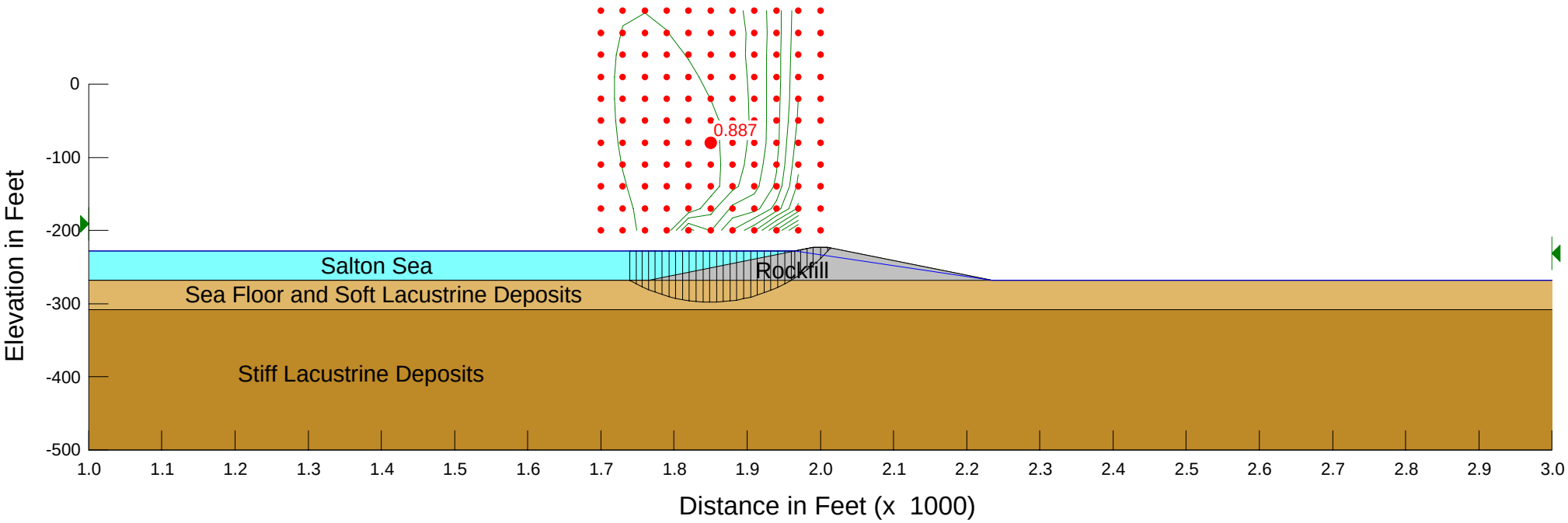
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 0.9 (Spencer's Method)



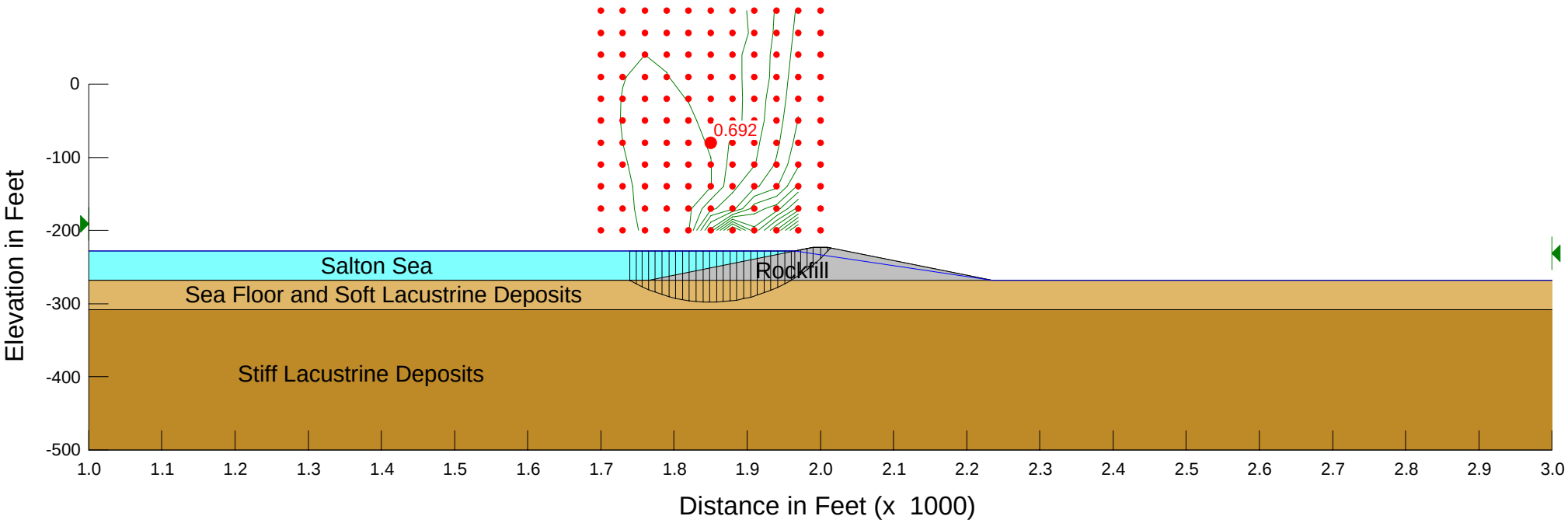
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 0.7 (Spencer's Method)



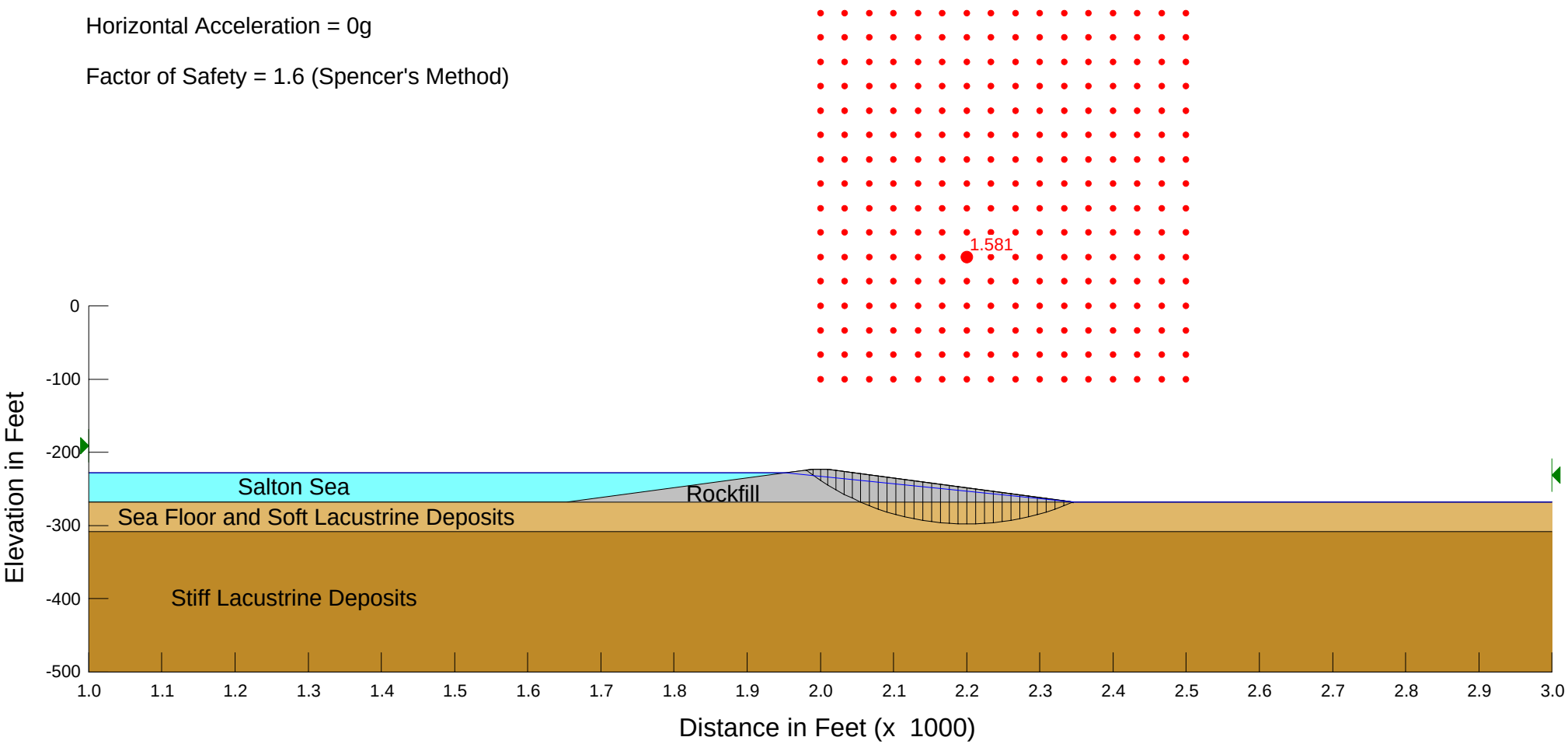
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 7.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 1.6 (Spencer's Method)



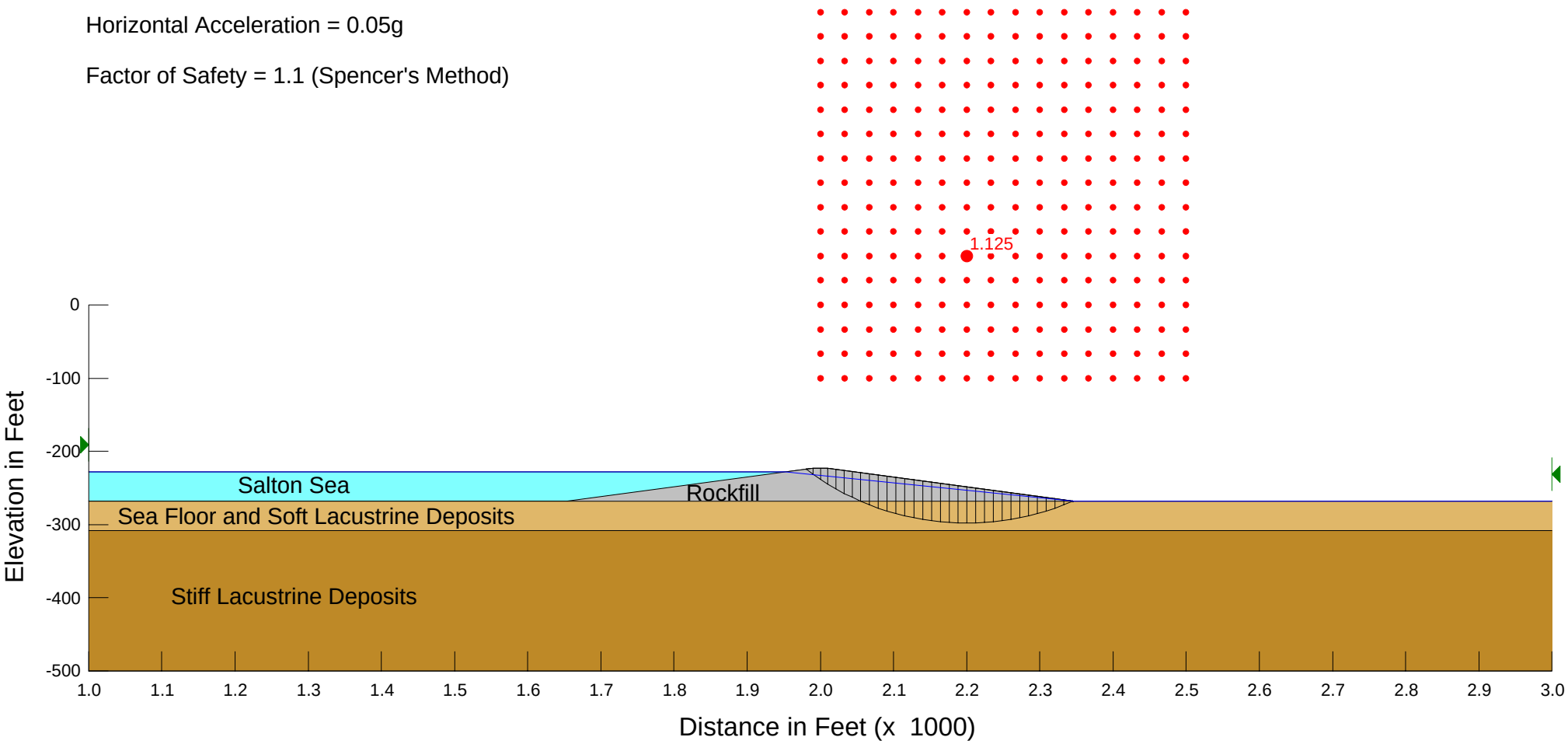
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 7.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 1.1 (Spencer's Method)



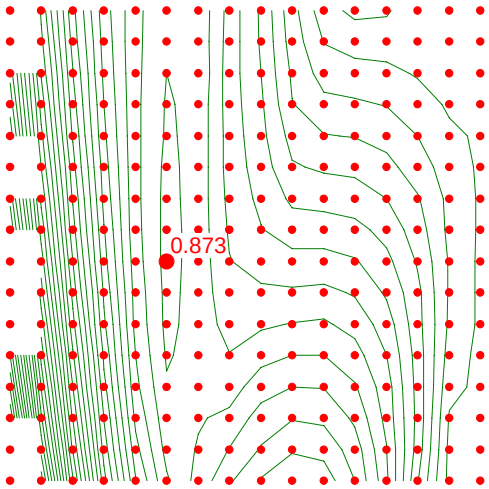
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

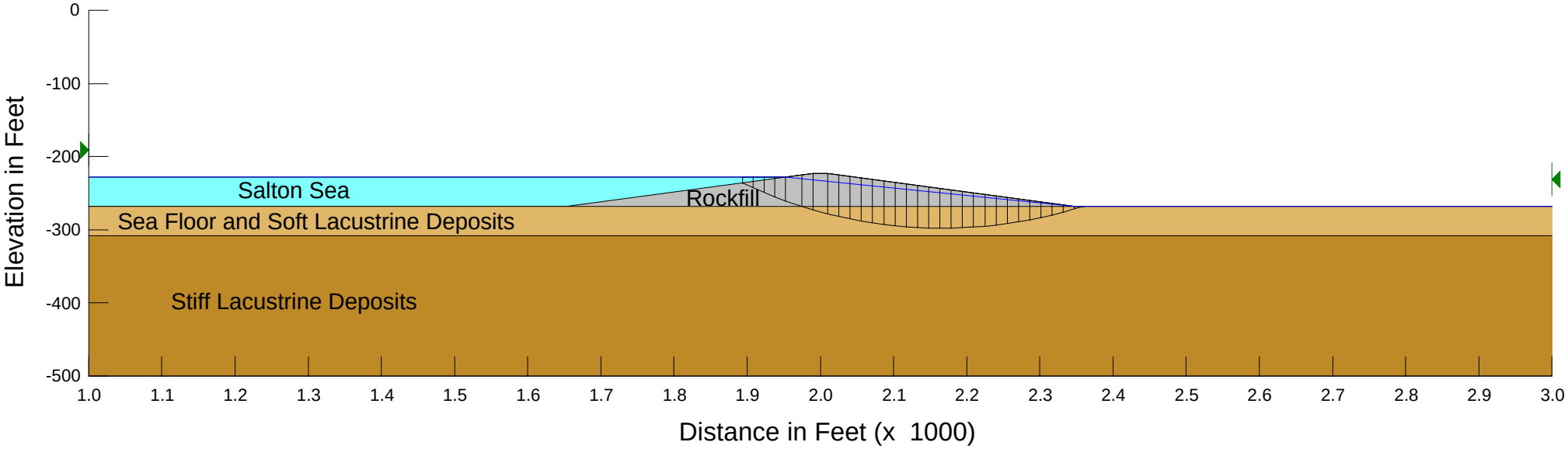
45' High Barrier with 7.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 0.9 (Spencer's Method)





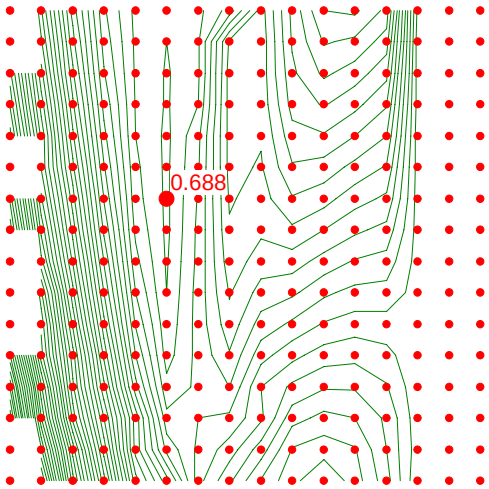
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

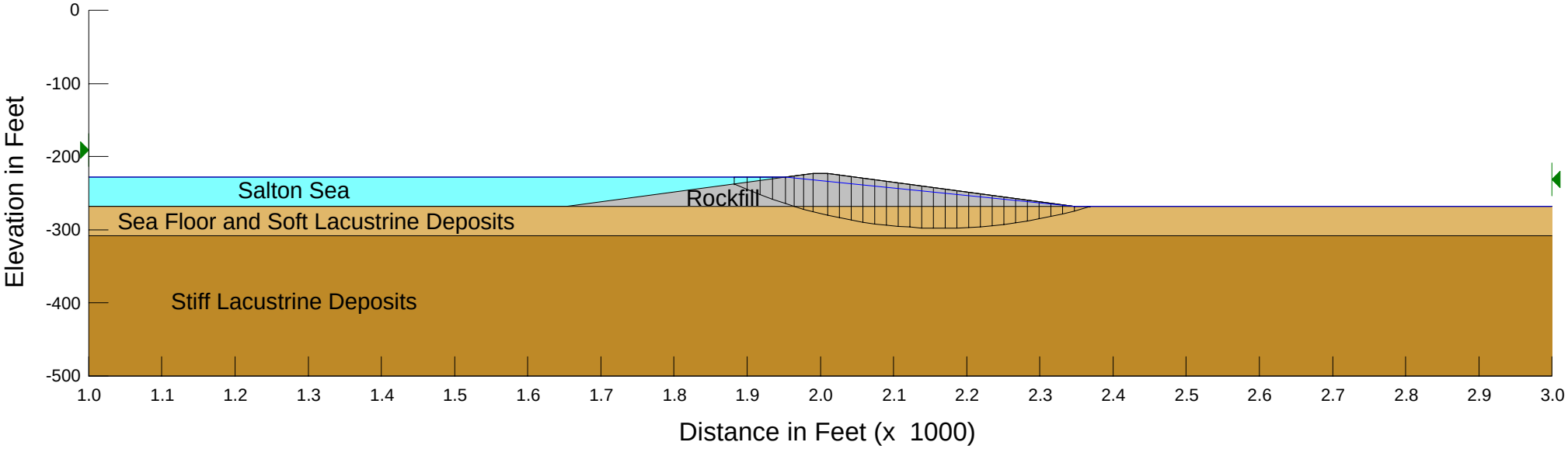
45' High Barrier with 7.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 0.7 (Spencer's Method)





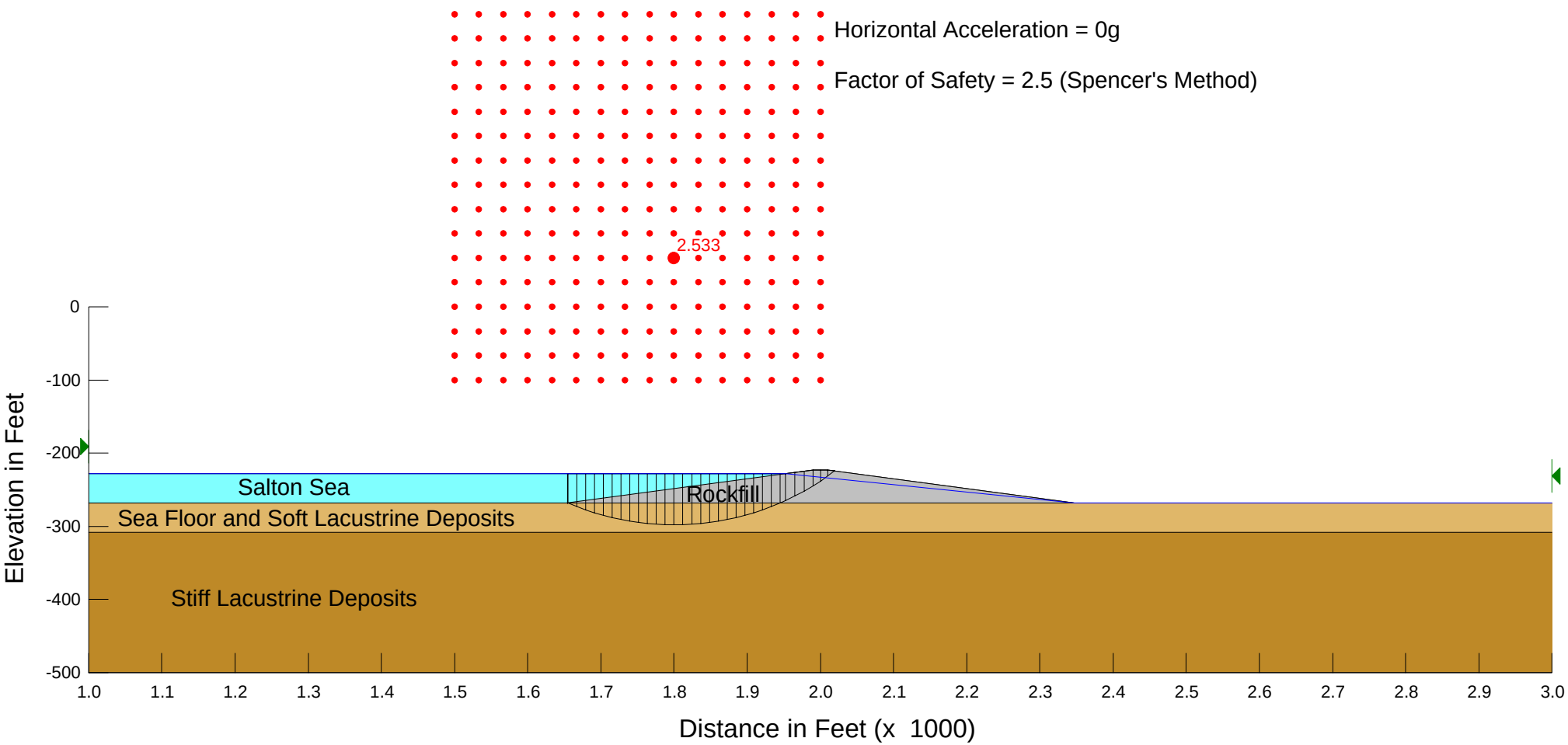
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 7.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 2.5 (Spencer's Method)



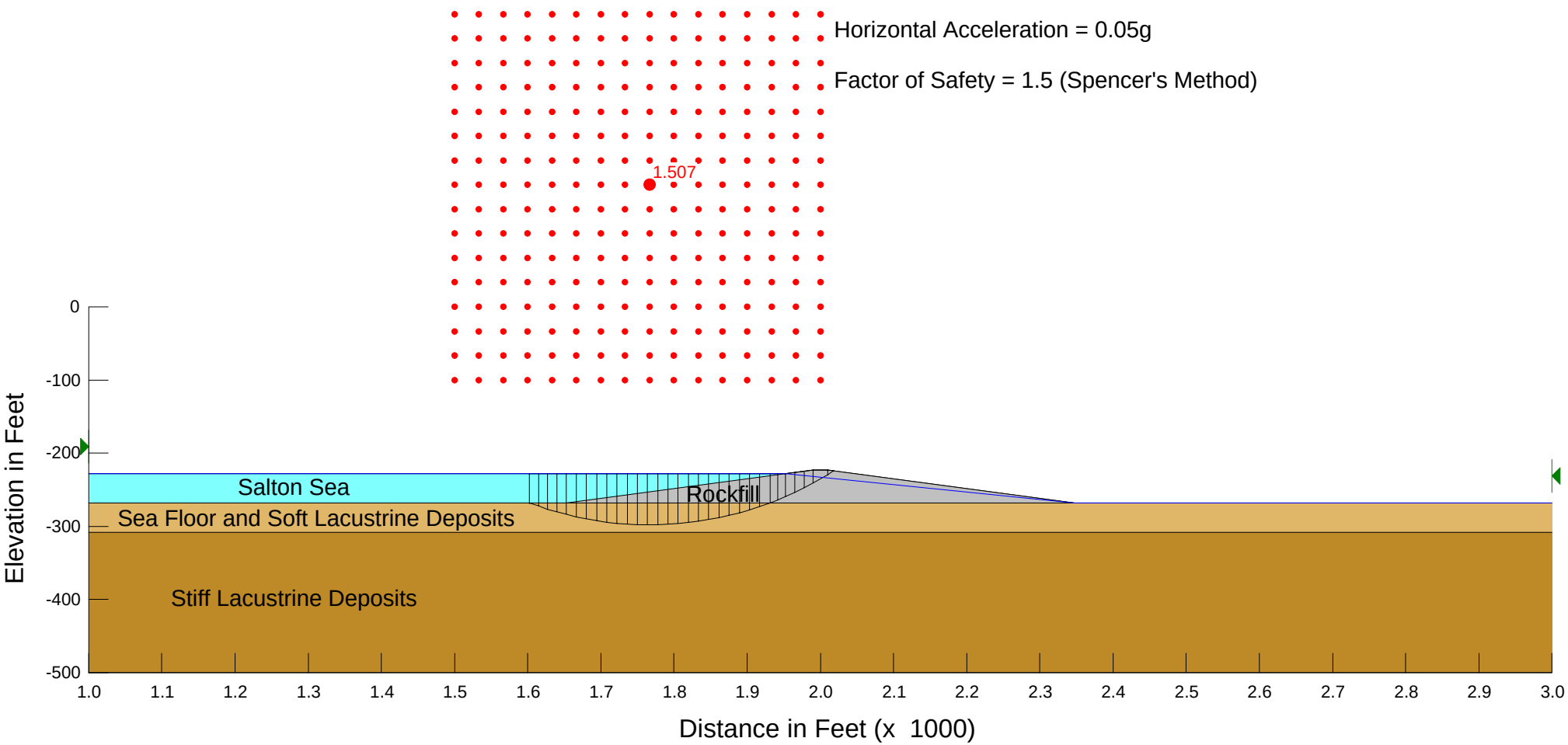
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 7.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 1.5 (Spencer's Method)



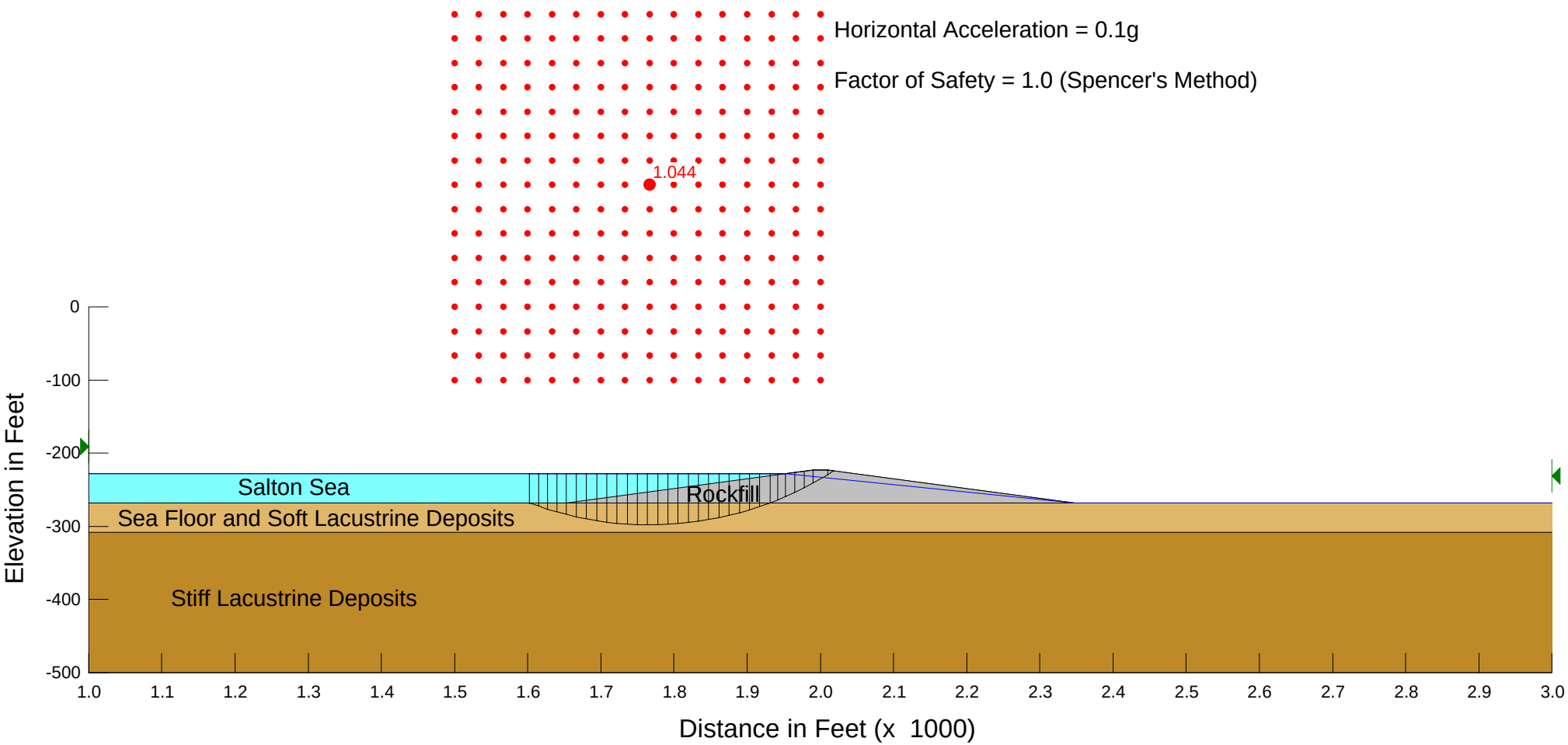
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 7.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 1.0 (Spencer's Method)



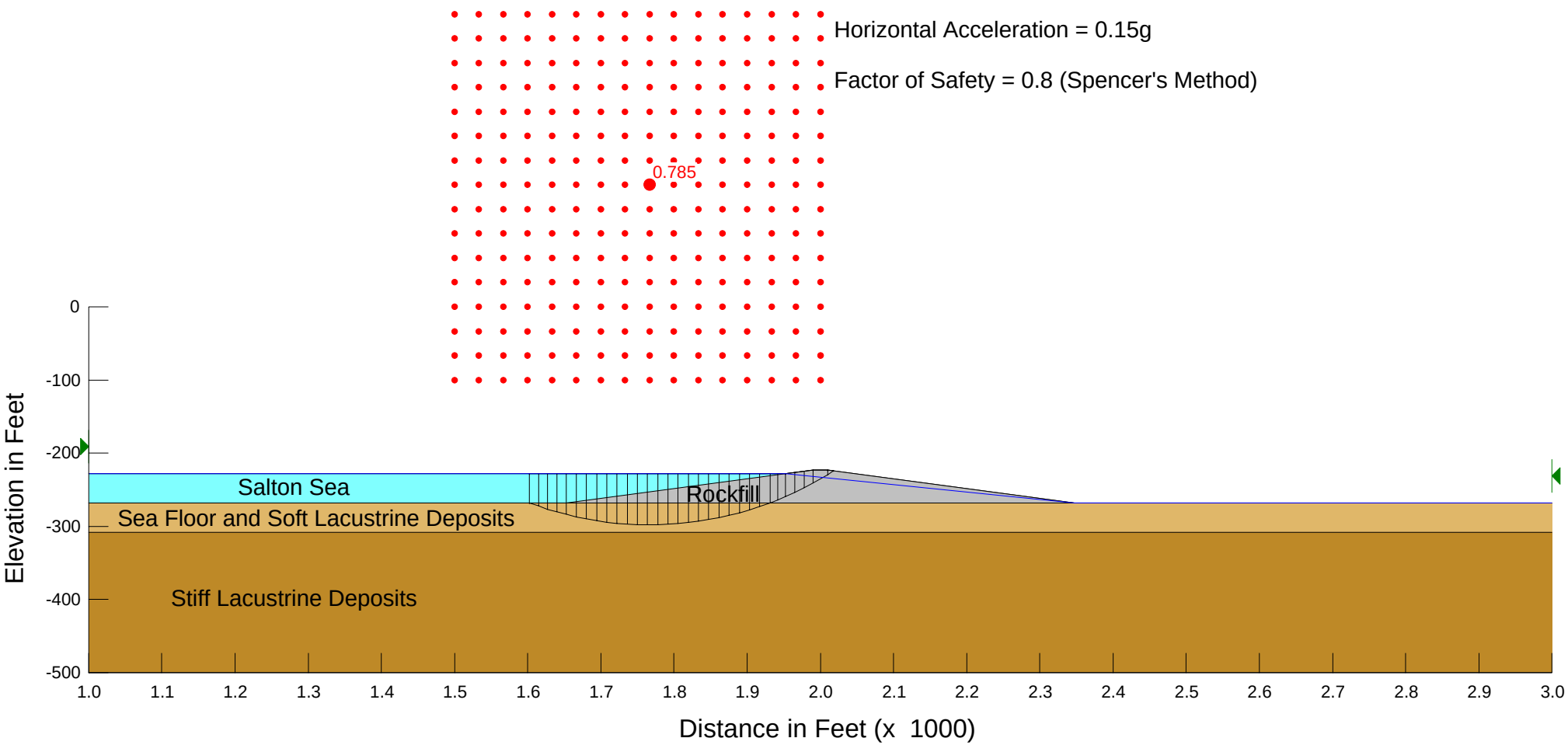
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 7.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 0.8 (Spencer's Method)



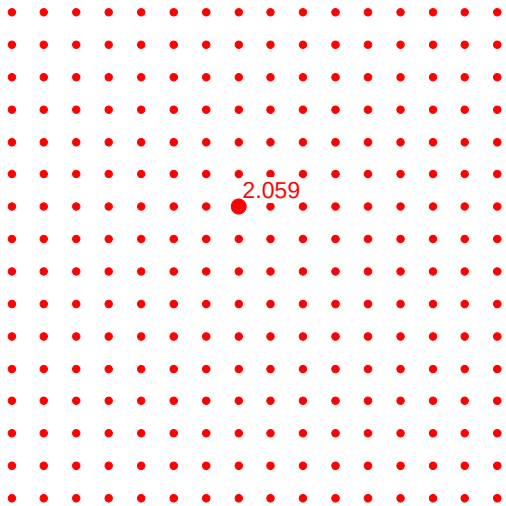
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

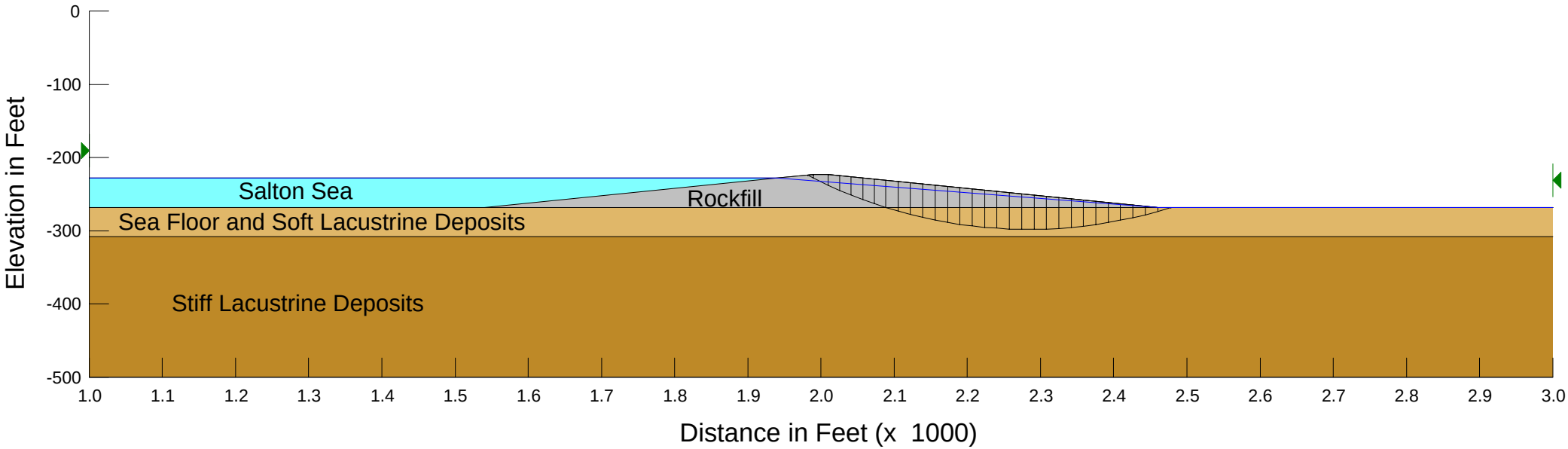
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 2.1 (Spencer's Method)





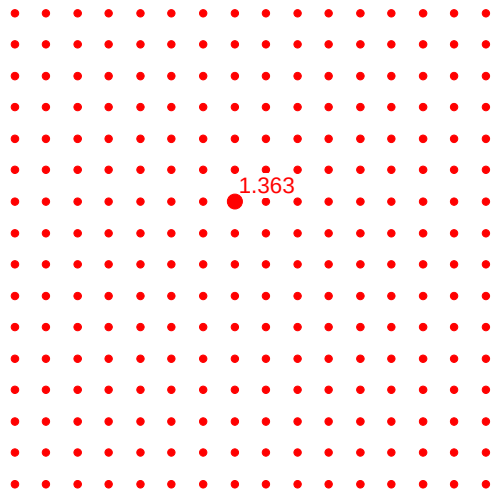
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

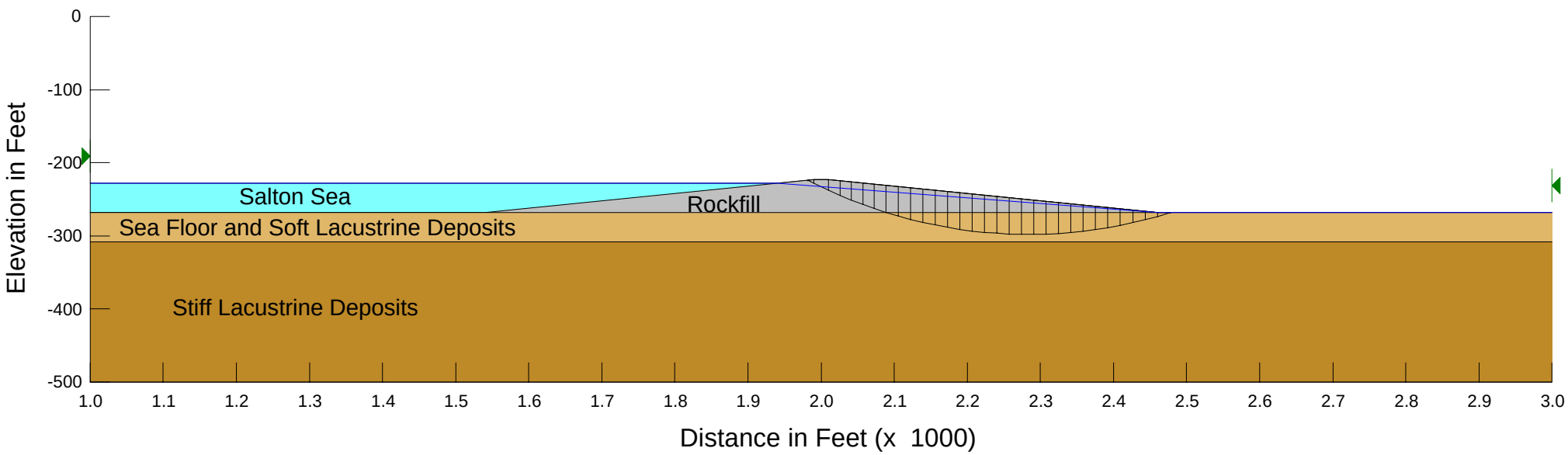
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 1.4 (Spencer's Method)





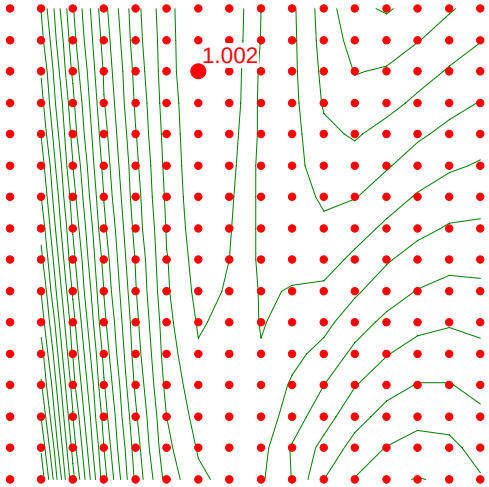
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

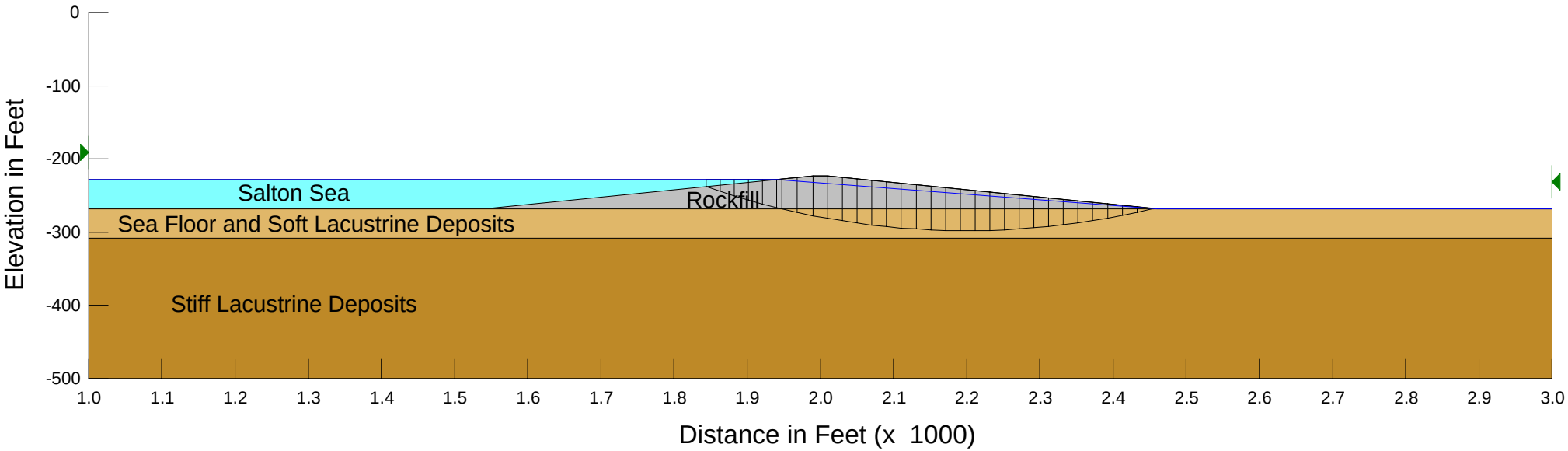
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 1.0 (Spencer's Method)





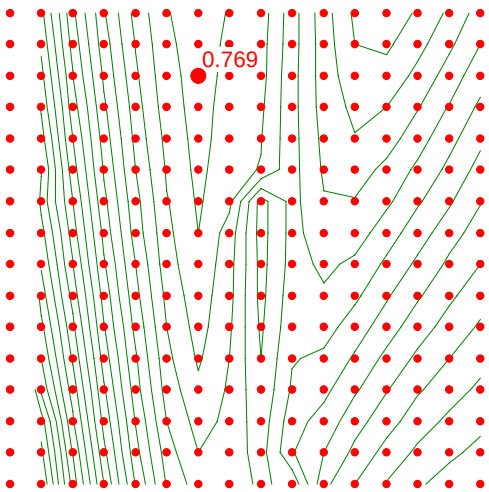
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

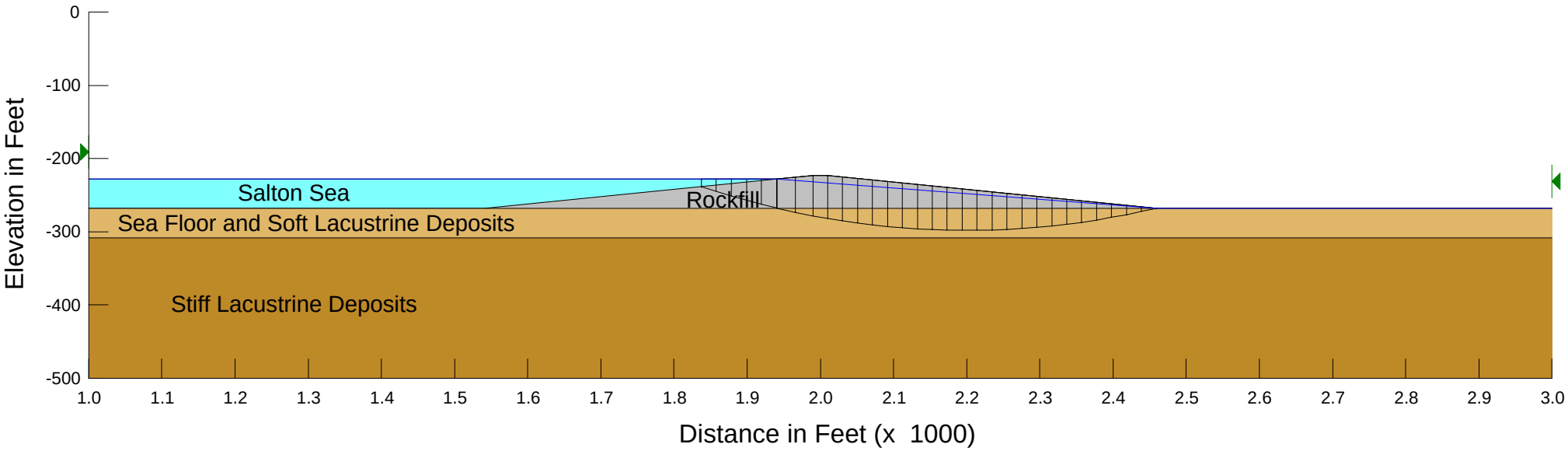
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 0.8 (Spencer's Method)





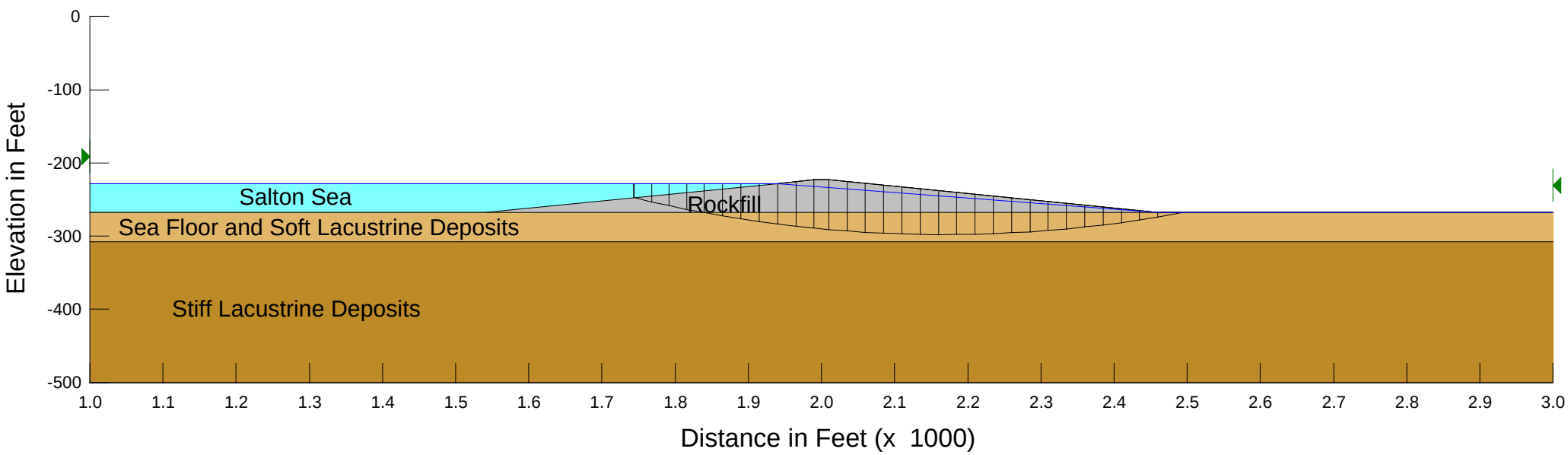
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.2g

Factor of Safety = 0.595 (Spencer's Method)



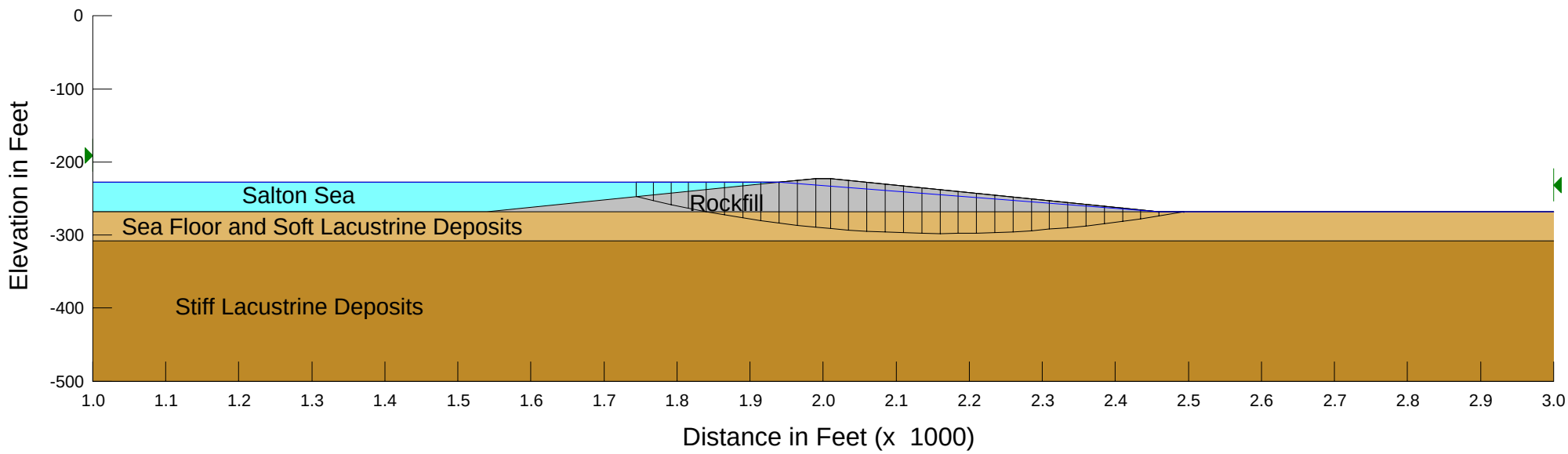
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.25g

Factor of Safety = 0.491 (Spencer's Method)



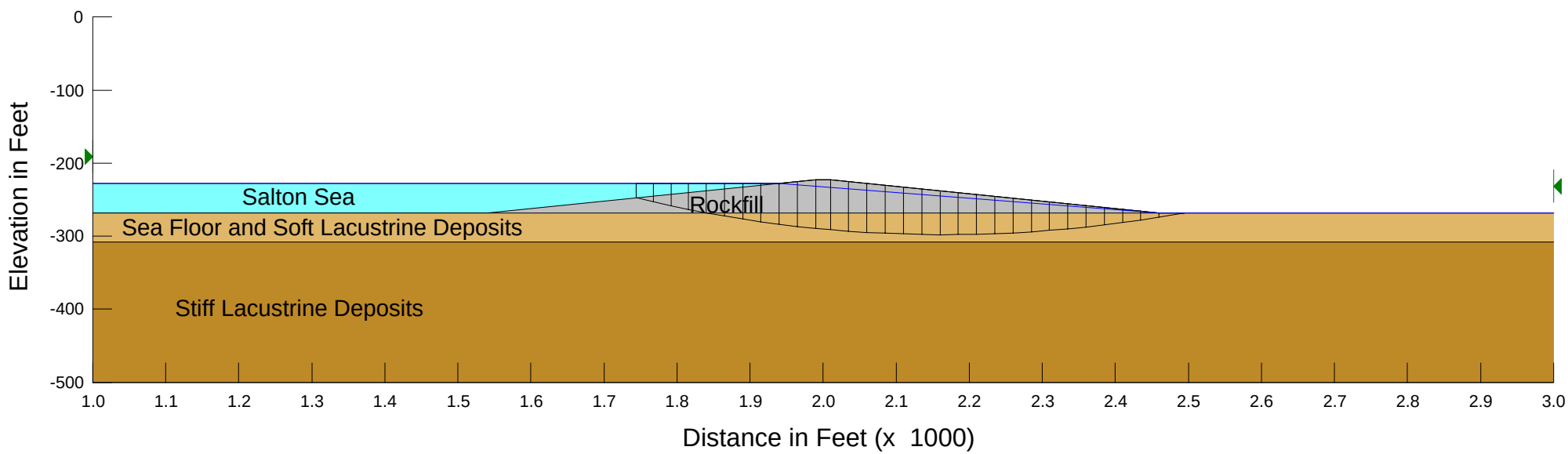
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.3g

Factor of Safety = 0.418 (Spencer's Method)



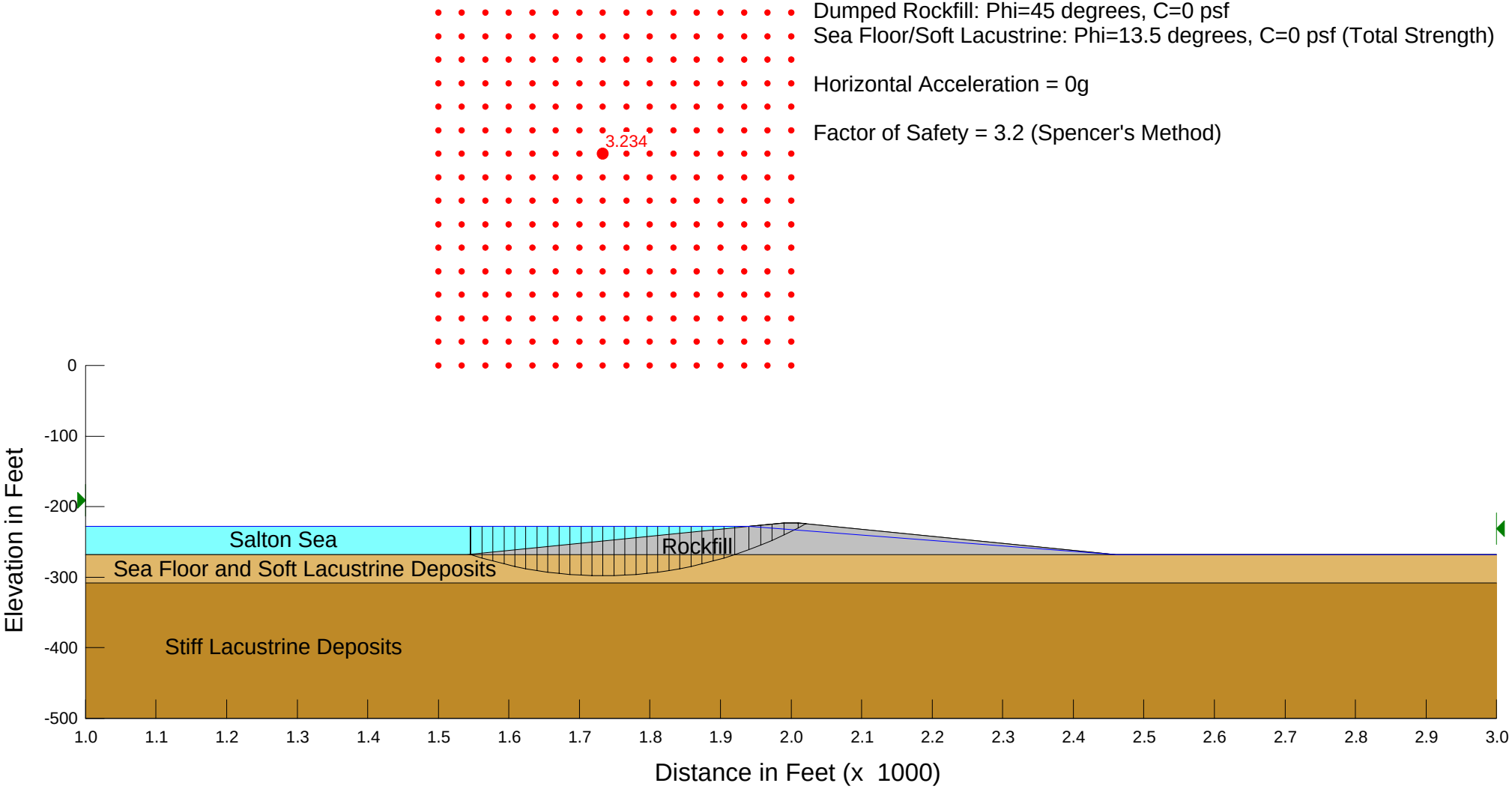
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 3.2 (Spencer's Method)



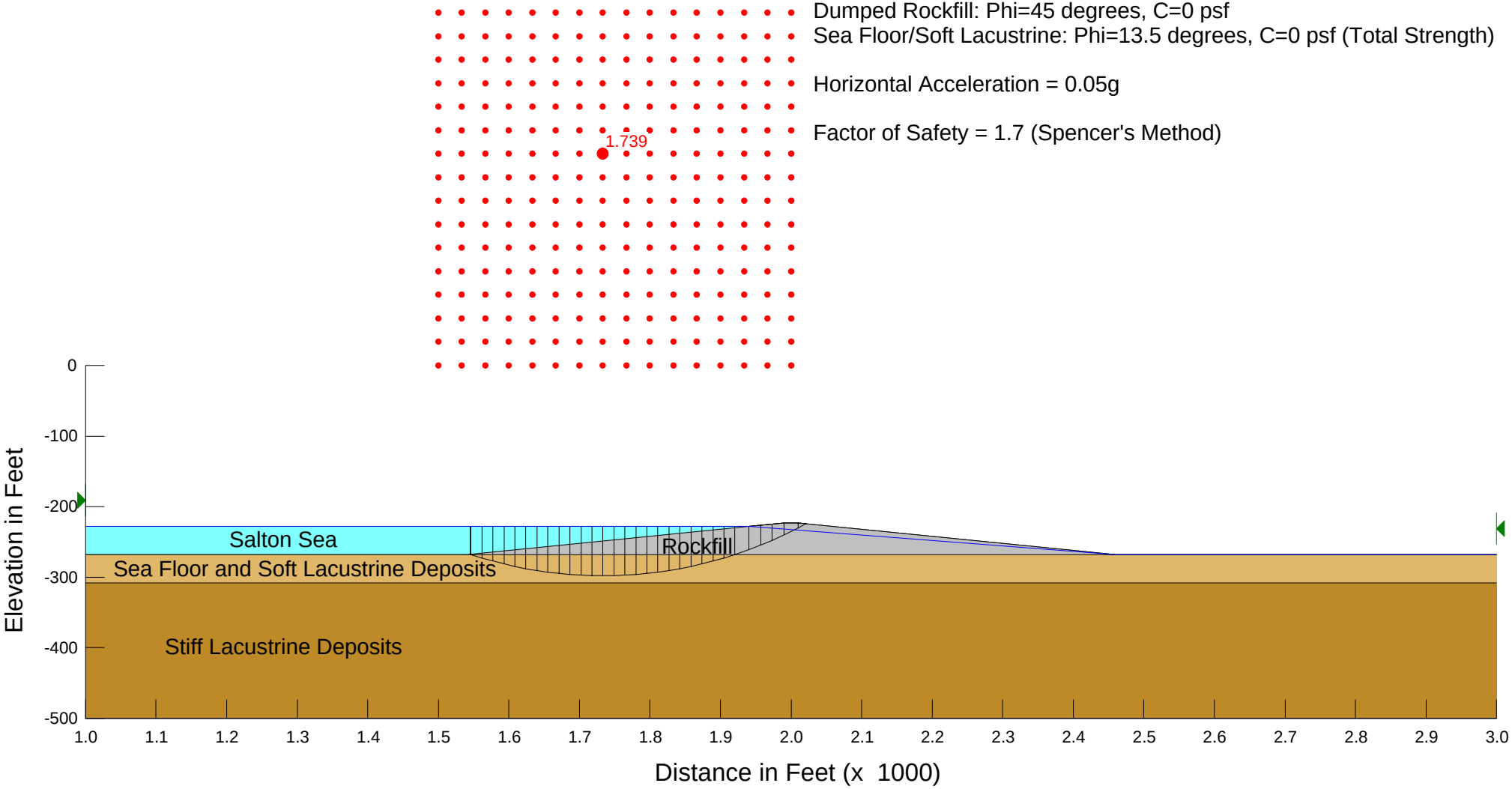
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 1.7 (Spencer's Method)



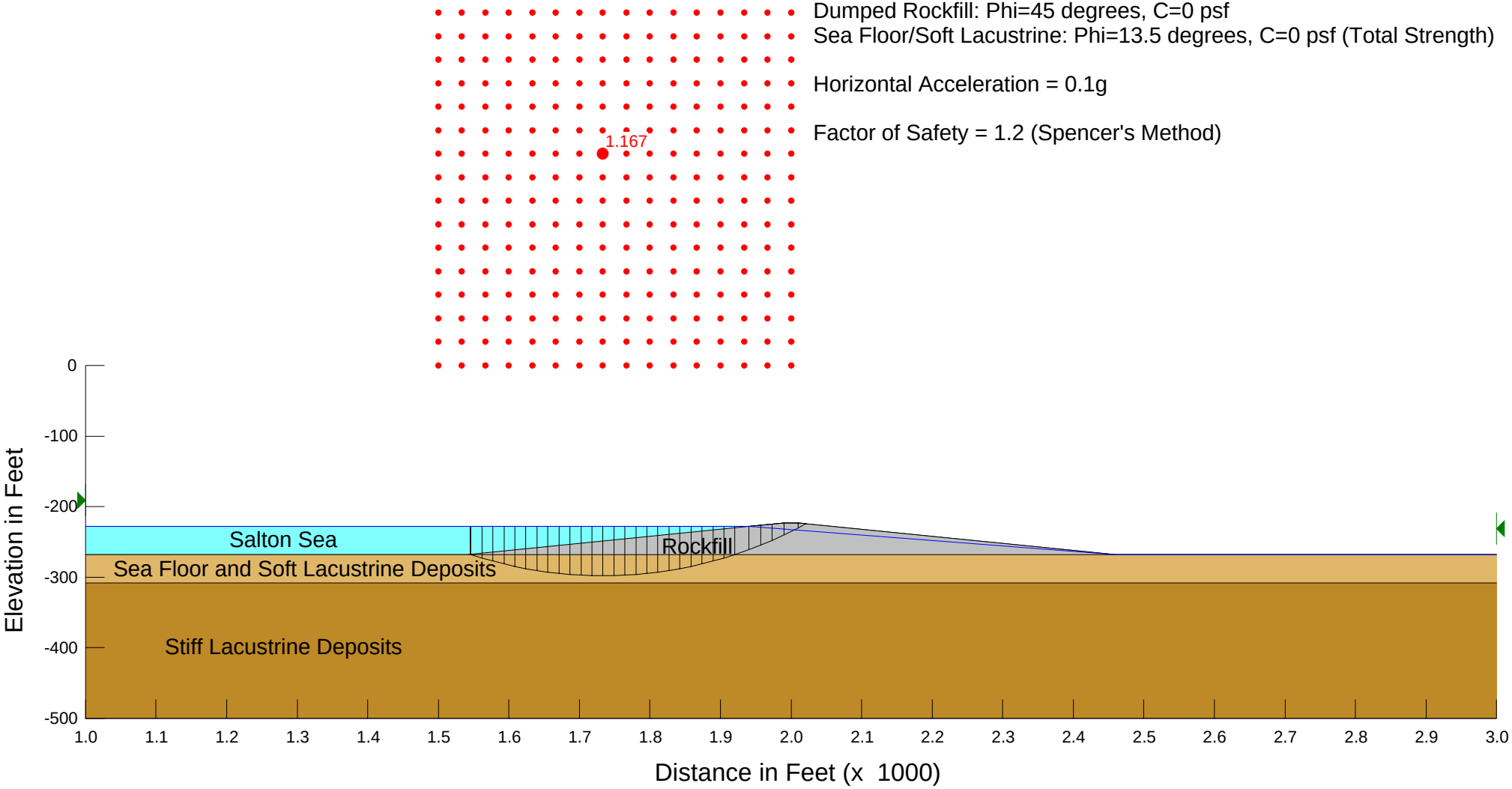
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 1.2 (Spencer's Method)



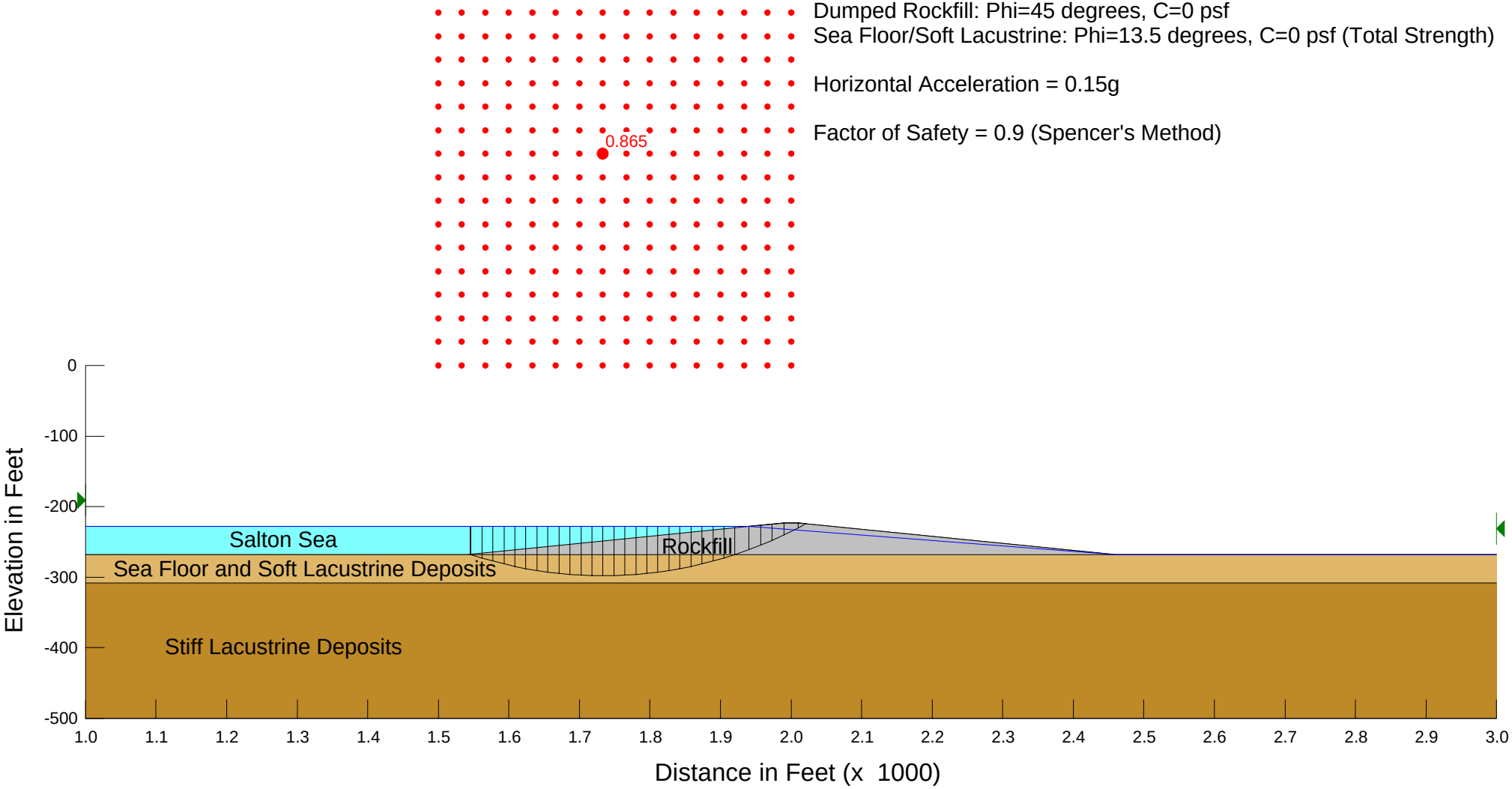
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 0.9 (Spencer's Method)



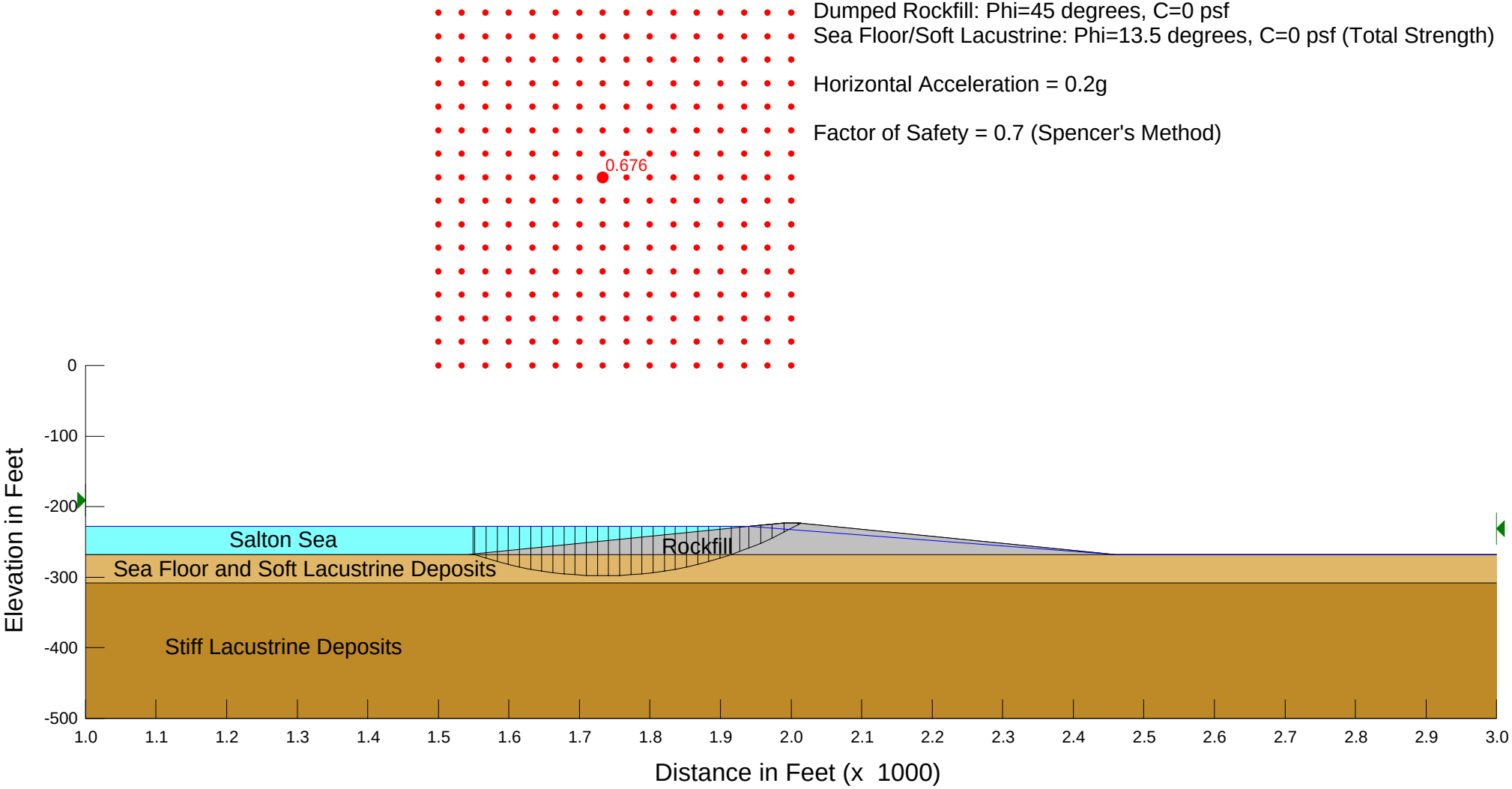
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.2g

Factor of Safety = 0.7 (Spencer's Method)



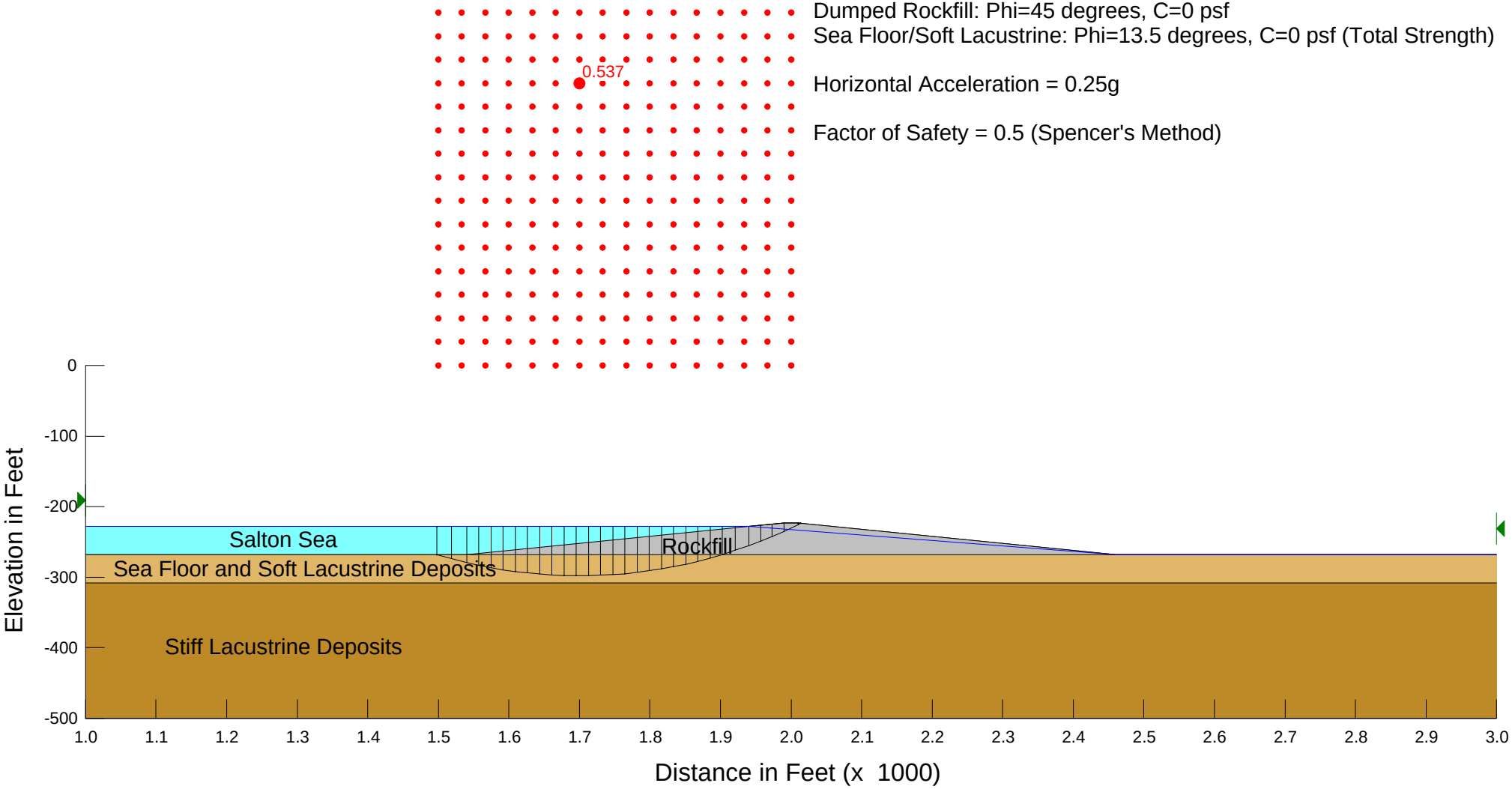
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.25g

Factor of Safety = 0.5 (Spencer's Method)



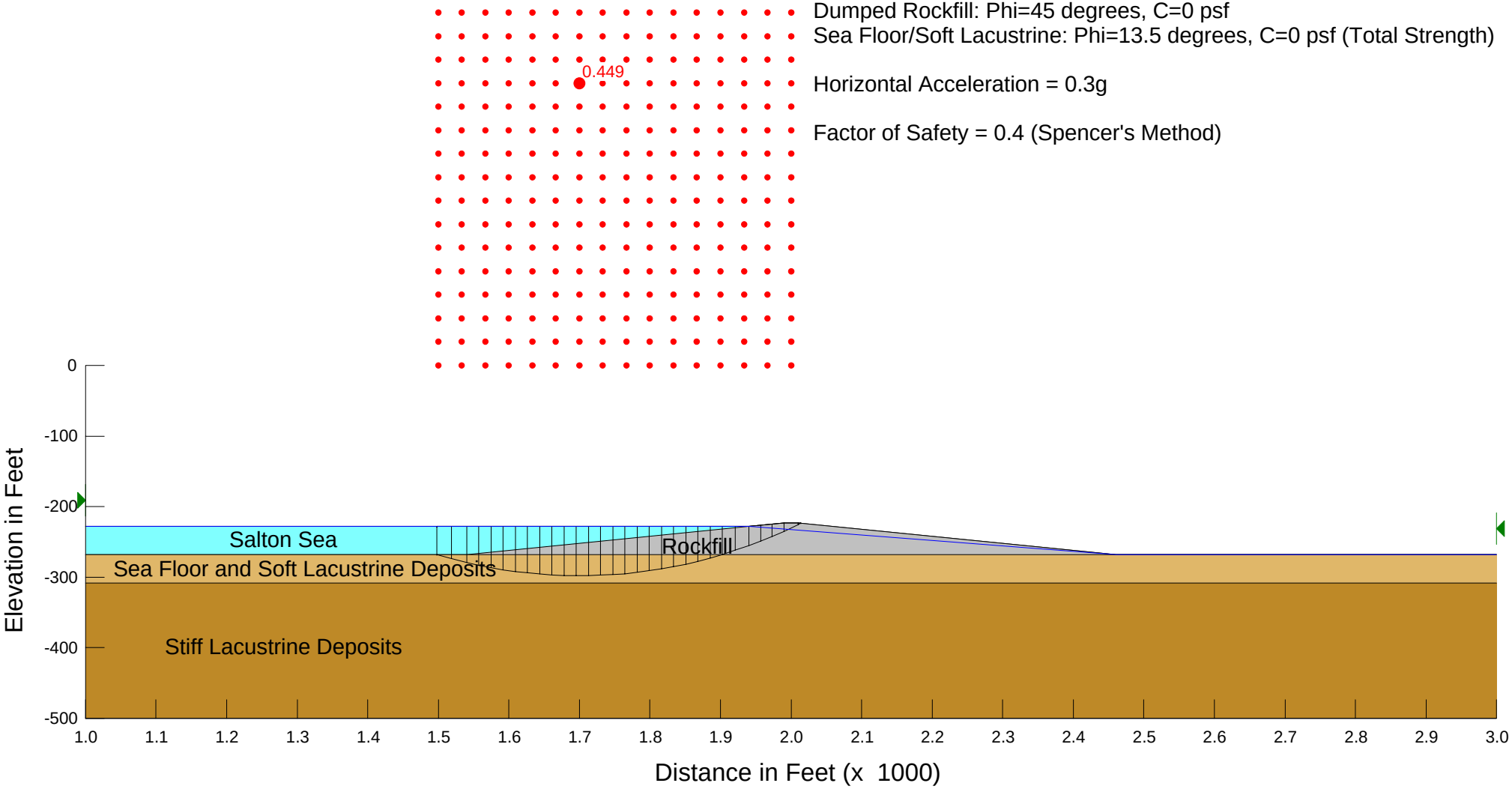
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.3g

Factor of Safety = 0.4 (Spencer's Method)



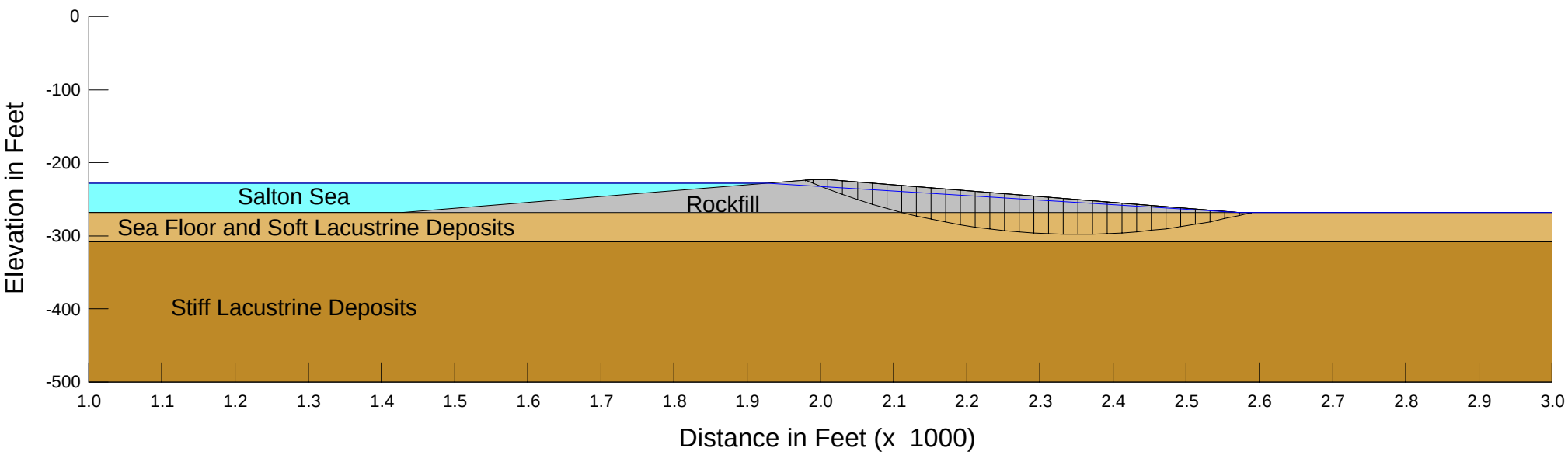
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 12.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 2.553 (Spencer's Method)



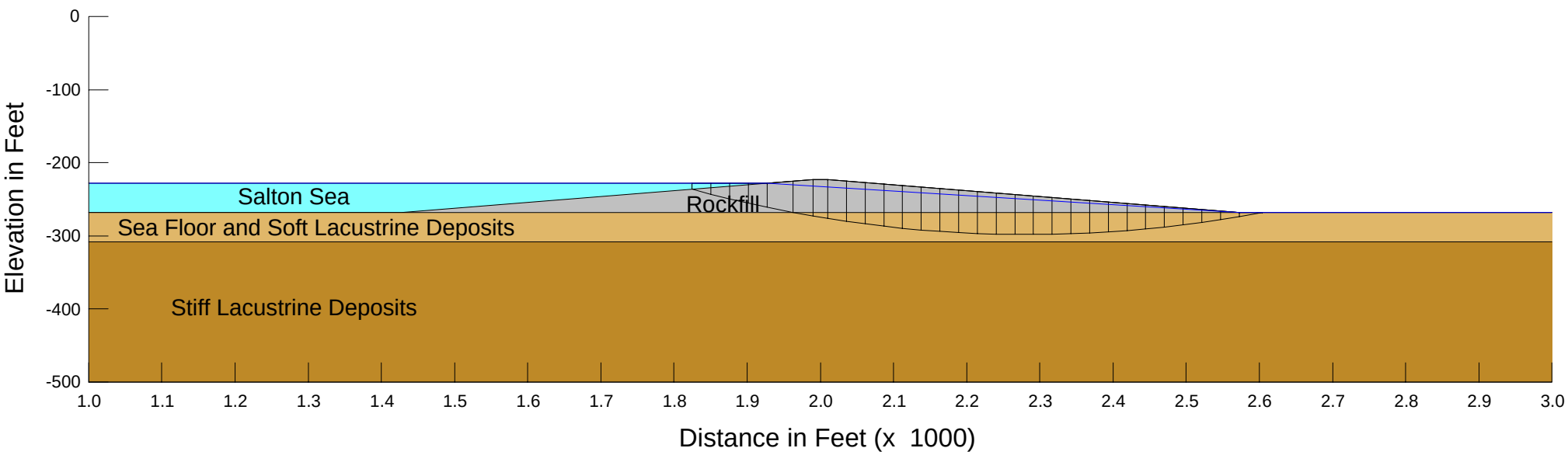
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 12.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 1.605 (Spencer's Method)



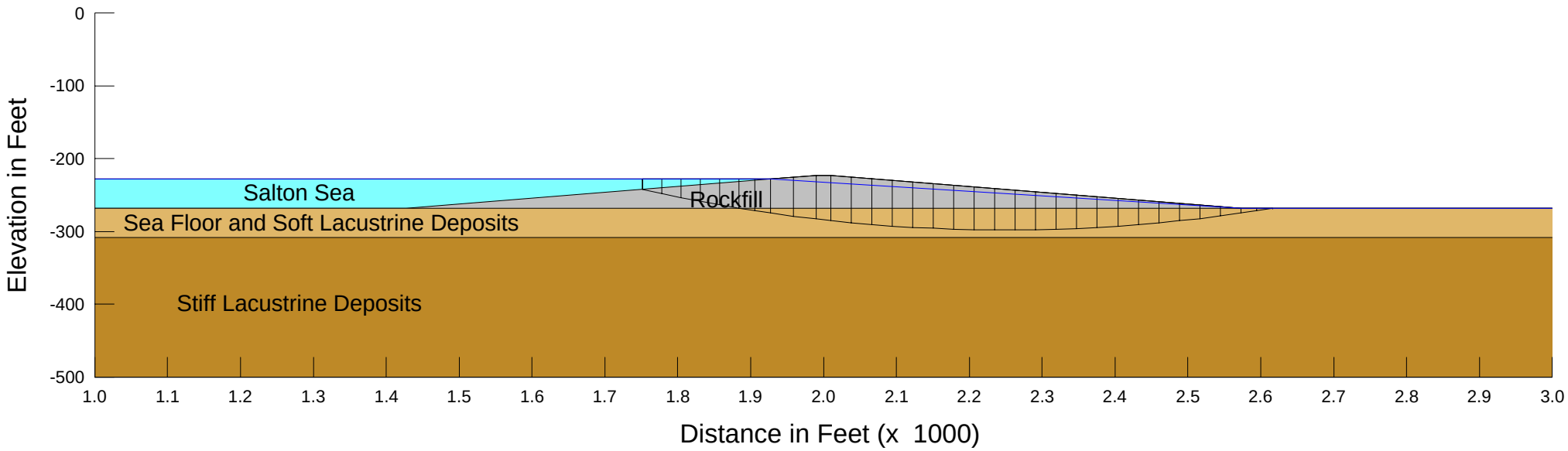
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 12.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 1.080 (Spencer's Method)



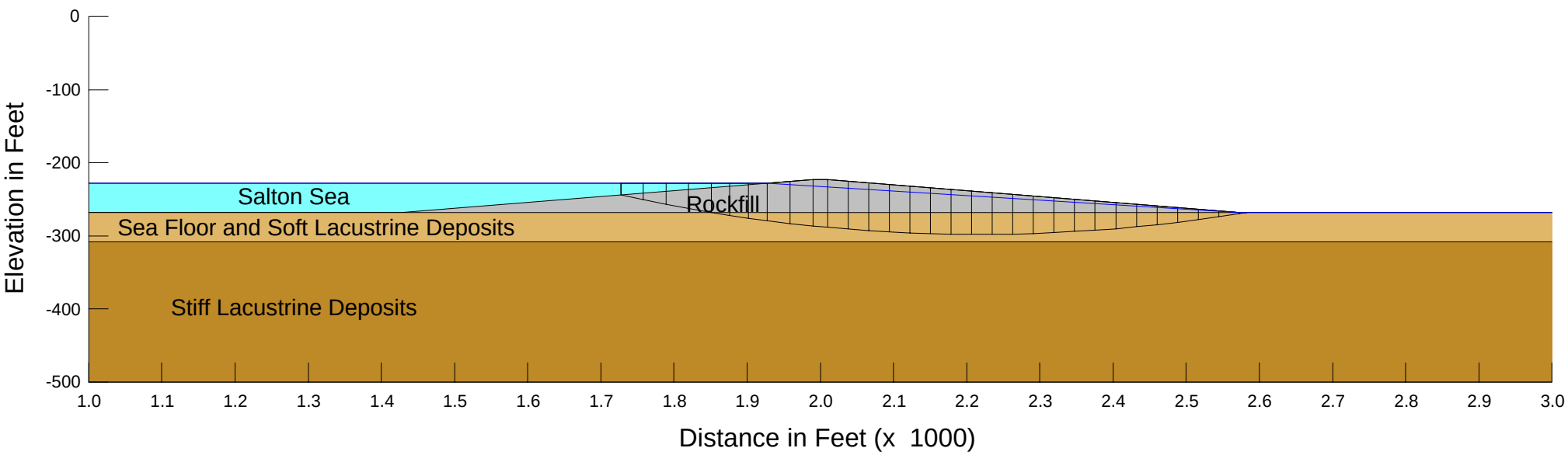
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

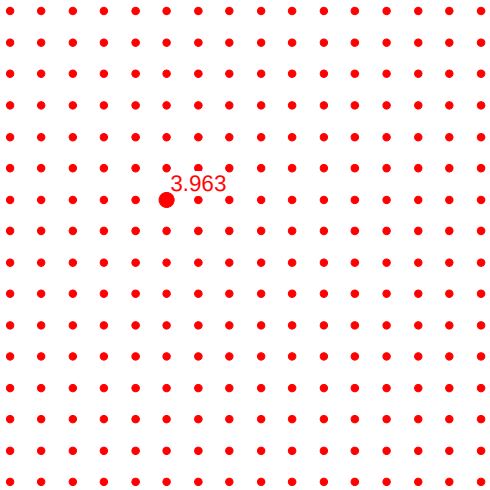
45' High Barrier with 12.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 0.800 (Spencer's Method)





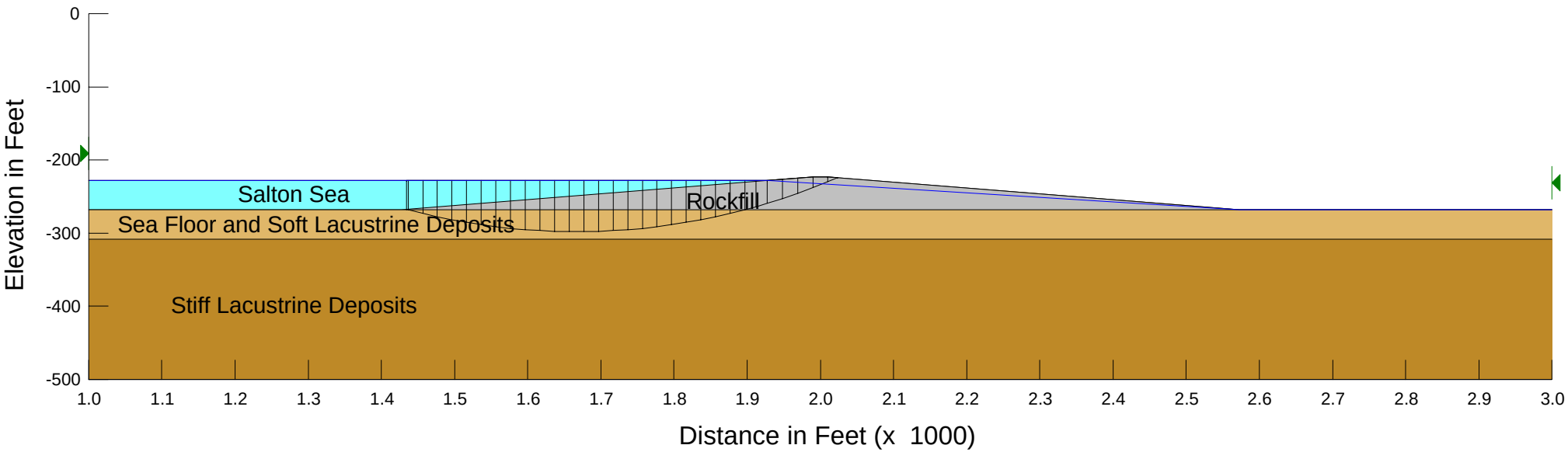
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

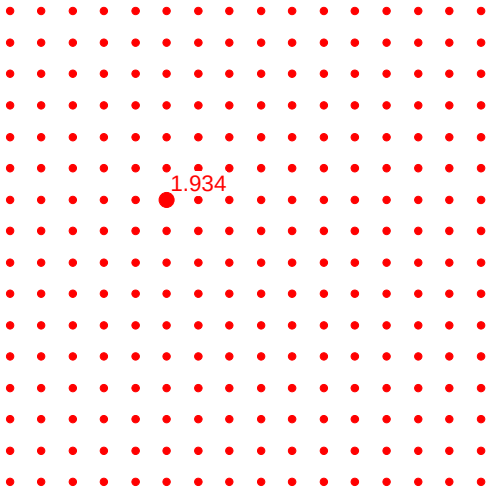
45' High Barrier with 12.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 4.0 (Spencer's Method)





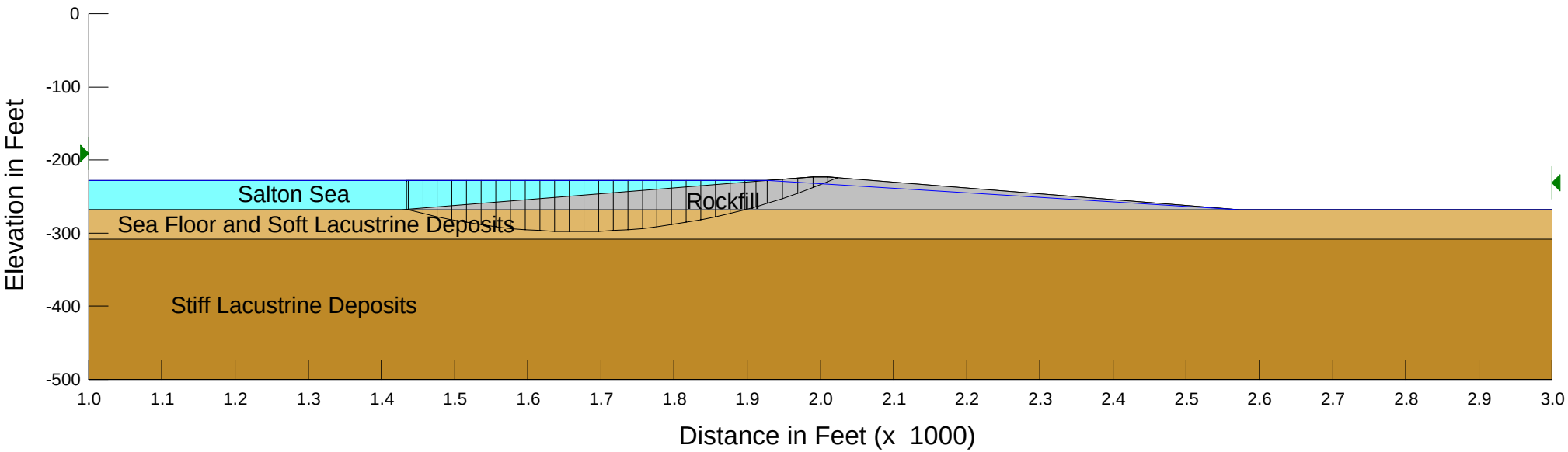
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

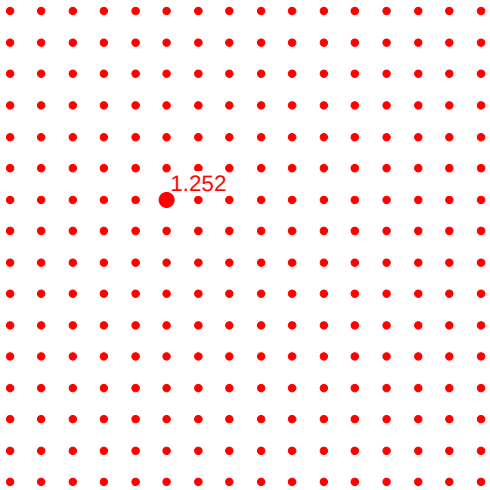
45' High Barrier with 12.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 1.9 (Spencer's Method)





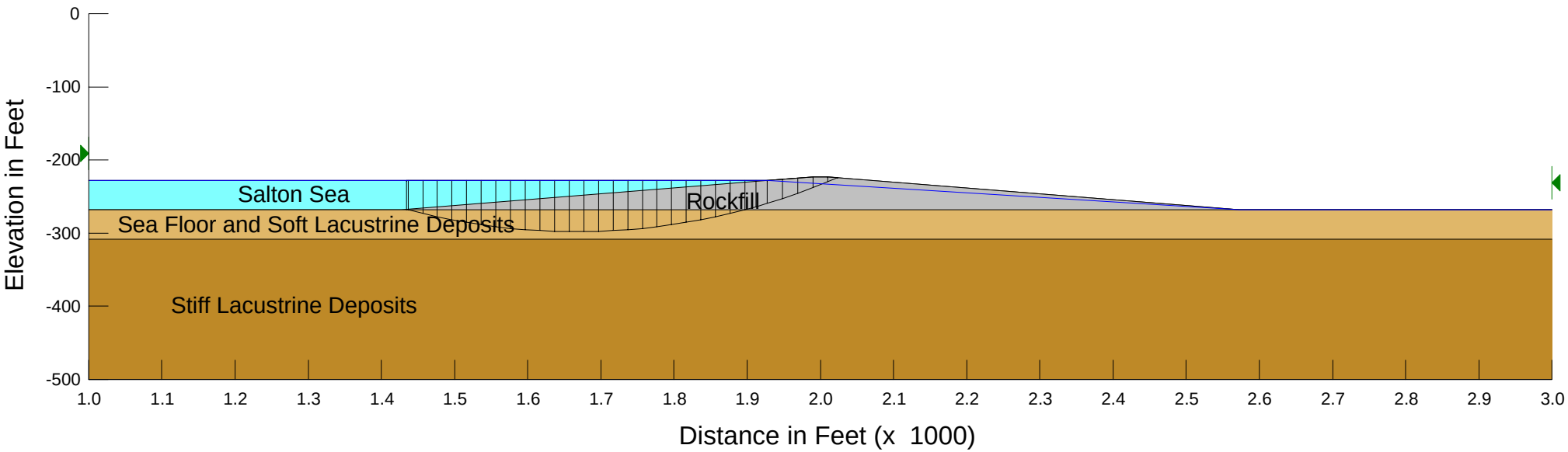
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

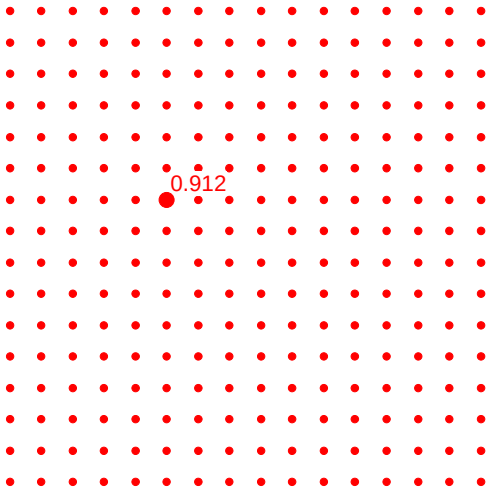
45' High Barrier with 12.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 1.3 (Spencer's Method)





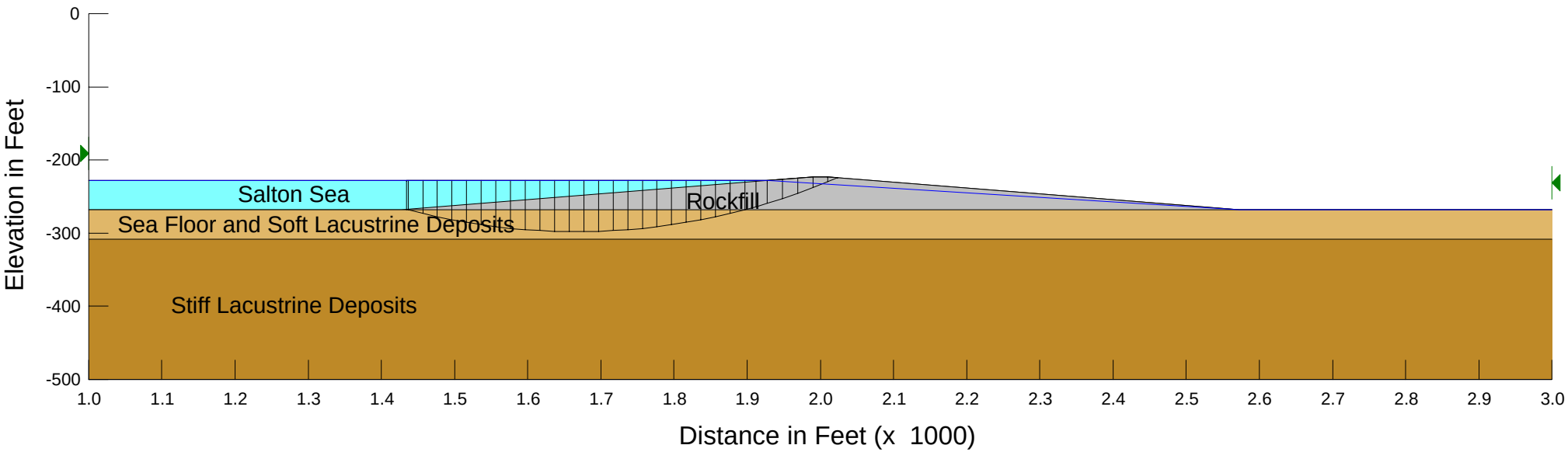
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 12.5H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 0.9 (Spencer's Method)



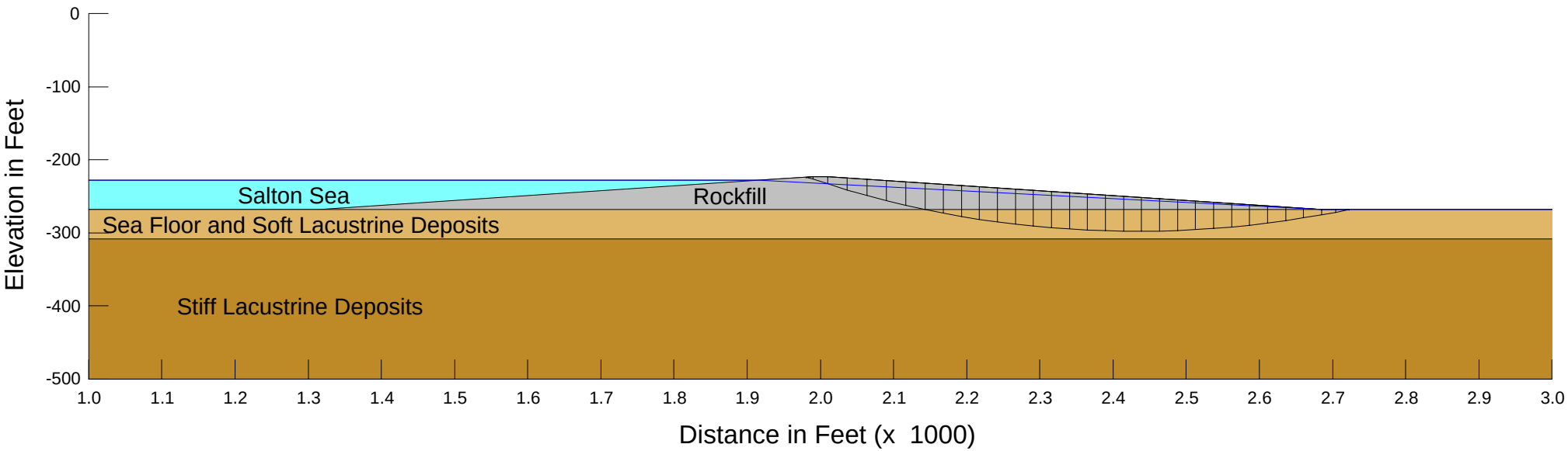
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 15H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 3.045 (Spencer's Method)



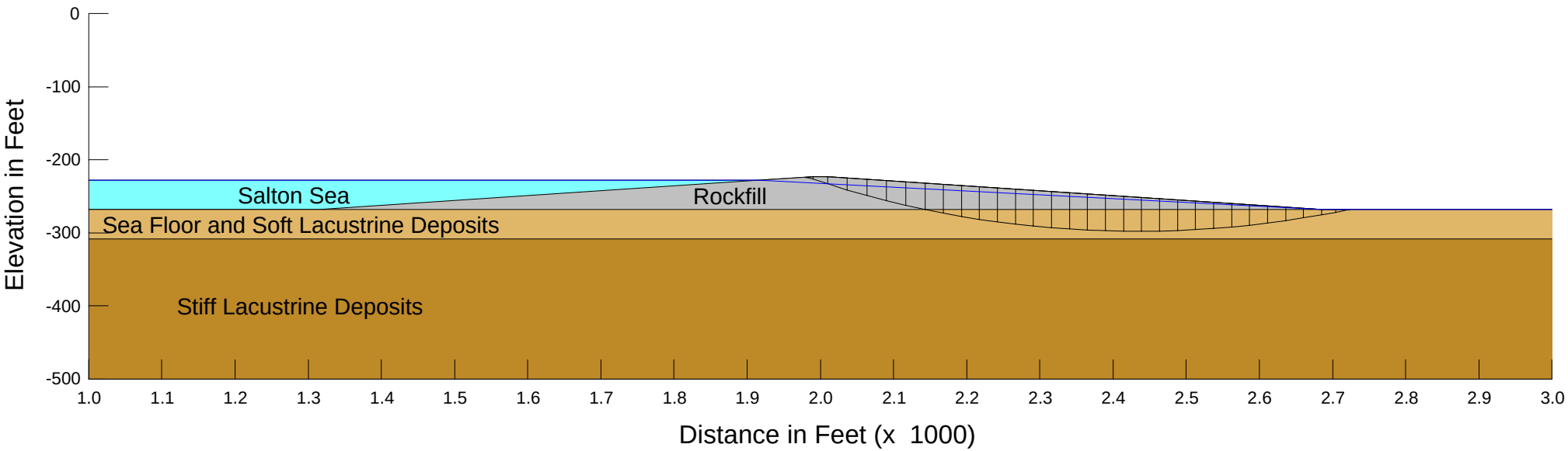
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 15H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 1.763 (Spencer's Method)



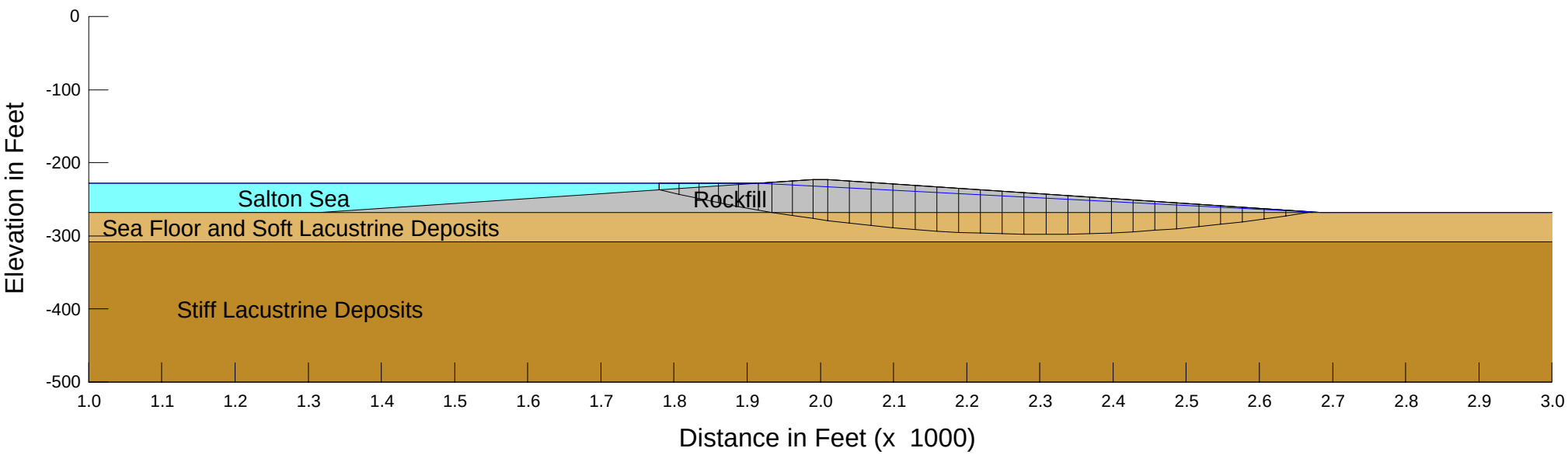
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 15H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 1.161 (Spencer's Method)



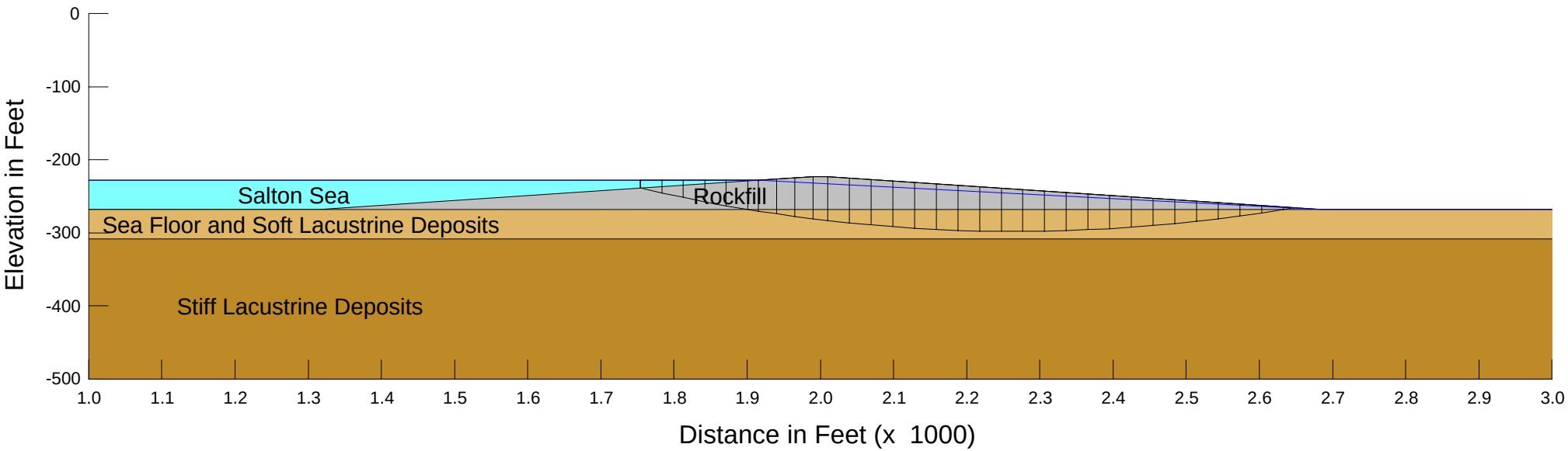
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 15H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 0.855 (Spencer's Method)



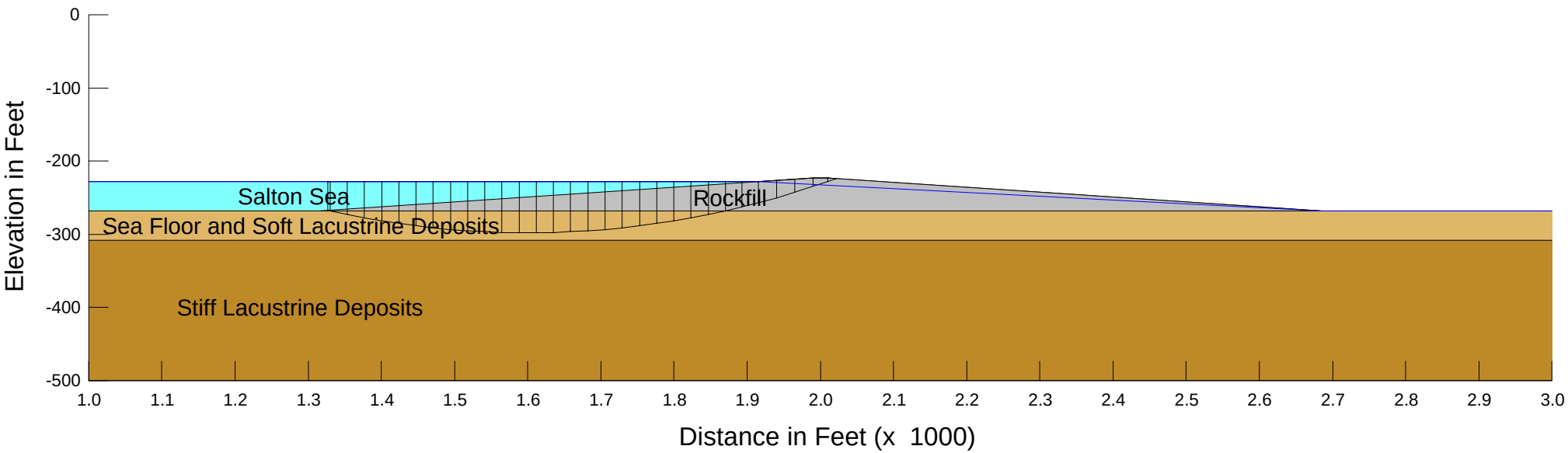
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 15H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 4.662 (Spencer's Method)



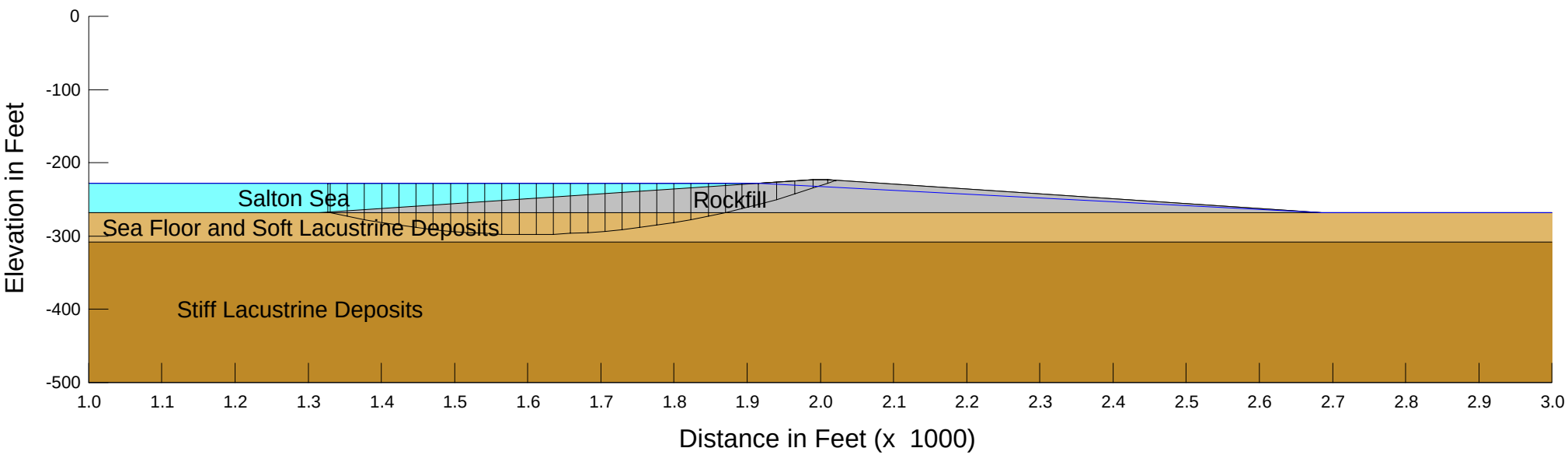
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 15H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 2.081 (Spencer's Method)



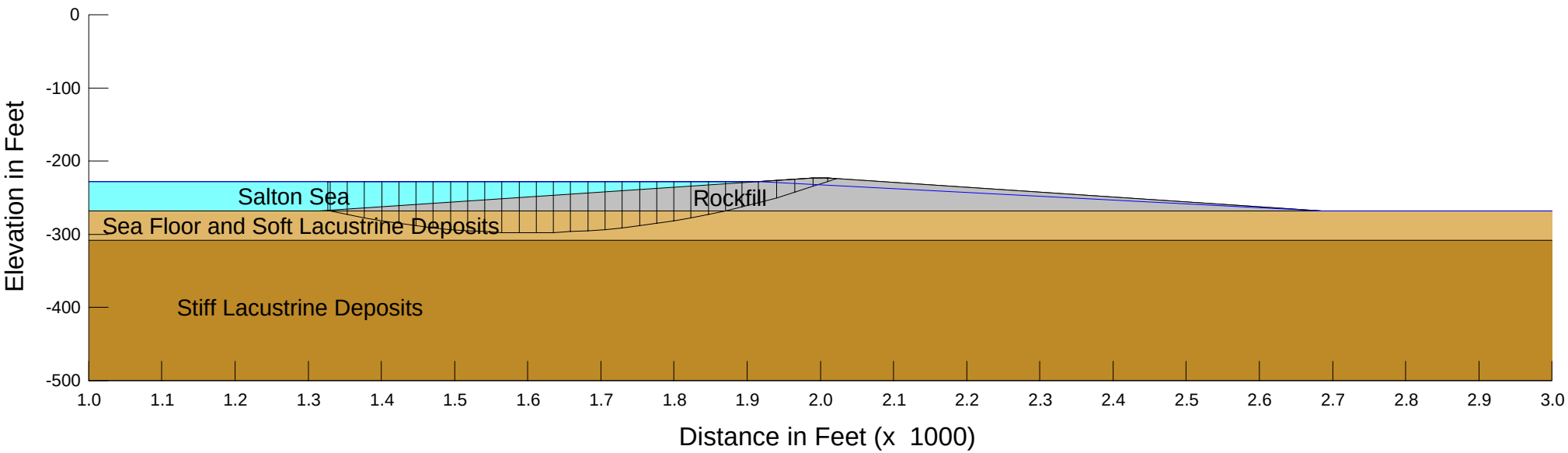
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 15H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 1.313 (Spencer's Method)



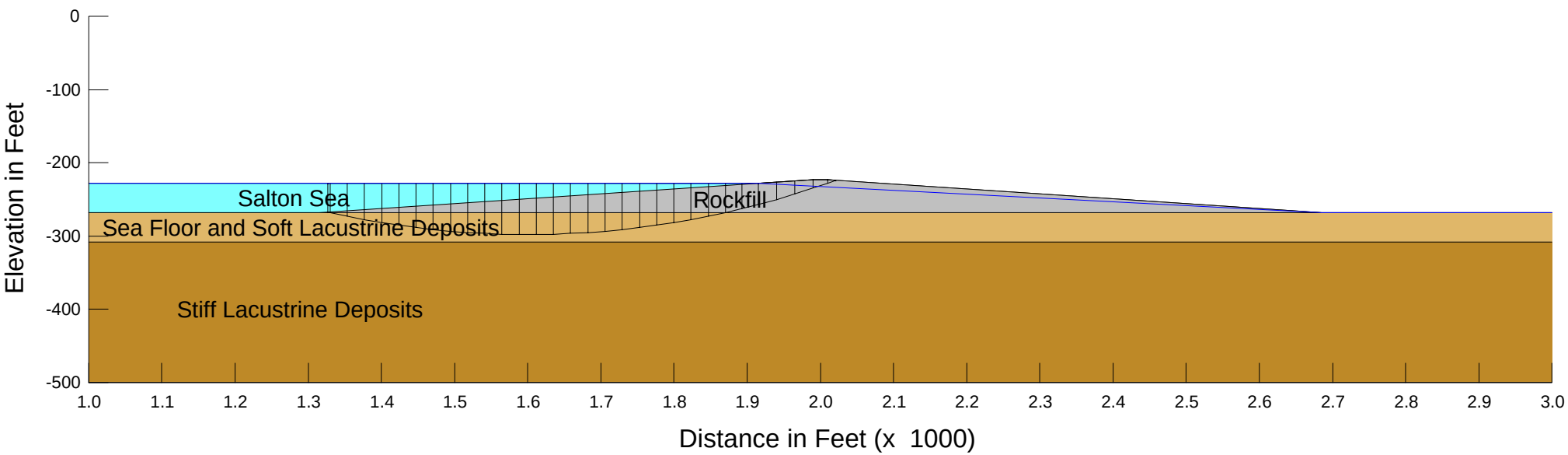
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 15H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 0.946 (Spencer's Method)



LONG-TERM BARRIER STABILITY WITH
STEADY-STATE SEEPAGE CONDITIONS AND EFFECTIVE STRENGTHS
BARRIER SLOPES = 10H:1V
BARRIER HEIGHT = 25 FEET

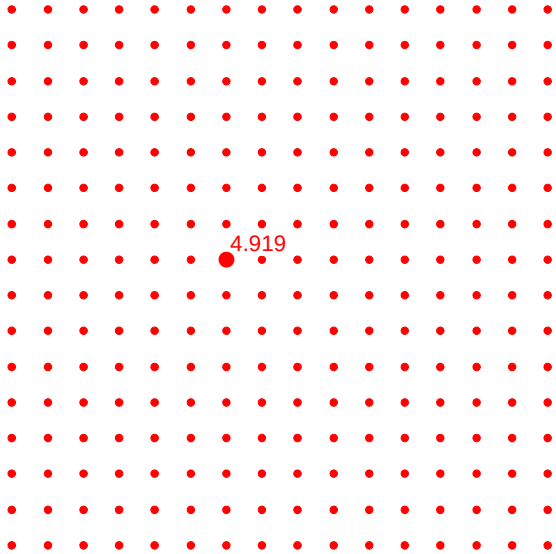
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

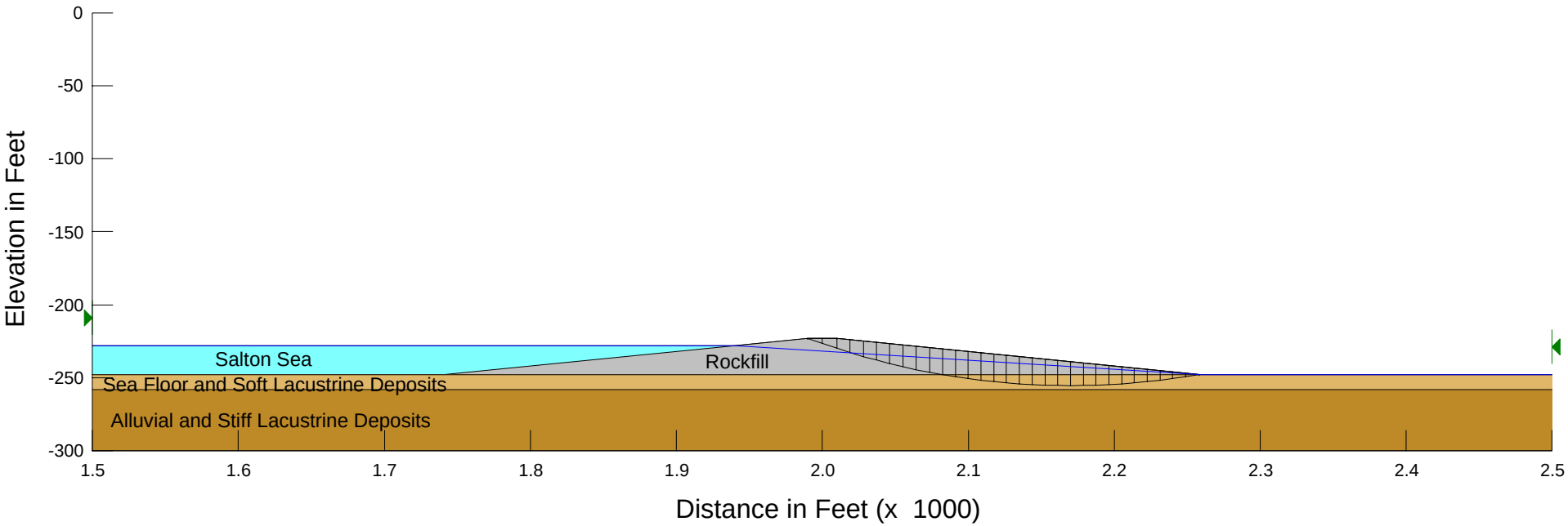
25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29 degrees, C=0 psf (Effective Strength)

Horizontal Acceleration = 0g

Factor of Safety = 4.9 (Spencer's Method)





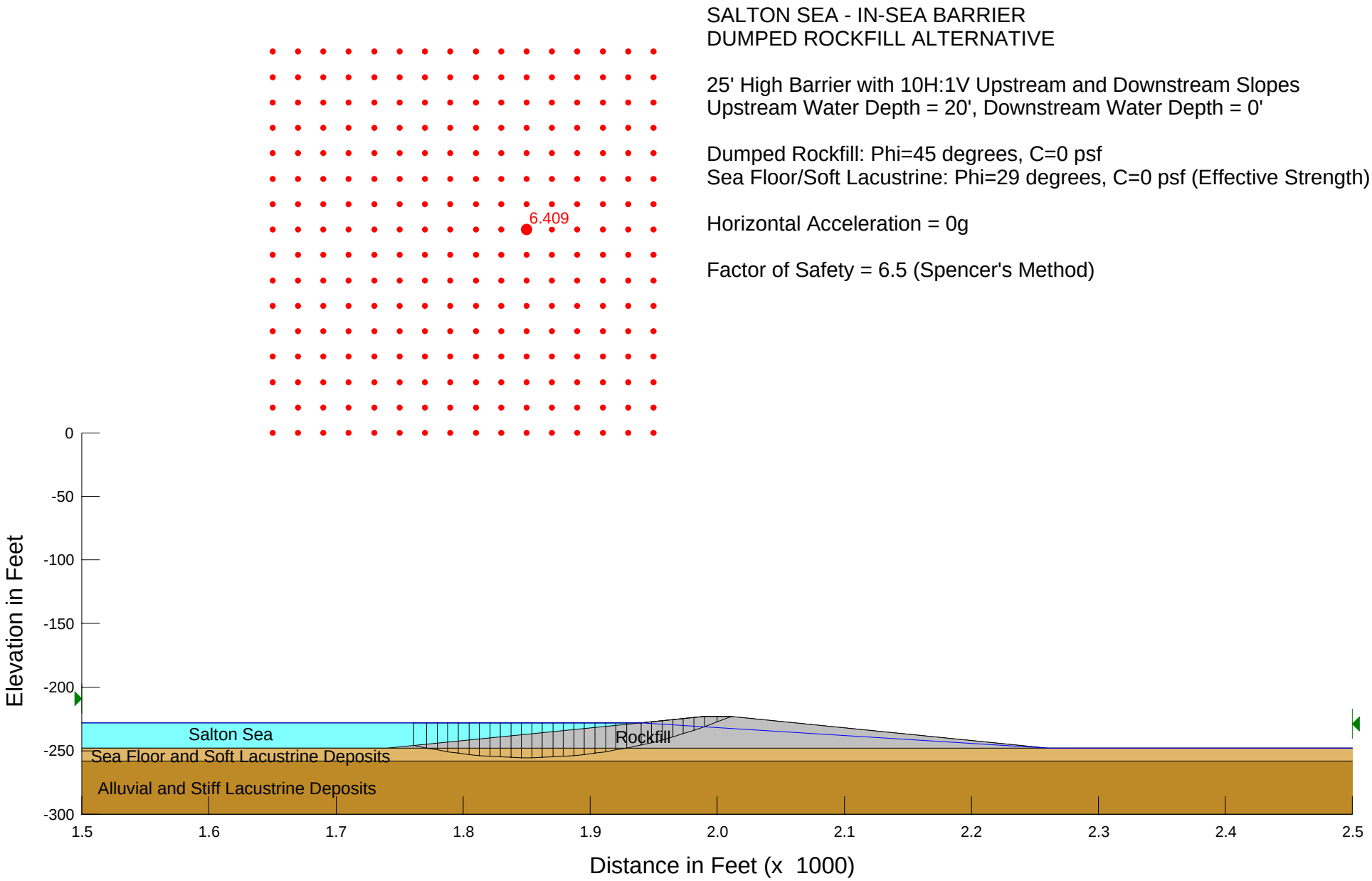
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29 degrees, C=0 psf (Effective Strength)

Horizontal Acceleration = 0g

Factor of Safety = 6.5 (Spencer's Method)



PSEUDO-DYNAMIC BARRIER STABILITY WITH
HORIZONTAL SEISMIC LOADS AND TOTAL STRENGTHS
BARRIER SLOPES = 10H:1V
BARRIER HEIGHT = 25 FEET

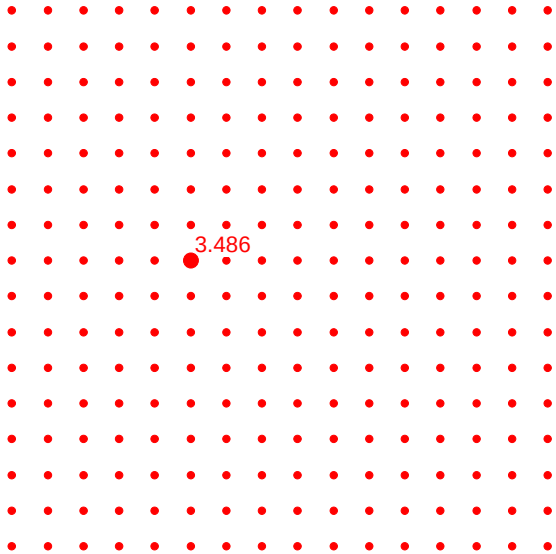
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

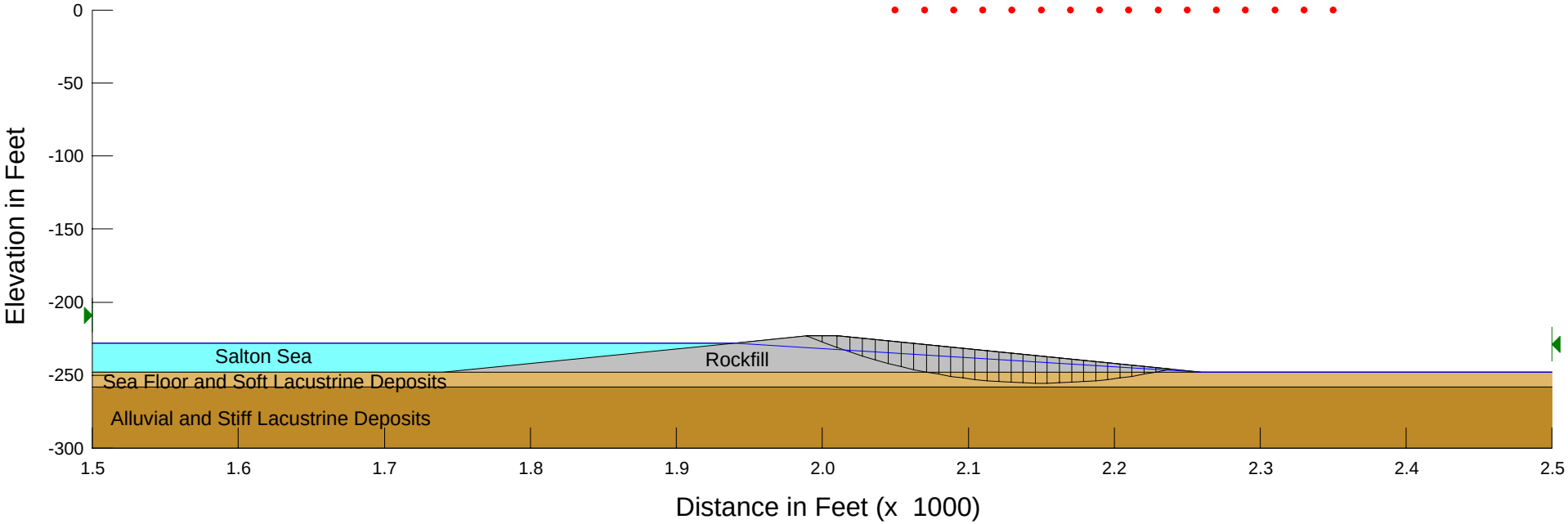
25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 3.5 (Spencer's Method)





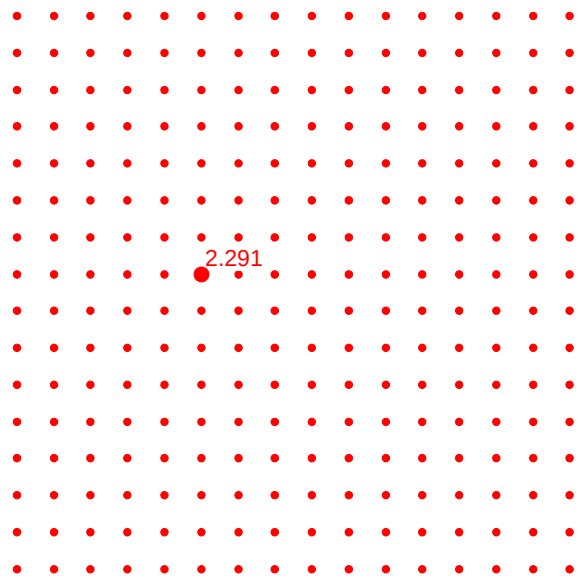
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

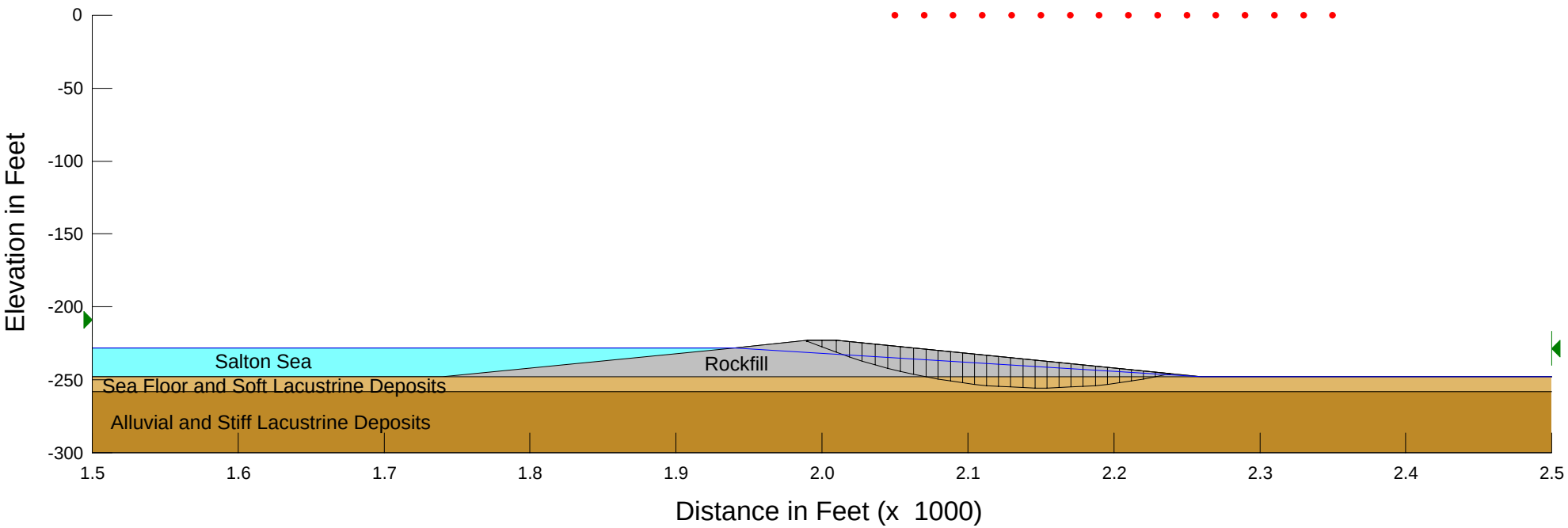
25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 2.3 (Spencer's Method)





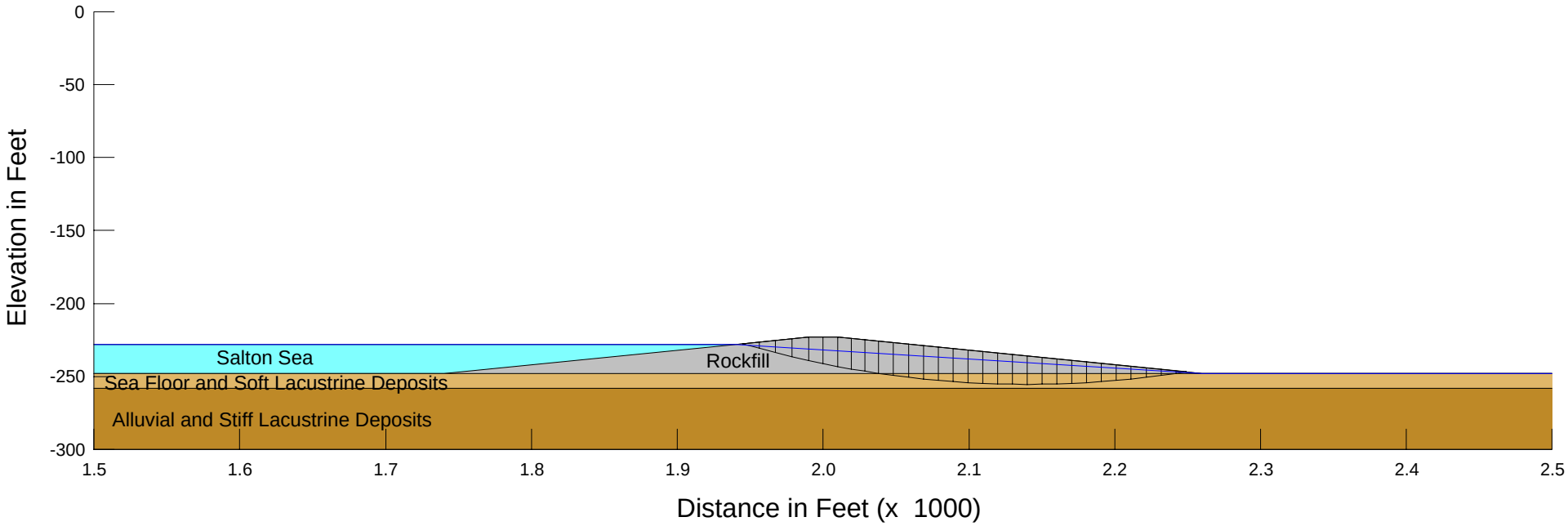
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 1.642 (Spencer's Method)



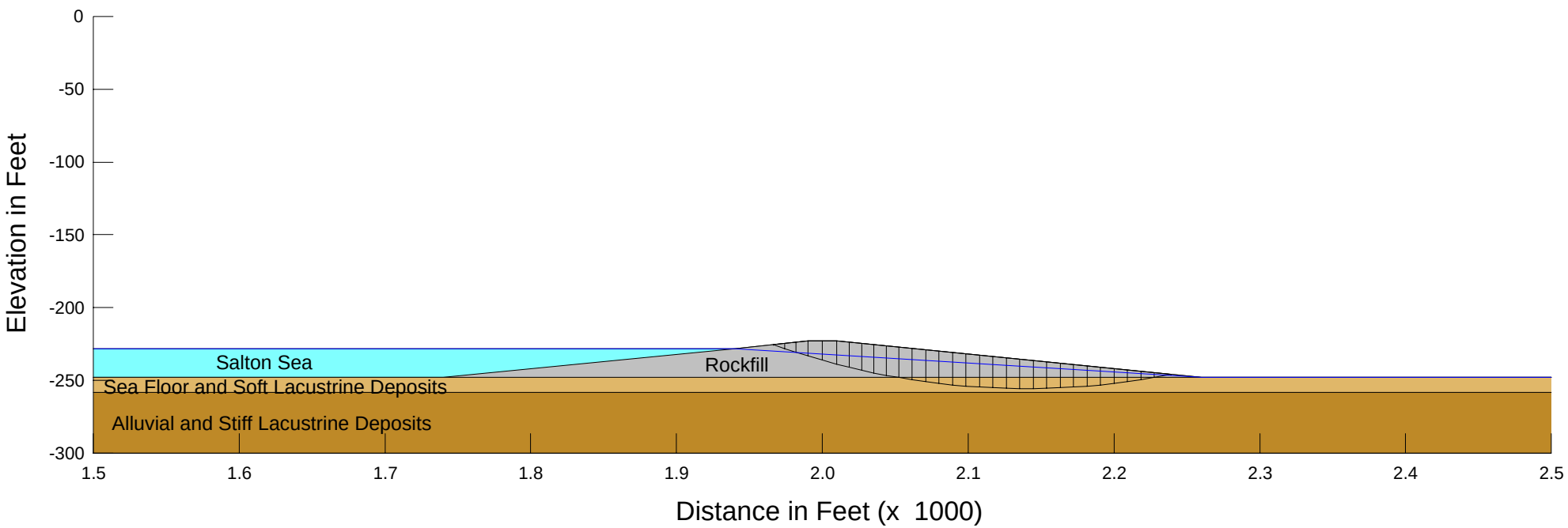
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 1.260 (Spencer's Method)



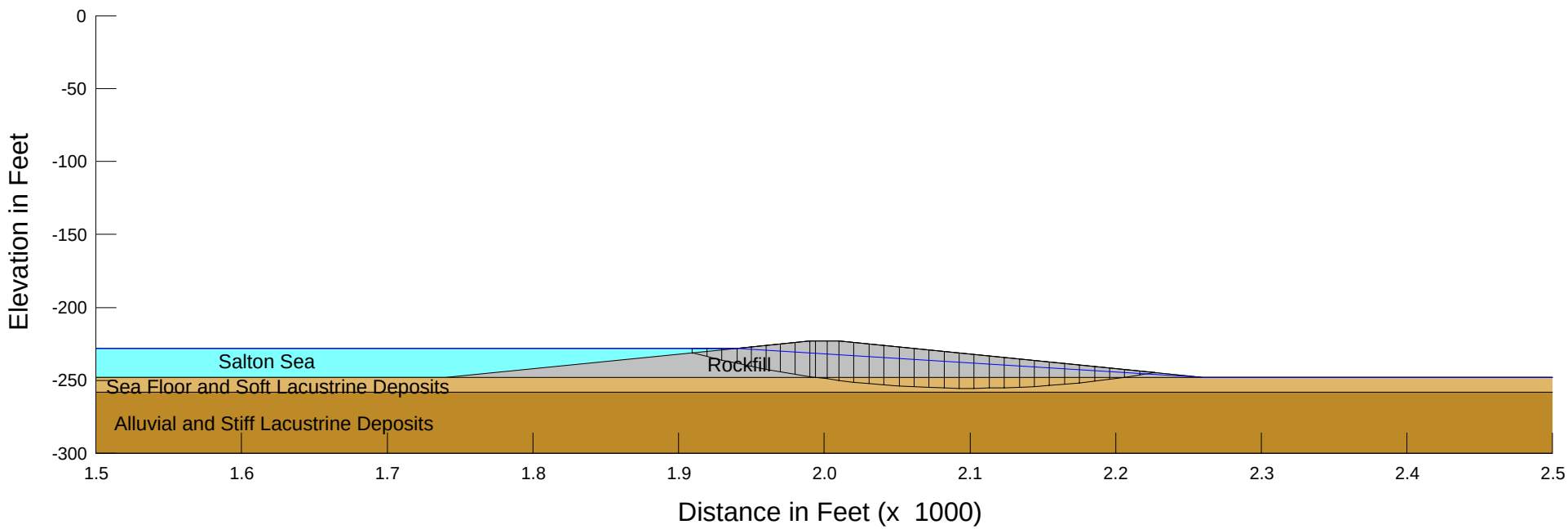
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.2g

Factor of Safety = 0.999 (Spencer's Method)



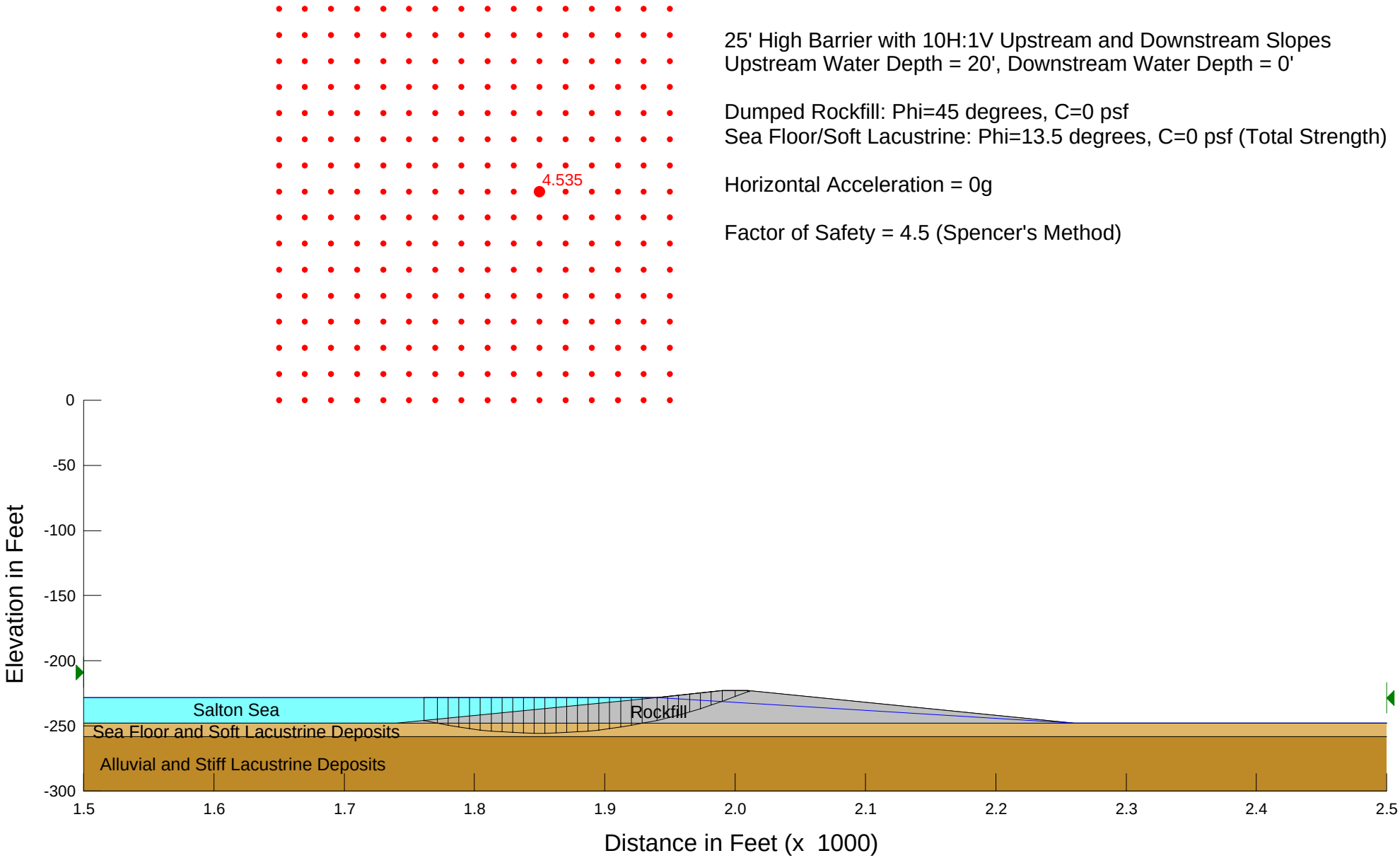
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 4.5 (Spencer's Method)



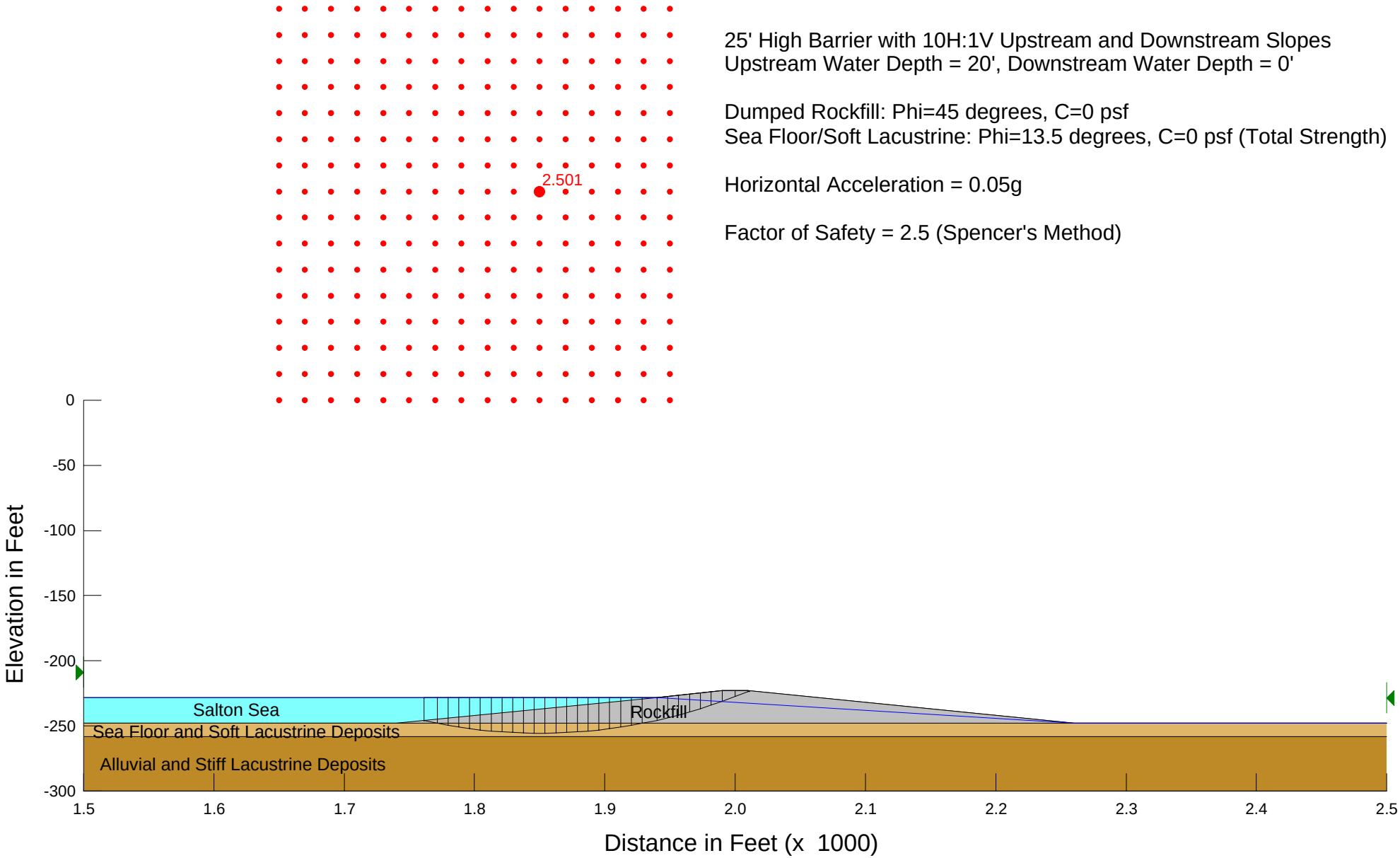
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 2.5 (Spencer's Method)



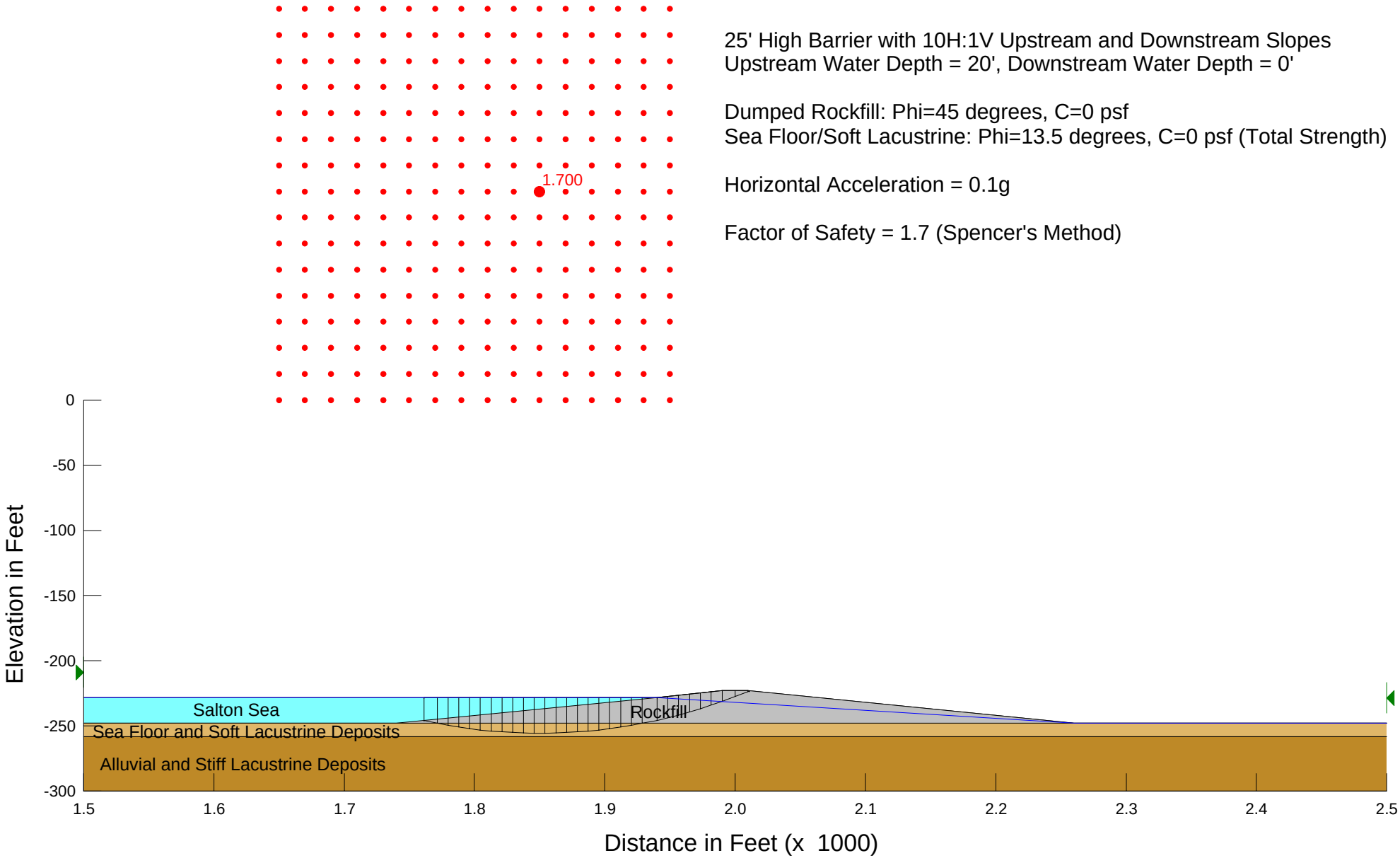
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 1.7 (Spencer's Method)



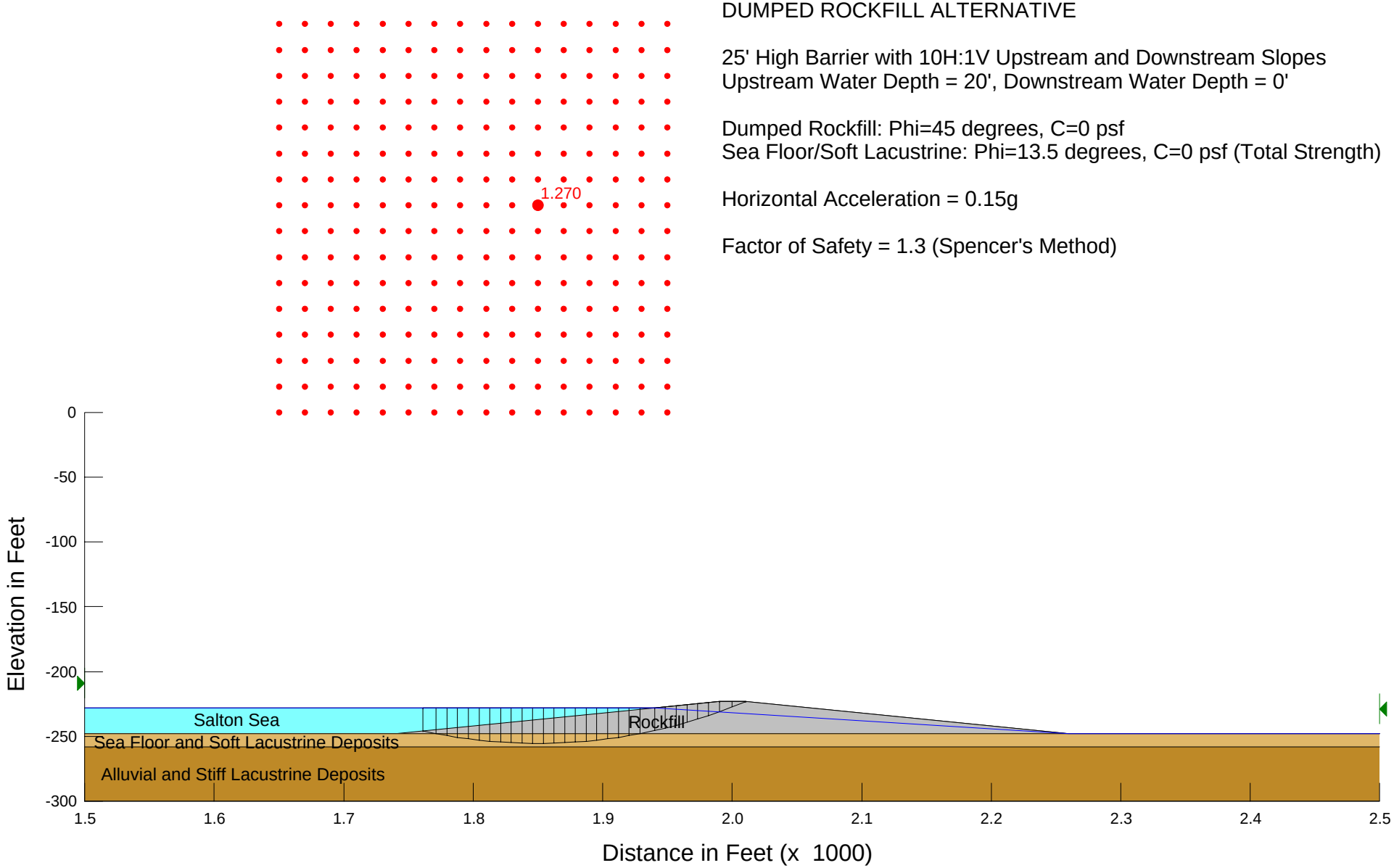
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 1.3 (Spencer's Method)



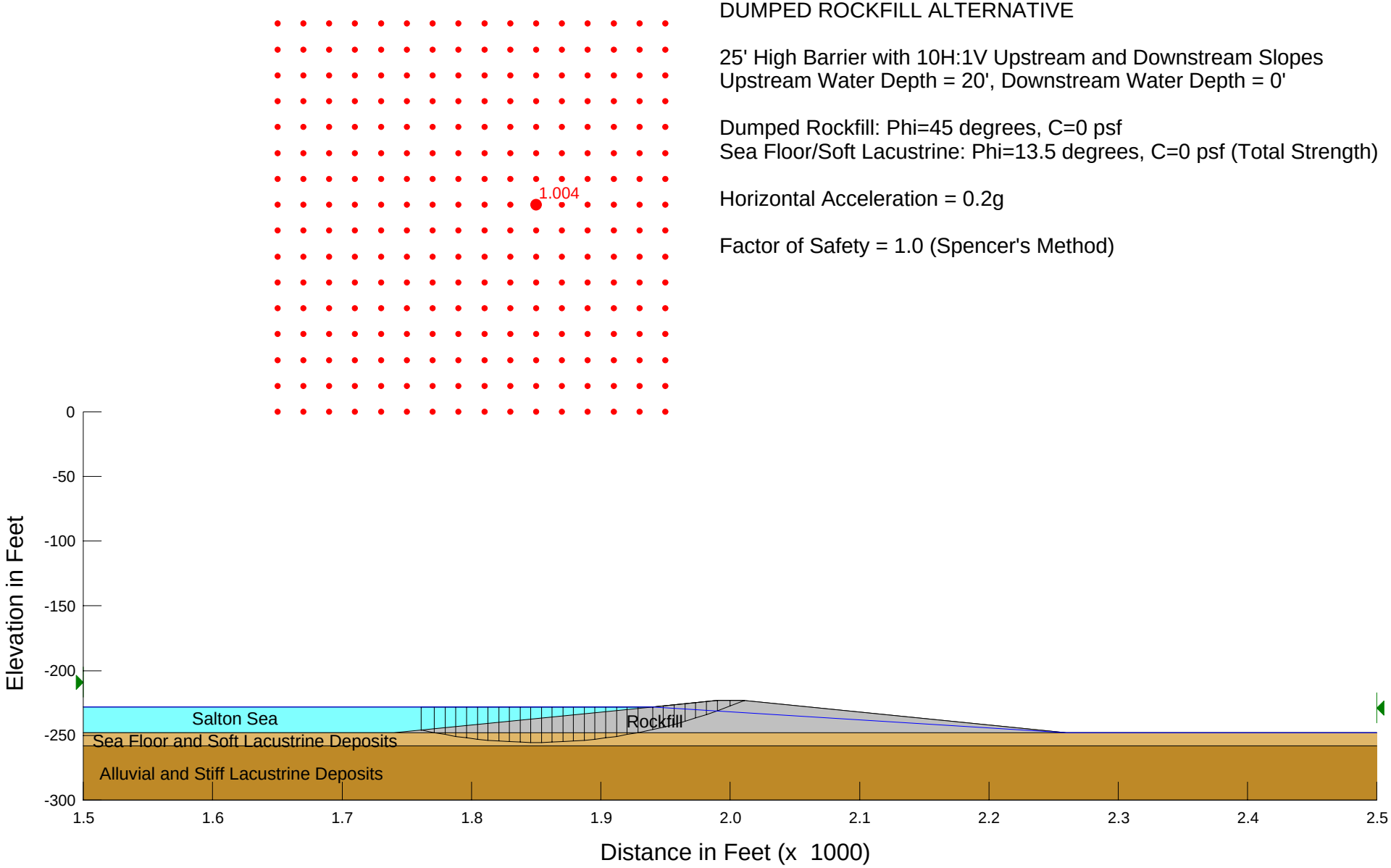
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

25' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 20', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.2g

Factor of Safety = 1.0 (Spencer's Method)



LONG-TERM BARRIER STABILITY WITH
STEADY-STATE SEEPAGE CONDITIONS AND EFFECTIVE STRENGTHS
BARRIER SLOPES = 10H:1V
BARRIER HEIGHT = 15 FEET

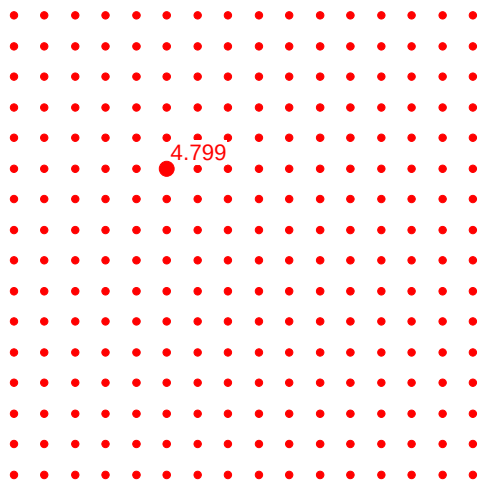
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

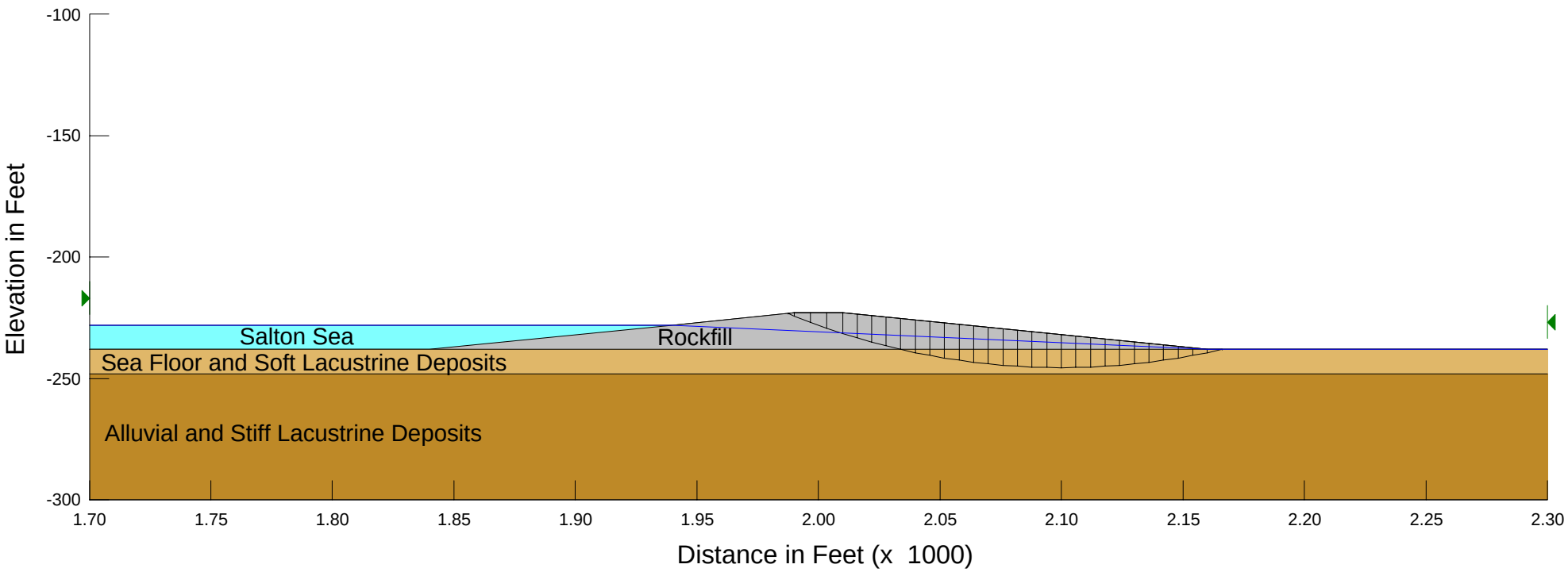
15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29 degrees, C=0 psf (Effective Strength)

Horizontal Acceleration = 0g

Factor of Safety = 4.8 (Spencer's Method)





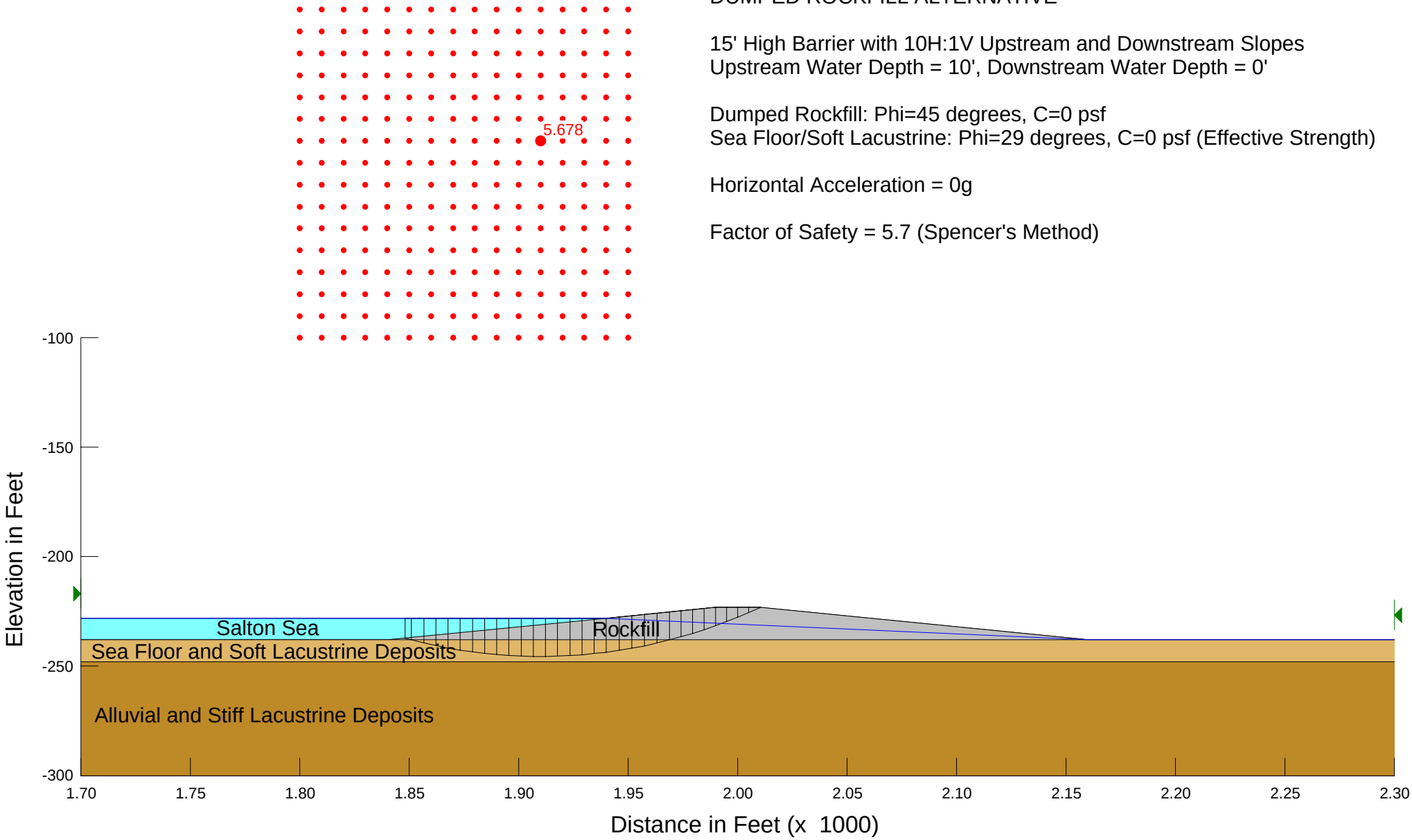
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=29 degrees, C=0 psf (Effective Strength)

Horizontal Acceleration = 0g

Factor of Safety = 5.7 (Spencer's Method)



PSEUDO-DYNAMIC BARRIER STABILITY WITH
HORIZONTAL SEISMIC LOADS AND TOTAL STRENGTHS
BARRIER SLOPES = 10H:1V
BARRIER HEIGHT = 15 FEET

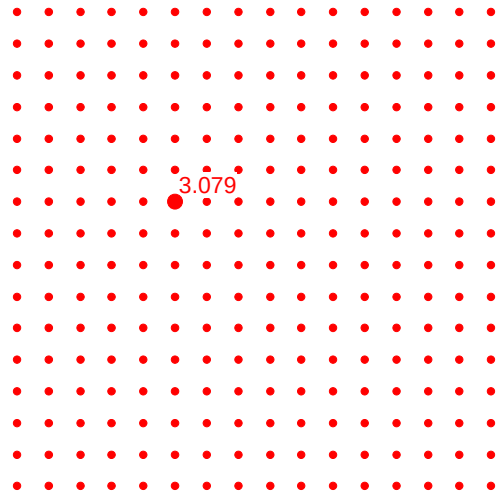
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

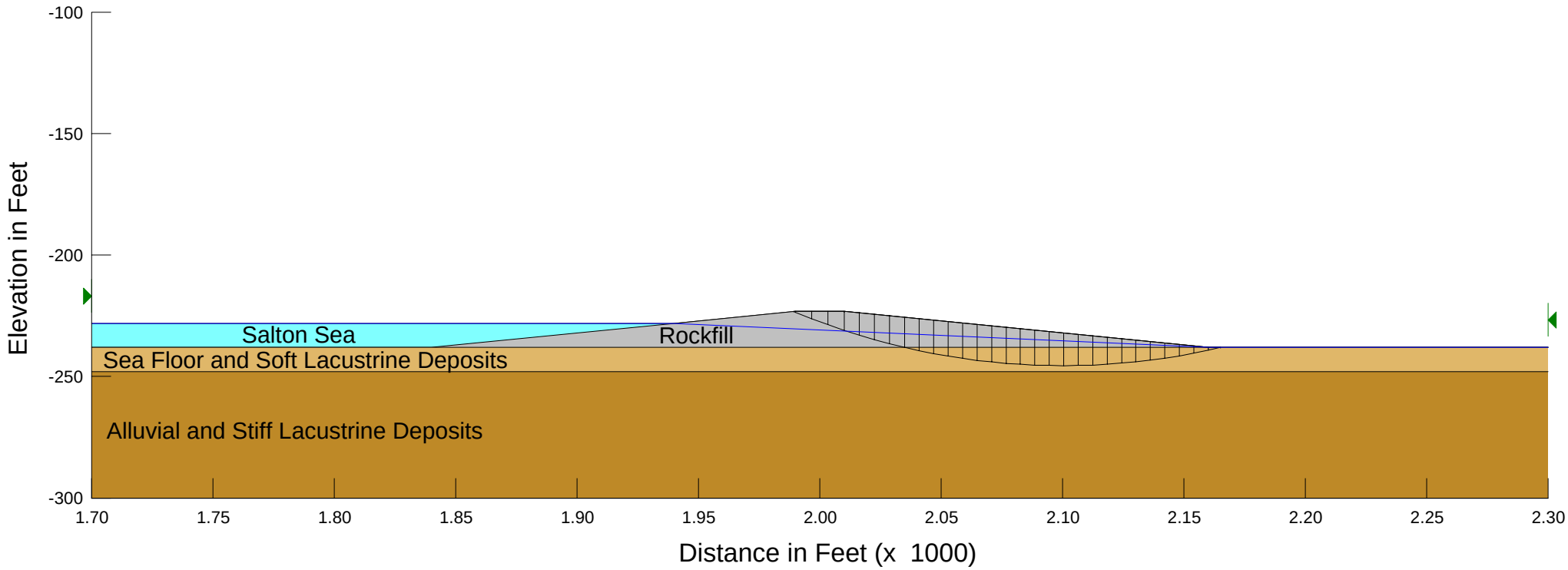
15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 3.1 (Spencer's Method)





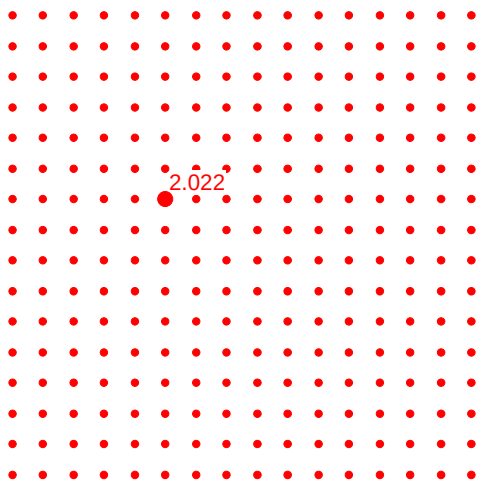
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

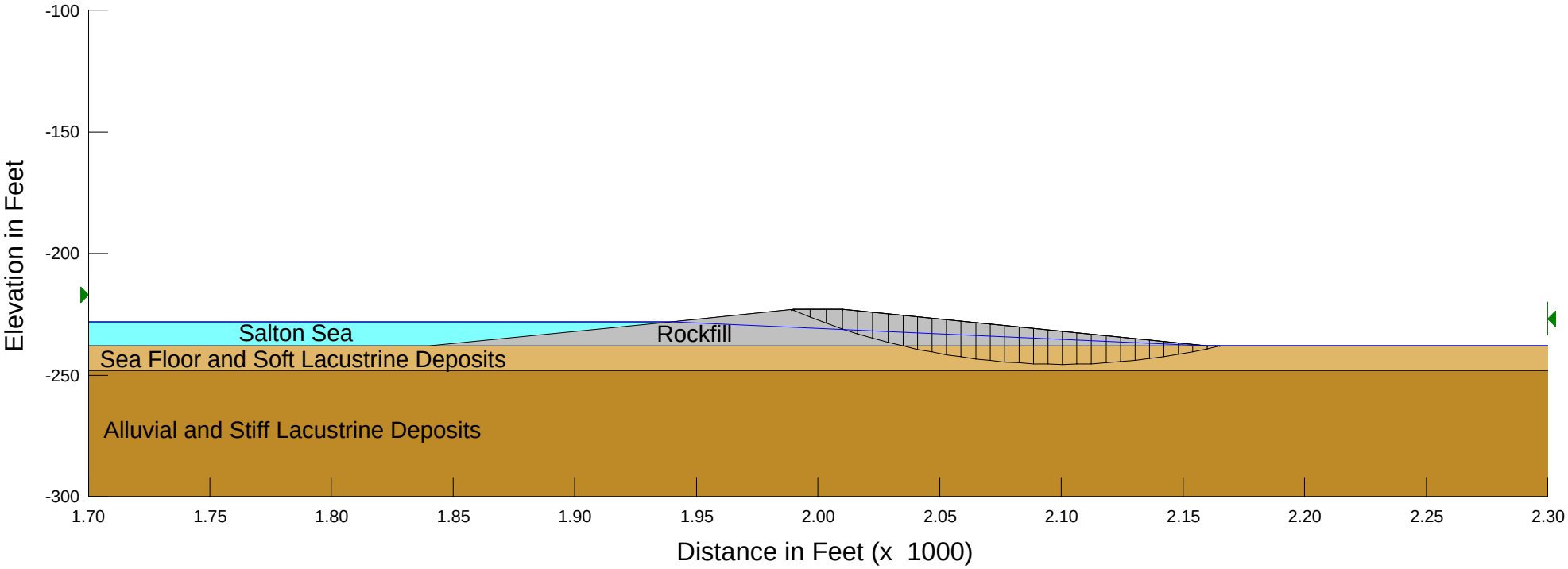
15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 2.0 (Spencer's Method)





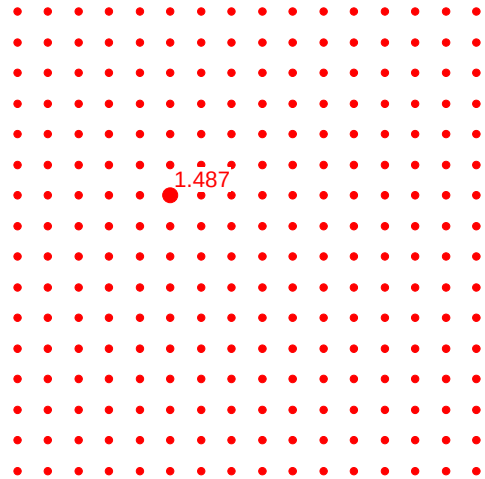
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

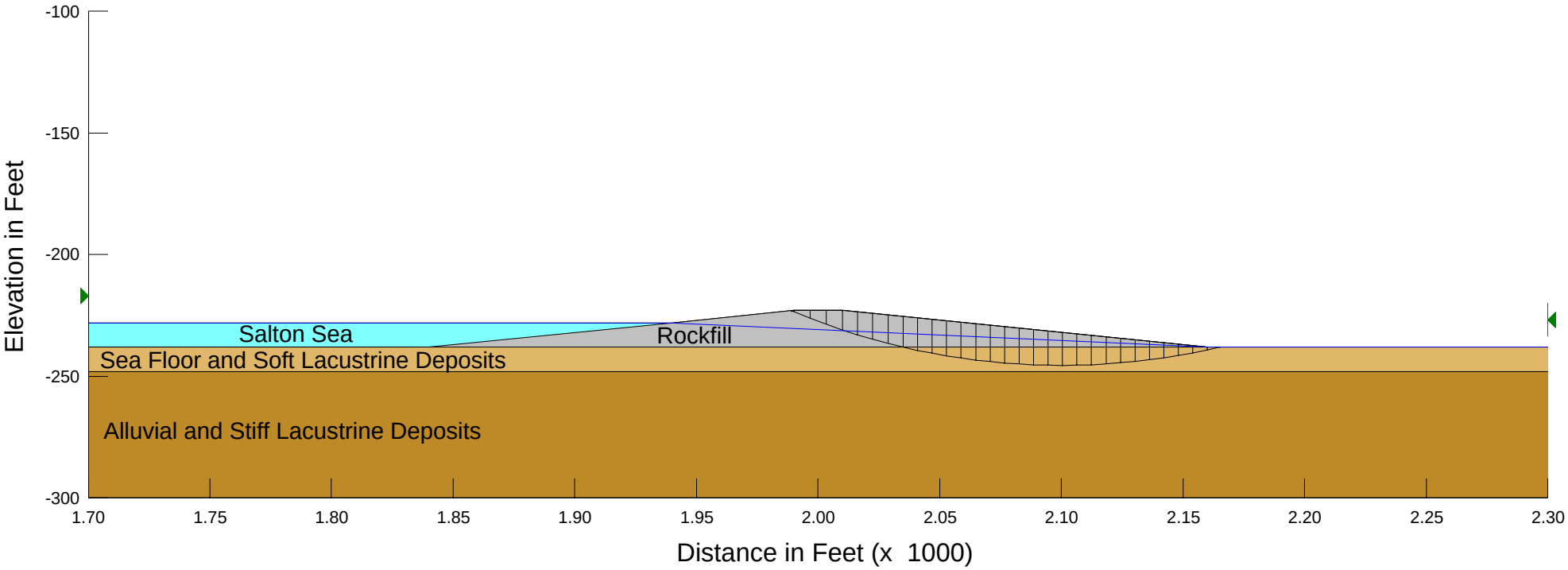
15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 1.5 (Spencer's Method)





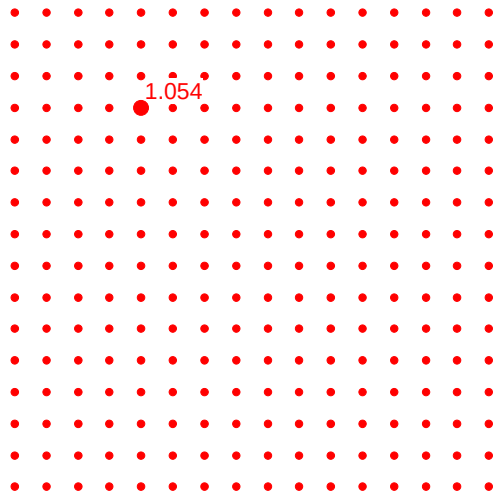
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

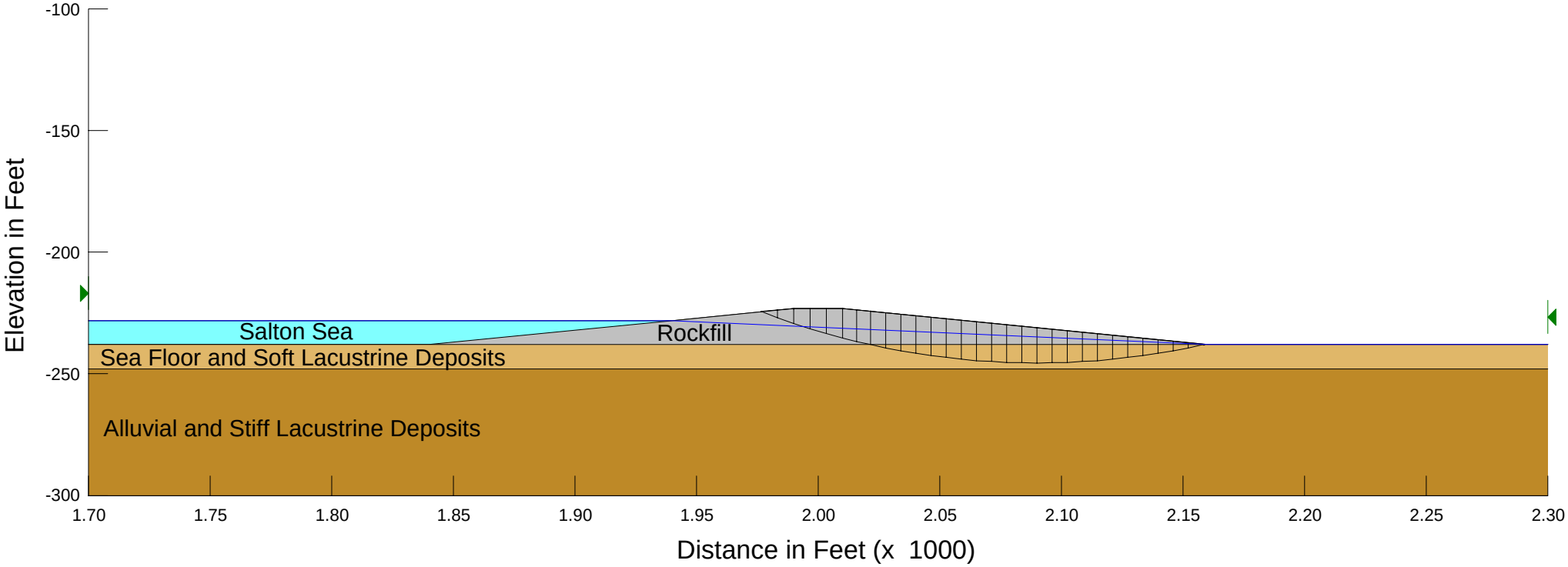
15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 1.1 (Spencer's Method)





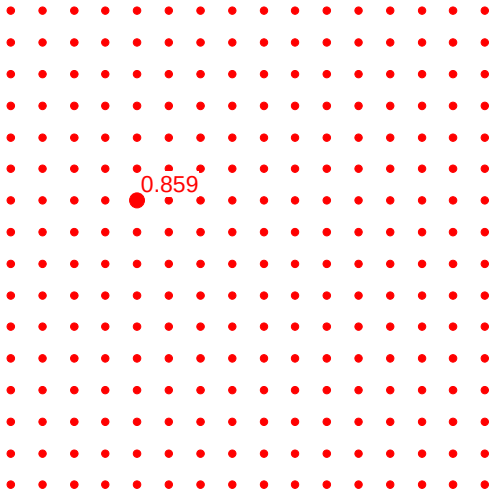
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

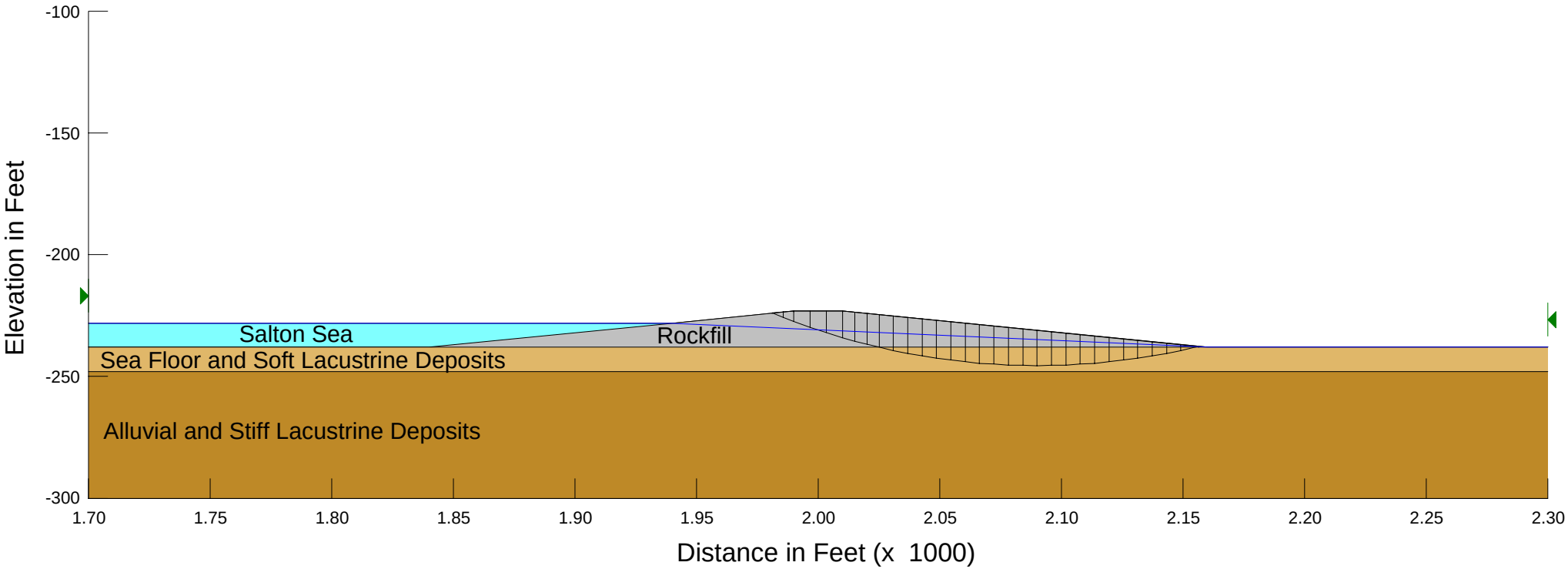
15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.2g

Factor of Safety = 0.9 (Spencer's Method)





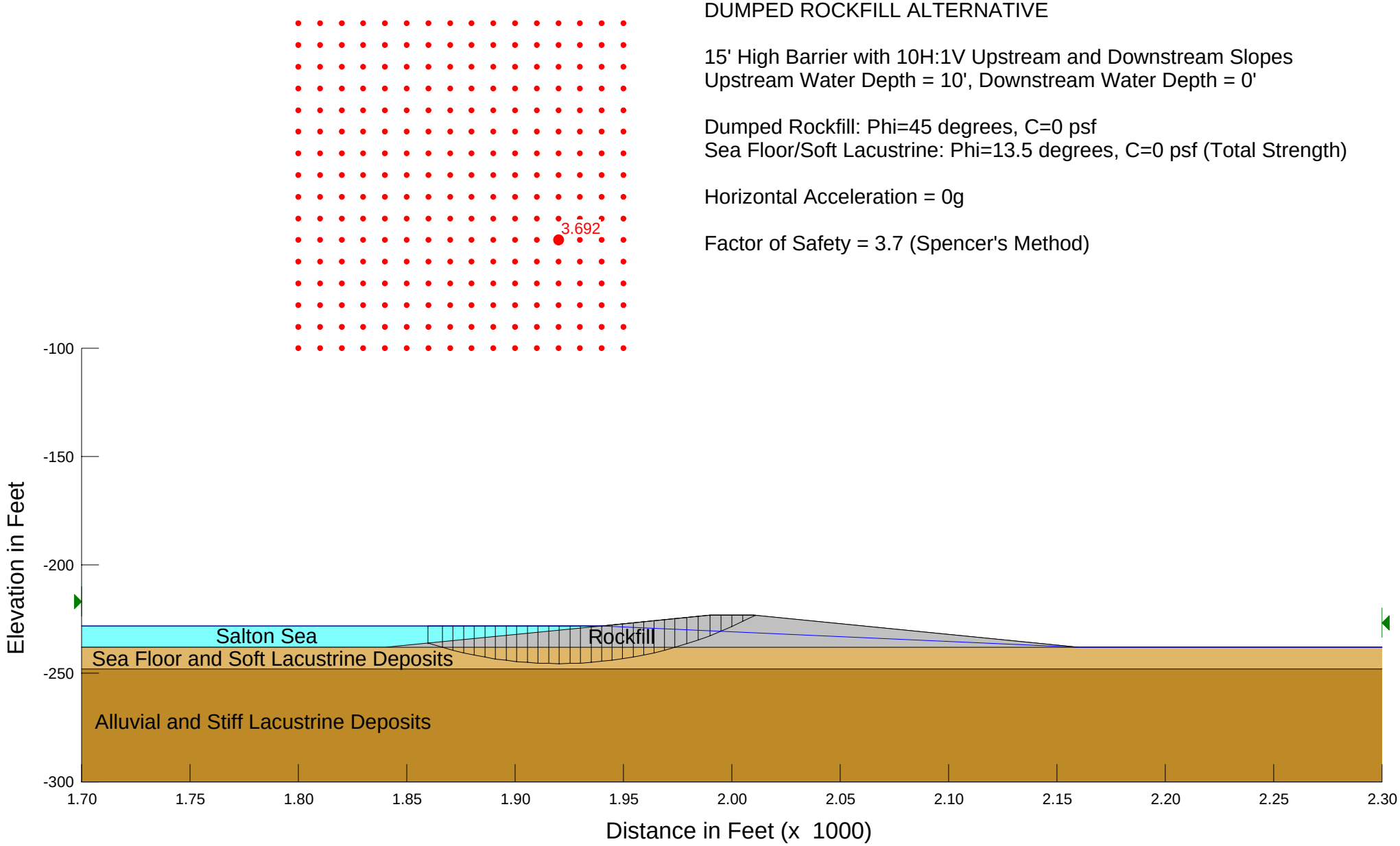
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0g

Factor of Safety = 3.7 (Spencer's Method)



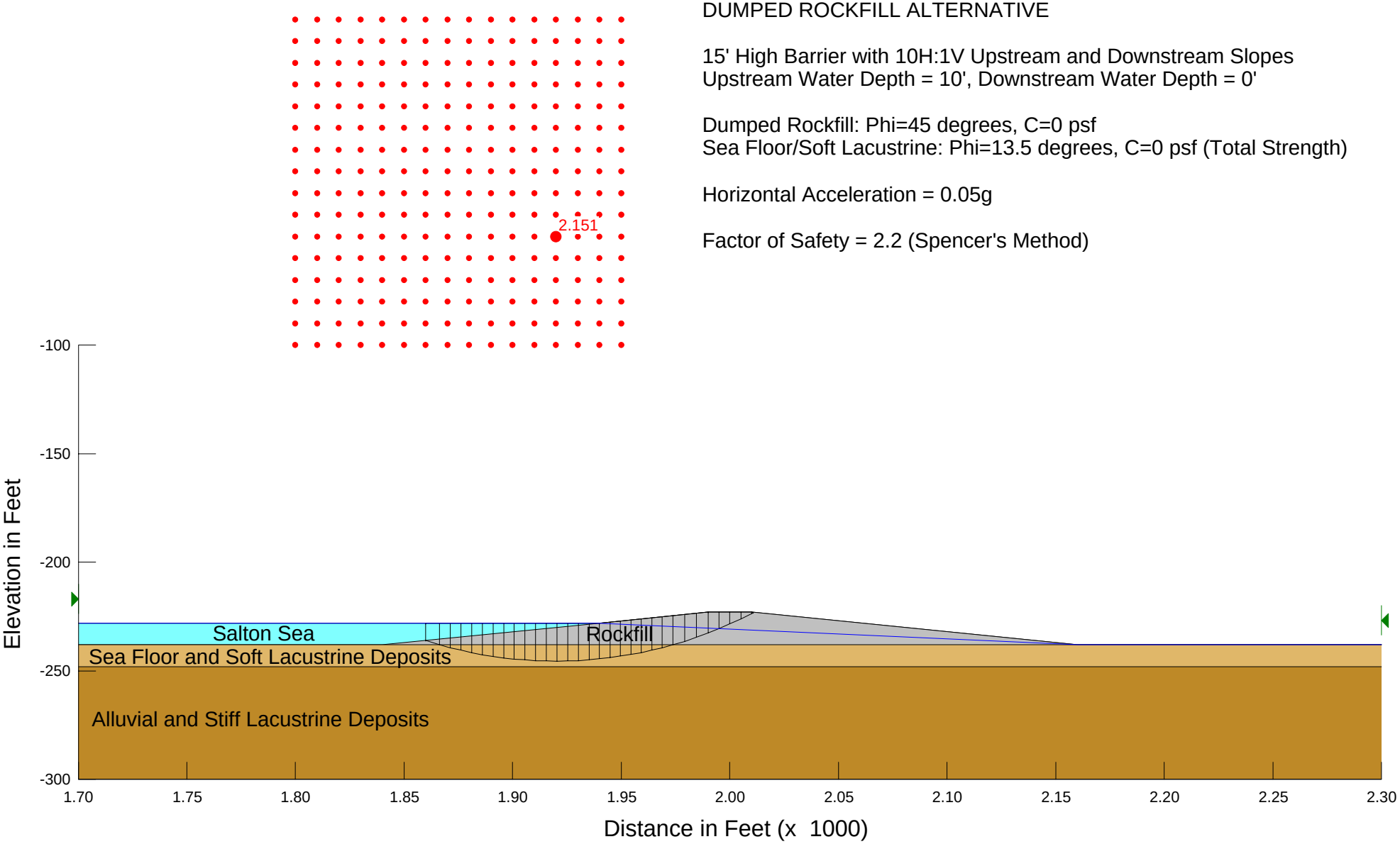
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.05g

Factor of Safety = 2.2 (Spencer's Method)



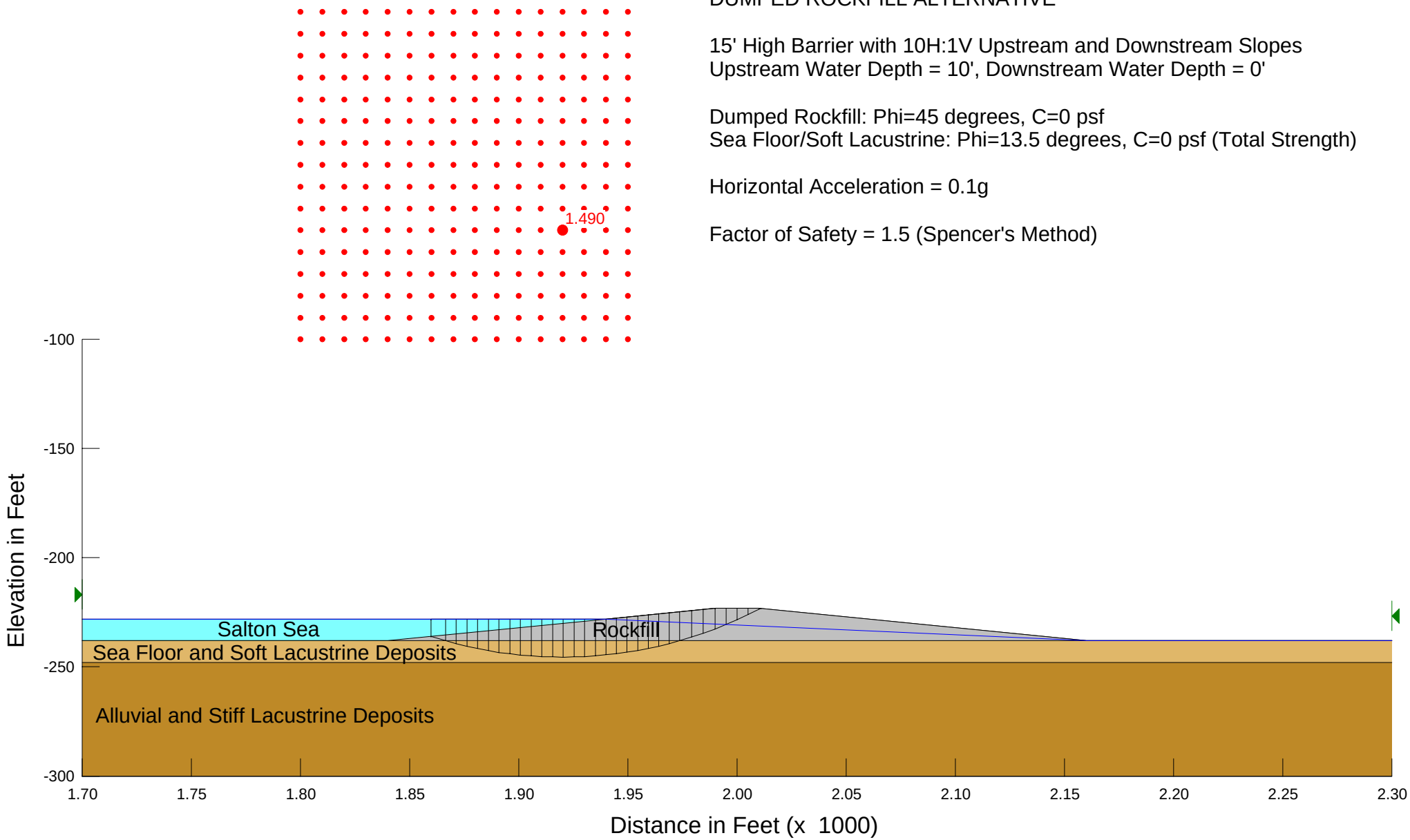
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.1g

Factor of Safety = 1.5 (Spencer's Method)



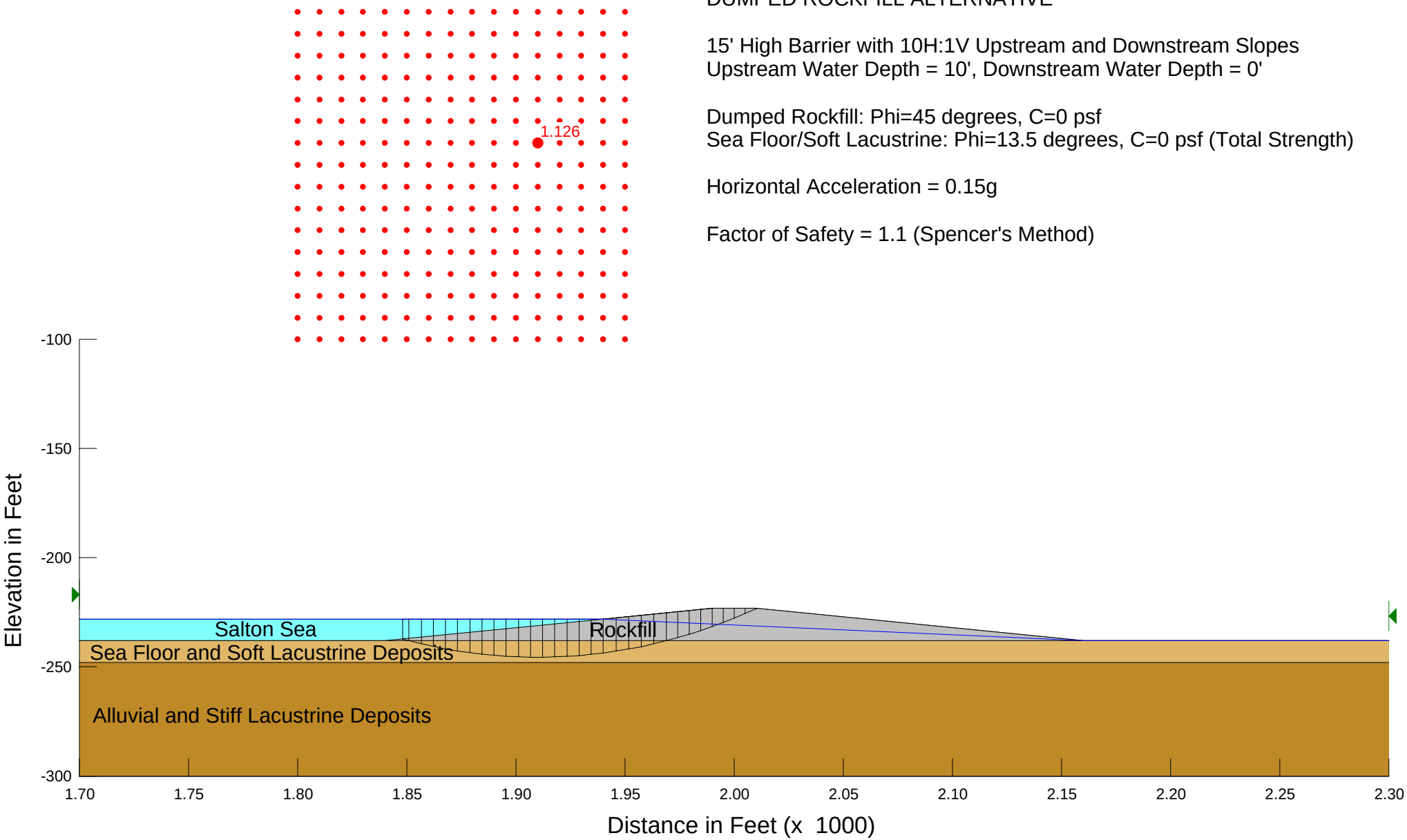
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.15g

Factor of Safety = 1.1 (Spencer's Method)



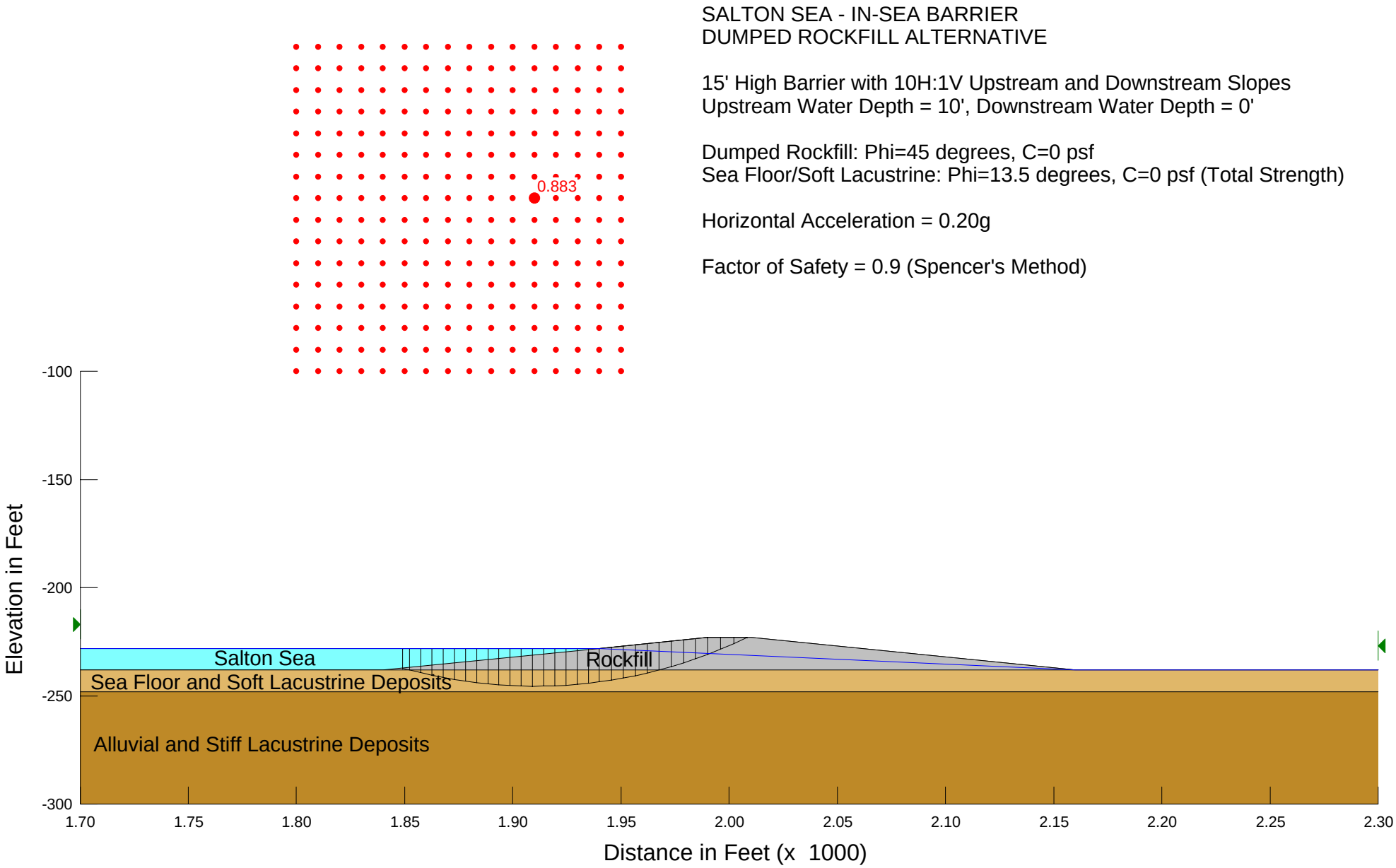
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

15' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Phi=13.5 degrees, C=0 psf (Total Strength)

Horizontal Acceleration = 0.20g

Factor of Safety = 0.9 (Spencer's Method)



PSEUDO-DYNAMIC BARRIER STABILITY WITH
HORIZONTAL SEISMIC LOADS
SEA FLOOR AND SOFT LACUSTRINE UNDRAINED STRENGTH RATIO = 0.29
BARRIER SLOPES = 10H:1V
BARRIER HEIGHT = 45 FEET

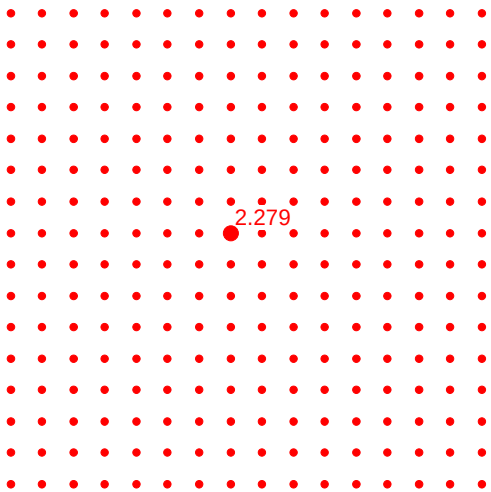
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

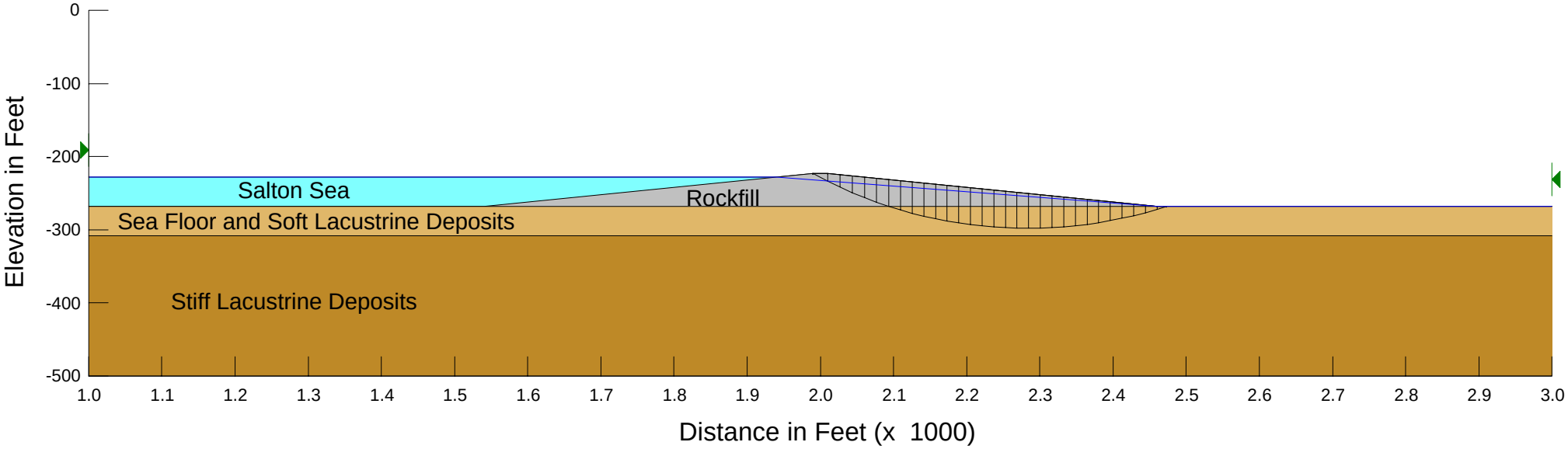
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0g

Factor of Safety = 2.3 (Spencer's Method)





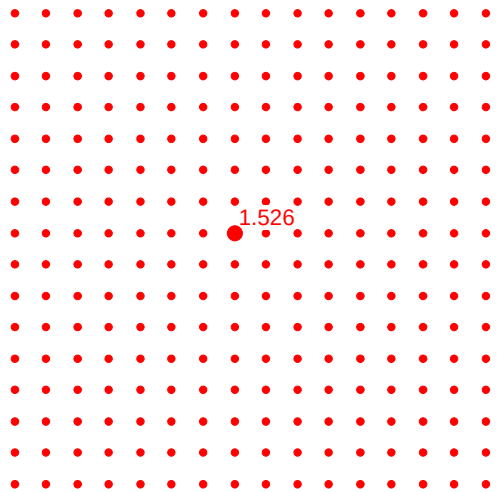
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

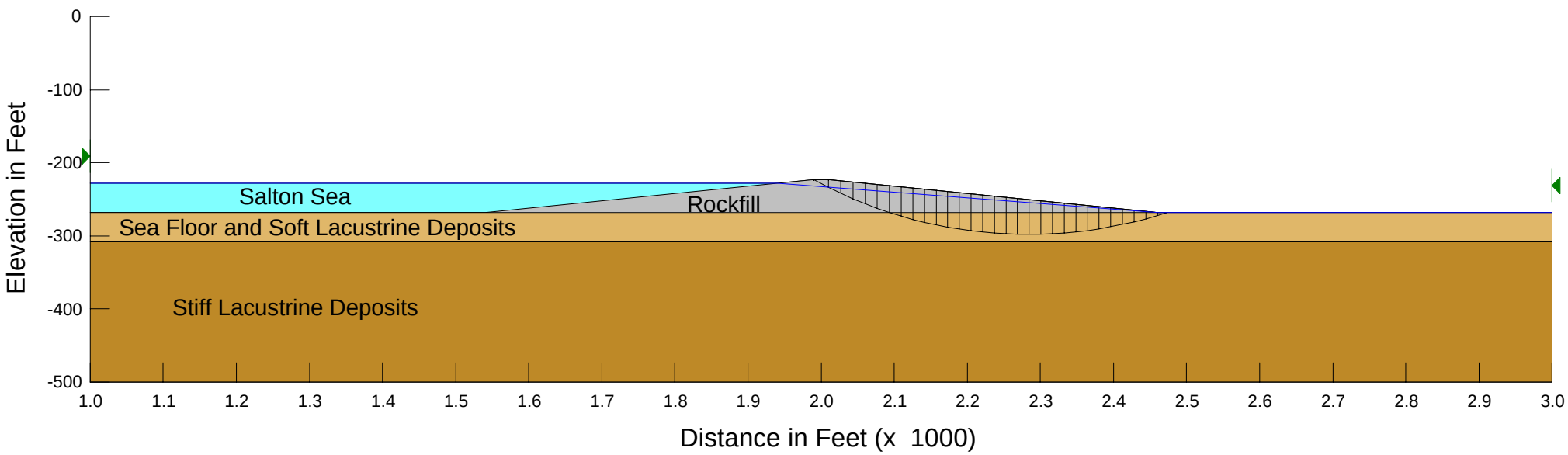
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.05g

Factor of Safety = 1.5 (Spencer's Method)





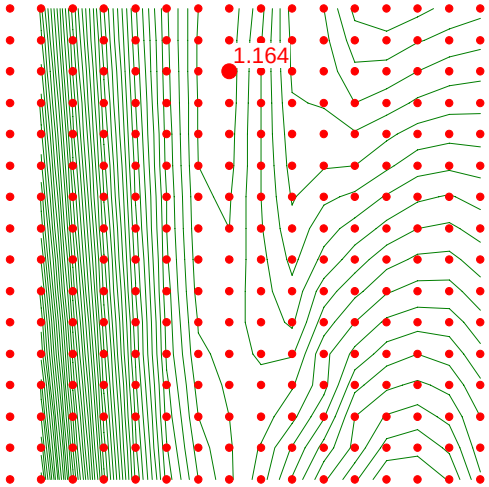
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

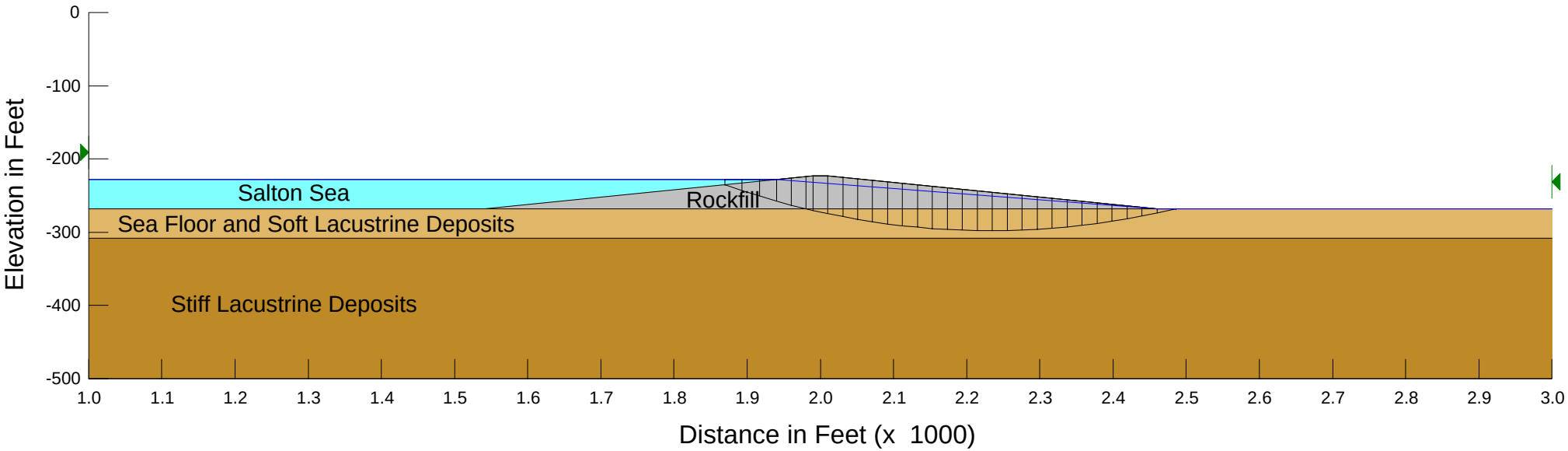
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.1g

Factor of Safety = 1.2 (Spencer's Method)





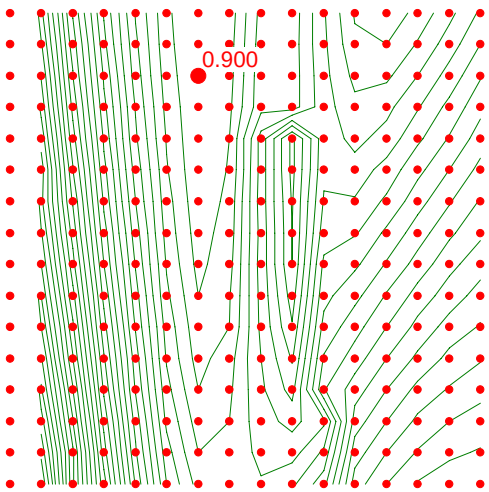
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

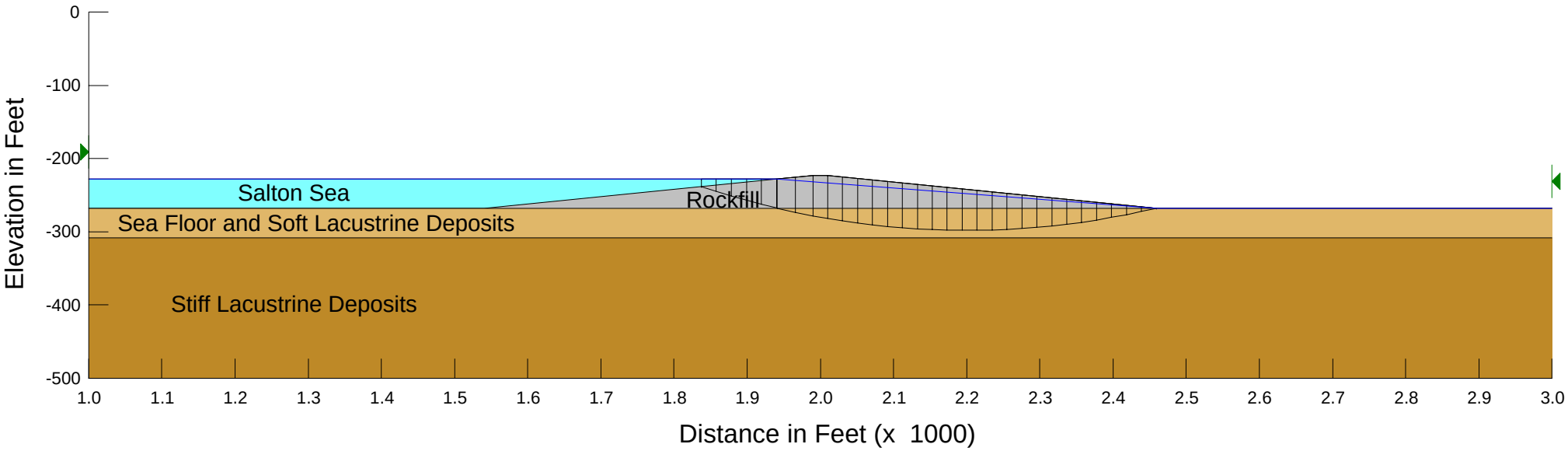
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.15g

Factor of Safety = 0.9 (Spencer's Method)





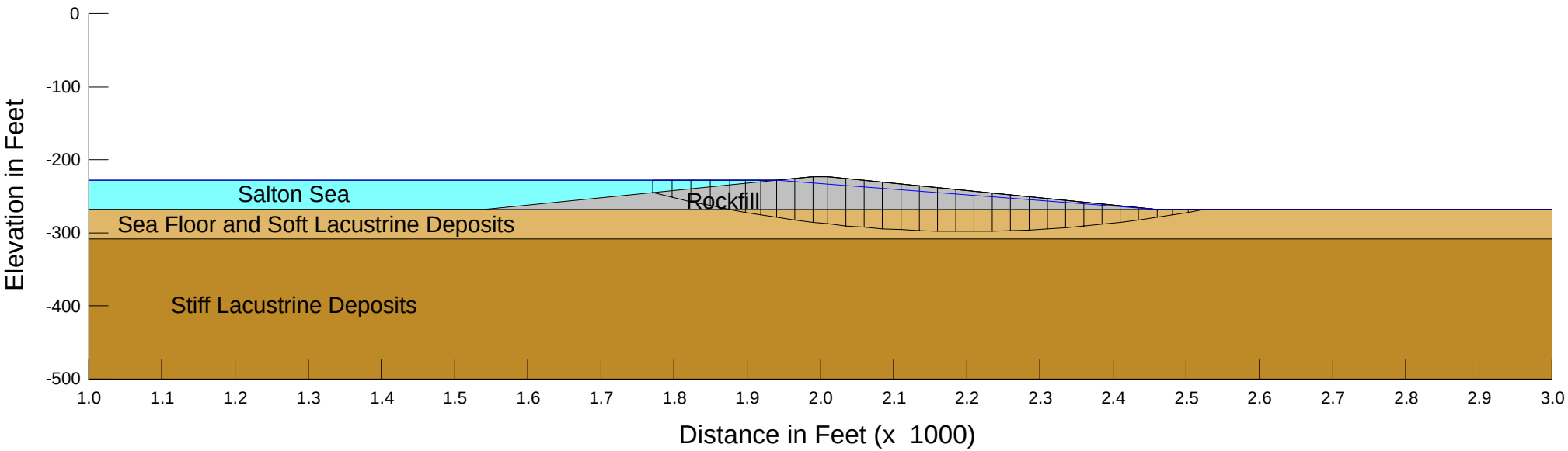
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.2g

Factor of Safety = 0.710 (Spencer's Method)



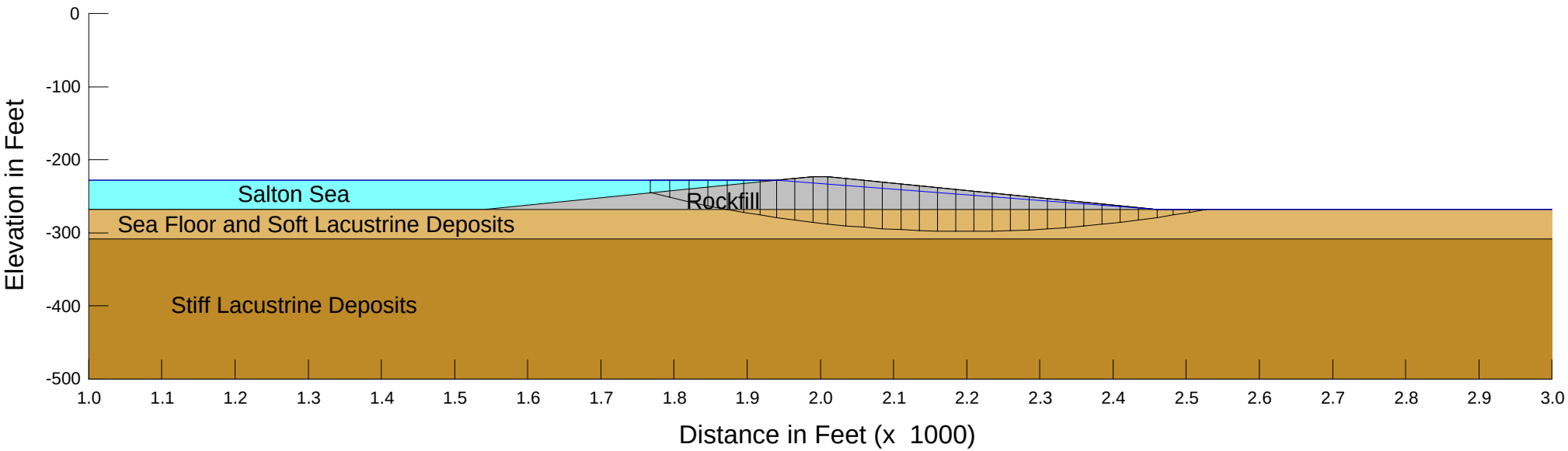
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.25g

Factor of Safety = 0.590 (Spencer's Method)



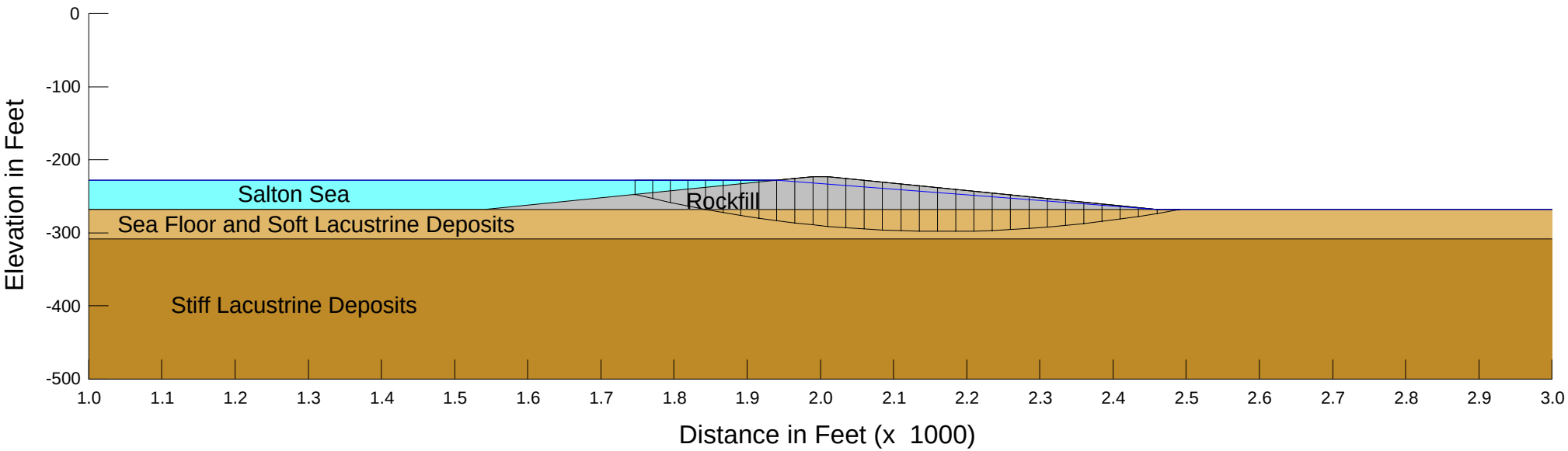
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.3g

Factor of Safety = 0.504 (Spencer's Method)



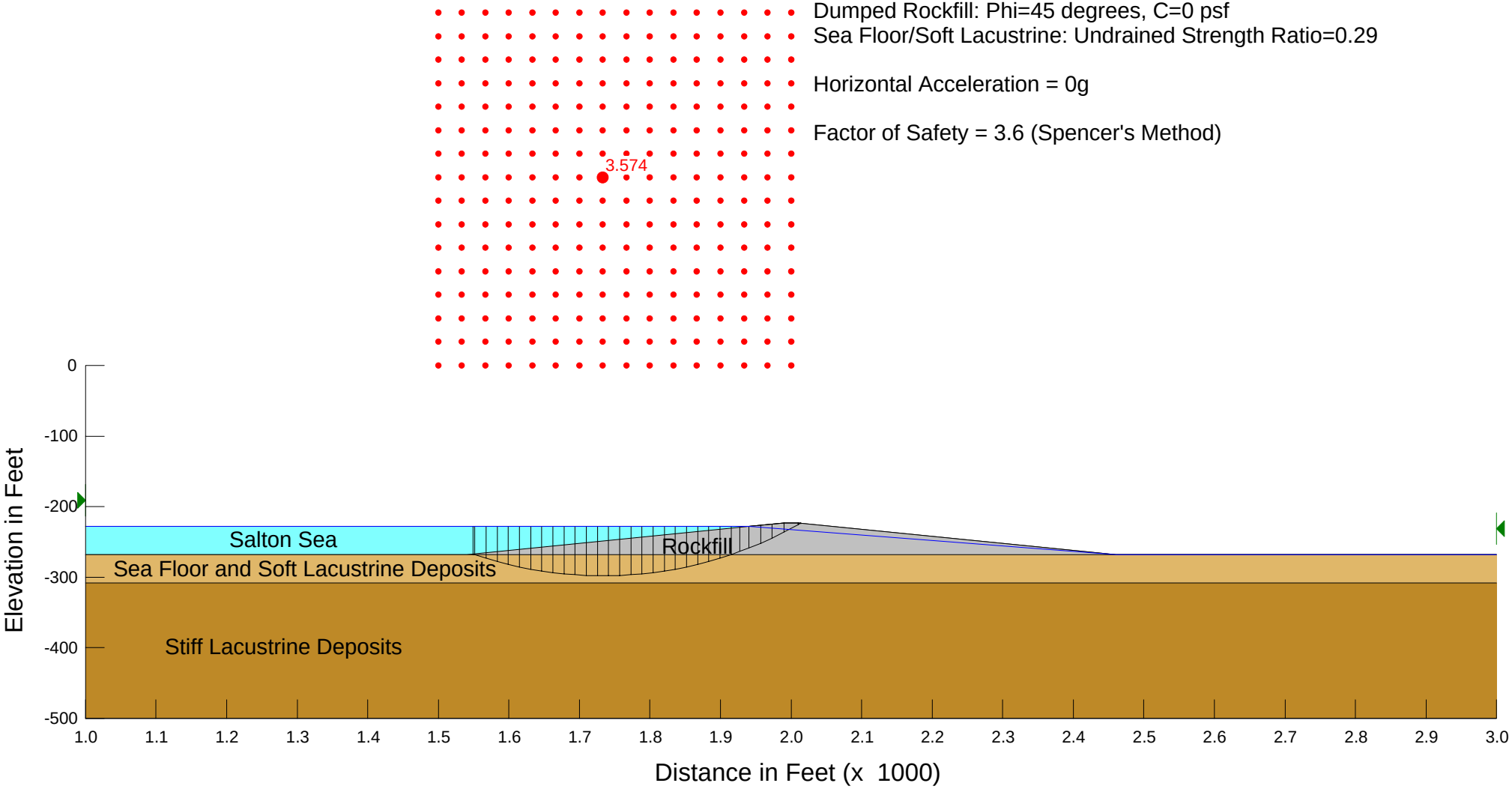
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0g

Factor of Safety = 3.6 (Spencer's Method)



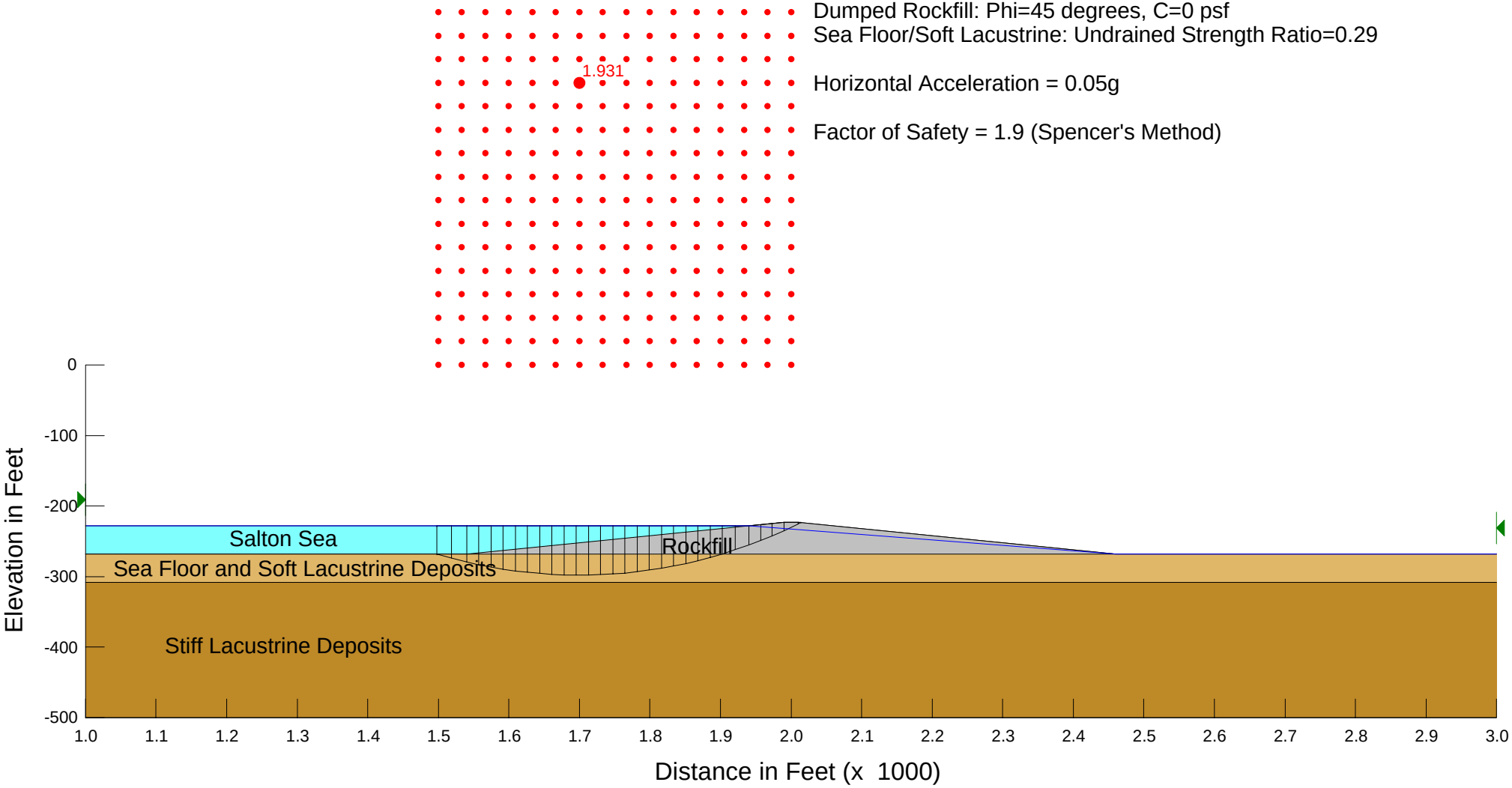
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.05g

Factor of Safety = 1.9 (Spencer's Method)



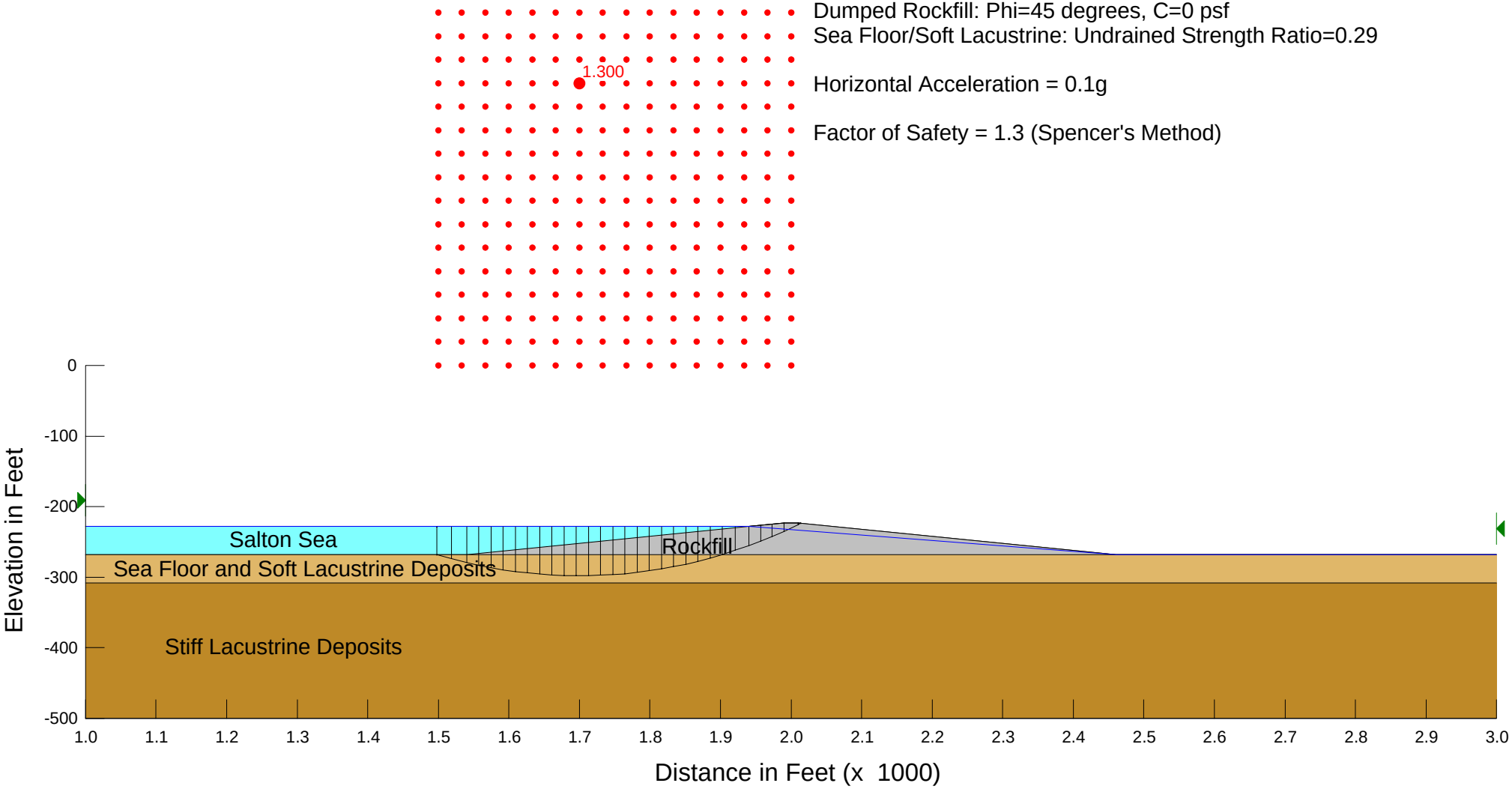
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.1g

Factor of Safety = 1.3 (Spencer's Method)



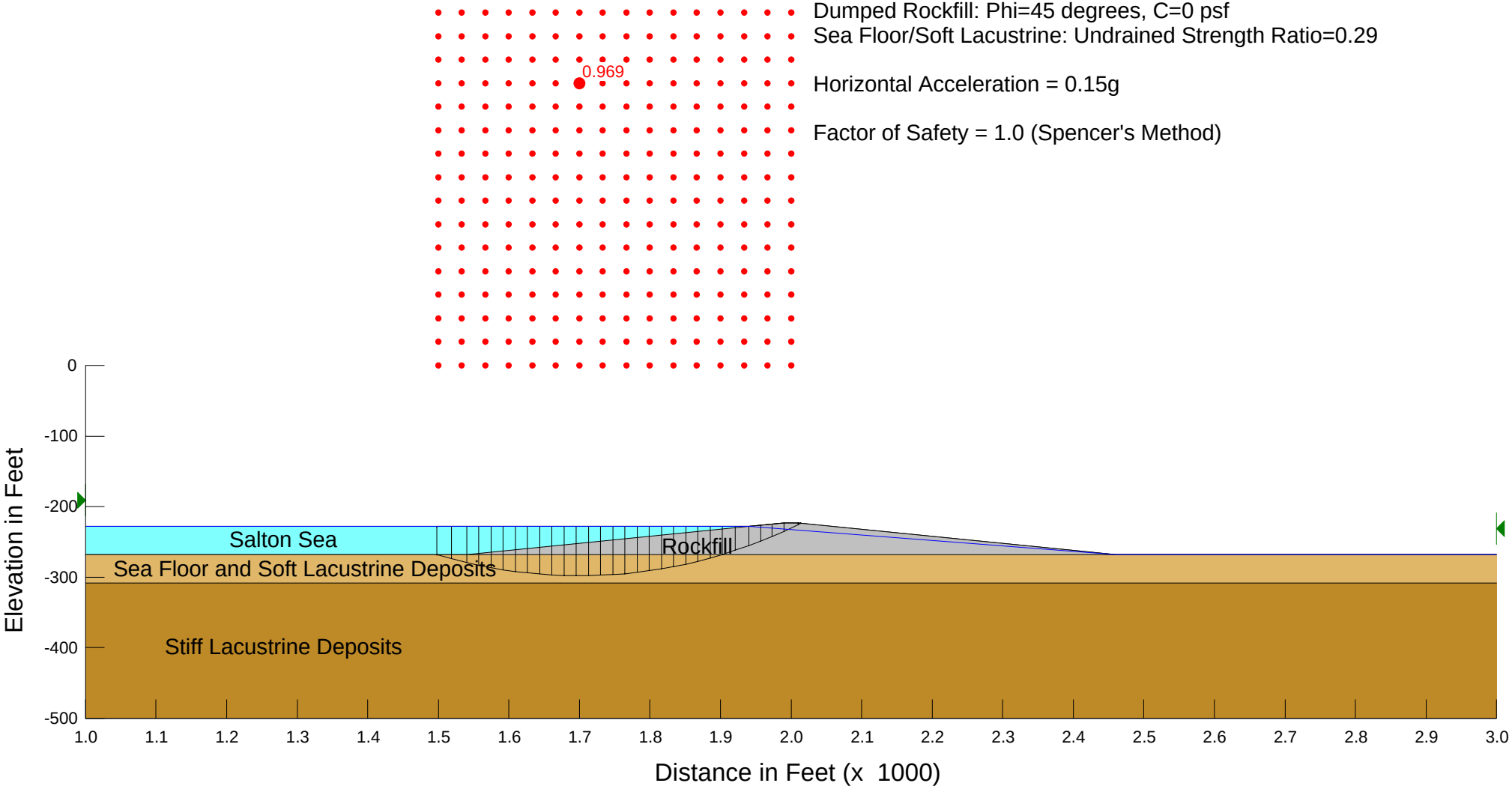
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.15g

Factor of Safety = 1.0 (Spencer's Method)



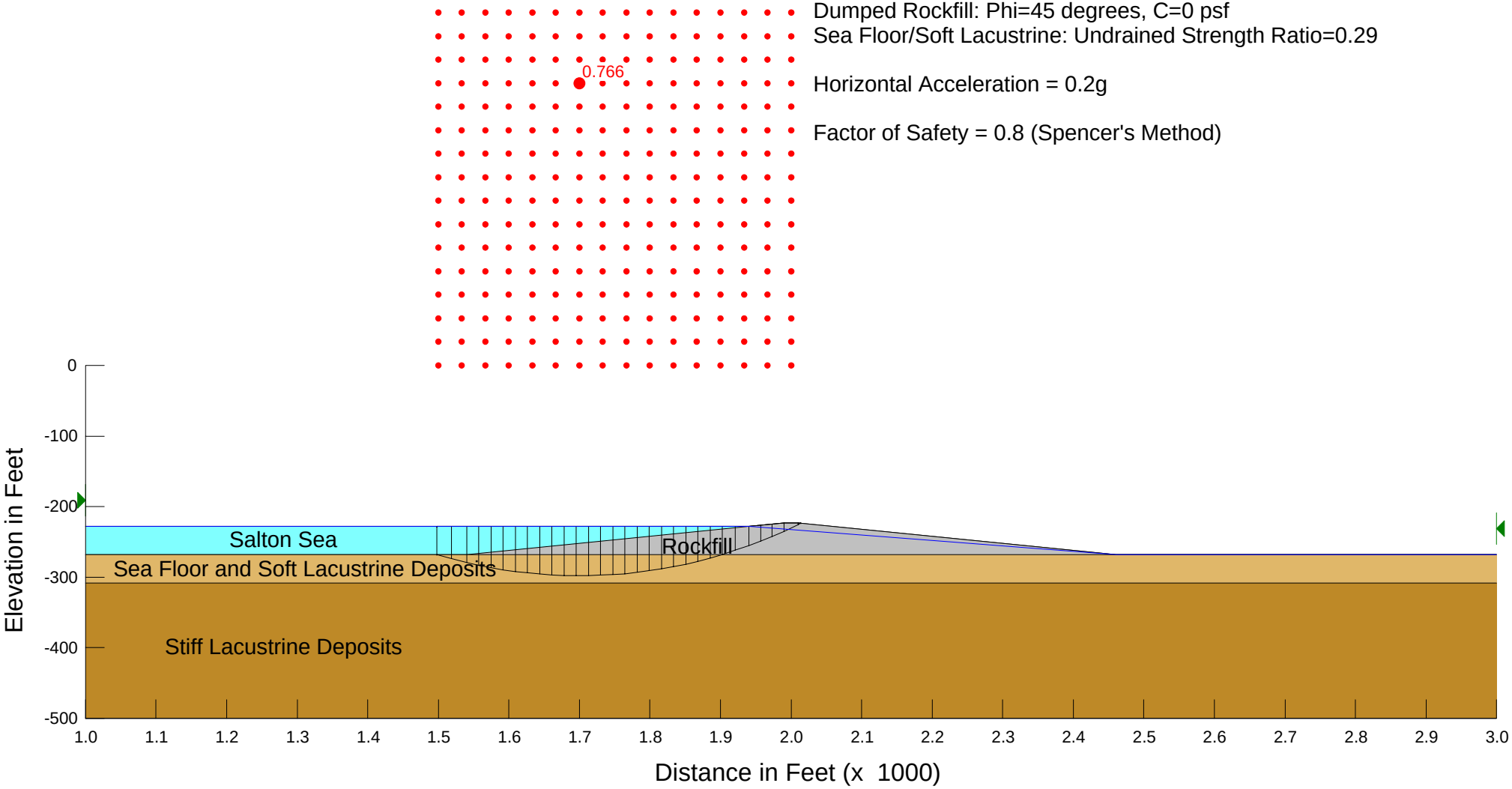
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.2g

Factor of Safety = 0.8 (Spencer's Method)



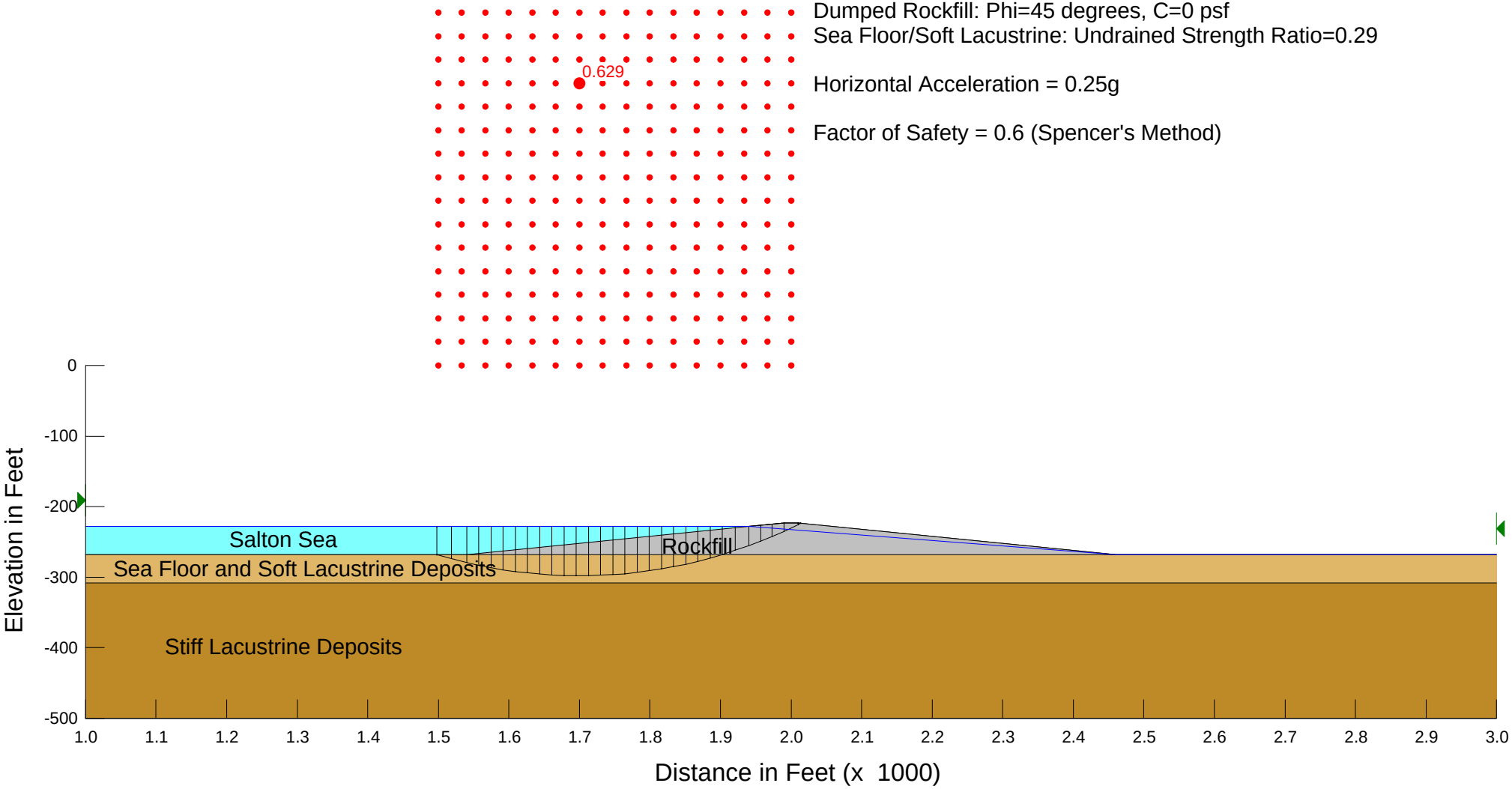
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.25g

Factor of Safety = 0.6 (Spencer's Method)



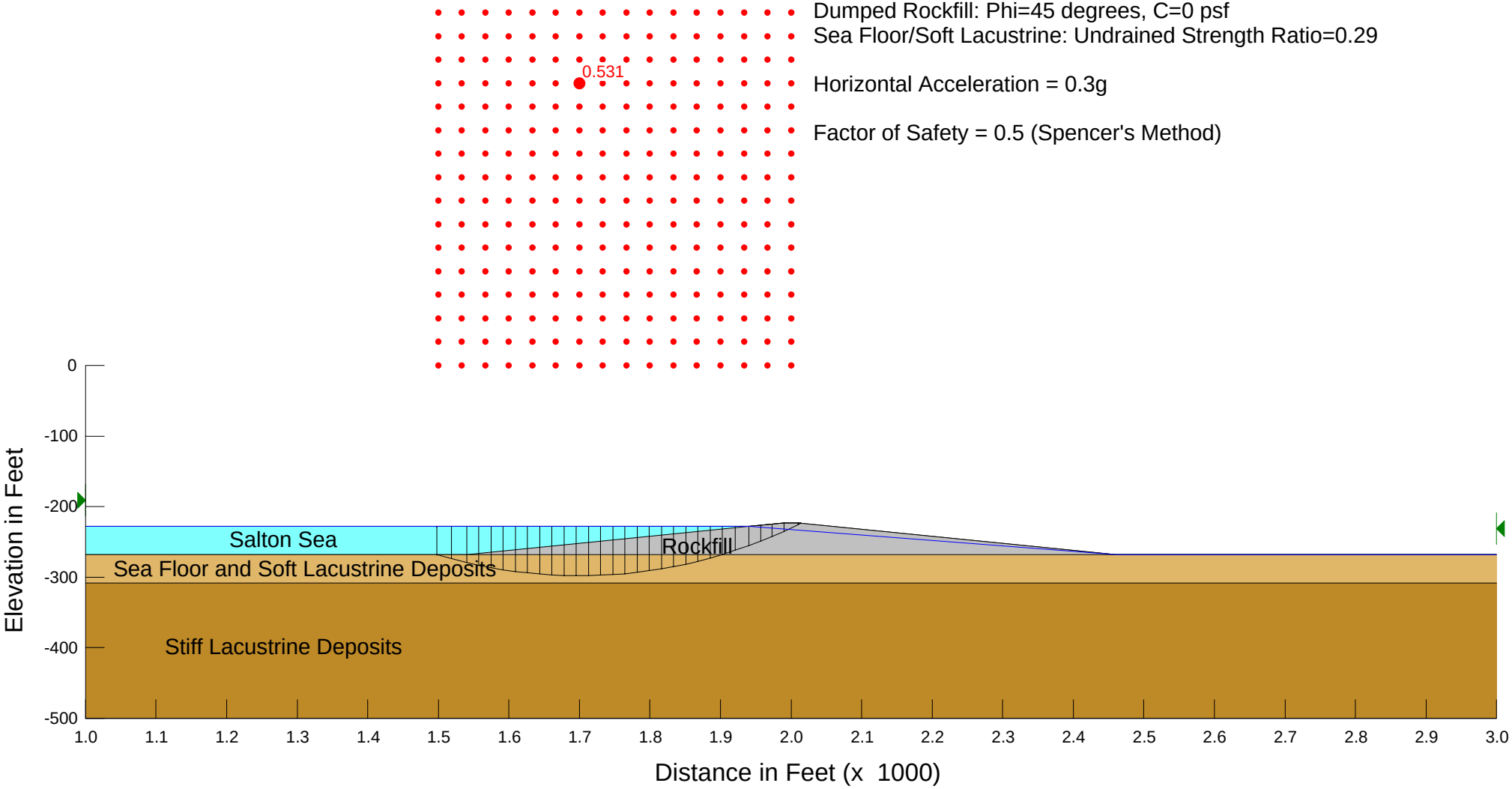
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0.3g

Factor of Safety = 0.5 (Spencer's Method)



PSEUDO-DYNAMIC BARRIER STABILITY WITH
HORIZONTAL SEISMIC LOADS
SEA FLOOR AND SOFT LACUSTRINE UNDRAINED STRENGTH RATIO = 0.6
BARRIER SLOPES = 10H:1V
BARRIER HEIGHT = 45 FEET

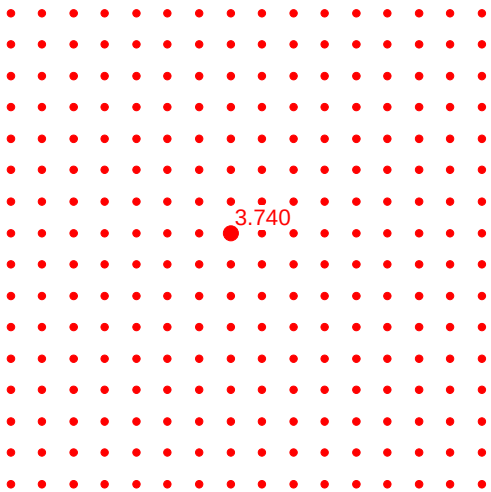
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

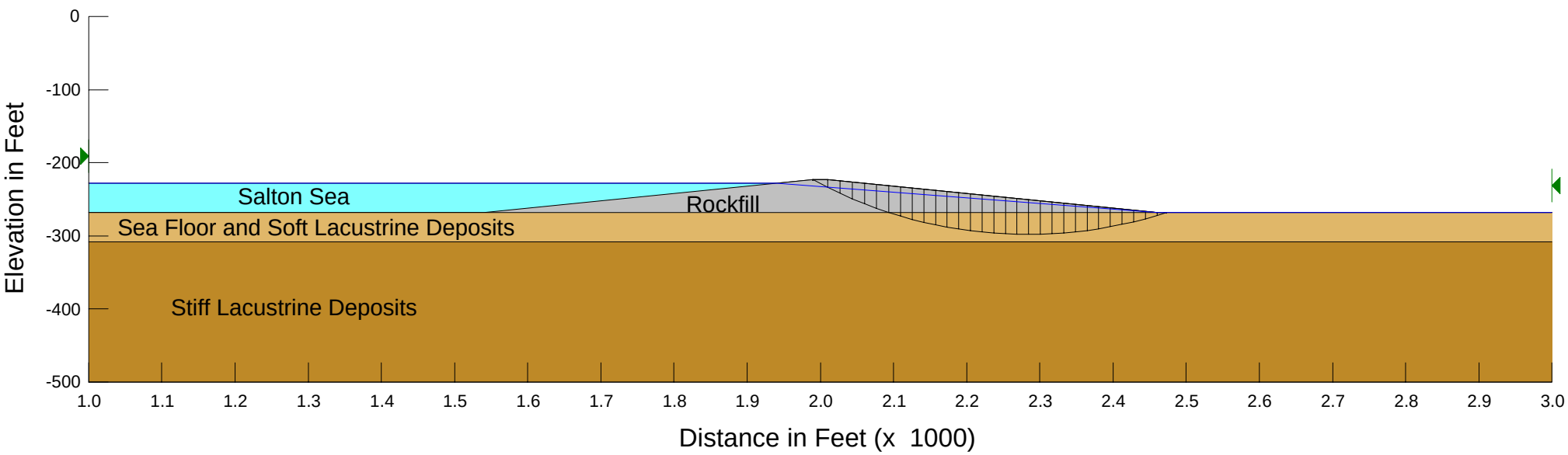
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0g

Factor of Safety = 3.7 (Spencer's Method)





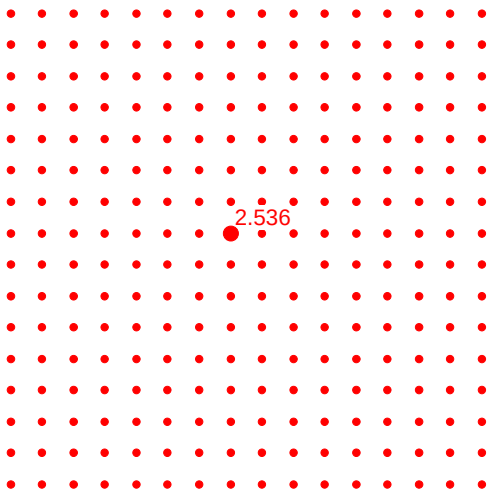
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

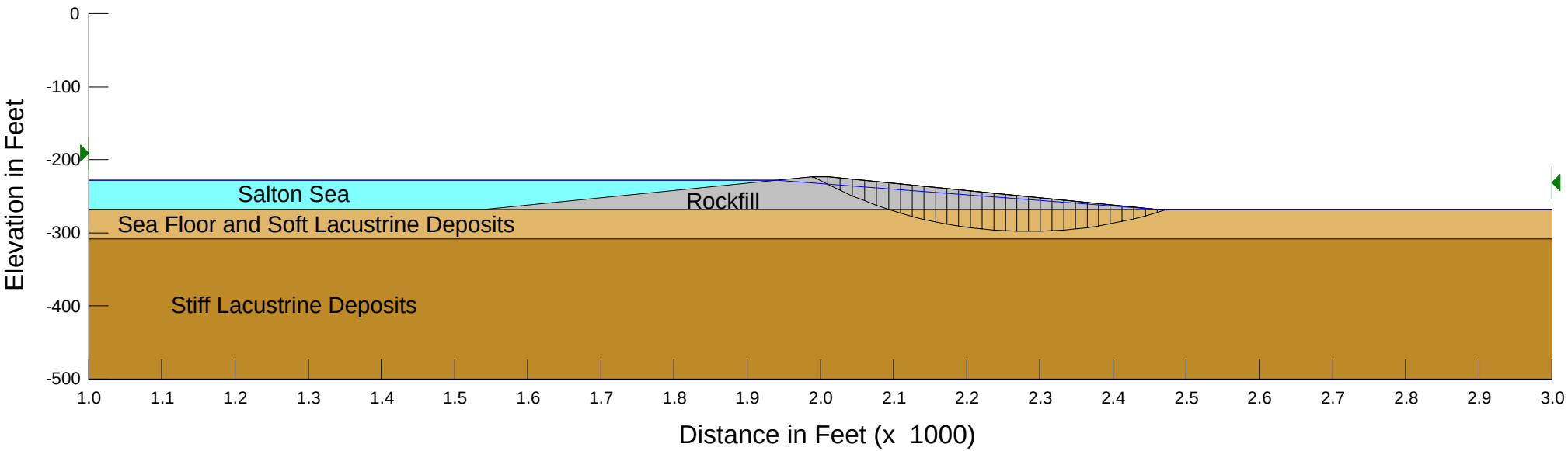
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.05g

Factor of Safety = 2.5 (Spencer's Method)





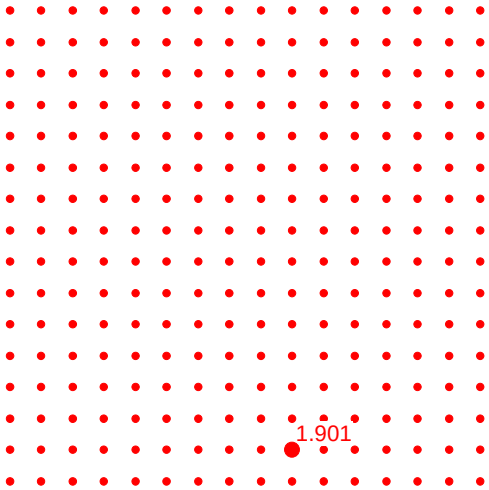
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

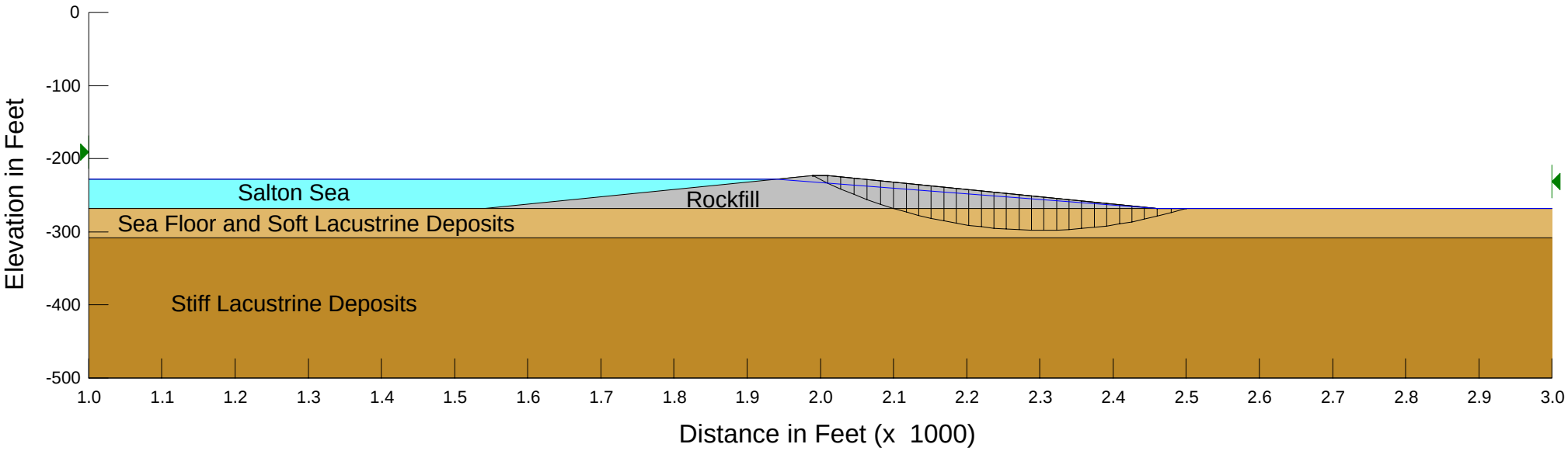
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.1g

Factor of Safety = 1.9 (Spencer's Method)





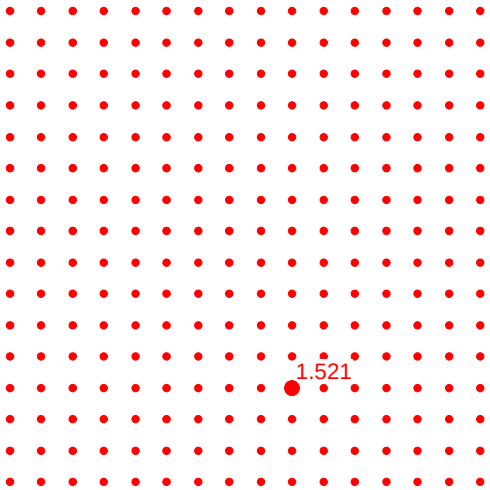
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

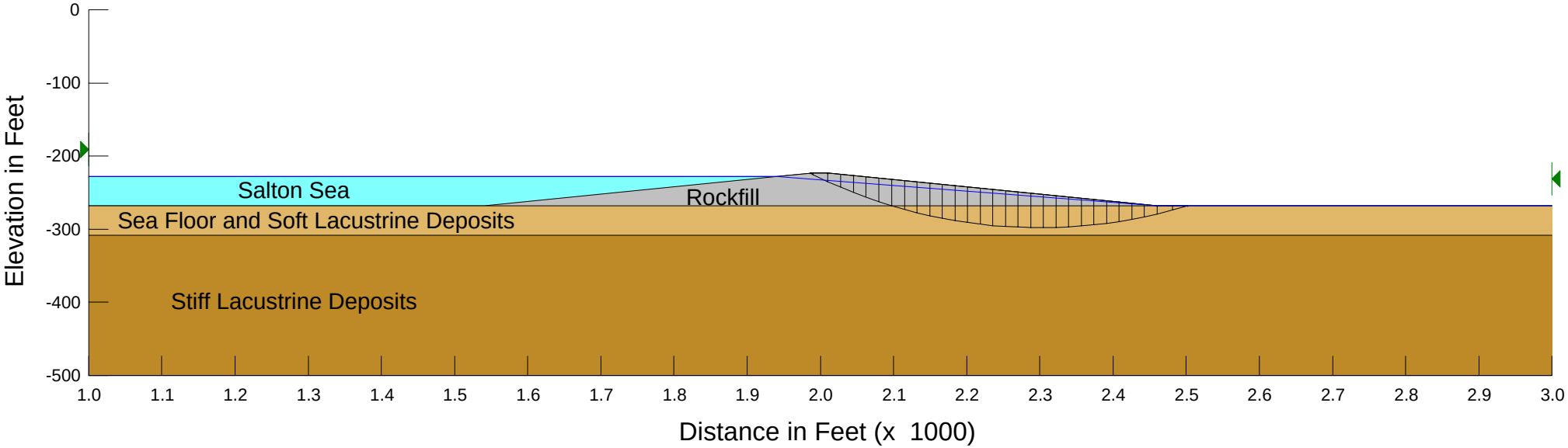
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.15g

Factor of Safety = 1.5 (Spencer's Method)





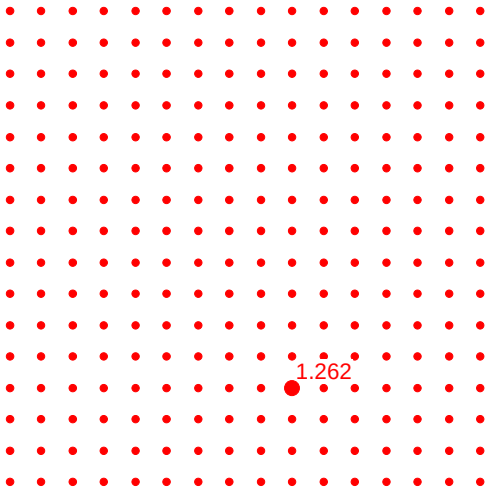
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

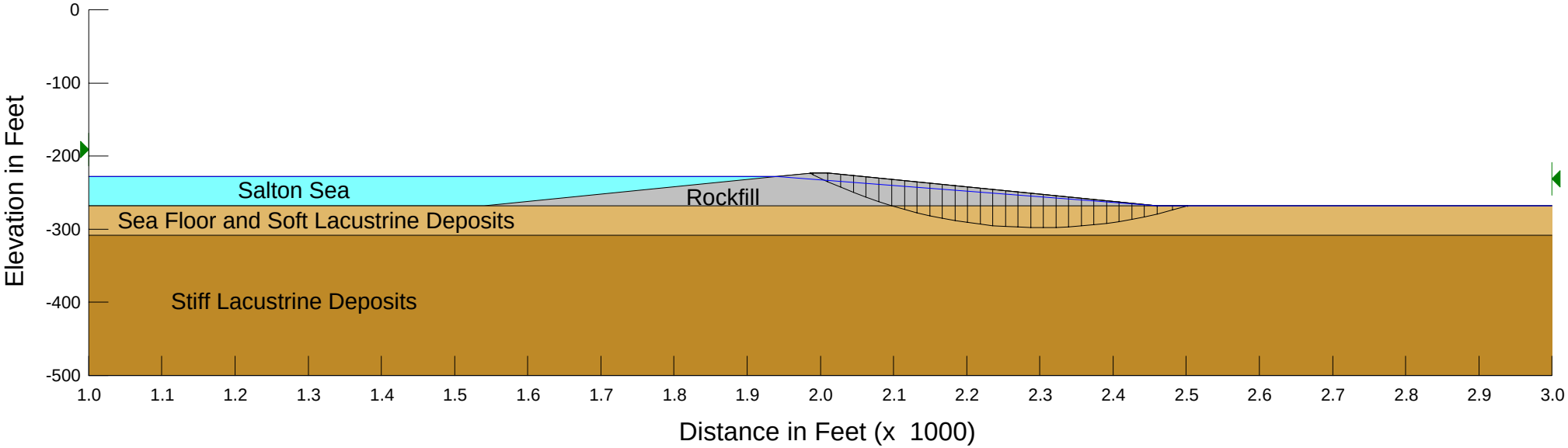
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.2g

Factor of Safety = 1.3 (Spencer's Method)





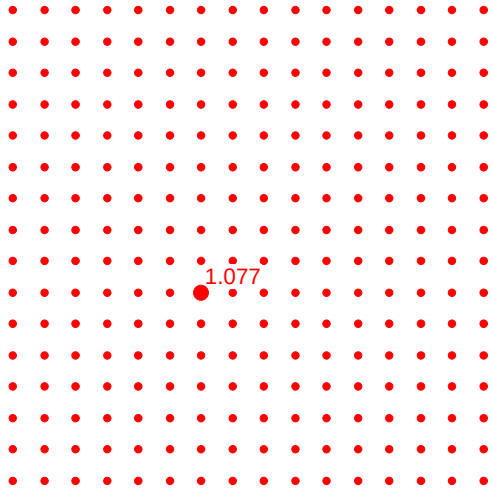
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

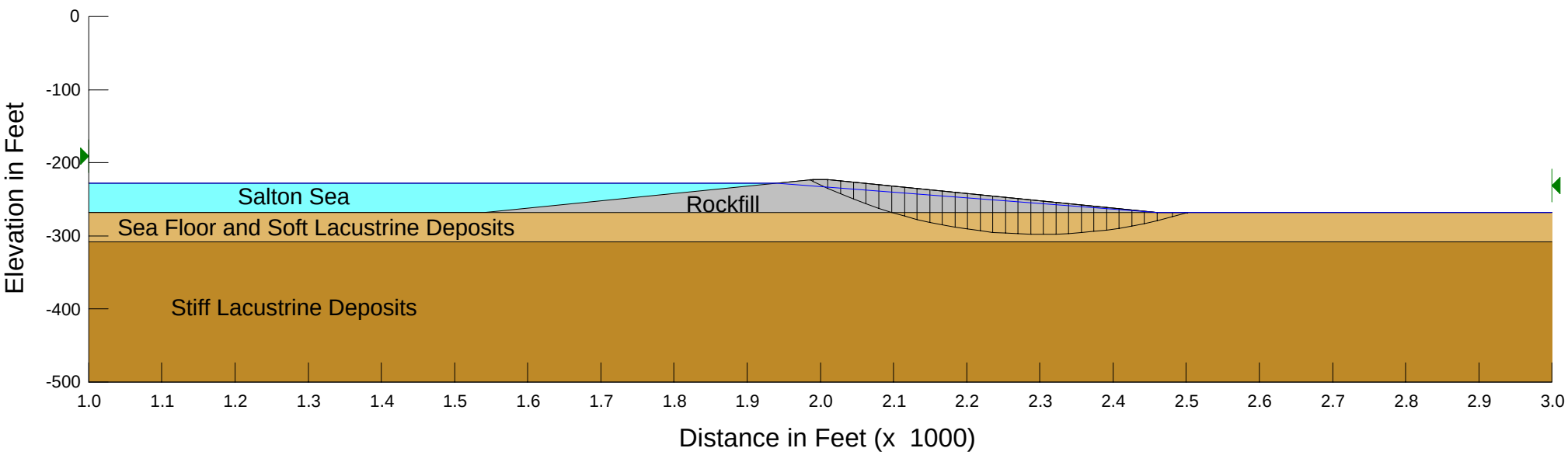
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.25g

Factor of Safety = 1.1 (Spencer's Method)





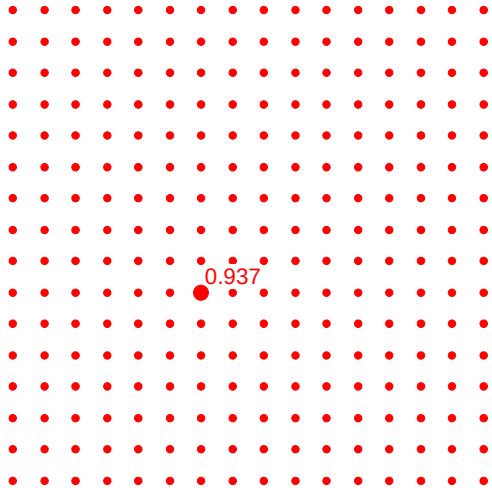
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

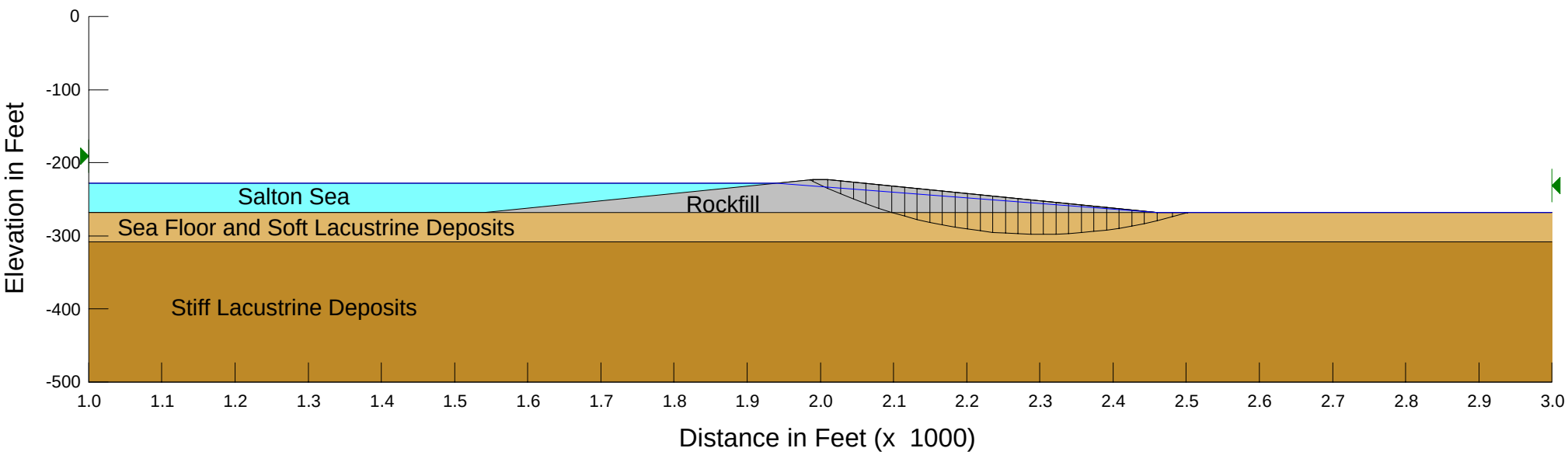
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.3g

Factor of Safety = 0.9 (Spencer's Method)





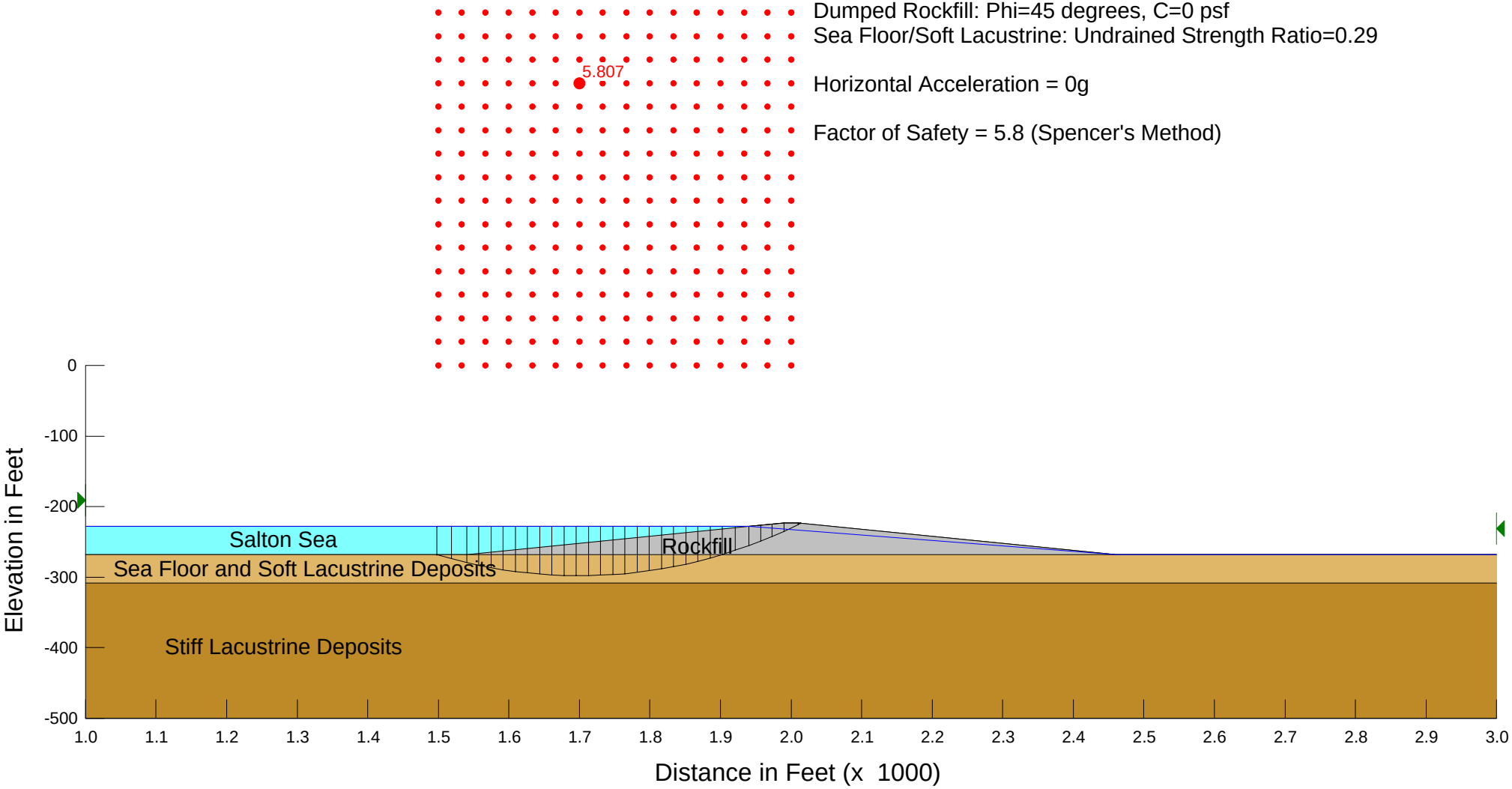
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.29

Horizontal Acceleration = 0g

Factor of Safety = 5.8 (Spencer's Method)



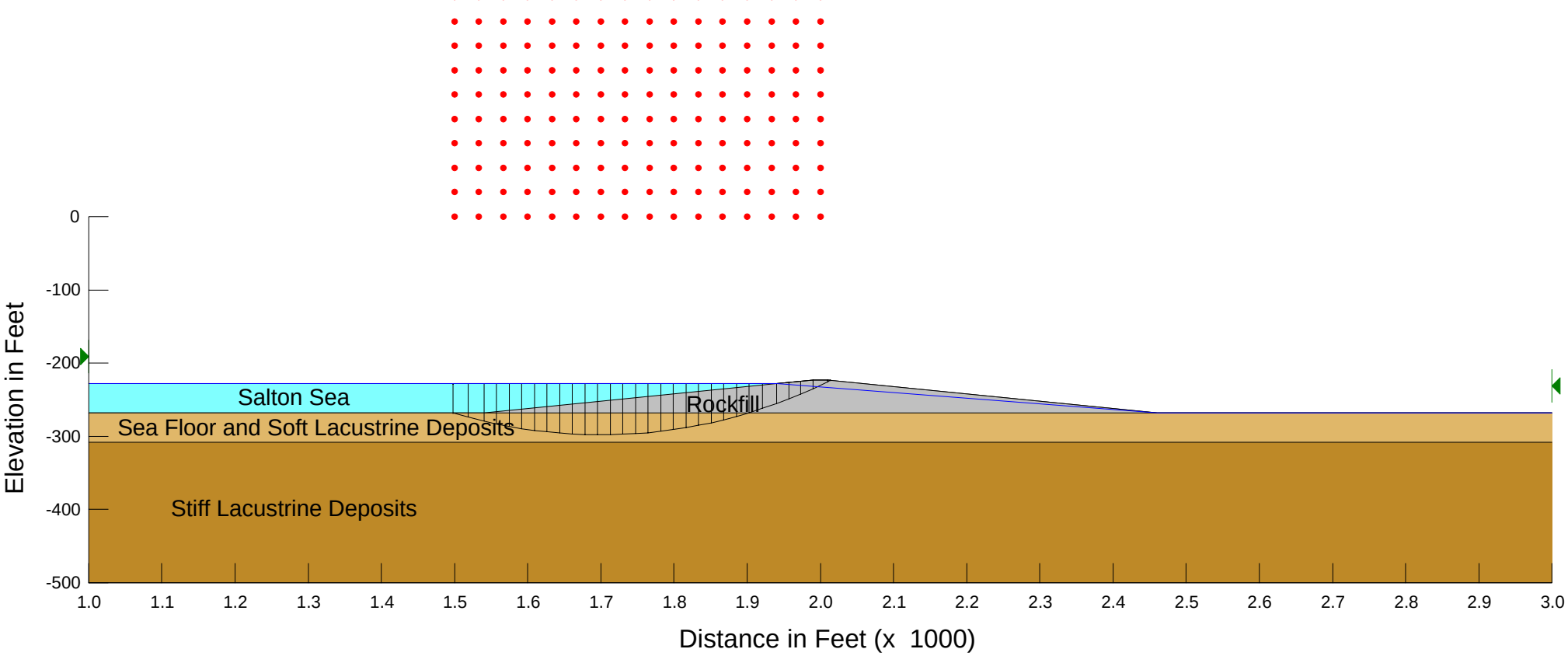
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.05g

Factor of Safety = 3.2 (Spencer's Method)



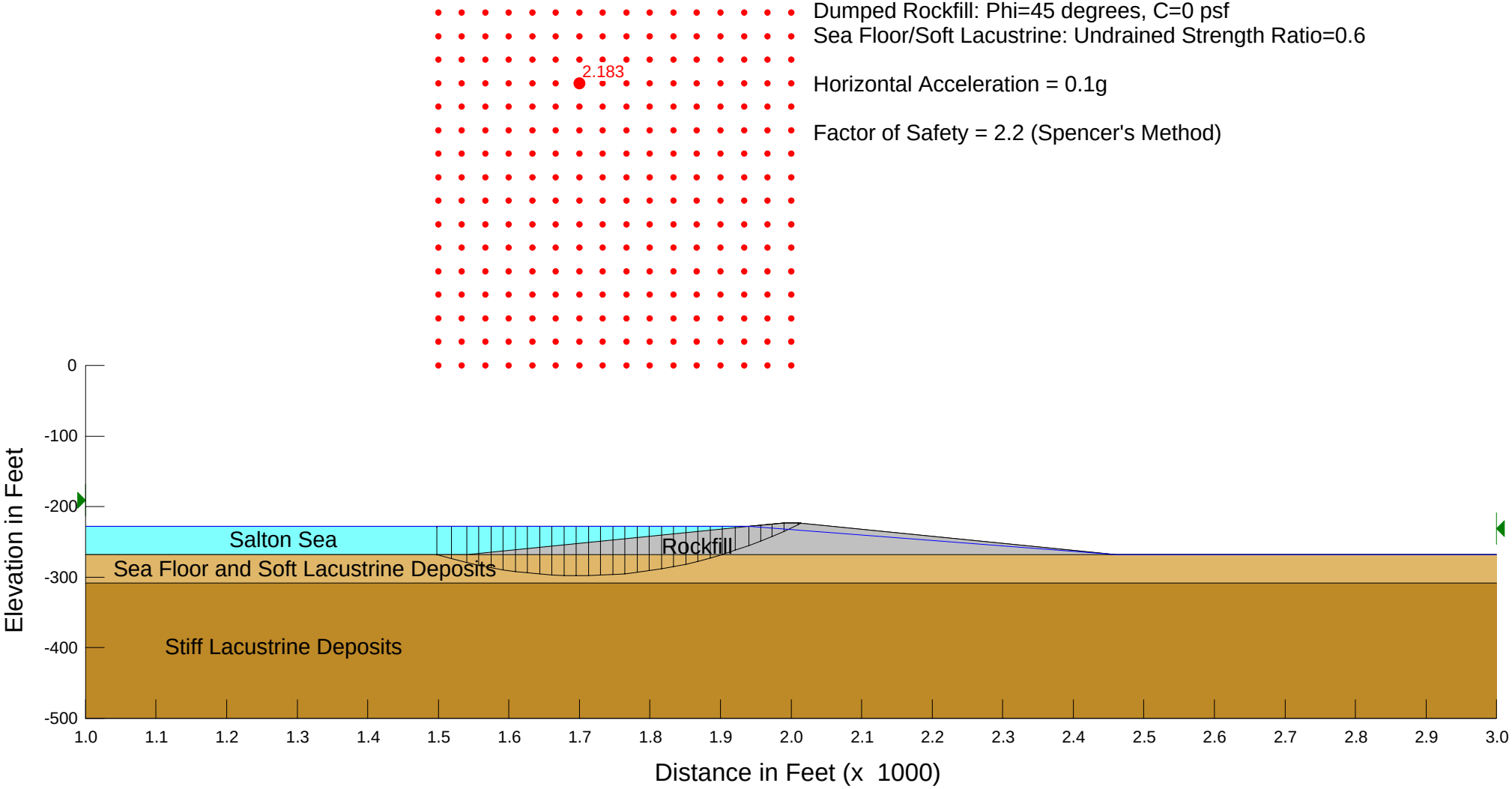
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

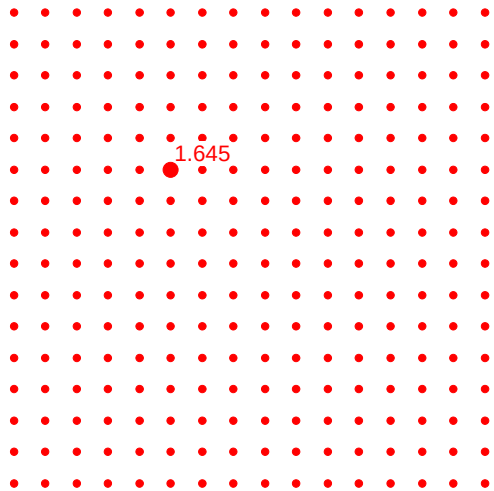
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.1g

Factor of Safety = 2.2 (Spencer's Method)





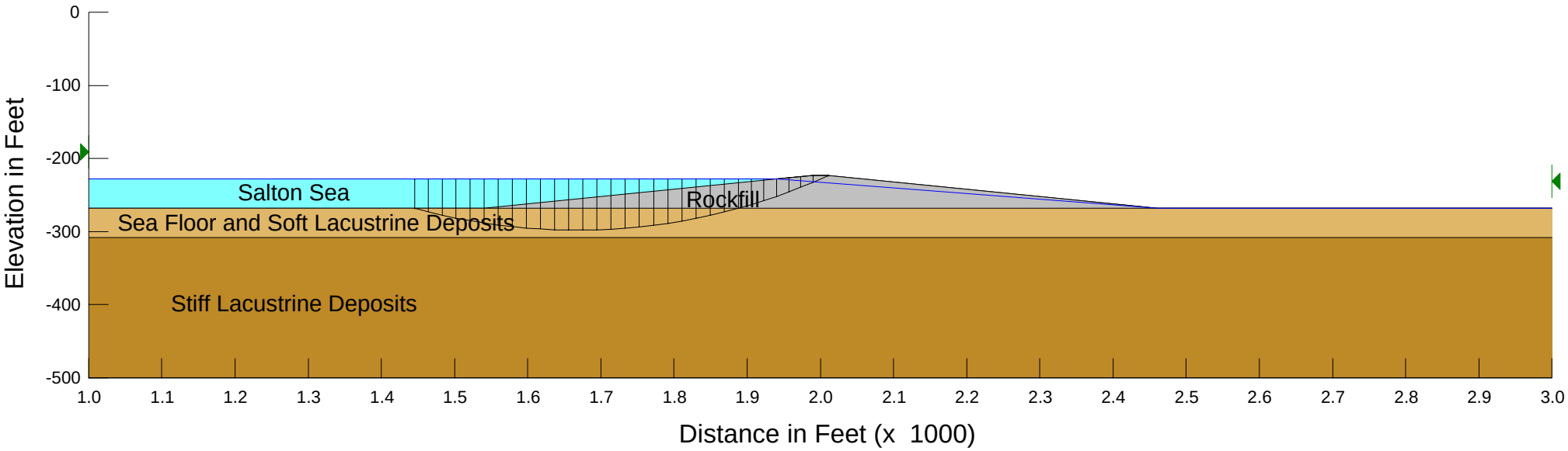
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

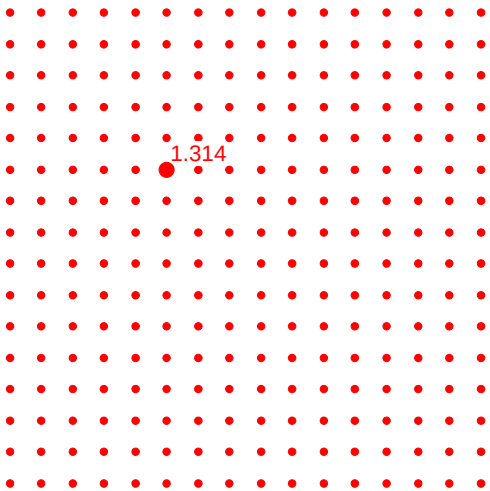
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.15g

Factor of Safety = 1.6 (Spencer's Method)





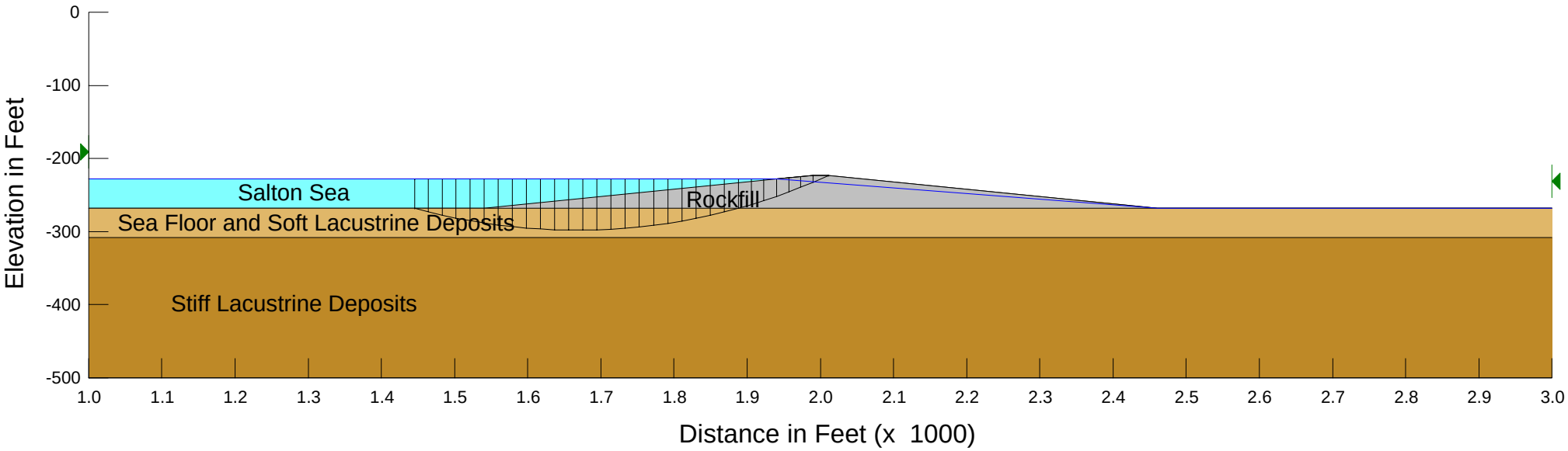
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

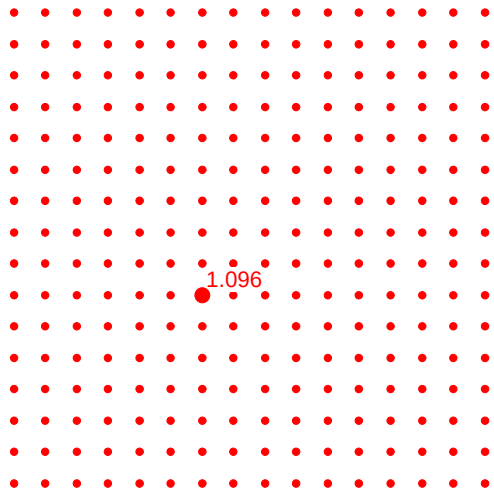
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.2g

Factor of Safety = 1.3 (Spencer's Method)





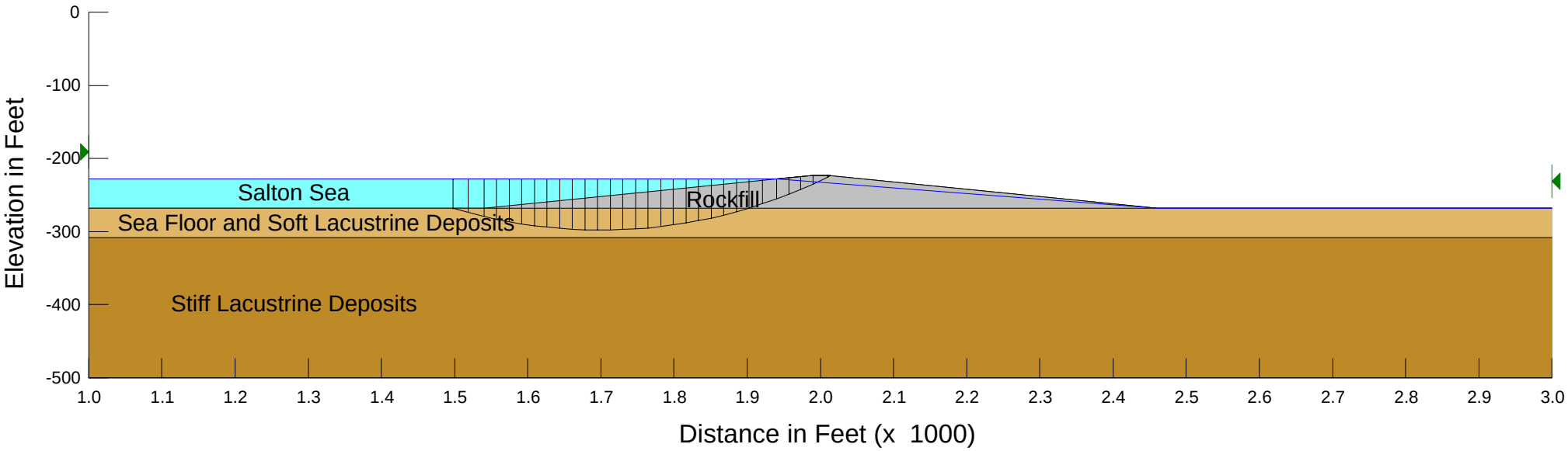
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

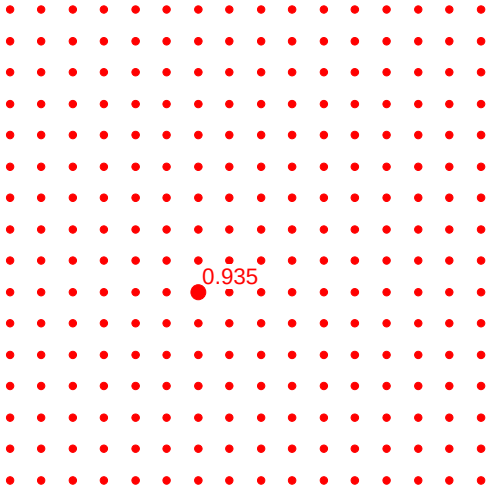
45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.25g

Factor of Safety = 1.1 (Spencer's Method)





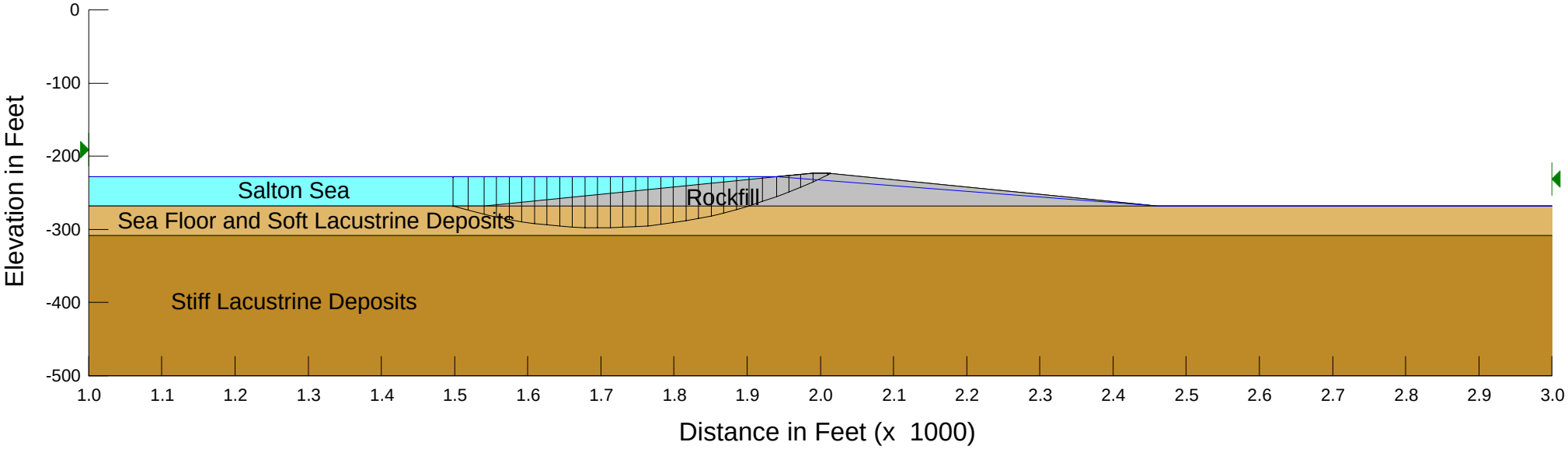
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Undrained Strength Ratio=0.6

Horizontal Acceleration = 0.3g

Factor of Safety = 0.9 (Spencer's Method)



POST-LIQUEFACTION (RESIDUAL STRENGTH) STABILITY WITH BARRIER SLOPES OF 10H:1V TO 15H:1V BARRIER HEIGHT = 45 AND 15 FEET

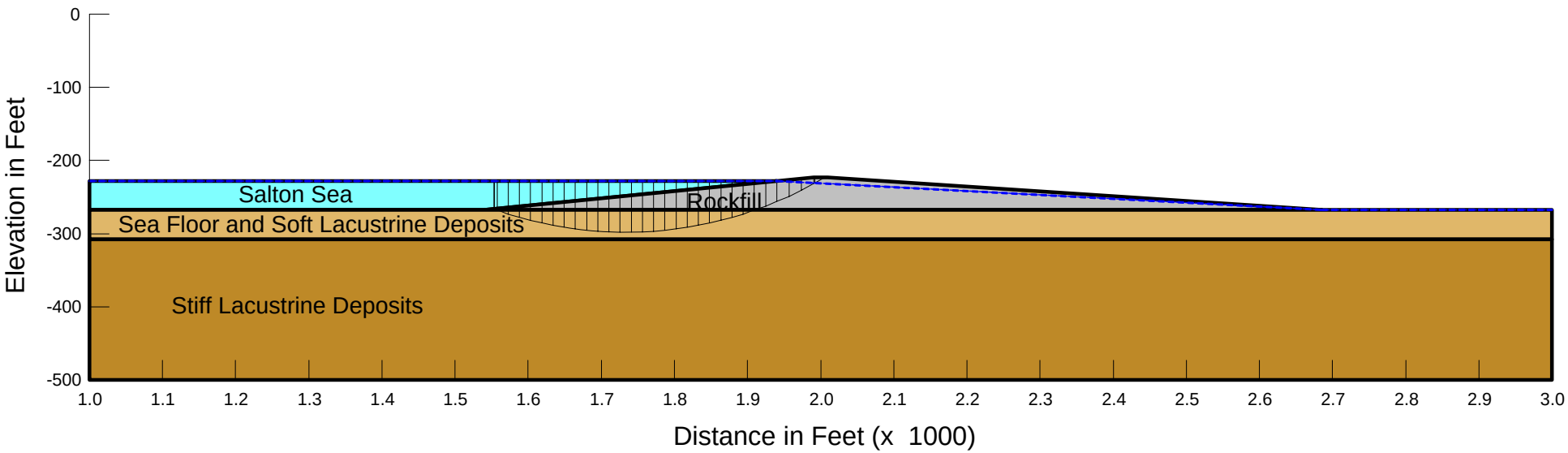
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and 15H:1V Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Residual Strength = 250 psf

Horizontal Acceleration = 0g

Factor of Safety = 2.0 (Spencer's Method)



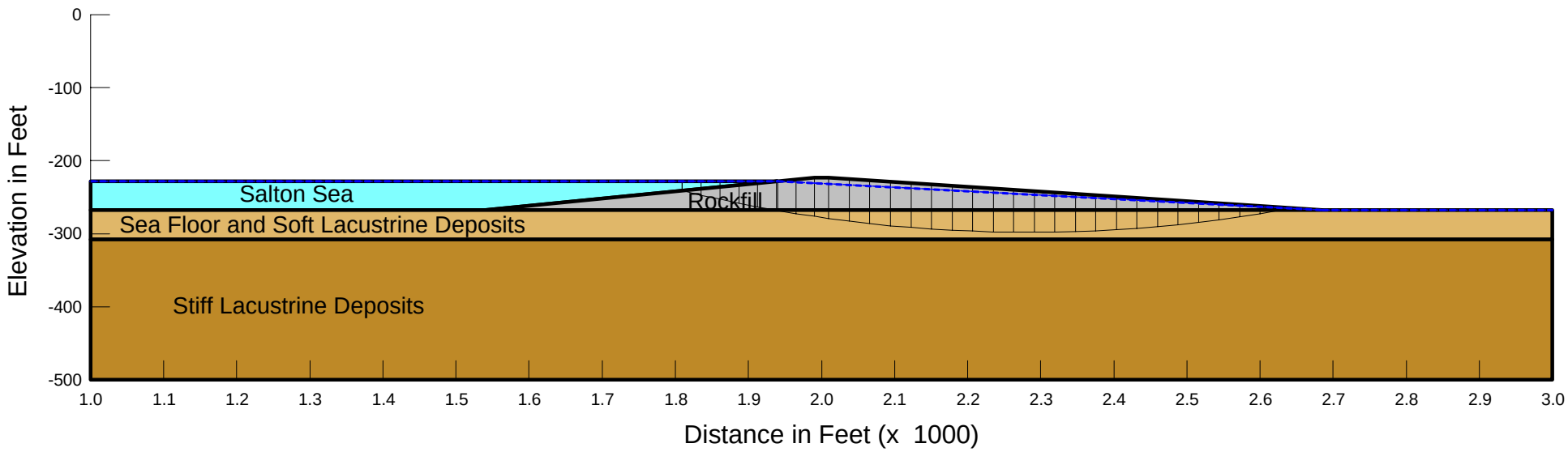
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

45' High Barrier with 10H:1V Upstream and 15H:1V Downstream Slopes
Upstream Water Depth = 40', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Residual Strength = 250 psf

Horizontal Acceleration = 0g

Factor of Safety = 1.8 (Spencer's Method)



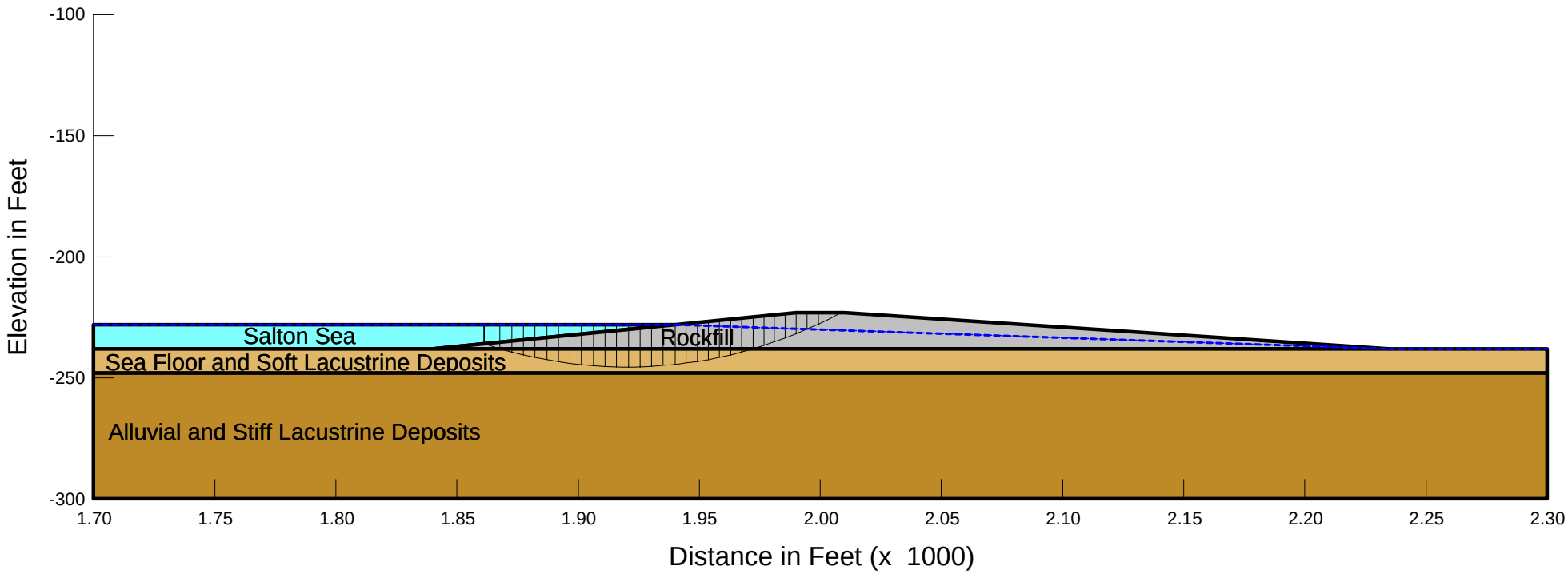
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

15' High Barrier with 10H:1V Upstream and 15H:1V Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Residual Strength =250 psf

Horizontal Acceleration = 0g

Factor of Safety = 3.8 (Spencer's Method)



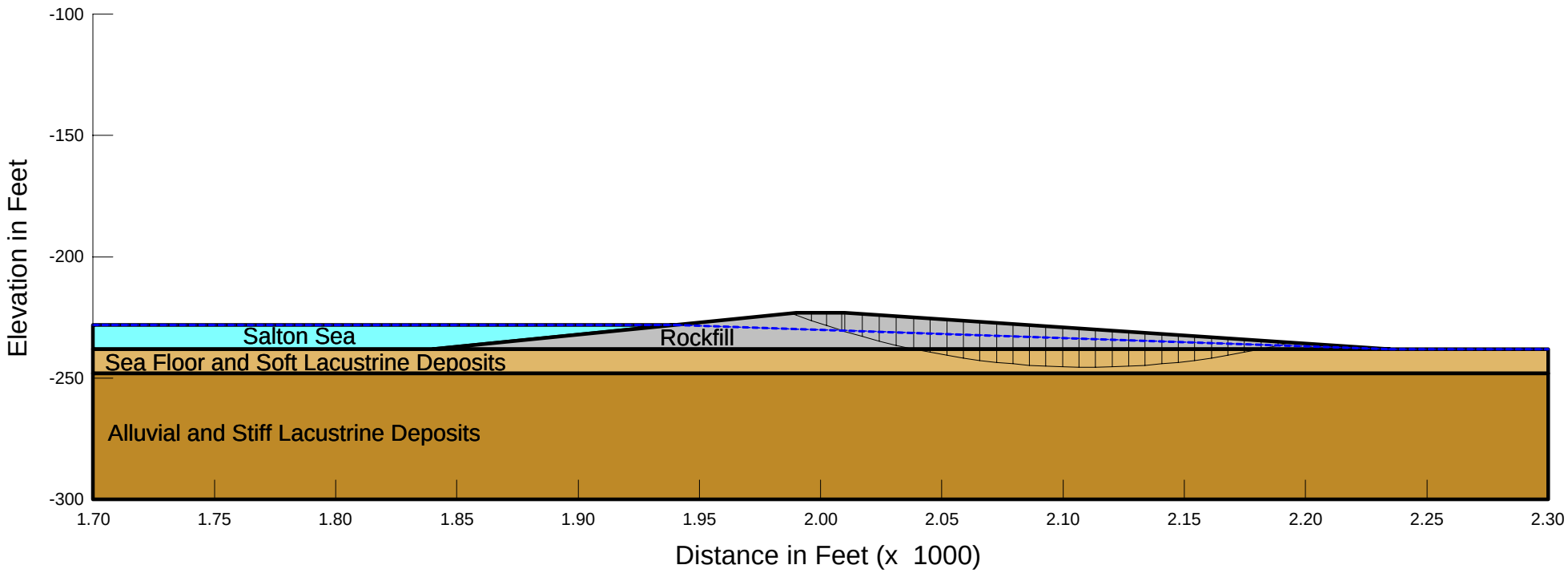
SALTON SEA - IN-SEA BARRIER
DUMPED ROCKFILL ALTERNATIVE

15' High Barrier with 10H:1V Upstream and 15H:1V Downstream Slopes
Upstream Water Depth = 10', Downstream Water Depth = 0'

Dumped Rockfill: Phi=45 degrees, C=0 psf
Sea Floor/Soft Lacustrine: Residual Strength =250 psf

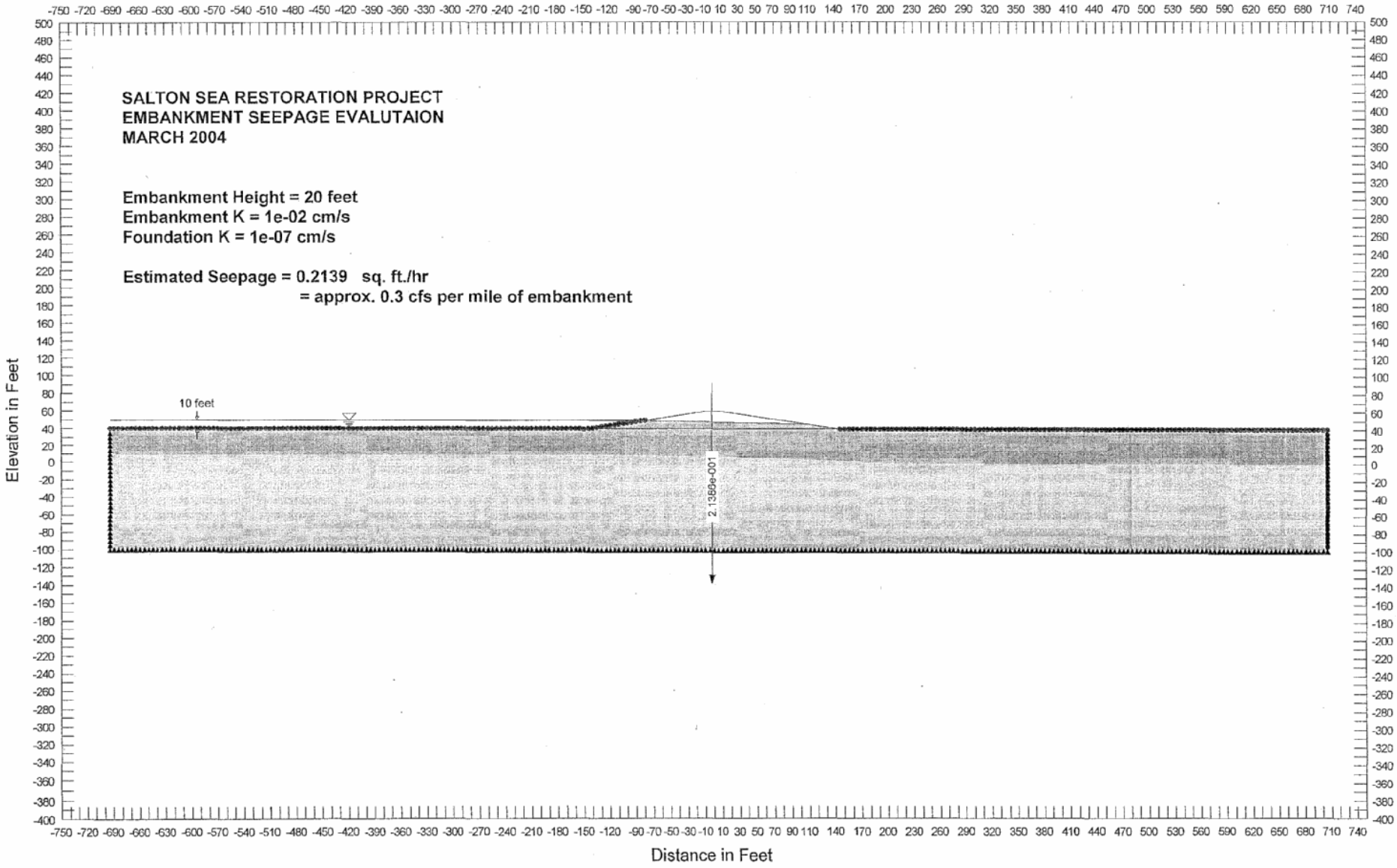
Horizontal Acceleration = 0g

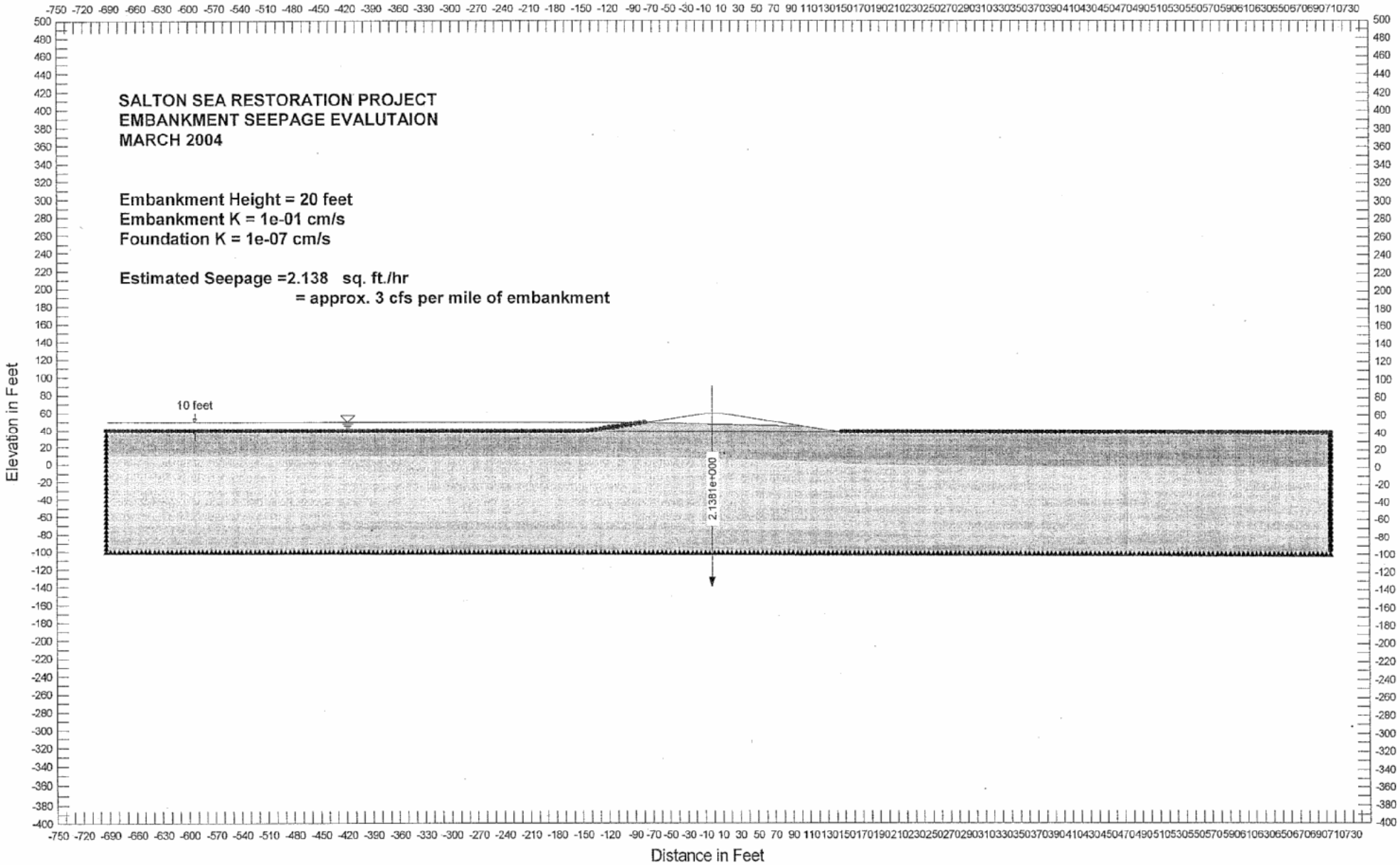
Factor of Safety = 4.2 (Spencer's Method)

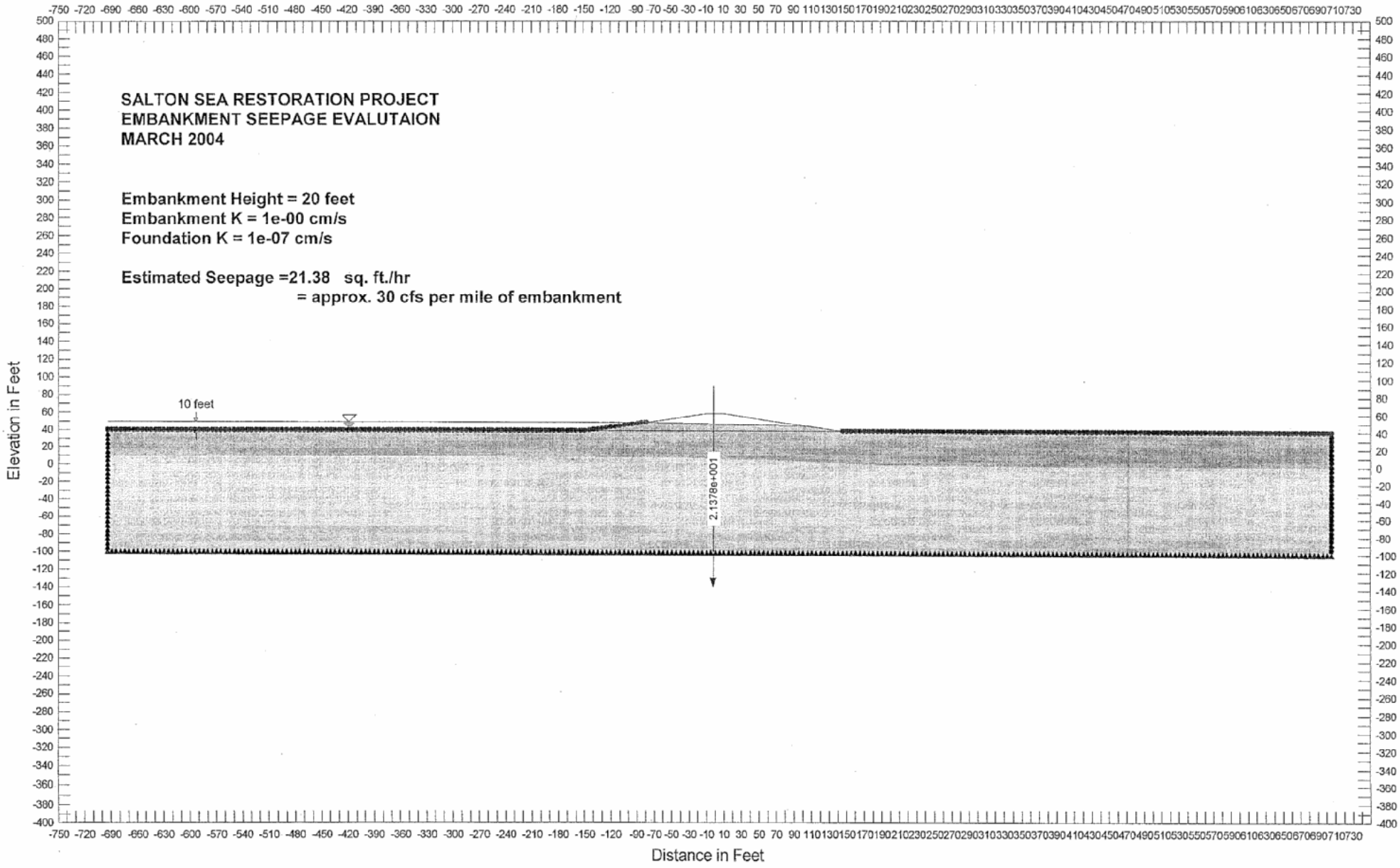


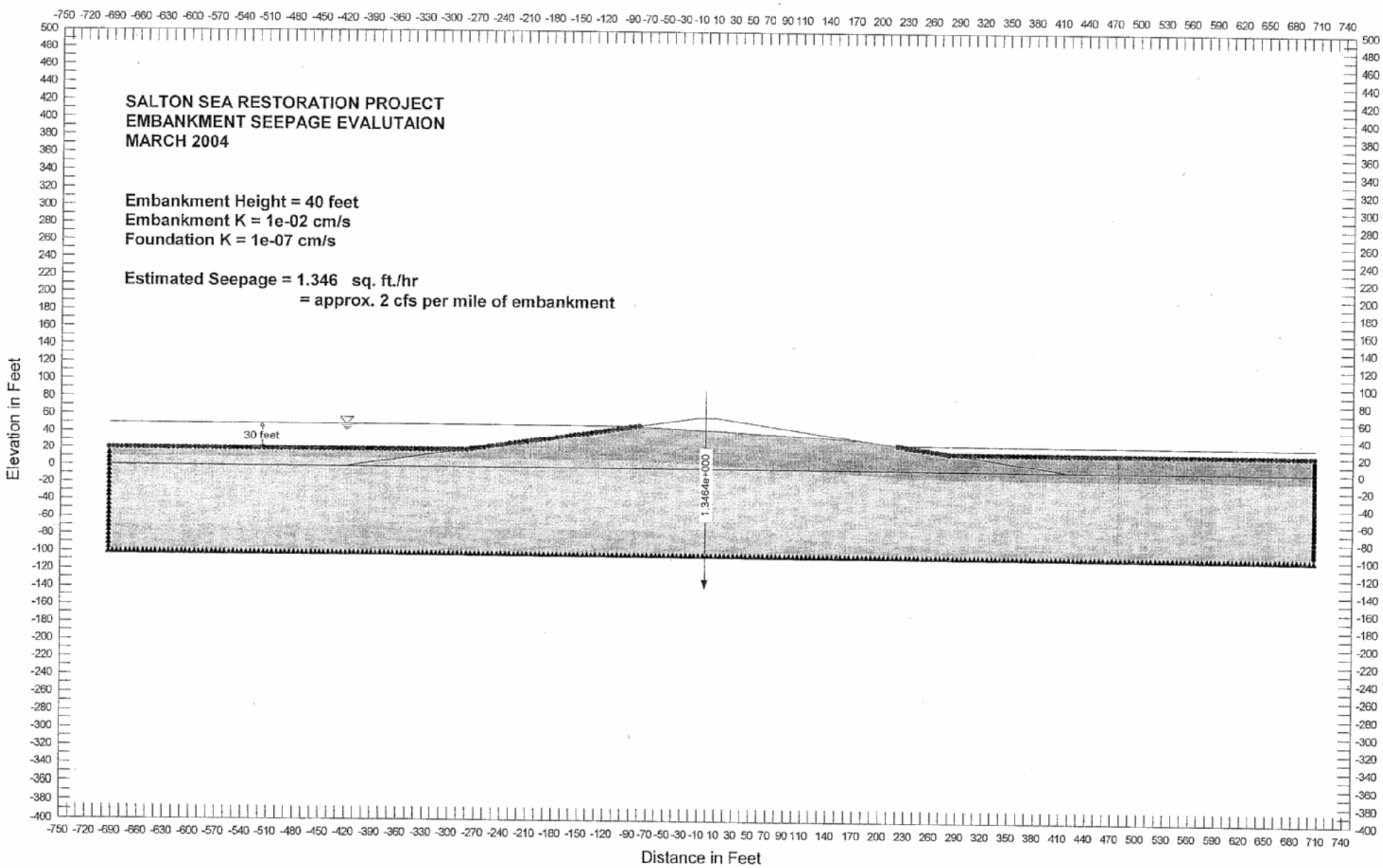
ATTACHMENT H4-2

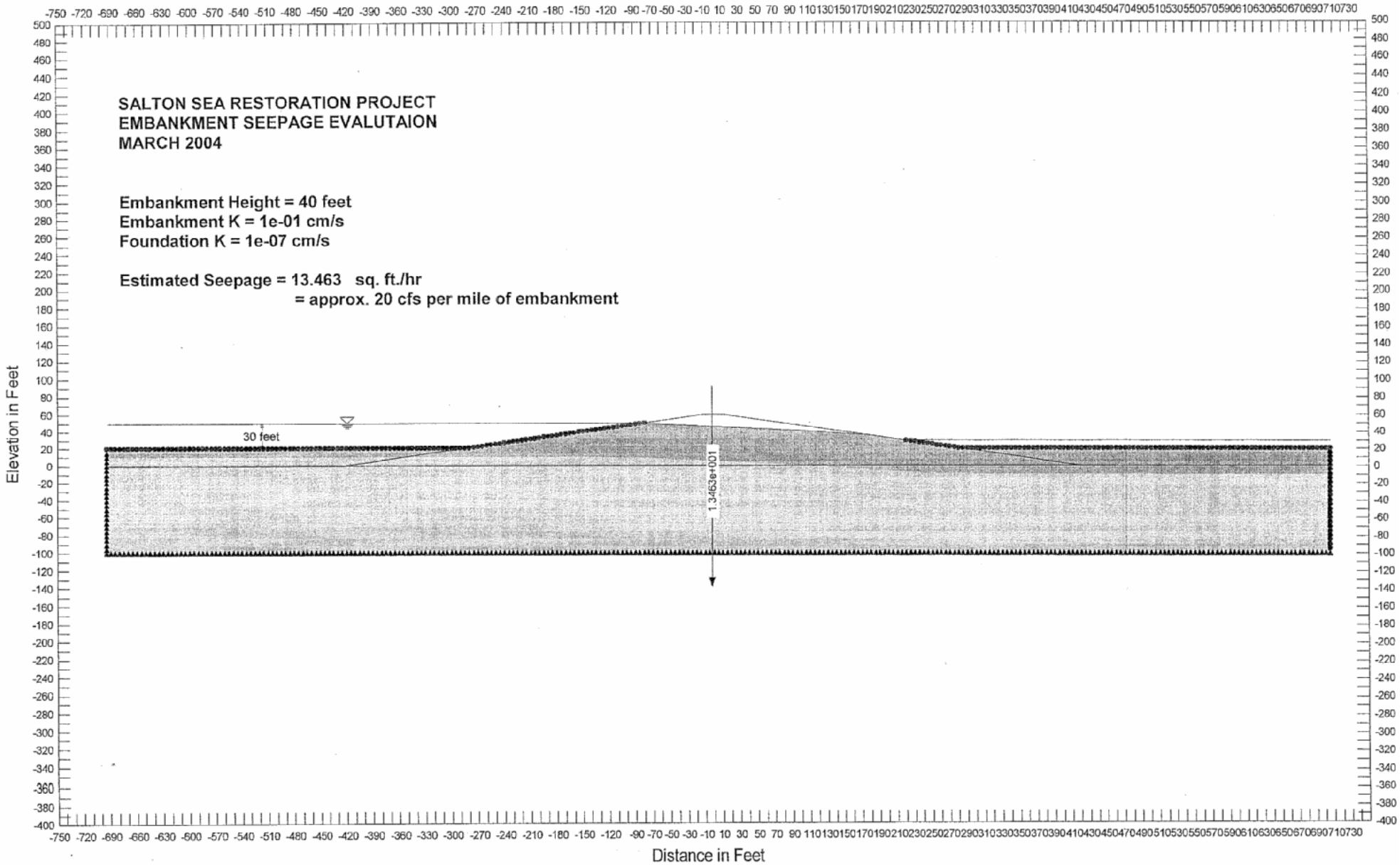
Seep/W Program Outputs

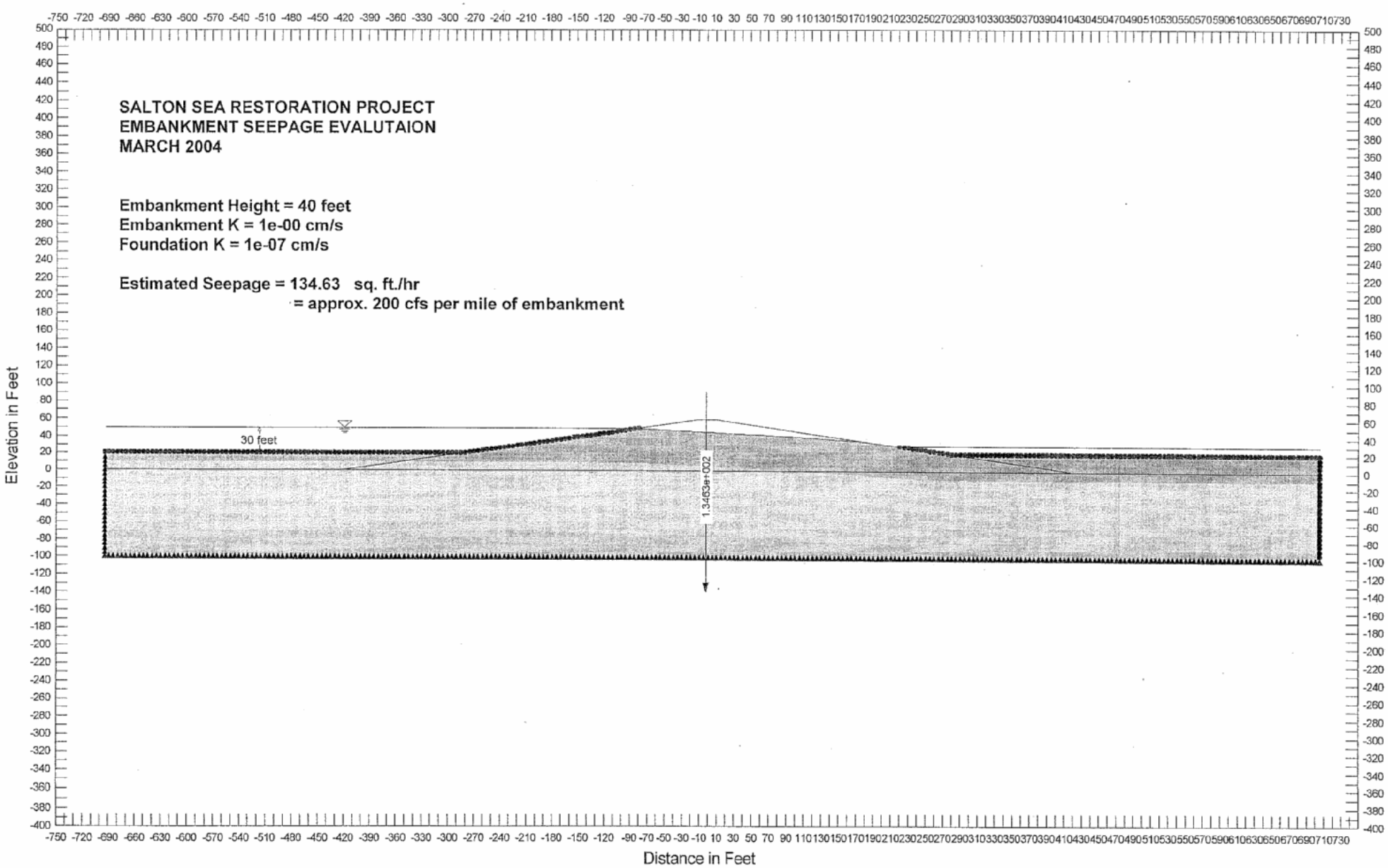

SALTON SEA RESTORATION PROJECT
EMBANKMENT SEEPAGE EVALUTAION
MARCH 2004
Embankment Height = 20 feet
Embankment K = 1e-02 cm/s
Foundation K = 1e-07 cm/s
Estimated Seepage = 0.2139 sq. ft./hr
= approx. 0.3 cfs per mile of embankment
10 feet
2.1369e-001
Elevation in Feet
Distance in Feet

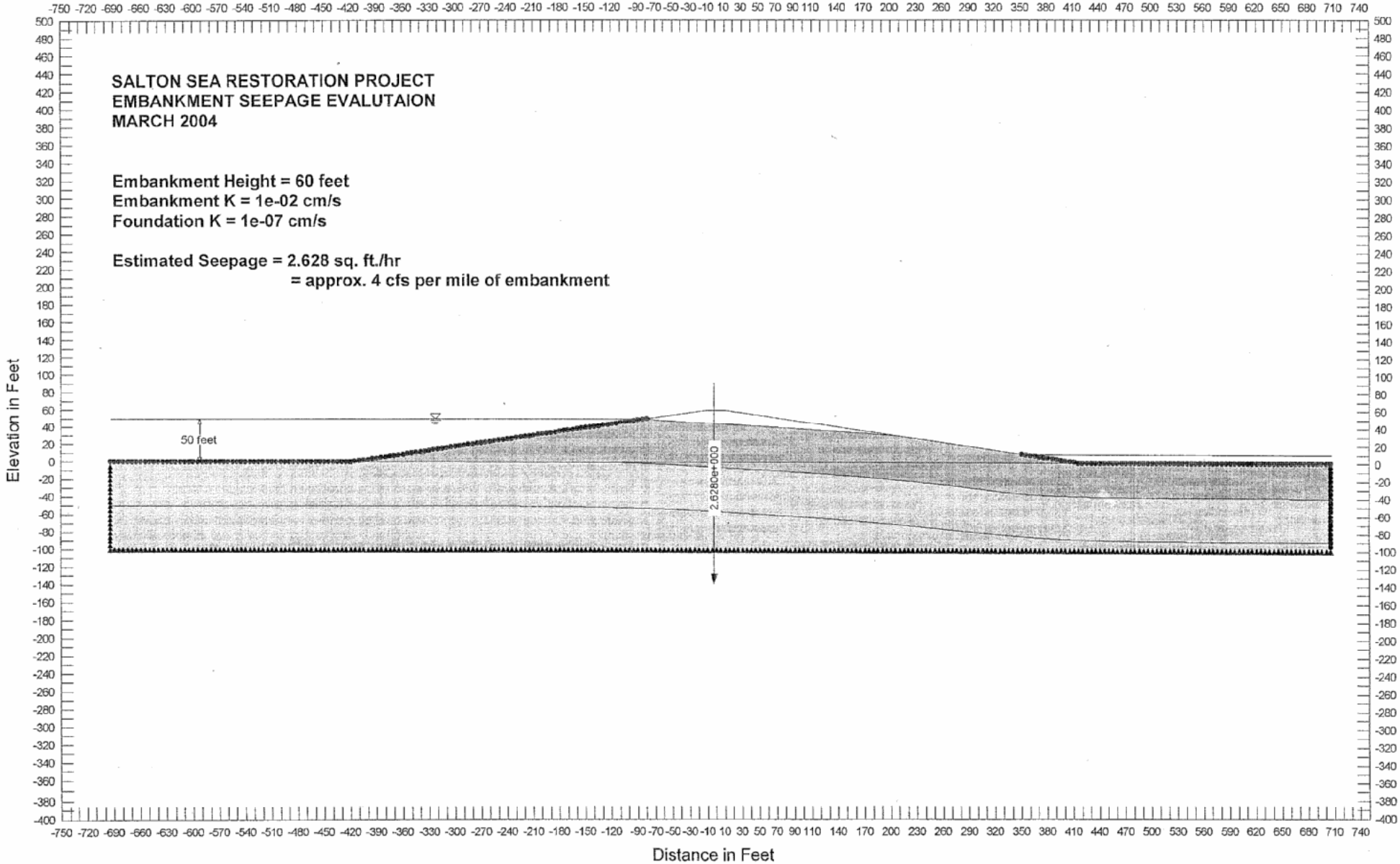

SALTON SEA RESTORATION PROJECT
EMBANKMENT SEEPAGE EVALUTAION
MARCH 2004
Embankment Height = 20 feet
Embankment K = 1e-01 cm/s
Foundation K = 1e-07 cm/s
Estimated Seepage =2.138 sq. ft./hr
= approx. 3 cfs per mile of embankment
10 feet
2.138 1e+000
Elevation in Feet
Distance in Feet

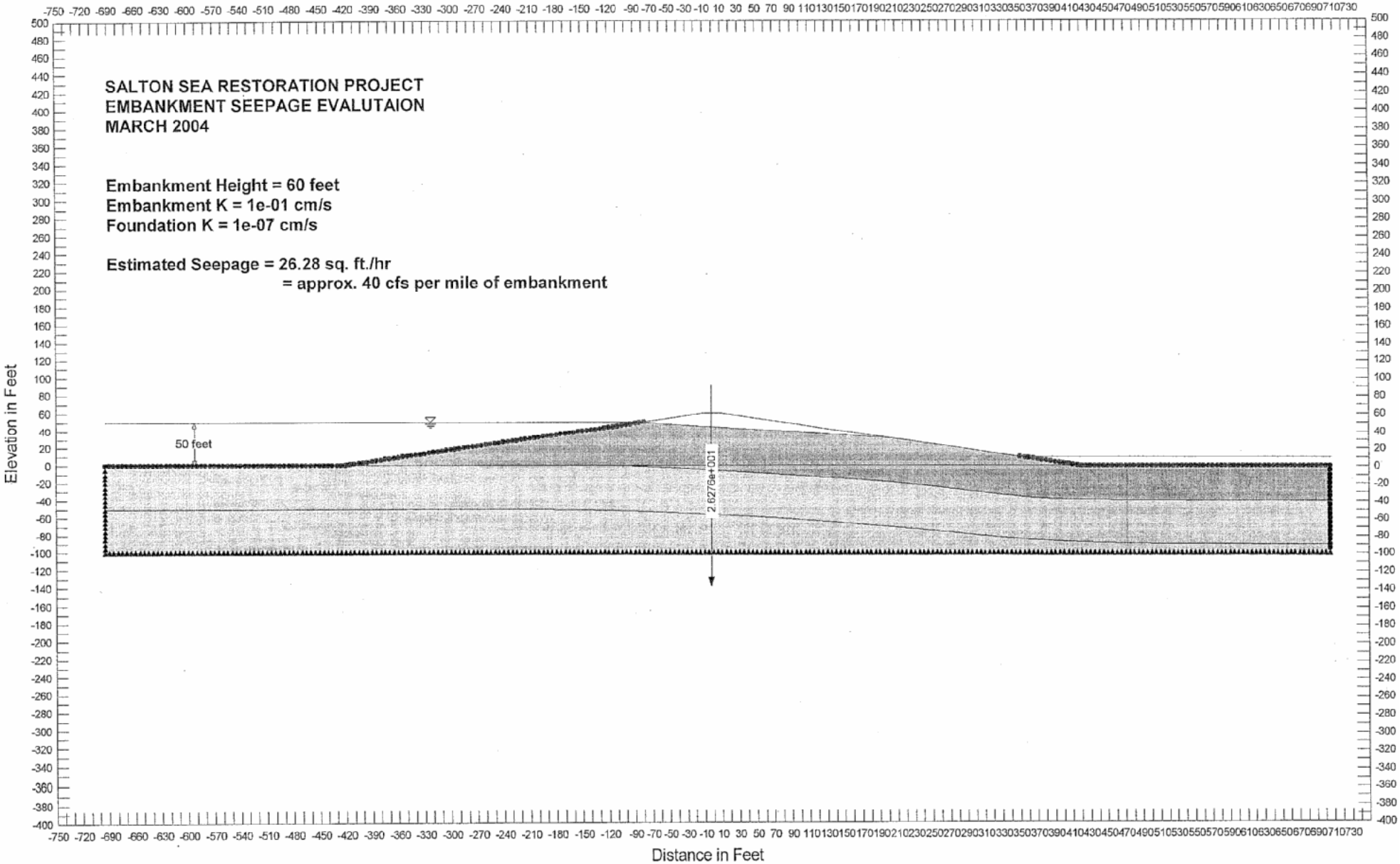

SALTON SEA RESTORATION PROJECT
EMBANKMENT SEEPAGE EVALUTAION
MARCH 2004
Embankment Height = 20 feet
Embankment K = 1e-00 cm/s
Foundation K = 1e-07 cm/s
Estimated Seepage =21.38 sq. ft./hr
= approx. 30 cfs per mile of embankment
10 feet
2.1378e+001
Elevation in Feet
Distance in Feet

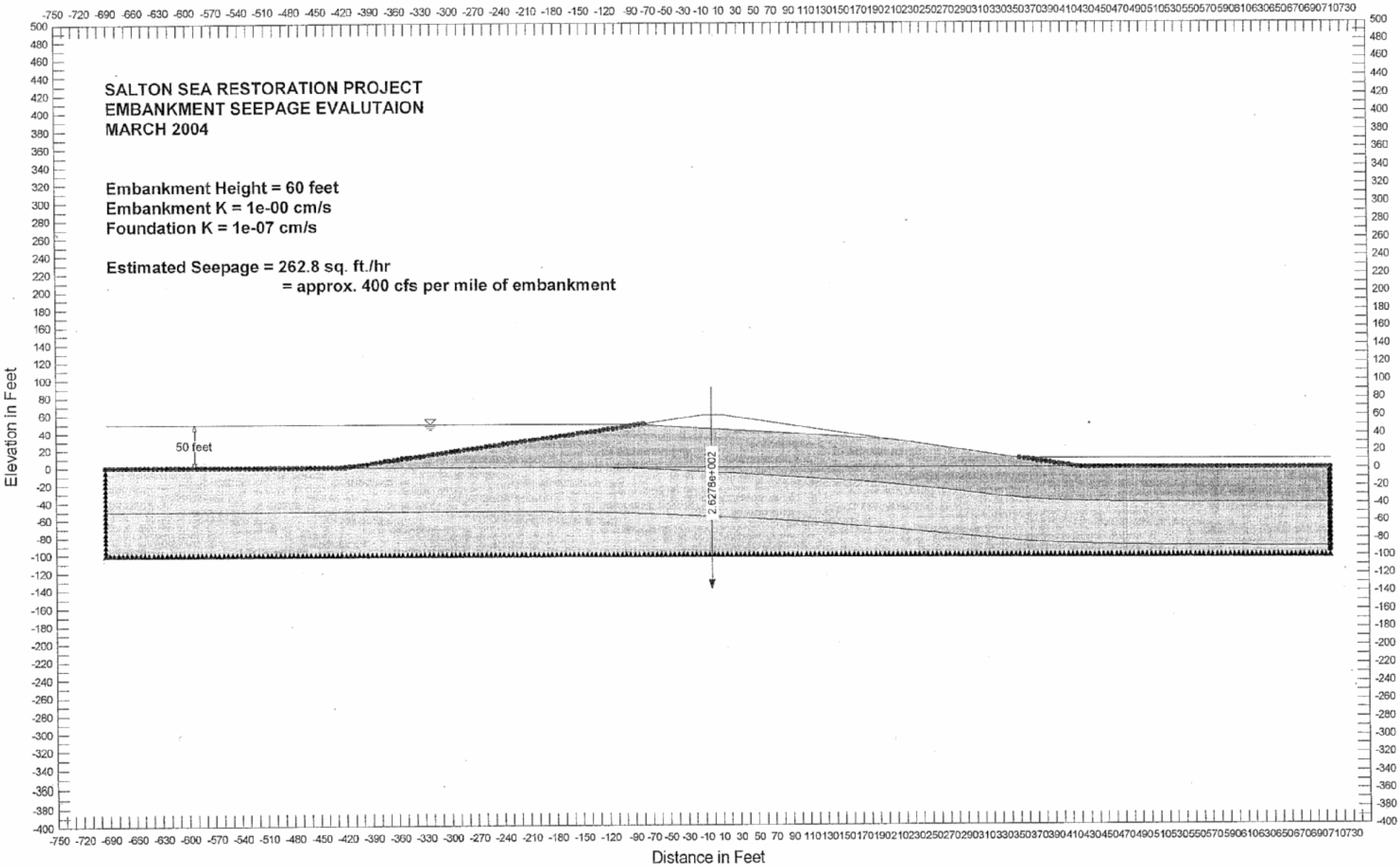

SALTON SEA RESTORATION PROJECT
EMBANKMENT SEEPAGE EVALUTAION
MARCH 2004
Embankment Height = 40 feet
Embankment K = 1e-02 cm/s
Foundation K = 1e-07 cm/s
Estimated Seepage = 1.346 sq. ft./hr
= approx. 2 cfs per mile of embankment
30 feet
1.3464e+000
Elevation in Feet
Distance in Feet


SALTON SEA RESTORATION PROJECT
EMBANKMENT SEEPAGE EVALUTAION
MARCH 2004
Embankment Height = 40 feet
Embankment K = 1e-01 cm/s
Foundation K = 1e-07 cm/s
Estimated Seepage = 13.463 sq. ft./hr
= approx. 20 cfs per mile of embankment
30 feet
1.3463e+001
Elevation in Feet
Distance in Feet


SALTON SEA RESTORATION PROJECT
EMBANKMENT SEEPAGE EVALUTAION
MARCH 2004
Embankment Height = 40 feet
Embankment K = 1e-00 cm/s
Foundation K = 1e-07 cm/s
Estimated Seepage = 134.63 sq. ft./hr
= approx. 200 cfs per mile of embankment
30 feet
1.3463e+002
Elevation in Feet
Distance in Feet


SALTON SEA RESTORATION PROJECT
EMBANKMENT SEEPAGE EVALUTAION
MARCH 2004
Embankment Height = 60 feet
Embankment K = 1e-02 cm/s
Foundation K = 1e-07 cm/s
Estimated Seepage = 2.628 sq. ft./hr
= approx. 4 cfs per mile of embankment
50 feet
2.628e+000
Elevation in Feet
Distance in Feet


SALTON SEA RESTORATION PROJECT
EMBANKMENT SEEPAGE EVALUTAION
MARCH 2004
Embankment Height = 60 feet
Embankment K = 1e-01 cm/s
Foundation K = 1e-07 cm/s
Estimated Seepage = 26.28 sq. ft./hr
= approx. 40 cfs per mile of embankment
50 feet
2.6275e+001
Elevation in Feet
Distance in Feet


SALTON SEA RESTORATION PROJECT
EMBANKMENT SEEPAGE EVALUTAION
MARCH 2004
Embankment Height = 60 feet
Embankment K = 1e-00 cm/s
Foundation K = 1e-07 cm/s
Estimated Seepage = 262.8 sq. ft./hr
= approx. 400 cfs per mile of embankment
50 feet
2.6278e+002
Elevation in Feet
Distance in Feet

ATTACHMENT H4-3

Acceleration Time History

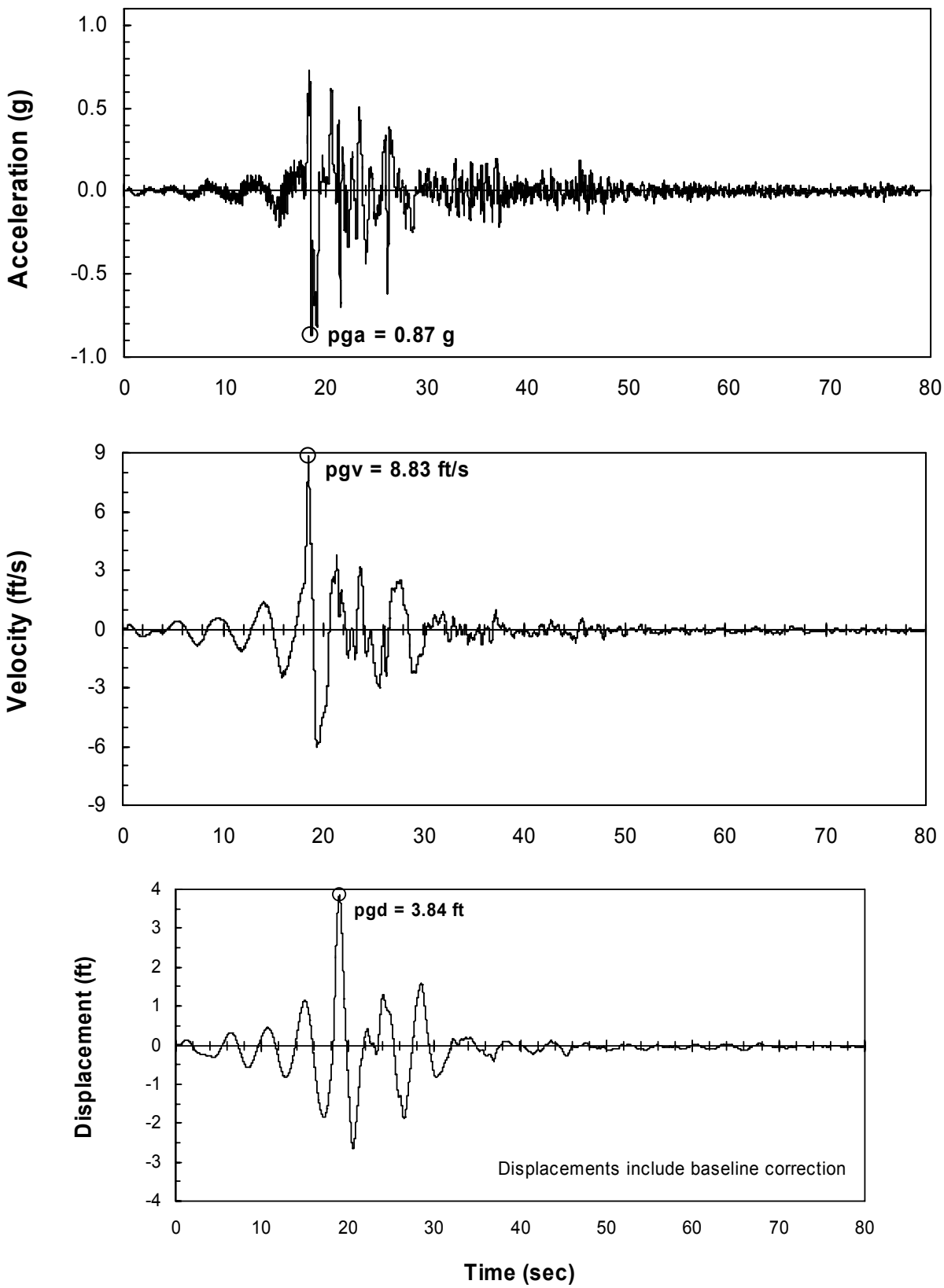


FIGURE H4-3-1
HISTORY PLOTS FOR PS10FN (CASE 1, EASTERN SHORE)

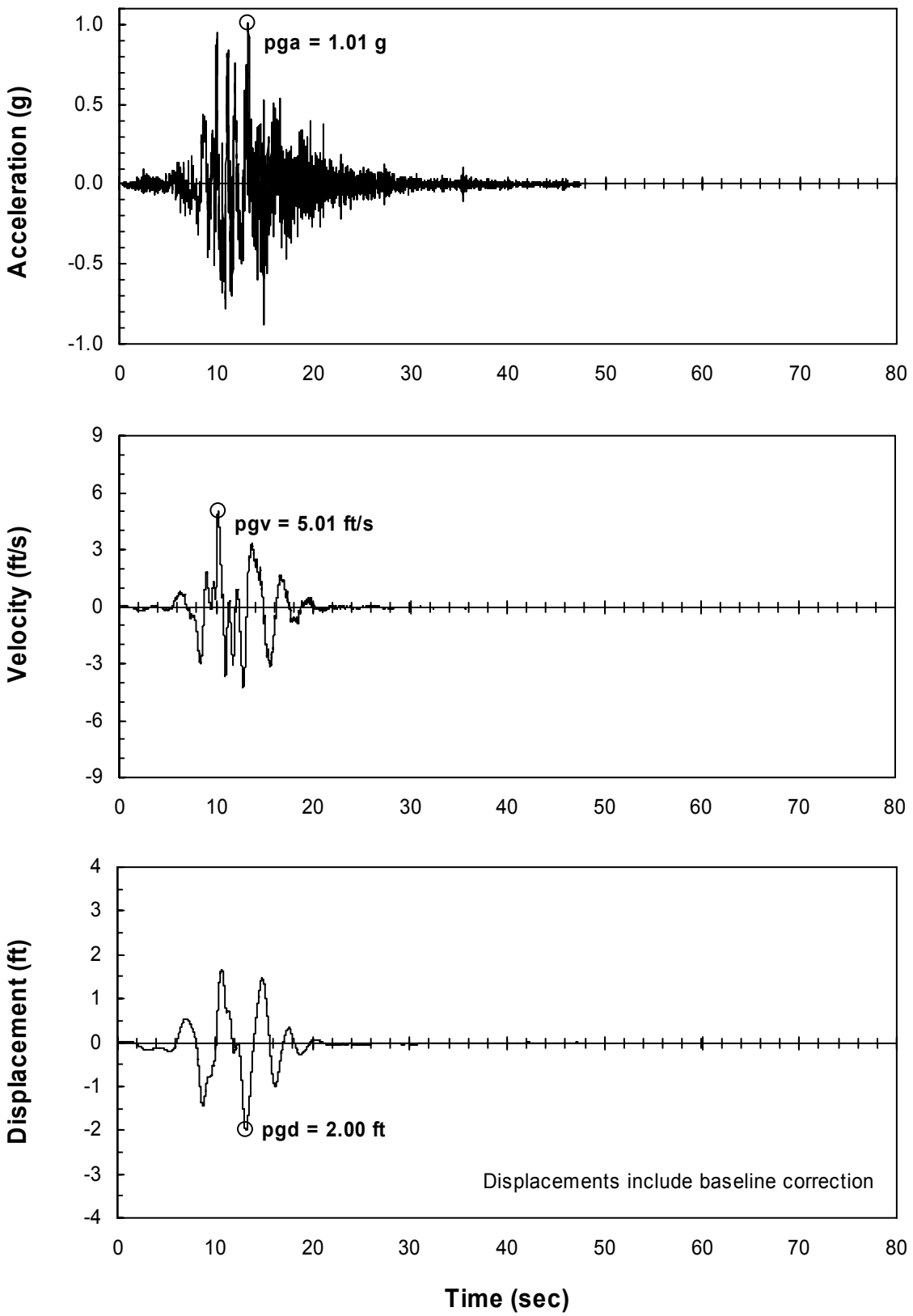


FIGURE H4-3-2
HISTORY PLOTS FOR LUC51 (CASE 1, EASTERN SHORE)

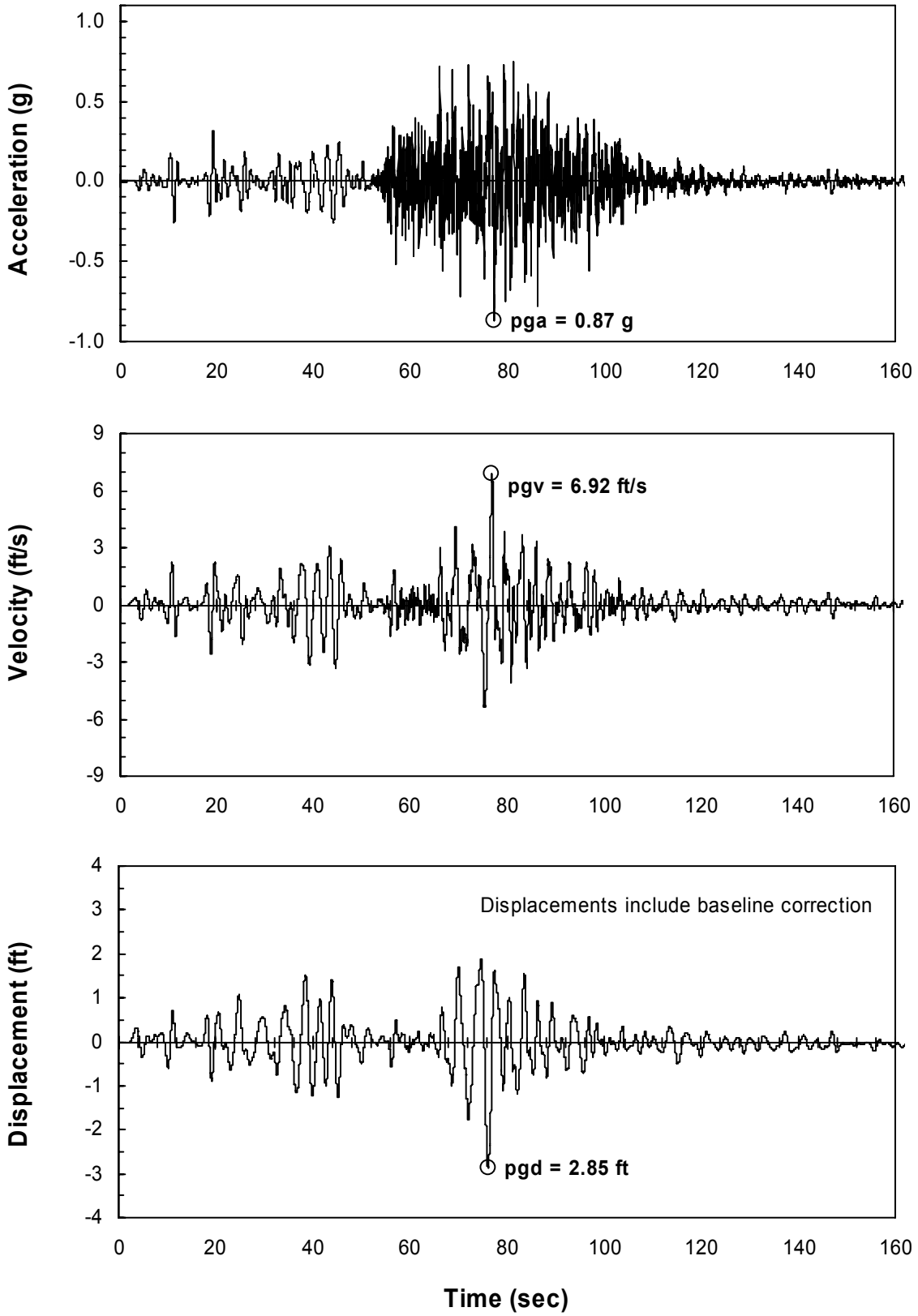


FIGURE H4-3-3
HISTORY PLOTS FOR R1390 (CASE 1, EASTERN SHORE)

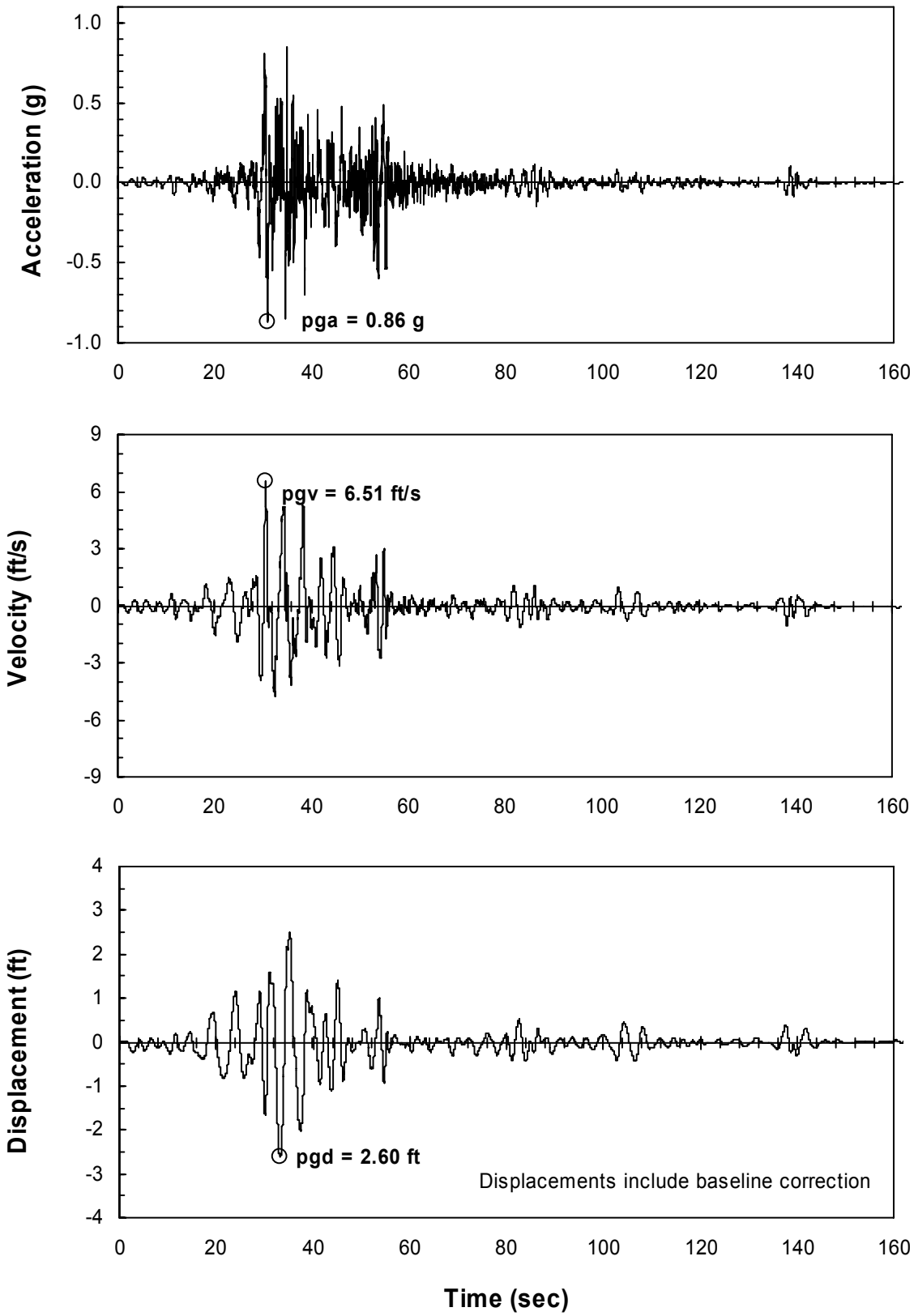


FIGURE H4-3-4
HISTORY PLOTS FOR R1590 (CASE 1, EASTERN SHORE)

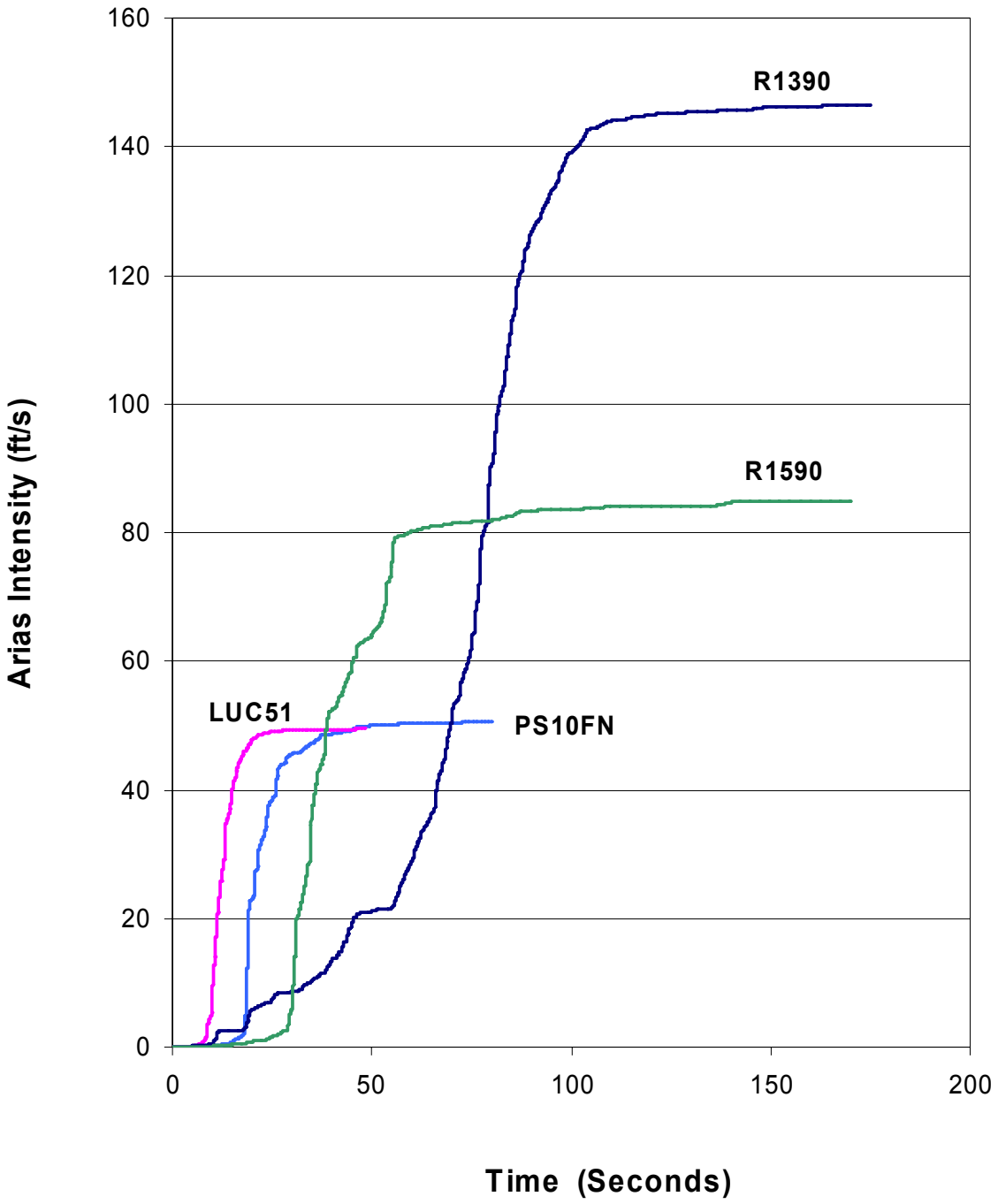


FIGURE H4-3-5
ARIAS INTENSITY PLOTS FOR CASE 1, EASTERN SHORE

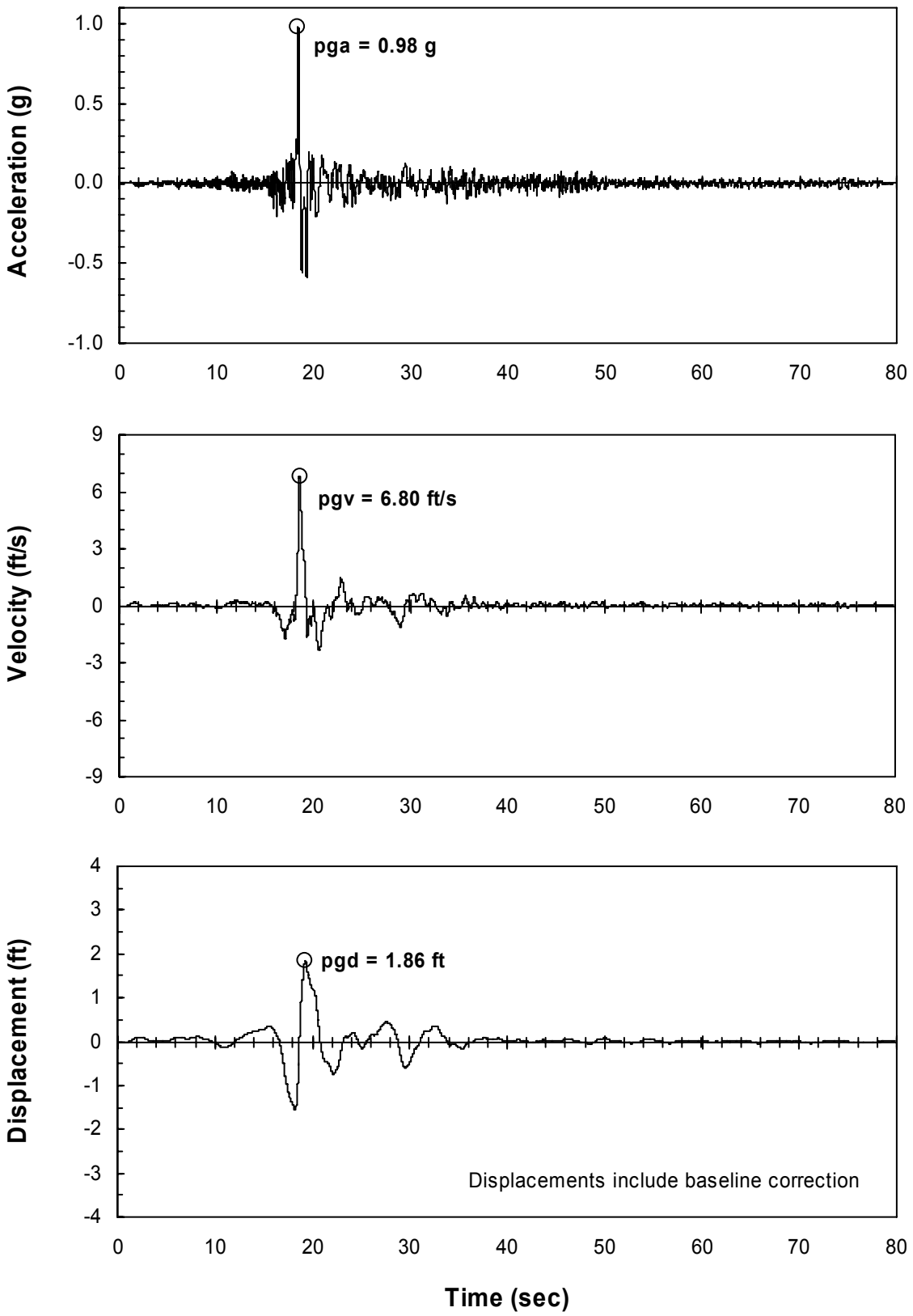


FIGURE H4-3-6
HISTORY PLOTS FOR PS10FP (CASE 2, EASTERN SHORE).

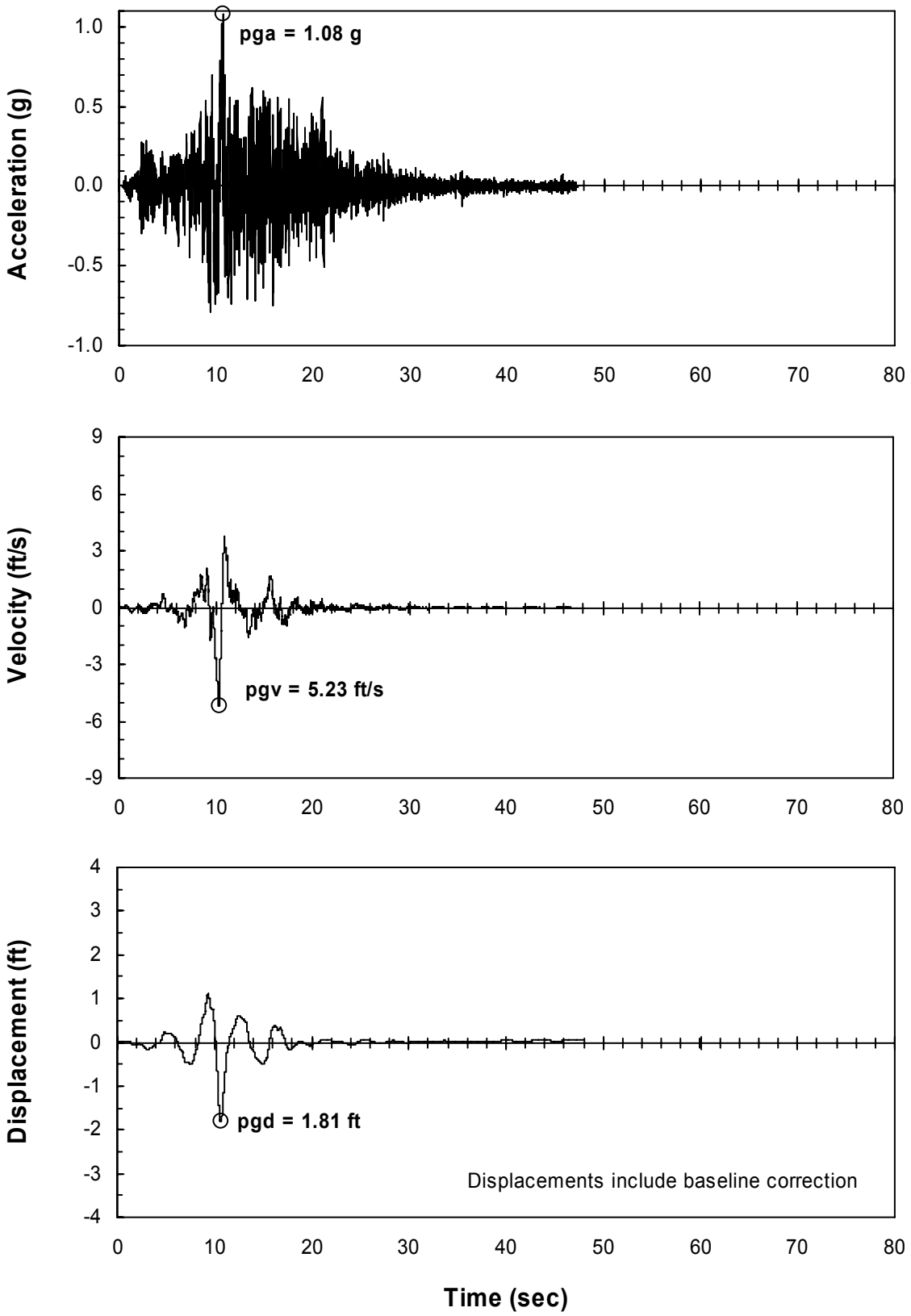


FIGURE H4-3-7
HISTORY PLOTS FOR LUC321 (CASE 2, EASTERN SHORE)

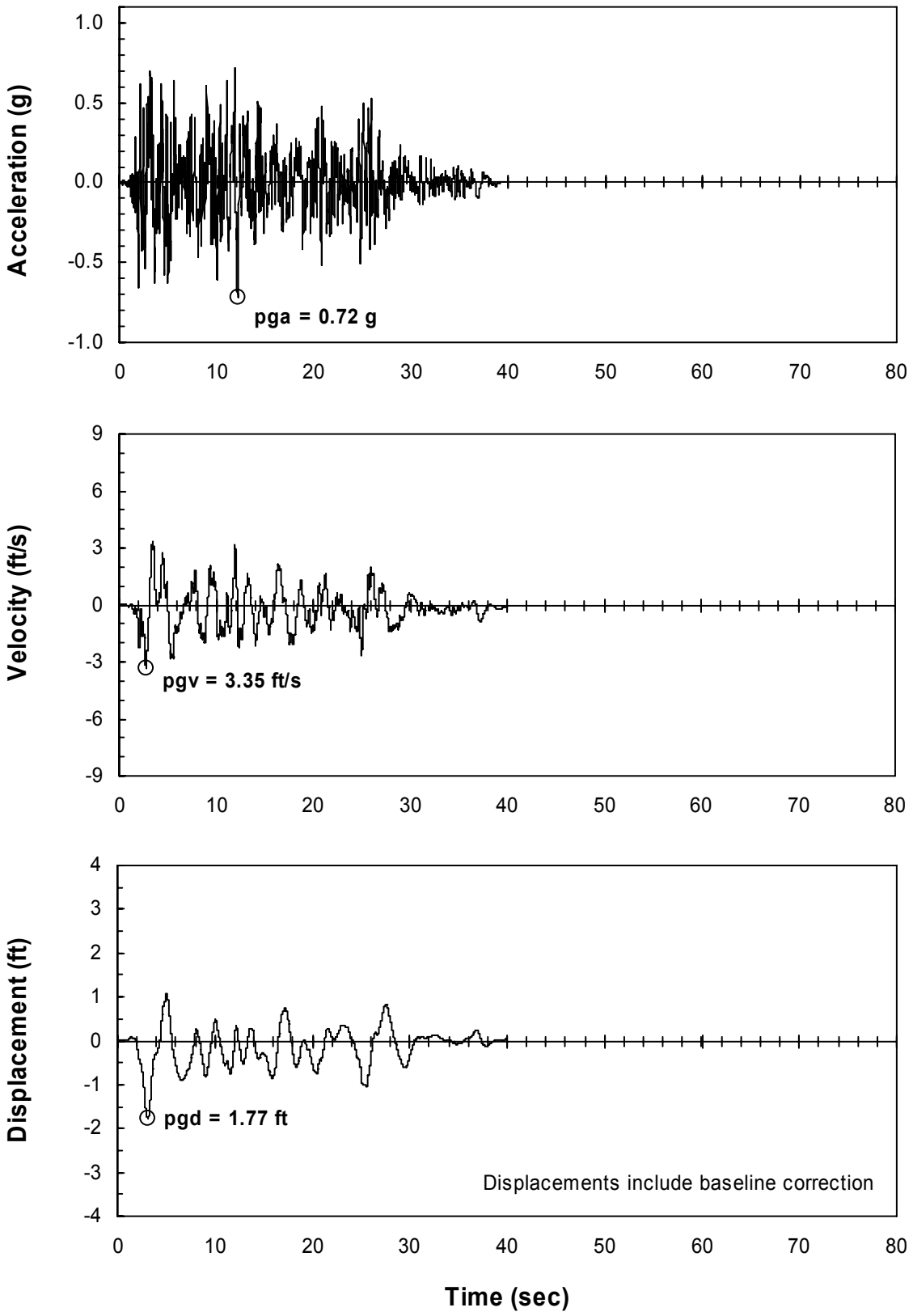


FIGURE H4-3-8
HISTORY PLOTS FOR ELC53 (CASE 2, EASTERN SHORE)

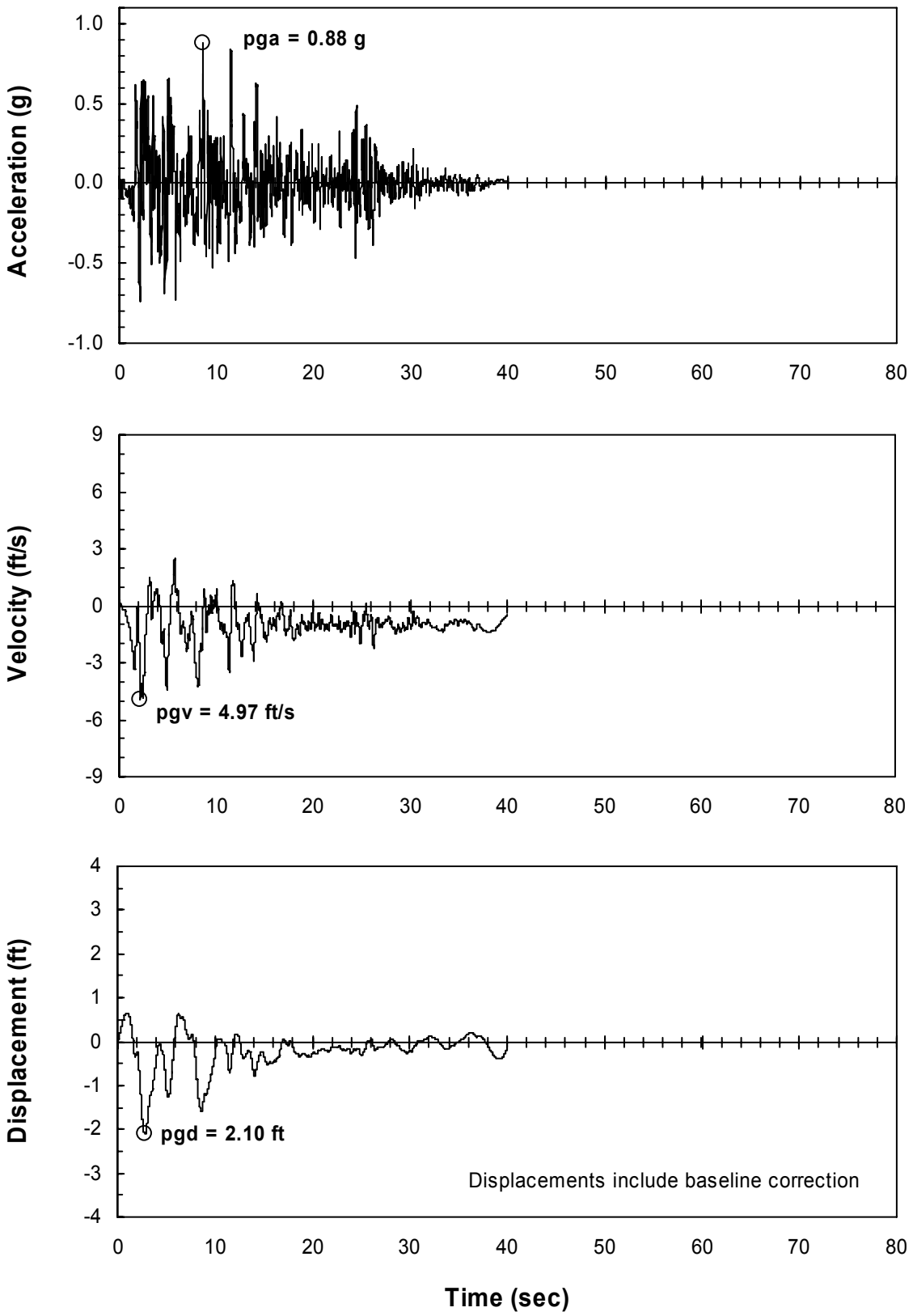


FIGURE H4-3-9
HISTORY PLOTS FOR ELC143 (CASE 2, EASTERN SHORE)

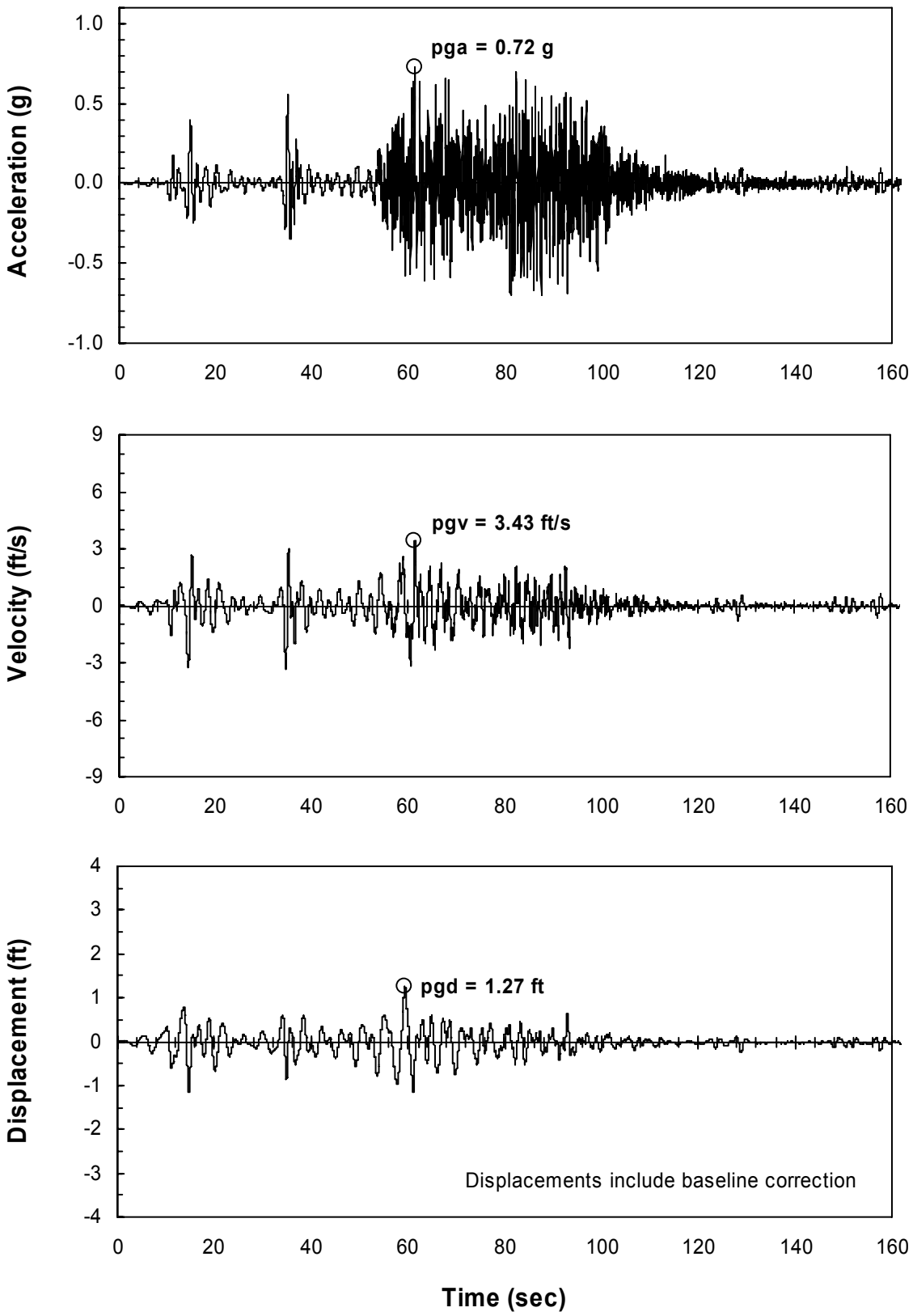


FIGURE H4-3-10
HISTORY PLOTS FOR R1300 (CASE 2, EASTERN SHORE)

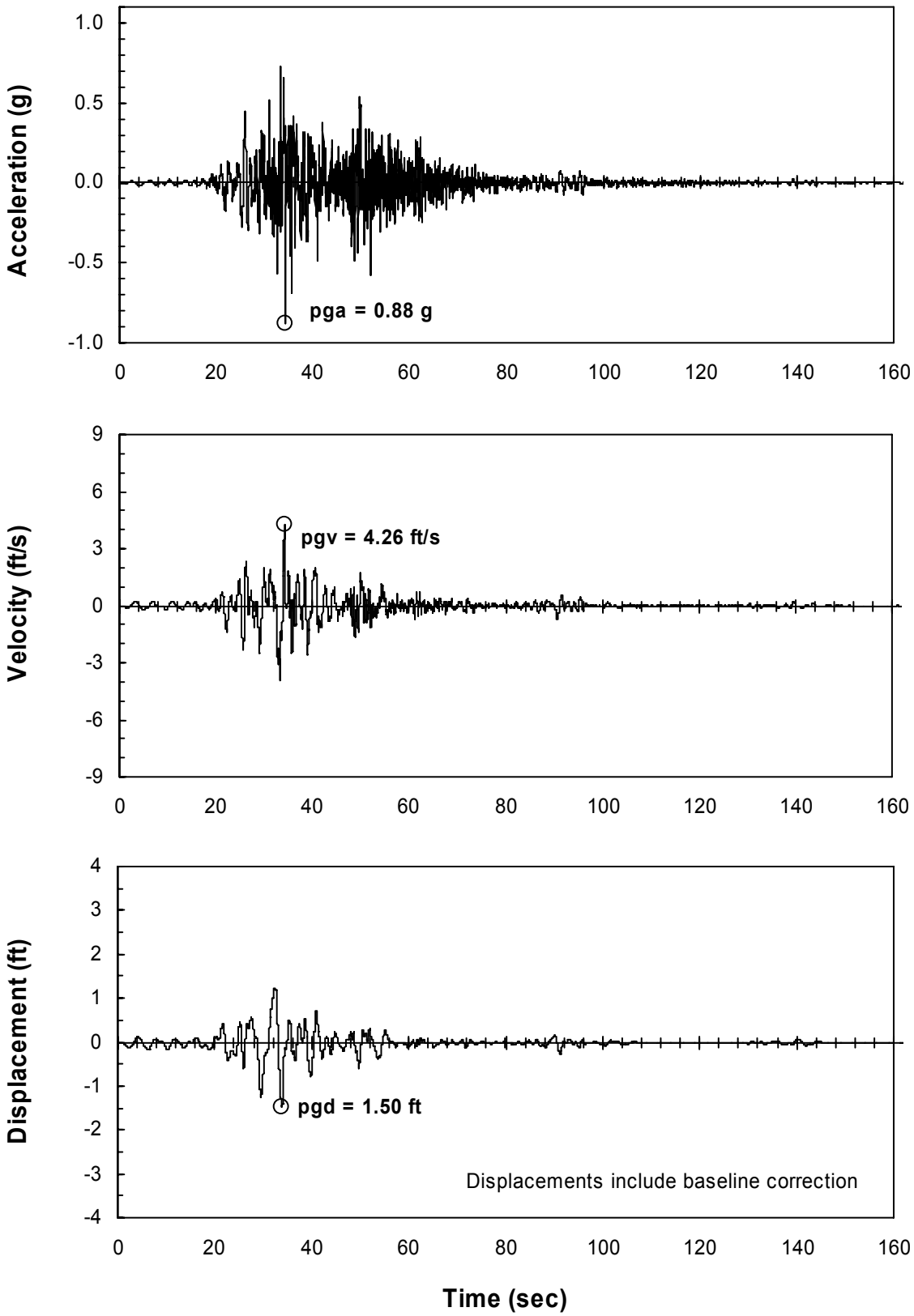
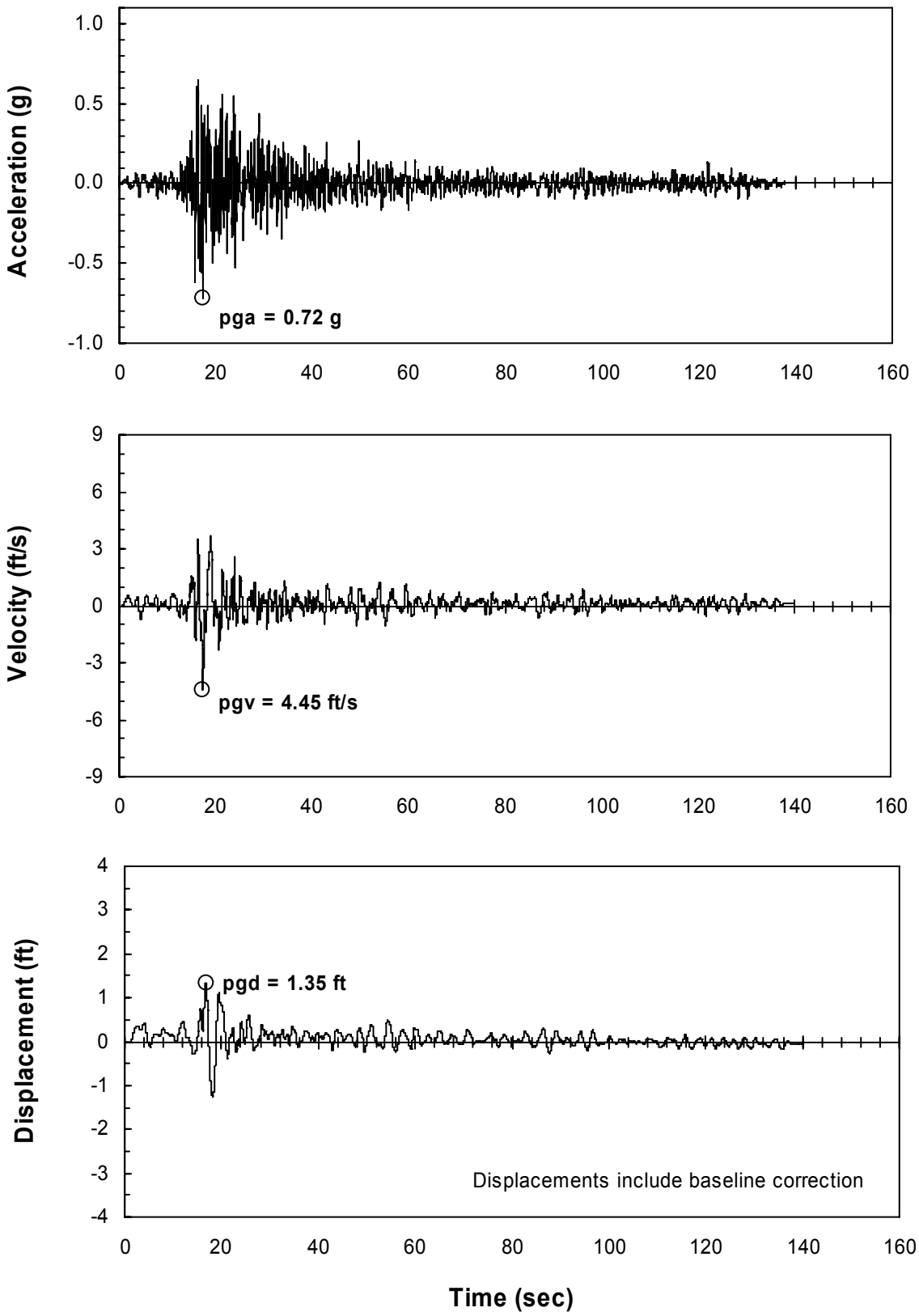
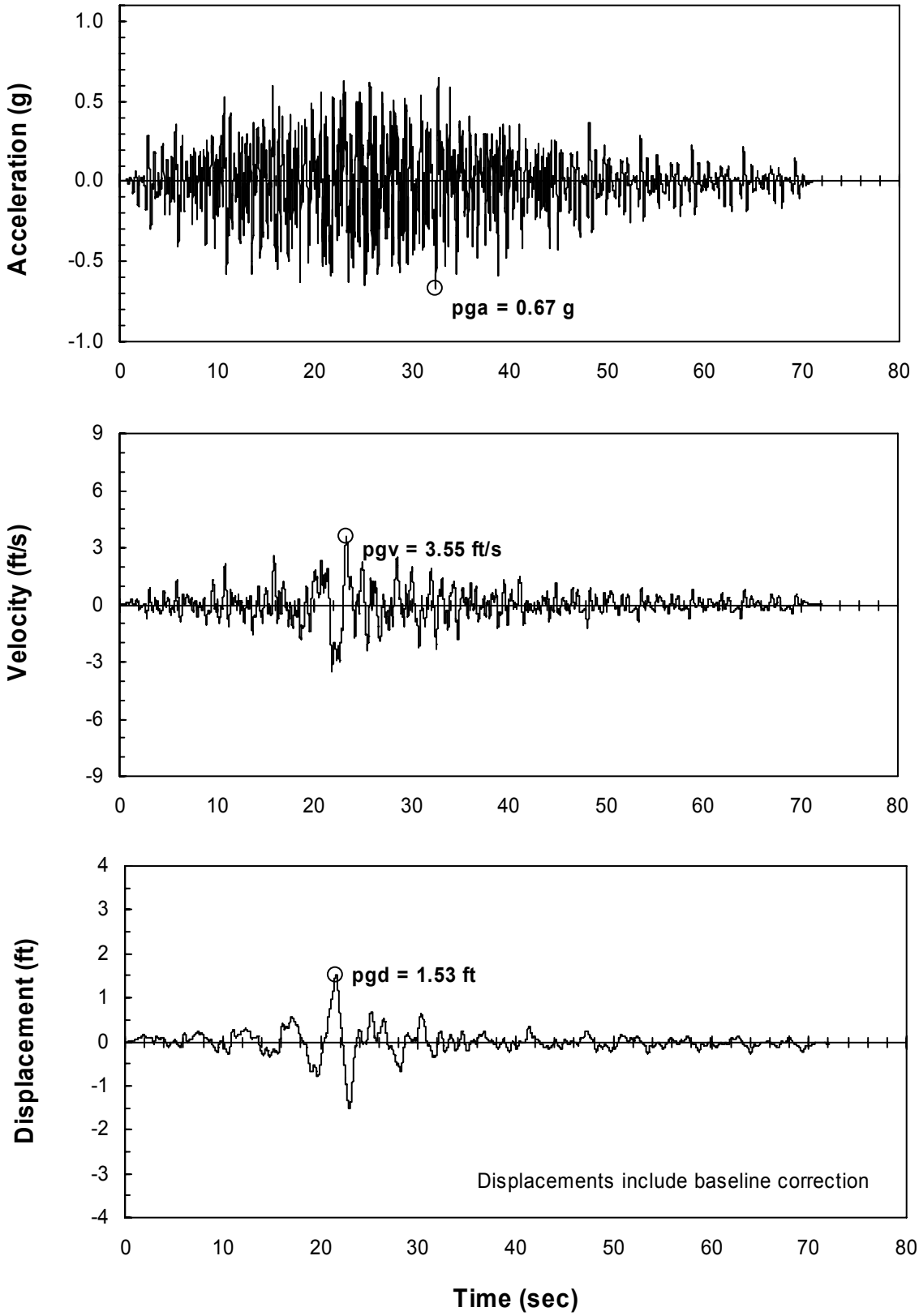


FIGURE H4-3-11
HISTORY PLOTS FOR R1500 (CASE 2, EASTERN SHORE)



X

FIGURE H4-3-12
HISTORY PLOTS FOR S1021M (CASE 2, EASTERN SHORE)



X

FIGURE H4-3-13
HISTORY PLOTS FOR SIM8 (CASE 2, EASTERN SHORE)

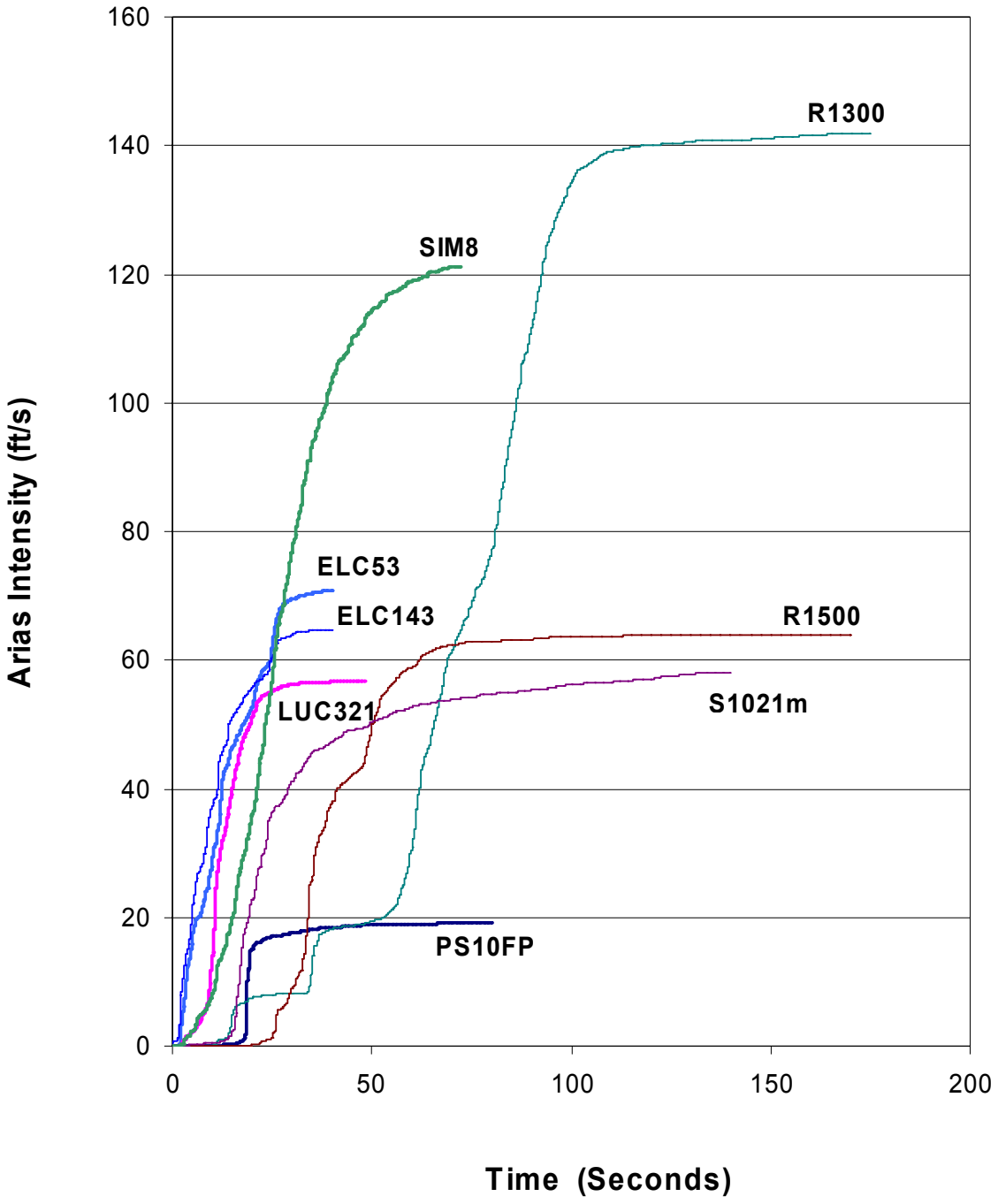
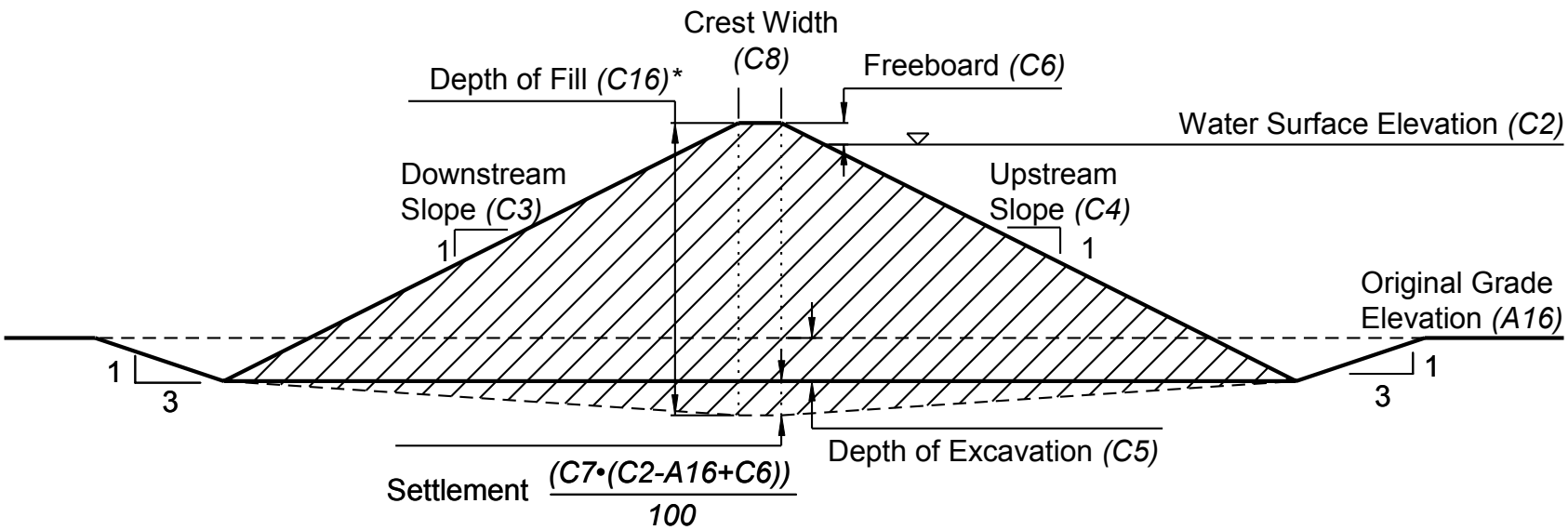


FIGURE H4-3-14
ARIAS INTENSITY PLOTS FOR CASE 1, EASTERN SHORE

ATTACHMENT H4-4

Volume Calculations

Barrier Volume Calculation



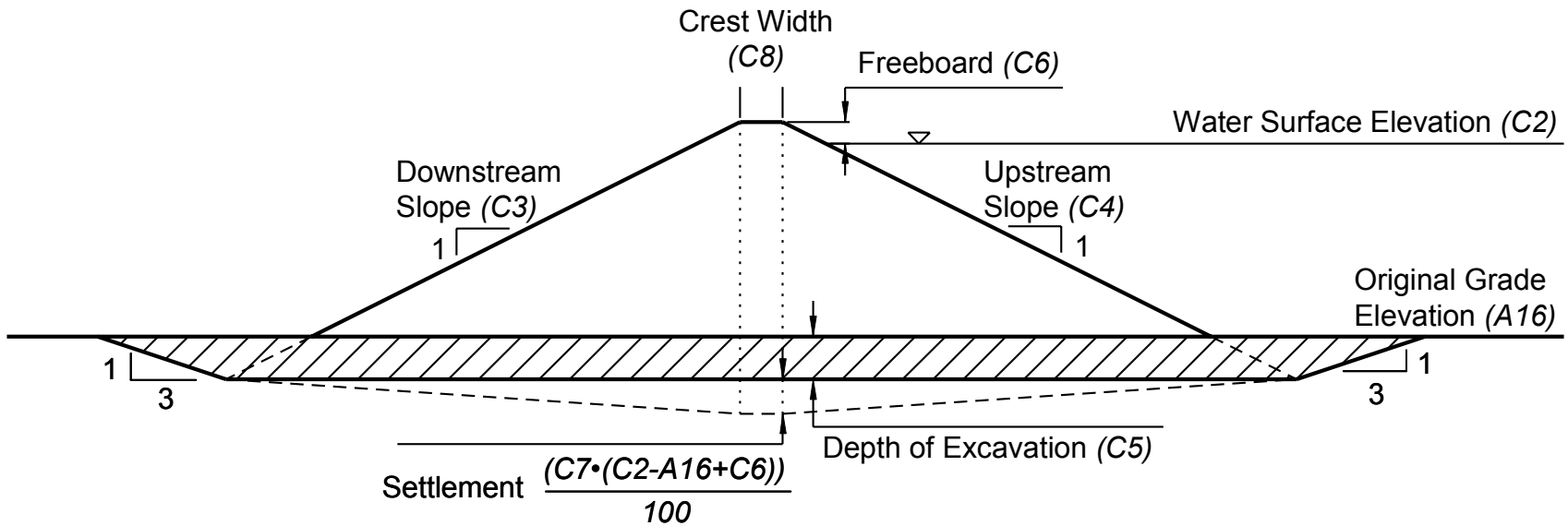
$$* C16 = ((C6 + C2 - A16) \bullet (1 + \frac{C7}{100}) + C5)$$

$$Area = \underbrace{[0.5 \bullet (C6 + C2 - A16 + C5) \bullet C3 \bullet C16]}_{\text{Downstream Slope Area}} + \underbrace{[C8 \bullet C16]}_{\text{Crest Width Area}} + \underbrace{[0.5 \bullet (C6 + C2 - A16 + C5) \bullet C4 \bullet C16]}_{\text{Upstream Slope Area}}$$

$$Volume = \frac{Area \bullet Length(B16)}{27 \frac{ft^3}{CY}}$$

Actual software equations may include **$** notation for absolute references

Excavation Volume Calculation



$$Area = \underbrace{\left[\left[(C5 + C6 + C2 - A16) \bullet C3\right] \bullet C5 + \frac{3 \bullet C5^2}{2}\right]}_{\text{Downstream Excavation Area}} + \underbrace{\left[C8 \bullet C5\right]}_{\text{Crest Width Excavation Area}} + \underbrace{\left[\left[(C5 + C6 + C2 - A16) \bullet C4\right] \bullet C5 + \frac{3 \bullet C5^2}{2}\right]}_{\text{Upstream Excavation Area}}$$

$$Volume = \frac{Area \bullet Length(B16)}{27 \frac{ft^3}{CY}}$$

Actual software equations may include **$** notation for absolute references