Fir Island Farm Final Design and Permitting
Draft Geotechnical Engineering Report
Skagit County, Washington

April 29, 2014

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Excellence. Innovation. Service. Value.

Since 1954.

Draft

Submitted To:
Mr. Brian Williams
Washington Department of Fish and Wildlife
111 Sherman Street
LaConner, Washington 98257

By:
Shannon & Wilson, Inc.
400 N 34th Street, Suite 100
Seattle, Washington 98103

21-1-12318-224

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
ALASKA • CALIFORNIA • COLORADO • FLORIDA • MISSOURI • OREGON • WASHINGTON • WISCONSIN

400 N. 34TH STREET, SUITE 100
P.O. BOX 300303
SEATTLE, WASHINGTON 98103
206•632•8020 FAX 206•695•6777

TRANSMITTAL

TO	Mr. Brian Williams	DATE	April 30, 2014
COMPANY	Washington Department of Fish and Wildlife	PHONE	
ADDRESS	111 Sherman Street	FAX	
	LaConner, Washington 98257	JOB NO.	21-1-12318-224
SUBJECT	**Fir Island Farm - Draft Geotechnical Engineering Report**		

THE FOLLOWING ITEMS ARE TRANSMITTED:

DATE	NO. COPIES	DESCRIPTION
4/29/2014	10 PAPER COPIES	FIR ISLAND FARM FINAL DESIGN AND PERMITTING, DRAFT GEOTECHNICAL ENGINEERING REPORT, SKAGIT COUNTY, WASHINGTON

☒ Per your request ☐ For your approval ☐ For your information ☒ For your files
☒ For your review ☐ For your action ☒ Return with comments ☐ Other

By: Brian S. Reznick, P.E. c: _____

Title: Senior Principal Engineer _____

Transmittal/wp/KPF

TABLE OF CONTENTS

Page

1.0 INTRODUCTION 1

2.0 PREVIOUS REPORTS AND STUDIES 1

3.0 SITE AND PROJECT DESCRIPTION 1

4.0 SCOPE OF SERVICES 2

5.0 DIKE DESIGN STANDARDS 3

6.0 SUBSURFACE CONDITIONS 4
- 6.1 Subsurface Explorations 4
- 6.2 In Situ and Laboratory Testing 4
- 6.3 Subsurface Soil Conditions and Geologic Units 5
- 6.4 Hydrogeologic Conditions 6

7.0 ENGINEERING RECOMMENDATIONS AND CONCLUSIONS 6
- 7.1 Dike Section 6
- 7.2 Fill Material 7
  - 7.2.1 Dike Select Fill 7
  - 7.2.2 Common Fill 7
  - 7.2.3 Topsoil 8
- 7.3 Dike Analyses 8
  - 7.3.1 Seepage Analysis Results (Exit Gradients) 9
  - 7.3.2 Stability Analysis Results 9
  - 7.3.3 Settlement Results 10
- 7.4 Liquefaction Analysis 10
  - 7.4.1 Ground Motions 11
  - 7.4.2 Liquefaction Analysis Results 11
  - 7.4.3 Potential Liquefaction-induced Risks 11
  - 7.4.4 Liquefaction Mitigation Measures 12
- 7.5 Geosynthetic Basal Reinforcement 12
- 7.6 Pipe Crossings 13
  - 7.6.1 Tide Gate and Outfall Pipes 13
  - 7.6.2 Seepage and Piping Mitigation 13
- 7.7 Storage Pond 14
- 7.8 Riprap Design 14
- 7.9 Filter Design 14
- 7.10 Temporary Access Roads 15

Draft

TABLE OF CONTENTS (cont.)

Page

7.10.1 Landside Access Roads 15
7.10.2 Waterside Access Roads 16

8.0 CONSTRUCTION CONSIDERATIONS 17
8.1 Site Preparation and Grading 17
8.2 Reuse of On-site Soil 17
8.3 Fill Placement and Compaction 18
8.4 Utilities 18
8.5 Geosynthetic Installation 18
8.6 Temporary Excavation Slopes 19
8.7 Temporary Surface Water and Groundwater Control 20
8.8 Wet Weather and Wet Condition Considerations 21

9.0 ADDITIONAL SERVICES 21

10.0 LIMITATIONS 22

11.0 REFERENCES 25

Draft

TABLES

1 Seepage Analysis (Exit Gradients) Summary
2 Global Stability Analysis Summary

FIGURES

1 Site and Exploration Plan
2 Geologic Profile Legend and Notes
3 Generalized Subsurface Profile A-A'
4 Generalized Subsurface Profile B-B' (2 sheets)
5 Generalized Subsurface Profile C-C'
6 Typical Dike Section
7 Filter Diaphragm

TABLE OF CONTENTS (cont.)

APPENDICES

A Slope Stability and Seepage Analyses and Results
B Liquefaction Analyses and Results
C Important Information About Your Geotechnical/Environmental Report

Draft

FIR ISLAND FARM FINAL DESIGN AND PERMITTING
DRAFT GEOTECHNICAL ENGINEERING REPORT
SKAGIT COUNTY, WASHINGTON

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering studies for the Fir Island Farm restoration project in Skagit County, Washington. The purpose of this study is to summarize our geotechnical engineering analyses, present our results and conclusions, and provide geotechnical recommendations for the design and construction of the setback dike.

Our services were conducted in general accordance with the contract and scope of services stated in the Washington State Department of Fish and Wildlife (WDFW) Contract No. 10-1431, Amendment No. 7, dated May 1, 2013.

2.0 PREVIOUS REPORTS AND STUDIES

In addition to the geotechnical engineering studies presented in this report, our services include preparing a geotechnical and hydrogeologic data report, performing coastal engineering analyses, and evaluating interior drainage for the project. Shannon & Wilson, Inc. (Shannon & Wilson) prepared the following reports for WDFW under our current contract:

- September 11, 2013 – Draft Geotechnical and Hydrogeologic Data Report
  This report summarizes current and past subsurface explorations, hydrogeologic studies, geotechnical laboratory testing, and site geology (Shannon & Wilson, 2013).
- January 10, 2014 – Draft Coastal Engineering Report
  This report summarizes our hydrodynamic, erosion and sediment analyses. Design recommendations for dike crest height, erosion protection, and scour mitigation are included in the report (Shannon & Wilson, 2014a).
- February 26, 2014 - Draft Interior Drainage Engineering Report
  This report summarizes the drainage basin, hydrologic monitoring, and our seepage and surface water analyses for the project. Design alternatives to mitigate anticipated interior drainage effects are identified for further study (Shannon & Wilson, 2014b).

3.0 SITE AND PROJECT DESCRIPTION

The Fir Island Farm restoration project is located on the WDFW Snow Goose Reserve at Fir Island. The project site is located south of Fir Island Road, approximately 3.1 miles west of Conway, Washington (Figure 1). The WDFW Snow Goose Reserve is comprised of 245 acres

that are currently farmed with special farming provisions to meet the reserve's snow goose management objectives. The reserve has four major interior drainage areas: Brown Slough – located west of the project site, Claude O. Davis and No Name Slough – located in the project site, and Dry Slough – located east of the project site.

Natural tidal exchange to the site has been eliminated by the historic construction of coastal dikes along Skagit Bay. Brown, No Name, and Dry Sloughs each have two, 48-inch-diameter top-hinge flap gates through the existing coastal dikes that provide interior farm drainage and block the inflow of tidal water to the farms. Brown Slough also has a 48-inch-diameter tide gate for drainage west of the Fir Island Farm parking area.

The project will reclaim approximately 127 acres of tidal marsh land and restore the tidal process to the site. The project will include:

- Breaching the existing coastal dike along Skagit Bay to restore tidal influences.
- Using the breach material to fill existing channels adjacent to the coastal dike.
- Raising the existing coastal dike near Brown Slough to a design crest elevation of +15 feet (North American Vertical Datum of 1988 [NAVD88]).
- Constructing a new setback dike. The new dike will extend about 3,200 feet to the north from Skagit Bay on the west side of Dry Slough. It will then turn east, extending about 2,150 feet to an existing parking lot. After which, the setback dike will turn south for about 500 feet and connect with the raised coastal dike. The new dike will have a design crest elevation of +15 feet (NAVD88).
- Dredging and expanding historical tidal channels and excavating marsh pilot channels to enhance restoration and limit adverse erosion effects.
- Constructing a storage pond and pump station north of the new setback dike to facilitate drainage from behind the new dike.
- Replacing two tide gates on No Name Slough and installing one tide gate and two slide gates on Dry Slough to facilitate drainage from the new storage pond and the existing sloughs.

The approximate locations of these elements are indicated in Figure 1, Site and Exploration Plan.

4.0 SCOPE OF SERVICES

As part of the 60 percent design submittal, our scope of services includes evaluating the geotechnical aspects for the setback dike design. Specific tasks include:

Draft

- Recommending materials for dike construction.
- Evaluating groundwater seepage through and beneath the setback dike.
- Evaluating stability of the setback dike for design conditions.
- Estimating settlement of the subgrade beneath the setback dike.
- Identifying potential geologic hazards for the site, their impacts to the setback dike, and potential mitigation strategies for these hazards.
- Evaluating the earthquake ground motions at the site and the potential for liquefaction to occur during a design-level earthquake.
- Evaluating filter design between the dike and riprap erosion protection.
- Evaluating temporary haul road sections for constructing the setback dike and for accessing the existing coastal dike.
- Providing design recommendations for utilities crossing under the dike.
- Providing dike construction considerations.

5.0 DIKE DESIGN STANDARDS

The Fir Island Farm dike evaluations and recommendations in this report follow the applicable U.S. Department of Agriculture (USDA) and U.S. Army Corps of Engineers (USACE) practice standards and engineering manuals for design and construction of dikes. The following manuals and guidelines were used for this project:

- USDA Natural Resources Conservation Service (NRCS), 2008, National Engineering Handbook, Section 16 “Drainage of Agricultural Land”
- USDA NRCS, WA, 2001, Conservation Practice Standard “Dike Code 356”
- USDA NRCS, 2007, National Engineering Handbook, Part 628 Dams, Chapter 45 “Filter Diaphragms”
- USACE, 2000, EM 1110-2-1913 “Design and Construction of Levees”
- USACE, 2003, EM 1110-2-1902 “Slope Stability”
- USACE, 2005, ETL 1110-2-569 “Design Guidance for Levee Underseepage”
- USACE, 1995, ER 1110-2-1806 “Earthquake Design and Evaluation for Civil Works Projects”

Draft

6.0 SUBSURFACE CONDITIONS

6.1 Subsurface Explorations

The results of the subsurface explorations for this project are presented in the Draft Geotechnical and Hydrogeologic Data Report (Shannon & Wilson, 2013) and include:

- Nine borings, designated B-1-13 through B-9-13, drilled along the proposed dike footprint and north of the proposed dike. Boring depths ranged from about 3 to 80 feet. Groundwater observation wells were installed in select borings and are designated with a "w" (e.g., B-4w-13).
- Twelve geoprobes, designated GP-01-10 through GP-12-10, pushed along the proposed dike footprint, near the existing drainage areas north of the proposed dike, and in the future restored tidal marshland south of the proposed dike. Geoprobe depths ranged from about 30 to 40 feet.
- Eight cone penetrometer tests (CPTs), designated CPT-1-13 through CPT-8-13, advanced along the proposed dike footprint. Advancement depths were about 50 feet.
- Thirty-seven test pits, designated TP-1-13 through TP-37-13, excavated along the proposed dike footprint, the proposed interior drainage pond, future marsh pilot channels, and in the existing dike. Excavation depths ranged from about 3 to 11 feet.

The approximate locations of the subsurface explorations are shown in Figure 1.

6.2 In Situ and Laboratory Testing

The results of the in situ and laboratory testing for this project are presented in the Draft Geotechnical and Hydrogeologic Data Report (Shannon & Wilson, 2013) and include:

- Seventy-three (73) Standard Penetration Tests (SPTs).
- Two hundred twenty-one (221) water content determinations
- Eight (8) fines content, 45 gradation (sieve only), and 17 combined gradation (sieve and hydrometer) tests.
- Seventeen (17) Atterberg limits tests.
- Four organic content tests.
- Two consolidation tests.
- Two consolidated-undrained triaxial compression tests
- Three compaction tests

Draft

- Twelve (12) falling head and 12 rising head slug tests performed in wells.
- Fourteen CPT pore pressure dissipation.

6.3 Subsurface Soil Conditions and Geologic Units

Our interpretation of the subsurface soil and groundwater conditions is based on the borings, geoprobes, CPTs, and test pits performed for this project. We interpret five soil units: dike fill (Hf), an agricultural layer, Holocene mudflat deposits (Hm), Holocene alluvium (Ha), and Holocene estuarine deposits (He). A general description of the subsurface conditions is presented below. A generalized subsurface profile of the soil units and groundwater encountered along the proposed north and east dike alignments is included as Figures 3 and 4, respectively. A generalized subsurface profile cross-section through the proposed dike and interior drainage pond is included as Figure 5. The generalized subsurface profile legend and notes are shown in Figure 2.

Dike fill (Hf) at the site primarily consists of locally derived, near-surface soil that was pushed into place during dike construction. Hf generally consists of a ½- to 2-foot-thick medium dense to dense, silty, sandy gravel layer with cobbles underlain by a 5- to 7-foot-thick loose or soft silt layer with variable amounts of clay and sand with iron-oxide staining. Test pits TP-19-13, TP-20-13, TP-22-13, and TP-26-13 encountered sand layers with variable amounts of silt. Portions of the dike have been repaired using imported riprap, sand, and gravel.

An agricultural layer is present across most of the site. The agricultural layer consists of 1 to 4 feet of soft to stiff, slightly clayey to clayey silt with a disturbed texture. The disturbance is likely the result of farming practices, such as tilling. Dike district managers report and historic photographs show that former Skagit River distributary channels were filled at the site along the margins of Brown, Claude O. Davis and Dry Sloughs.

The agricultural layer is underlain by a 1- to 7-foot-thick layer of very soft to soft, slightly clayey to clayey silt and very loose to loose, fine sandy silt, locally interbedded with layers of silty, fine sand. In this report, we assign the geologic unit Holocene mudflat (Hm) to these deposits. Test pits encountered layers and pockets of organic silt within Hm layers that are composed primarily of fibrous organics. Hm represents deposits that accumulated from a combination of tidal flat, estuarine, and alluvial overbank deposition. The unit is analogous to the Quaternary nearshore (Qn) unit of Dragovich and others (2002).

Holocene alluvium (Ha) underlies Hm. Ha typically consists of very loose to medium dense, clean to slightly silty sand. Locally, the deposits contain layers of silty sand, scattered shell

Draft

fragments, and wood debris. The dark gray color of the sand suggests its origin is largely volcanic, and that the Ha may be derived from Glacier Peak lahars. The explorations typically encountered Ha layers below about elevation 0 to 5 feet. Below about elevation -10 to -20 feet, Ha layers are interbedded with underlying estuarine deposits.

Holocene estuarine (He) deposits form layers of very loose silt, sandy silt, and silty sand, and very soft, slightly clayey to clayey silt that are interlayered with Ha. The explorations encountered interbedded He and Ha typically between elevations -10 and -40 feet. We observed shells and shell fragments in some He layers.

6.4 Hydrogeologic Conditions

Our interpretations of the hydrogeologic conditions at the site are based on our geologic interpretation, groundwater level observations, and in situ testing. The hydrogeologic conditions include three layers: an overlying confining layer (Hm), a confined aquifer (Ha), and the base of the aquifer (He). The top of the Ha aquifer is between elevation 0 and 5 feet. The base of the aquifer at the contact between Ha and He is between elevation -10 and -20 feet. The Ha aquifer is continuous across the site.

Groundwater levels determined from the pore pressure measurements made while advancing the CPTs on June 19, 2014, ranged from about 2 to 5 feet below the ground surface. Groundwater levels measured in the observation wells on June 21 and 27, 2013, were 2.6 to 2.9 feet below grade for the wells further inland (B-4w-13 through B-7w-13) and 0.8 to 0.9 foot below grade for the wells closer to the bay (B-8w-13 and B-9w-13). The difference in the piezometric head between the wells installed in the Ha layer (B-4w-13, B-6w-13, and B-8w-13) and those installed in the Hm layer (B-5w-13, B-7w-13, and B-9w-13) was less than 0.3 foot for the paired wells.

The groundwater levels in the Ha layer were consistently at a higher elevation compared to the Hm layer, which shows the Ha layer is confined. The groundwater elevation at the project site and the piezometric head in different soil units are influenced by the season and bay water levels.

7.0 ENGINEERING RECOMMENDATIONS AND CONCLUSIONS

7.1 Dike Section

A typical dike section is provided in Figure 6. This figure provides our recommendations for the proposed setback dike dimensions, geometries, and material. The recommendations are based on

Draft

our understanding of the project, our analyses results, and our experience with similar dike setback projects.

A basal reinforcement geosynthetic is recommended beneath the dike and access roads. Installing a geosynthetic will help facilitate dike construction, and will improve stability to meet USACE global stability factors of safety (FS) requirements. A geosynthetic will aid in subgrade stabilization for haul route operations and for future dike system maintenance activities. Design information for the basal reinforcement geosynthetic is discussed later in this report.

7.2 Fill Material

7.2.1 Dike Select Fill

Dike Select Fill should be well-graded soil with a minimum 25 percent passing the No. 200 sieve, and be free of organic and deleterious materials. Dike fill in contact with the basal reinforcement geosynthetic should have a maximum particle size of 1¼ inch. We recommend that dike fill placed elsewhere not exceed a particle size of 3 inches.

Soil with a fines content equal to or greater than 25 percent is sensitive to moisture content at the time of compaction. We recommend that soil delivered to the site for use as dike fill be within 2 percent of the optimum moisture content when delivered to the site, so that the soil can be placed and compacted with minimal on-site processing. Soil stockpiled on-site should be maintained within 2 percent of the optimum moisture content for compaction.

We recommend the aggregate for the dike road surfacing be crushed surfacing base course (CSBC) meeting the criteria defined in Section 9-03.9(3) of the 2014 Washington State Department of Transportation (WSDOT) Standard Specifications (WSDOT, 2014).

7.2.2 Common Fill

Excavated soil that does not meet the requirements of Dike Select Fill could be used as Common Fill for grading in the marshland on the waterside of the setback dike. Common Fill likely will be moisture-sensitive and could prove difficult to handle and place during wet weather or wet conditions. Moisture conditioning of the Common Fill is not anticipated, as compaction requirements for fill used in the grading of the marshland will be limited to constructing a stable grade and applying several passes with a roller.

Draft

7.2.3 Topsoil

Topsoil placed on the dike slopes, above the dike fill and riprap, should meet the requirements of WSDOT Standard Specification 9-14.1(2) for Topsoil Type B (WSDOT, 2014). We anticipate topsoil stripped from the site, and peat layers if encountered, may be suitable for reuse as topsoil at the site, provided that the material does not contain contaminants to a degree that precludes its use for this application.

7.3 Dike Analyses

Three dike cross sections were selected for seepage and stability analyses. One section was located at the proposed tide gates at No Name Slough (approximate Station 21+00). The remaining two cross sections were selected to represent typical soil conditions along the dike alignment, differing dike geometries, anticipated scour, and proximity to tidal channels or interior drainage features. These two sections are located at approximate Stations of 34+15 (Pond) and 48+90 (Dry Slough). The approximate dike design height and base widths for each cross section are summarized in the table below:

Selected Dike Cross Sections			
Cross Section Designation	**Approximate Dike Station**	**Dike Design Height, H[1] (ft)**	**Dike Base Width (ft)**
Tide Gates	21+00	13	80
Pond	34+15	9	60
Dry Slough	48+90	9	60

Notes:
[1] Dike design crest elevation is +15.0 feet (North American Vertical Datum of 1988). Dike design height is based on surveyed existing ground surface and design crest elevation.
ft = feet

Based on USACE guidelines, the following conditions were evaluated for each of the three dike cross sections:

- Case 1 – End of Construction
- Case 2 –Storm Drawdown
- Case 3 – Daily Tidal Drawdown
- Case 4 – Interior Flooding

Results of our dike global stability and seepage analyses are discussed in the following sections. The methodology and supporting documentation for the analyses are summarized in

Appendix A. The model geometries for the different cases are shown as global stability analysis output figures, and included as Figures A-1 through A-24 in Appendix A.

We did not perform seismic and post-seismic stability analyses for the proposed setback dike. Dikes are typically not designed to resist seismic loads due to the low probability of an earthquake coinciding with periods of high water (USACE, 2000). We did perform liquefaction analyses on the dike foundation soil. This analyses and the associated risks for liquefaction-induced hazards, is discussed in Section 7.4.

7.3.1 Seepage Analysis Results (Exit Gradients)

Upward exit hydraulic gradients, i_v, and seepage flow rates for steady state flow conditions during the design flood (Case 2), are summarized in Table 1. The USACE Technical Letter ETL 1110-2-569 *Design Guidance for Levee Underseepage* (USACE, 2005) recommends that dikes be designed to achieve a FS against piping (quick condition) of 1.6. The FS is equal to the critical gradient (i_c) divided by the calculated upward hydraulic gradient (i_v). We estimate that the critical gradient to cause a quick condition in the Agricultural Layer and Mudflat Deposits (Hm) is approximately 0.73. Therefore, the maximum i_v for a FS of 1.6 or greater at the dike toe, interior drainage pond, or sloughs during a design flood condition must be less than 0.73/1.6 = 0.46. The calculated i_v values were less than 0.46 for the dike design section shown in Figure 6.

7.3.2 Stability Analysis Results

Minimum FS values for each design case and each analysis cross section are summarized in Table 2, and described in Appendix A. Recommended minimum FS values (design criteria) presented in the USACE's Levee Design and Construction Manual EM 1110-2-1913 (USACE, 2000) and Slope Stability Manual EM 1110-2-1902 (USACE, 2003) for the design cases are shown in the bottom row of Table 2. Our analyses indicate that the dike design presented in Figure 6 satisfies the minimum recommended FS criteria for the cases identified. A basal reinforcement geosynthetic is required to meet global stability FS for Case 1 (End of Construction). Long-term global stability FS values (Cases 2 through 4) do not require the use of a geosynthetic. Specific recommendations regarding the reinforcement geosynthetic are provided in Section 7.5.

Draft

7.3.3 Settlement Results

We calculated settlement along the proposed dike using the commercial program Settle3D (Rocscience, Inc., Version 2.018, 2013). Settle3D calculates three-dimensional stresses and one-dimensional displacements of a subgrade due to applied surface loads.

Dike geometry (height, crest width, and slope angles) and subsurface soil (soil type and relative density) are factors that contribute to the magnitude and distribution of settlement along the length of the dike. We analyzed the "Pond" section, at approximate Station 34+15, and the "Dry Slough" section, at approximate Station 48+90, to characterize differing dike geometry and subsurface soil conditions. The subgrade soil beneath the proposed dike alignment is mostly granular. Therefore, we anticipate most of the settlement will be elastic and will occur essentially as the dike fill is placed and compacted. Our calculations show that the total settlement at the analyzed locations would range from about 5 to 7 inches. We estimate that approximately 90 percent of the settlement will occur during construction as the dike fill is placed, with less than 1 inch occurring between 1 and 20 years following construction.

In our opinion, settlement mitigation measures are not needed at the site.

7.4 Liquefaction Analysis

Liquefaction is a phenomenon which occurs in loose, saturated, mostly granular soil when the water pressure in the pore spaces increases to a level that is sufficient to separate the soil grains from each other. When a saturated soil experiences partial or full liquefaction, porewater pressure between the soil grains increases. This causes a reduction in the soil's effective stress, strength, and stiffness.

The liquefaction potential along the proposed setback dike alignment was evaluated based on the anticipated design life of the dike and the Operating Basis Earthquake, as described in the USACE ER 1110-2-1806, *Engineering and Design, Earthquake Design and Evaluation for Civil Works Projects* (USACE, 1995). A ground motion level corresponding to a 50 percent probability of exceedence in 100 years, or about a 144-year return period, was used in our analyses. The determination of the site ground motions and the results of the liquefaction analyses are discussed in the following sections. Plots of the FS against liquefaction versus depth and a discussion of the analysis methods are included in Appendix B.

Draft

7.4.1 Ground Motions

The mean and modal magnitude and soft rock peak ground acceleration (PGA) for the design ground motion level were determined based on results of the 2008 U.S. Geologic Survey (USGS) probabilistic seismic hazard analyses (Petersen and others, 2008). Based on the USGS interactive deaggregation and project location, we estimate a design mean and modal magnitude of 6.7 and 9.0, respectively, and a soft rock PGA of 0.14g. The mean magnitude represents a crustal earthquake, and the modal magnitude represents a subduction zone earthquake. Our liquefaction analyses evaluated both probable earthquake occurrences.

For design purposes, the soft rock PGA is modified for subsurface conditions within 100 feet of the ground surface. Based on the subsurface conditions encountered in the explorations, we recommend that the site be classified as Site Class E in accordance with the definition from the 2012 American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design Bridge Design Specifications (AASHTO, 2012). Based on a Site Class E, we recommend the design PGA be 0.21g.

7.4.2 Liquefaction Analysis Results

Our liquefaction analyses show that the alluvial deposits (Ha) beneath the dike footprint are potentially liquefiable under the design seismic ground motions. According to our liquefaction analyses of the CPT data, the upper estuarine deposits (He) contain scattered, potentially liquefiable seams and sublayers. Our liquefaction analyses of the SPT data indicate that the upper estuarine deposits, except for the sand seams, should not liquefy. We anticipate that for the design seismic event, the alluvial deposits will fully liquefy and that the upper estuarine deposits will undergo a loss of shear strength due to elevated porewater pressures, but will not fully liquefy.

7.4.3 Potential Liquefaction-induced Risks

Potential effects of liquefaction include settlement, a reduction in soil shear strength, and potential embankment instability or lateral spreading.

We estimate that liquefaction induced settlements of 2 to 24 inches could occur during a design-level earthquake. This amount of settlement could reduce the flood level of protection of the dike until the dike is built back to the design crest elevation.

A loss of shear strength below the dike would reduce the global stability FS, and possibly lead to localized global instability of the dike. Lateral spreading could occur along the north and

Draft

east parts of the dike where adjacent grades are lower due to the existing sloughs and a proposed storage pond. Liquefaction-induced hazards may occur over a small area or over hundreds of feet. The costs associated with such a failure could be great because repair could require complete replacement of the failed section.

7.4.4 Liquefaction Mitigation Measures

Mitigation alternatives to address seismic vulnerability could include one or more of the following:

- Do not increase the seismic resistance. Perform repairs as needed following an earthquake.
- Increase the strength of the basal reinforcement geosynthetic beneath the dike.
- Install pile foundations below the dike.
- Perform jet grouting or deep soil mixing to increase soil strength.
- Densify the alluvial deposits using vibratory techniques.

As the project progresses, we will work with WDFW to identify their selected mitigation alternative. In our experience, most dike owners select the first alternative and perform repairs as needed following an earthquake.

7.5 Geosynthetic Basal Reinforcement

To meet recommended End of Construction global stability FS values, we recommend a reinforcement geosynthetic be installed at the base of the dike. The minimum recommended short-term design strength (STDS) is provided in Table 2. The STDS is the strength required during fill placement and compaction. We recommend the STDS include reduction factors for construction damage and creep assuming 1 year of loading.

The reinforcement geosynthetic should be placed from dike toe to dike toe such that the machine direction is oriented perpendicular to the dike alignment, and each geotextile panel should be continuous with no seams or overlap. Adjacent reinforcement panels should overlap a minimum of 12 inches.

Draft

7.6 Pipe Crossings

7.6.1 Tide Gate and Outfall Pipes

We understand that two 48-inch-diameter pipes with tide gates (tide gate pipes) will be installed at No Name Slough, and three 12-inch-diameter pump station outfall pipes will be installed near the center one-third of the storage pond. The design invert for the tide gate pipes is at existing slough grade; 2.0 feet (NAVD88). The design invert for the outfall pipes is still under consideration, but will likely be located within the new dike.

Because the tide gate pipes are located in an existing slough, a cofferdam, localized dewatering, and the temporary rerouting of the slough may be required for construction. Additional evaluation of the construction options and the evaluation of dewatering and slough flow rates will be performed by us during dike plan preparation.

7.6.2 Seepage and Piping Mitigation

Utilities and utility backfill can create paths for seepage and piping beneath the dike. We understand that planned utilities crossing under or through the new or existing dike include the two tide gate pipes at No Name Slough and the three pump station outfall pipes near the center one-third of the storage pond.

Selection of proper backfill material for the pipe trenches is critical for long-term functionality of the dike system at this location. Preferential water flow pathways could develop through the trench backfill or along the pipe if the backfill is poorly compacted, develops cracks, consists of poorly graded or coarse material, or is more permeable than the surrounding dike material. This could lead to internal piping of the soil, which could erode soil around the pipe, compromise the integrity of the dike, and may eventually lead to a breach.

We recommend that the tide gate and outfall pipe trench backfill soil meet the criteria for Dike Select Fill. As recommended by the USDA NRCS, a filter diaphragm and an 18-inch annular thickness of drainage soil should be installed around the pipe on the landside of the dike, as shown in Figure 7. The trench backfill and filter soil should be compacted to at least 95 percent of its modified Proctor maximum dry density (ASTM International [ASTM] D1557) (ASTM, 2013).

Draft

7.7 Storage Pond

A storage pond is proposed on the landside of the new dike north segment (Figure 1). This pond will collect water from No Name Slough and its tributaries to the north and from the parking lot to the west. The water level in the pond will be controlled by the pump station and tide gates discharging to the marshland on the waterside of the dike. The bottom of the pond will be at elevation -1 foot (NAVD88). We recommend permanent cut slopes for the pond be 3H:1V or flatter.

As discussed in Section 7.3.1, transient tidal and storm flow exit hydraulic gradients from groundwater underseepage are sufficiently small that mitigation for piping (quick condition) along the pond lining is not required. Surface water runoff from seepage into the pond, wind waves on the pond, and regular drawdown of the pond water could lead to erosion along the pond slopes. To mitigate these effects, we recommend the pond slopes be protected from erosion using Best Management Practices, such as hydroseeding with a tackifier.

7.8 Riprap Design

The proposed dike will be subject to erosion and scour due to wind waves, tidal channel development, and impacts from floating large wood debris. To mitigate the effects of scour and erosion, riprap will be placed on the waterside face of the new dike. The proposed riprap will be 2 to 3 feet thick, extend 3 to 4 feet below the dike toe, and meet the gradation and particle weight requirements for Class VII riprap as defined in the 2009 Federal Highway Administration Hydraulic Engineering Circular No. 23. Further details about the riprap design are presented in the Draft Coastal Engineering Report (Shannon & Wilson, 2014a).

7.9 Filter Design

Where riprap is placed in contact with the finer-grained dike fill or native soil, groundwater flow between the riprap and underlying soil could cause soil movement and internal erosion. This soil movement and erosion could cause undermining and failure of the armoring and subgrade soil. To mitigate this, we recommend placing an aggregate filter (bedding layer) and separation geotextile between the riprap and underlying soil. The aggregate filter layer will serve to protect the underlying geotextile from damage during riprap placement and installation.

We recommend that the aggregate filter material be free of deleterious materials and have the following gradation:

Draft

Sieve Size	Percent Passing	
	Minimum	Maximum
16 inches	100	100
12 inches	80	100
8 inches	55	95
6 inches	35	85
4 inches	15	75

The aggregate filter should be at least 16 inches thick. If placed under water, we recommend the thickness be multiplied by 1.5 due to the uncertainty in measuring placed material below the water line.

A separation geotextile should be placed between the aggregate filter material and the finer-grained dike fill and native soil. We recommend the geotextile be a non-woven, high survivability material. It should meet or exceed the geotextile properties listed for a Permanent Erosion Control geotextile in Table 4 of Section 9-33.2(1) of the 2014 WSDOT Standard Specifications.

7.10 Temporary Access Roads

Temporary access roads will be built at the site to construct the project. These access roads will be used as pathways to haul material, transport construction equipment, and allow worker movement. The roads will be built on soft to medium stiff Agricultural and Hm layers. These soils are sensitive to weather and vehicle traffic, and will likely deteriorate with frequent use and when exposed to water. The following sections provide our recommendations for temporary access road construction on the land- and water-side of the new dike.

7.10.1 Landside Access Roads

The design of the landside access roads depends on the size of equipment being used, the axle loads, the number of passes, and the strength of the subgrade. For our analyses, we assumed 10,000 passes with a loaded on-highway haul truck, 70 pounds per square inch tire pressure, and 3 inches of allowable rut. Based on our analyses, we recommend the landside access roads be constructed with 24 inches of crushed rock supported on a geosynthetic placed directly over the existing subgrade. The access roads should be at least 15 feet wide at the crest and have 2H:1V or flatter side slopes.

Draft

The crushed rock should meet the gradation requirements of Section 9-03.9(3), CSBC, of the 2014 WSDOT Standard Specifications (WSDOT, 2014). In lieu of the 24 inches of CSBC, the crushed rock could consist of 6 inches of CSBC overlain with 18 inches of crushed ballast. The crushed ballast should meet the gradation requirements of Section 9-03.9(1) of the 2014 WSDOT Standard Specifications (WSDOT, 2014). The crushed rock should be placed in loose lifts not to exceeding 12 inches, and compacted to a minimum 95 percent of the modified Proctor maximum dry density (ASTM D 1557) with at least two passes of a heavy equipment compactor.

The geosynthetic used should be high survivability, separation geosynthetic to help facilitate the removal of temporary access roads where required. We recommend the geosynthetic have a minimum short term design tensile strength of 3,000 pounds per foot. Geosynthetic installation recommendations are include in Section 8.5.

The Contractor should be required to maintain the access roads throughout the life of the project. Because of subgrade soil strength variability, weather, and vehicle operations, ruts that develop may exceed 3 inches after repetitive use. These ruts should be filled-in periodically by adding CSBC or ballast to the rut, and not filled in by grading the roadway surface.

7.10.2 Waterside Access Roads

Crushed rock materials and geosynthetics used to construct access roads within the marsh areas are not aligned with the project restoration objectives, unless removed from the project area after construction. To reduce the potential volume of temporary rock placed in the marsh restoration areas, we recommend limitations be placed on construction access and that environmentally sensitive road material, such as hog-fuel and salvaged wood chips, be used where practical.

The required depth of hog-fuel placed over native subgrade to develop access roads is primarily a performance-based decision depending upon the Contractor's earthwork and hauling means and methods. This requirement should be made the Contractor's responsibility in the contract specifications. For bidding and probable cost comparison purposes, we recommend assuming that the depth of the hog-fuel placed for each waterside access road will be 4 feet and that each road will be 15 feet wide at the crest.

We recommend that the contract documents include provisions that make the Contractor responsible for all equipment access, hauling, removal of crushed rock and geosynthetic reinforcement, and restoration of damaged areas within the marsh area.

Draft

8.0 CONSTRUCTION CONSIDERATIONS

8.1 Site Preparation and Grading

Clearing and grubbing for the proposed dike should be done in accordance with Section 7-2, Foundation Preparation and Treatment, of the 2000 USACE EM 1110-2-1913, *Design and Construction of Levees* (USACE, 2000). Site preparation should commence by collecting and diverting surface water into storm drainage and/or treatment facilities. We anticipate that this work will include constructing temporary erosion and sedimentation control measures, and draining the ponded water on the site.

Following the demolition of existing structures where present, the ground should be cleared of trees, brush, and existing fill or debris. The area should then be grubbed of stumps and large roots, and stripped of the topsoil or underlying soil, which contains significant amounts of roots or other objectionable debris and organic material. We recommend assuming the average stripping depth will be 10 inches for cost estimating; however, stripping should occur to the depth needed to remove topsoil, sod, and roots greater than ½-inch-diameter. We recommend that organic-rich soil be stockpiled for later use as topsoil.

Following stripping, the exposed soil should be graded to a uniform, smooth surface. Soft, loose, or wet zones that inhibit construction of the basal reinforcement geosynthetic should be removed, reworked, and adequately compacted until a soil mass is produced that allows the installation of the geosynthetic.

8.2 Reuse of On-site Soil

We anticipate excavations for the dike subgrade preparation will consist primarily of stripping topsoil. Deeper excavations made for the storage pond, tide gate pipes, and riprap launch aprons will be made in the Agricultural layer and Mudflat deposits (Hm) and the upper Alluvium deposit (Ha). These excavations will be made mostly below the groundwater table; therefore, excavated soil will have high moisture content at the time they are excavated.

In our opinion, some of the Agricultural layer and Mudflat deposits could be suitable as Dike Select Fill. Layers with high organic content should be expected in the Agricultural layer and Mudflat deposits. These organic layers should be segregated and not used as Dike Select Fill if they have an organic content exceeding 1 percent by dry unit weight. The Contractor should be advised that the on-site soil will likely require moisture conditioning before placement and compaction. If the Contractor proposes to mix imported soil with on-site soil to provide Dike

Draft

Select Fill, the different soils should be thoroughly blended and moisture conditioned prior to placement on the dike, and the organic content of the blended material should not exceed 1 percent by dry unit weight. The Contractor should perform tests to show that the moisture content of the mixed soil is suitable for compaction.

8.3 Fill Placement and Compaction

We recommend that Dike Select Fill be compacted to a minimum 90 percent of the modified Proctor maximum dry density (ASTM D 1557). We recommend that the CSBC and crushed ballast for the access roads and the dike road surfacing be placed and compacted in accordance with Section 4-04 of the 2014 WSDOT Standard Specifications. The loose lift thickness for the fill before compaction should not exceed 8 inches with heavy equipment compactors and 4 inches for hand-operated compaction equipment.

All fill should be placed and compacted in uniform horizontal lifts. Where the new dike ties into the existing spur dike section to remain, the fill should be keyed into the slope by excavating a bench into the soil as recommended in Section 2-03.3(14), Embankment Construction, of the 2014 WSDOT Standard Specifications.

Topsoil should be placed in lift thicknesses of 6 inches or less and be hand rolled for compaction.

8.4 Utilities

We understand that the only utilities crossing under or through the new or existing dike will be the two tide gate pipes at No Name Slough, and the three pump station outfall pipes near the center one-third of the storage pond. However, we recommend that that the Contractor check with utility owners and collect as-built information in the work area prior to construction for confirmation. If other utilities are present, they should be relocated and/or specifically addressed where they could be affected by dike construction or could affect dike performance.

During backfill of the tide gate and outfall pipes, we recommend 2 feet of fill be placed above the pipe crown prior to using large compaction equipment. Proper equipment should be selected by the Contractor to prevent damage to the pipes during backfill placement and compaction.

8.5 Geosynthetic Installation

The Contractor should take care to protect the geosynthetic from damage during installation. Installation of the geosynthetic should be done in accordance with the manufacturer's recommendations. The Contractor should be responsible for selecting equipment and operations

Draft

that do not damage the geosynthetic. In general, we recommend a minimum 24 inches of fill be placed over the geosynthetic prior to wheeled construction equipment operating over it. Track rigs and rollers could operate above the geosynthetic with a minimum of 8 inches of fill placed over it.

The geosynthetic reinforcement layer under the proposed dike and access roads should be placed on top of the prepared subgrade with its machine direction perpendicular to the dike and access road alignment. It should be stretched tight using hand labor and held with stakes prior to placing backfill. Backfill should not be pushed onto the geosynthetic, but dumped from an excavator or loader bucket. To reduce damage, soil should not be dropped from greater than 3 feet. Fill may be spread after the geosynthetic is covered with a minimum of 8 inches of soil.

8.6 Temporary Excavation Slopes

Temporary excavation slopes should be the responsibility of the Contractor because the Contractor is responsible for its own means and methods, and is continuously at the site and able to observe the nature and conditions of the soil and groundwater encountered. All current and applicable safety regulations regarding excavation slopes and shoring should be followed. Because the proposed construction may require temporary excavations that differ from the geometries of the proposed project structures (e.g., storage pond and tide gate pipes), the contract documents should require a submittal in which the Contractor explains how it intends to construct those features.

For planning purposes, we recommend assuming that excavations below current grade will occur below the groundwater table. Temporary, unsupported, open-cut slopes excavated below the groundwater will depend on whether the excavations are:

- Dewatered such that seepage does not occur into the excavation or is greatly reduced,
- Dewatered using sumps during excavation such that seepage does occur, or
- Excavations are made in the wet without lowering the water level in the excavation below the groundwater table.

If the Contractor elects to dewater prior to excavating or makes the excavation in the wet, we anticipate temporary excavation slopes might be made no steeper than 1.75H:1V. Excavations that are not dewatered should be attempted only if:

Draft

- Backfill to be placed in the water is not settlement sensitive.
- The backfill does not need to be compacted. Densifying backfill below the water line is not practical unless the material is coarse.
- Precise line and grade control of the excavation is not required.

If the Contractor does not dewater prior to excavating and seepage into the excavation is removed using sumps, we anticipate temporary excavation slopes of 2.5H:1V or flatter could be required.

We recommend the Contractor excavate a test pit at the tide gate and pond locations prior to mass excavation to observe the existing subgrade conditions and assess appropriate cut slopes. Flatter cut slopes may be required where loose/soft soil or seepage is encountered or if wet weather conditions are present.

8.7 Temporary Surface Water and Groundwater Control

We recommend installing temporary surface and groundwater controls to provide proper drainage of excavations made for the proposed dike, pipeline crossings, storage pond, and other features associated with the project. In our opinion, temporary dewatering will be required to make relatively dry excavations at the site. Because of the high fines content in the upper agricultural layer and mudflat deposits, and a groundwater near the surface, we anticipate drainage from the soil will be poor. We expect the surficial soils to become saturated during rainstorms, resulting in overland flow. We also expect seeps and possible water pressure-induced instabilities along the floor of deeper excavations, and that performing these excavations will require advance dewatering, and special dewatering equipment and construction methods.

The Contractor is typically responsible for dewatering using their own means and methods. We recommend the construction contract include a performance-based temporary dewatering specification that includes submittal requirements that require the Contractor to demonstrate their understanding of the soils and groundwater conditions, and require the Contractor to develop a dewatering plan that meets the requirements of the construction specifications. We caution against using a prescriptive specification such as, “The Contractor shall fully dewater all excavation and fill areas to a minimum 1.0 foot below the soil surface,” as this simplified approach can cause several issues such as requiring dewatering over large areas that may not be necessary or difficult to enforce. We recommend the specifications include a requirement that the Contractor monitor performance of their dewatering system, have appropriate equipment and

backup systems, and submit daily reports on the dewatering system performance and groundwater conditions.

A Contractor's dewatering plan may include an array of dewatering provisions including:

- Drainage ditches, pipes and diversion structures used to intercept and redirect flow from construction areas
- Sumps and pumps
- Wells
- Wells points

The Contractor's dewatering plan should include provisions that are appropriately matched to the proposed construction features, soil and groundwater conditions, and construction methods. The dewatering plan should include a discussion on how the dewatering system will work and how the dewatering system will be operated including any treatment proposed to meet applicable permit and regulatory requirements. The specifications should require that the Contractor submit dewatering plans complete with supporting engineering calculations and analyses. The dewatering plan calculations and analyses should be performed and stamped by a Washington State licensed professional engineer or hydrogeologist.

8.8 Wet Weather and Wet Condition Considerations

In the project area, wet weather generally begins about mid-October and continues through about May, although rainy periods may occur at any time of year. The soil for the proposed dike embankment contains sufficient fines that will produce an unstable mixture when wet. Such soil is highly susceptible to changes in water content and tends to become difficult or impossible to compact if its moisture content significantly exceeds the optimum. In addition, during wet weather months, groundwater levels could rise, resulting in seepage into the project area. Performing earthwork during dry weather would reduce the problems and costs associated with rainwater, trafficability, and the handling of wet soil. We recommend earthwork be scheduled for the dry weather months of June through September. The contract documents should include provisions for wet weather/wet condition earthwork.

9.0 ADDITIONAL SERVICES

Geotechnical recommendations that are used as a basis for design are developed from a limited number of explorations and tests. Consequently, there may be a need for adjustment in the field; therefore, we recommend that Shannon & Wilson be retained to observe the geotechnical aspects

Draft

of the construction. Construction observation should include site excavation, coastal dike breaching, backfilling of drainage channels, dike embankment placement and compaction, utility/pipeline installation, tide gate/slide gate installation, pump station foundation subgrade preparation, erosion control, groundwater control, and quality assurance and testing. Construction observation would allow us to evaluate the subsurface conditions and dike fill as they are exposed and placed during construction, to make recommendations as needed, and to determine that the work is accomplished in accordance with our recommendations.

10.0 LIMITATIONS

This report was prepared for the exclusive use of WDFW and other members of the design team for specific application to the design of the Fir Island Farm restoration project as it relates to the geotechnical aspects discussed in this report. It should be made available to prospective contractors and/or the Contractor for information only, and not as a warranty of subsurface conditions.

The interpretations, analyses, conclusions, and recommendations contained in this report are based on our observation of site conditions as they existed during our site visits, and our interpretation of subsurface conditions based on explorations we performed; geologic and hydrogeologic data for the project site; and information provided to us and documents we reviewed describing the construction, maintenance, and operation history of the facilities evaluated. The professional opinions, recommendations, and conclusions contained in this report for the dike system are valid for a period not greater than 10 years from the date of this report. This time limitation is included in recognition that the conditions of dike systems can and do change with time as do the conditions that lead to water surface elevation determinations. If new information becomes available to Shannon & Wilson, such as the performance during a significant flood event, the professional opinions, recommendations, and conclusions contained in this report may be modified by Shannon & Wilson.

Our interpretation of existing conditions and analyses, and resulting conclusions and recommendations, rely on data provided by others, including, but not limited to, survey data, subsurface data, dike geometry information (plans and cross sections), dike system design and construction data, dike system maintenance and operation data, and design flood water surface elevations and hydrographs. Shannon & Wilson makes no warranty, express or implied, as to the accuracy of the data relied on. Within the limitations of the scope, schedule, and budget of this project, the analyses, conclusion, and recommendations presented in this report were prepared in

Draft

accordance with generally accepted professional engineering principles and practices in use in the area of the project at the time this report was prepared.

We assume that our interpretations of subsurface conditions are representative of subsurface conditions at the site. Unanticipated soil and groundwater conditions are commonly encountered and cannot be fully determined by taking soil samples, drilling test borings, or pushing probes. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs. If subsurface conditions different from our observations or interpretation are encountered or appear to be present, or if dike or dike facility performance appears to be different than we observed or interpreted from information provided to us, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary.

If there is a substantial lapse of time between the issuance of this report and start of construction at the site, or if conditions have changed due to natural causes or construction at or near the site, we recommend that site conditions and this report be reviewed to determine the applicability of the conclusions and recommendations.

Dike systems are a collection of components that must function as a complete, integrated system to be effective. It is not practical or possible to completely know all of the engineering properties of dikes and their foundations. Consequently, uncertainty exists as to actual dike system behavior and performance. Robust regular inspections and high water monitoring for dikes, floodwalls, appurtenances, and features should be performed. Any deficiency should be remediated as appropriate based on observed conditions, uncertainty, and potential consequences.

It must be understood that some seepage is normal and acceptable when water is elevated. Uses incompatible with this seepage should not be allowed in areas protected by dike systems. Excavations near or in dikes and floodwalls could compromise the dike system and should not be performed without proper engineering and construction controls. The potential impact of these excavations depends on many factors, including, but not limited to subsurface and groundwater conditions, excavation depth, distance from dike toe, dike geometry, and difference in elevation of water on the waterside of the dike and the excavation. Penetrations through and below dikes should be assessed individually because penetrations have the potential to produce rapid failures of dikes as they can provide a preferential seepage path or an open conveyance for water.

Draft

The scope of our services for this report did not include any assessment or evaluations regarding the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air on or below or around the site.

We have prepared the document, "Important Information About Your Geotechnical/ Environmental Report," as Appendix C to assist you and others in understanding the use and limitations of our report.

SHANNON & WILSON, INC.

Brian S. Reznick, P.E.
Senior Principal Engineer

BSR:MDH:DRC:CAR/bsr

Draft

11.0 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO), 2012, AASHTO guide specifications for LRFD seismic bridge design (with 2013 interim specifications): Washington, D.C., American Association of State Highway and Transportation Officials, 1 v.

ASTM International (ASTM), 2013, Annual book of standards, construction, v. 4.08, soil and rock (I): D420 - D5876: West Conshohocken, Penn., ASTM International, 1 v.

Dragovich, J.D., Gilbertson, L.A., Norman, D.K., Anderson, G., and Petro, G.T., 2002, Geologic map of the Utsalady and Conway quadrangles, Skagit, Snohomish, and Island Counties, Washington: Washington Division of Geology and Earth Resources, OF 2002-5, scale 1:24,000.

Petersen, M.D.; Frankel, A.D.; Harmsen, S.C.; and others, 2008, Documentation for the 2008 update of the national seismic hazard maps: U.S. Geological Survey Open-File Report 08-118, available: http://pubs.usgs.gov/of/2008/1128.

Rocscience Inc., 2013, Settlement modeling with Settle3D version 2.018.

Shannon & Wilson, Inc. (Shannon & Wilson), 2013, Fir island farm final design and permitting, draft geotechnical and hydrogeologic data report, Skagit County, Washington: Report prepared by Shannon & Wilson, Inc., Seattle, Wash., 21-1-12318-215, for Washington Department of Fish and Wildlife (WDFW), September 11.

Shannon & Wilson, Inc. (Shannon & Wilson), 2014a, Fir island farm final design and permitting, draft coastal engineering report, Skagit County, Washington: Report prepared by Shannon & Wilson, Inc., Seattle, Wash., 21-1-12318-217, for Washington Department of Fish and Wildlife (WDFW), January 10.

Shannon & Wilson, Inc. (Shannon & Wilson), 2014b, Fir island farm final design and permitting, draft interior drainage engineering report, Skagit County, Washington: Report prepared by Shannon & Wilson, Inc., Seattle, Wash., 21-1-12318-216, for Washington Department of Fish and Wildlife (WDFW), February 26.

U.S. Army Corps of Engineers (USACE), Engineering and Design, 1995, Earthquake Design and Evaluation for Civil Works Projects, Engineering Manual 1110-2-1806: Washington, D.C., July.

U.S. Army Corps of Engineers (USACE), Engineering and Design, 2000, Design and Construction of Levees, Engineering Manual 1110-2-1913: Washington, D.C., April.

Draft

U.S. Army Corps of Engineers (USACE), Engineering and Design, 2003, Slope Stability, Engineering Manual 1110-2-1902: Washington, D.C., October.

U.S. Army Corps of Engineers (USACE), Engineering and Design, 2005, Design Guidance for Levee Underseepage, Engineering Technical Letter 1110-2-569: Washington, D.C., May.

U.S. Army Corps of Engineers (USACE), Engineering and Design, 2005, Design Guidance for Levee Underseepage, Engineering Technical Letter 1110-2-569: Washington, D.C., May.

U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), 2001, Conservation Practice Standard, Dike Code 356, Washington: Washington, D.C, January

U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), 2007, National Engineering Handbook, Part 628 Dams, Chapter 45 – Filter Diaphragms: Washington, D.C, January.

U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), 2008, National Engineering Handbook, Section 16 Drainage of Agricultural Land, Chapter 6 – Dikes: Washington, D.C, May.

Washington State Department of Transportation (WSDOT), 2014, Standard specifications for road, bridge, and municipal construction: Olympia, Wash., WSDOT, Manual M 41-10, 1 v., January, available: http://www.wsdot.wa.gov/Publications/Manuals/M41-10.htm.

Draft

TABLE 1
SEEPAGE ANALYSIS (EXIT GRADIENTS) SUMMARY

Analysis Location	Analysis Geometry		Storm Drawdown (Case 2) Transient Seepage Analysis
	Dike Design Height, H[1] (ft)	Dike Base Width (ft)	i_V [2, 3] (ft/ft)
Tide Gate (Station 21+00)	13	80	0.10
Pond (Station 34+15)	9	60	0.35
Dry Slough (Station 48+90)	9	60	0.10

Notes:

[1] Design dike crest elevation is +15 feet (North American Vertical Datum of 1988). Dike design height is a function of existing ground surface elevation and the design crest elevation.

[2] U.S. Army Corps of Engineers Technical Letter ETL 1110-2-569 (2005) recommends that levees (dikes) should be designed to maintain a factor of safety against a quick (piping) condition of 1.6. Based on our estimation of the density for the Hm layer, this corresponds to a required maximum upward exit gradient (i_v) of 0.46.

[3] Exit gradients presented in this table occur at the proposed drainage pond, an existing slough, or the landside toe of the dike.

ft = feet

i_V = calculated upward hydraulic exit gradients

TABLE 2
GLOBAL STABILITY ANALYSIS SUMMARY

Analysis Location	Basal Reinforcement[1]	Factor of Safety Against Global Instability[8]			
	Short-term Strength, T_{STDS} (lb/ft)	Case 1: End of Construction[2,3]	Case 2: Storm Drawdown[4,5] (Transient)	Case 3: Daily Tidal Drawdown[4,6] (Transient)	Case 4: Interior Flooding[4,7] (Transient)
Tide Gate (Station 21+00)	3,000	1.3	1.5	1.6	1.5
Pond (Station 34+15)	3,000	1.3	1.4	1.5	1.4
Dry Slough (Station 48+90)	3,000	1.5	1.4	1.6	1.4
USACE Recommended FS	**-**	**1.3**	**1.0-1.2**	**1.4**	**1.0-1.2**

Notes:

[1] Geosynthetic basal reinforcement was included to improve stability during construction. Short-term strength includes 1 year creep and construction damage reduction factors.

[2] Assumes that it takes approximately three months or longer to construct the dike fill.

[3] Groundwater elevation in Case 1 assumed to be at the ground surface. No water in Pond. Water in Slough is at elevation 3.23 feet (ft).

[4] Global stability of rapid drawdown conditions (Cases 2, 3 and 4) was calculated using transient seepage method. The lowest factors of safety (FS) from waterside and landside dike slope stability are reported.

[5] In Case 2, water on waterside rises from elevation 3.1 ft to 15 ft in 3 hours 15 minutes, then falls from elevation 15 ft to 3.2 ft in 8 hours. Water in Pond/Slough is at elevation 3.23 ft.

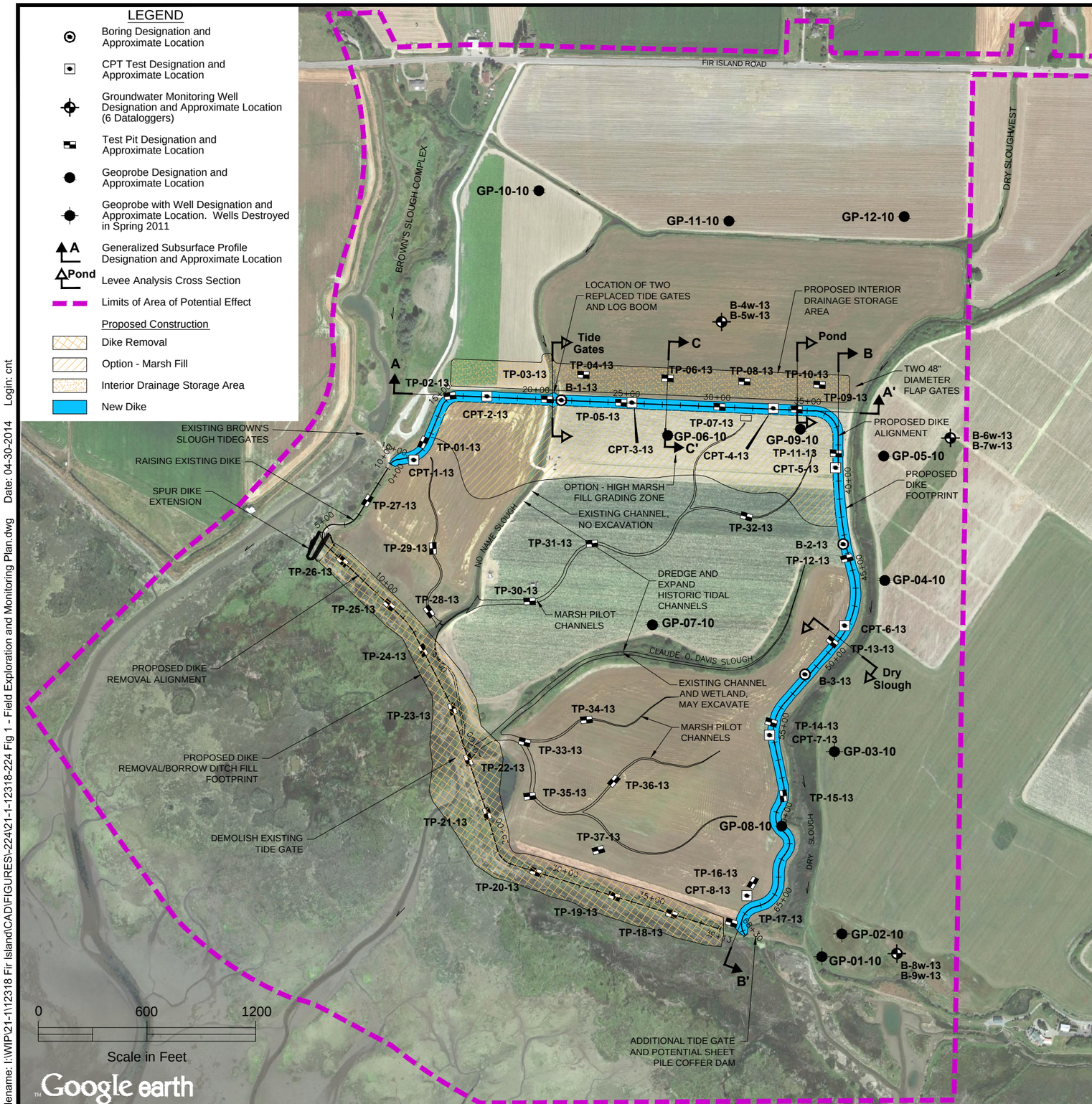
[6] In Case 3, water on waterside rises from elevation 3.1 ft to 9.3 ft in 3 hours 15 minutes, falls from elevation 9.3 ft to 3.2 ft in 8 hours, rises from elevation 3.2 ft to 6.7 ft in 3 hours 45 minutes, falls from elevation 6.7 ft to 3.2 ft in 5 hours 15 minutes, and then falls from 3.2 ft to 3.1 ft in 3 hours 45 minutes. Water in Pond/Slough is at elevation 3.23 ft.

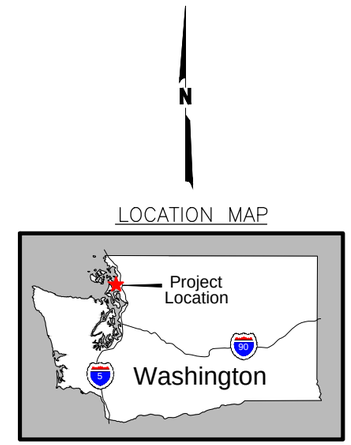
[7] In Case 4, water on waterside rises from elevation 3.1 ft to 15 ft in 3 hours 15 minutes, then falls from elevation 15 ft to 3.2 ft in 8 hours. Water on landside is at elevation 10 ft.

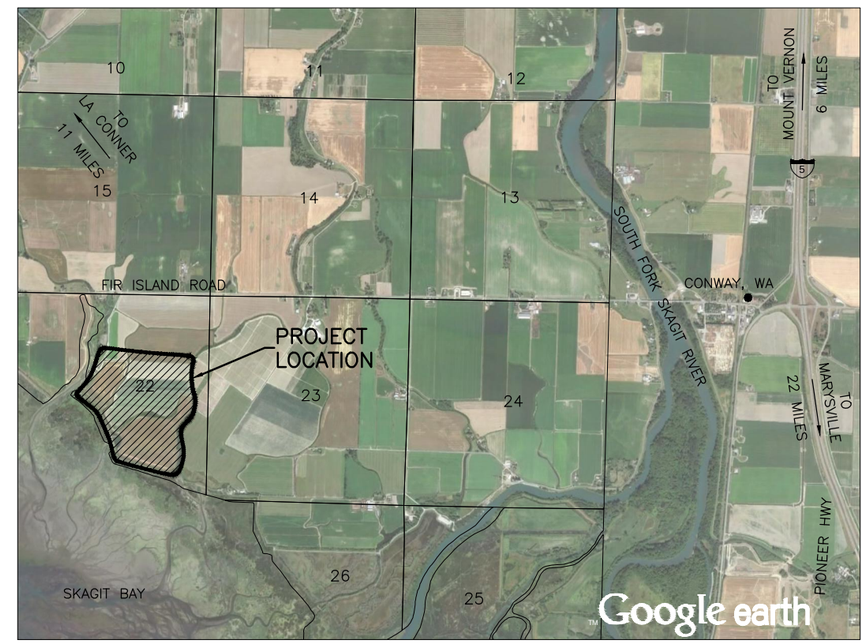
[8] FS from optimized non-circular failure surface analysis.

lb = pound

USACE = U.S. Army Corps of Engineers







VICINITY MAP

0 2000 4000

Scale in Feet

DRAFT

NOTE

This figure is adapted from *R004113c004.dwg*, dated December 2011. Aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth ™ Mapping Service.

Fir Island Farm
Final Design and Permitting
Skagit County, Washington

SITE AND EXPLORATION PLAN

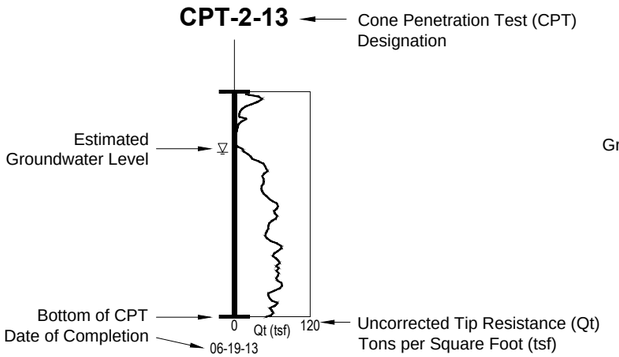
April 2014 | 21-1-12318-224

SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

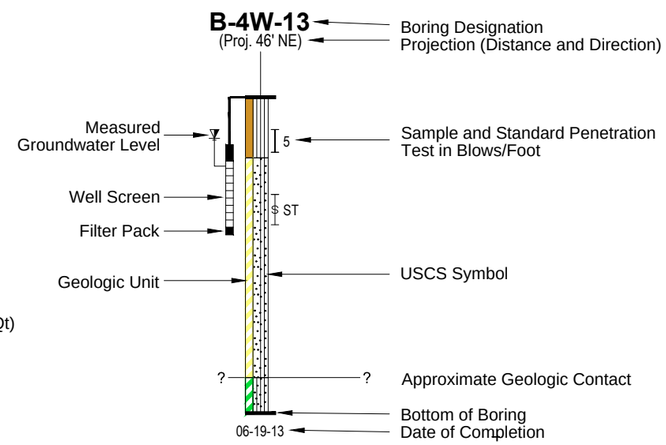
FIG. 1

Filename: I:\WIP\21-1\12318 Fir Island\CAD\FIGURES\-224\21-1-12318-224 Fig 1 - Field Exploration and Monitoring Plan.dwg Date: 04-30-2014 Login: cnt

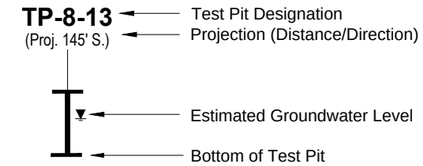
CONE PENETRATION TEST LEGEND



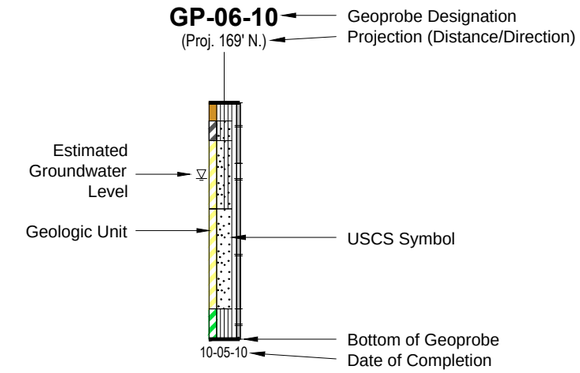
BORING LOG LEGEND



TEST PIT LEGEND



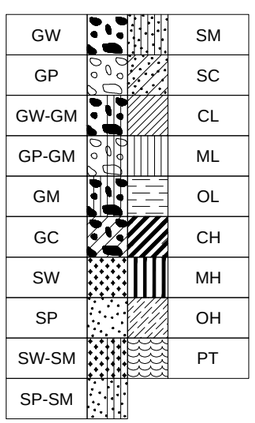
GEOPROBE LEGEND



SOIL OR ROCK AND SAMPLING LEGEND

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)

(From USACE Tech Memo 3-357)



SAMPLE TYPES

*	Sample Not Recovered
I	2" O.D. Split Spoon Sample with 140 lb. Hammer (standard penetration test - SPT)
ST	3" O.D. Direct Push Thin-Walled Sample
	3" O.D. Piston (Osterberg) Thin-Walled Sample
G	Grab Sample
	Geoprobe Run with Soil Recovery
	Geoprobe Run without Soil Recovery

1. Dual Symbols (symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND) are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.
2. Borderline symbols (symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND) indicate that the soil may fall into one of two possible basic groups, based on ASTM D 2488 Visual Manual Classification System. The graphic symbol of only the first group symbol is shown on the profile.

GENERALIZED GEOLOGY TYPE EXPLANATION

	AGRICULTURAL LAYER
Hm	**HOLOCENE MUDFLAT DEPOSITS**
Ha	**HOLOCENE ALLUVIUM**
He	**HOLOCENE ESTUARY DEPOSITS**

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED/COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	<2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

Reference: Oregon State Department of Transportation, 1987, Soil and Rock Classification Manual, p.18.

NOTES

1. This profile is constructed from surface elevations and project alignment, received from Washington Department of Fish and Wildlife, on 10-30-2010. The subsurface conditions shown are interpreted from the explorations conducted by Shannon & Wilson. Soil descriptions for CPTs are interpreted from the soil behavior type presented on In-Situ Engineering's CPT log and nearby borings. The geology, as encountered in the explorations, has been projected into the plane of the profile or section. Elevations and contacts should be considered approximate. Where layers are described as interbedded, the geologic unit observed to be more dominant is listed first and shown as the geologic symbol. Variations between the profile and actual conditions are likely to exist.
2. Water levels shown were measured on various dates. Groundwater fluctuations should be expected.
3. Estimated groundwater levels are based on observations made while drilling borings, excavation test pits and pushing geoprobes. Evidence of groundwater includes wet samples, visible seepage, and water standing in an open hole. We estimated the groundwater level in CPTs using porewater pressure measurements and pore pressure dissipation tests. The actual groundwater level may differ from our estimates.
4. The locations and extents of proposed structures should be considered approximate. Project alignments presented in exhibits were developed by Shannon & Wilson.

DRAFT

Fir Island Farm
Final Design and Permitting
Skagit County, Washington

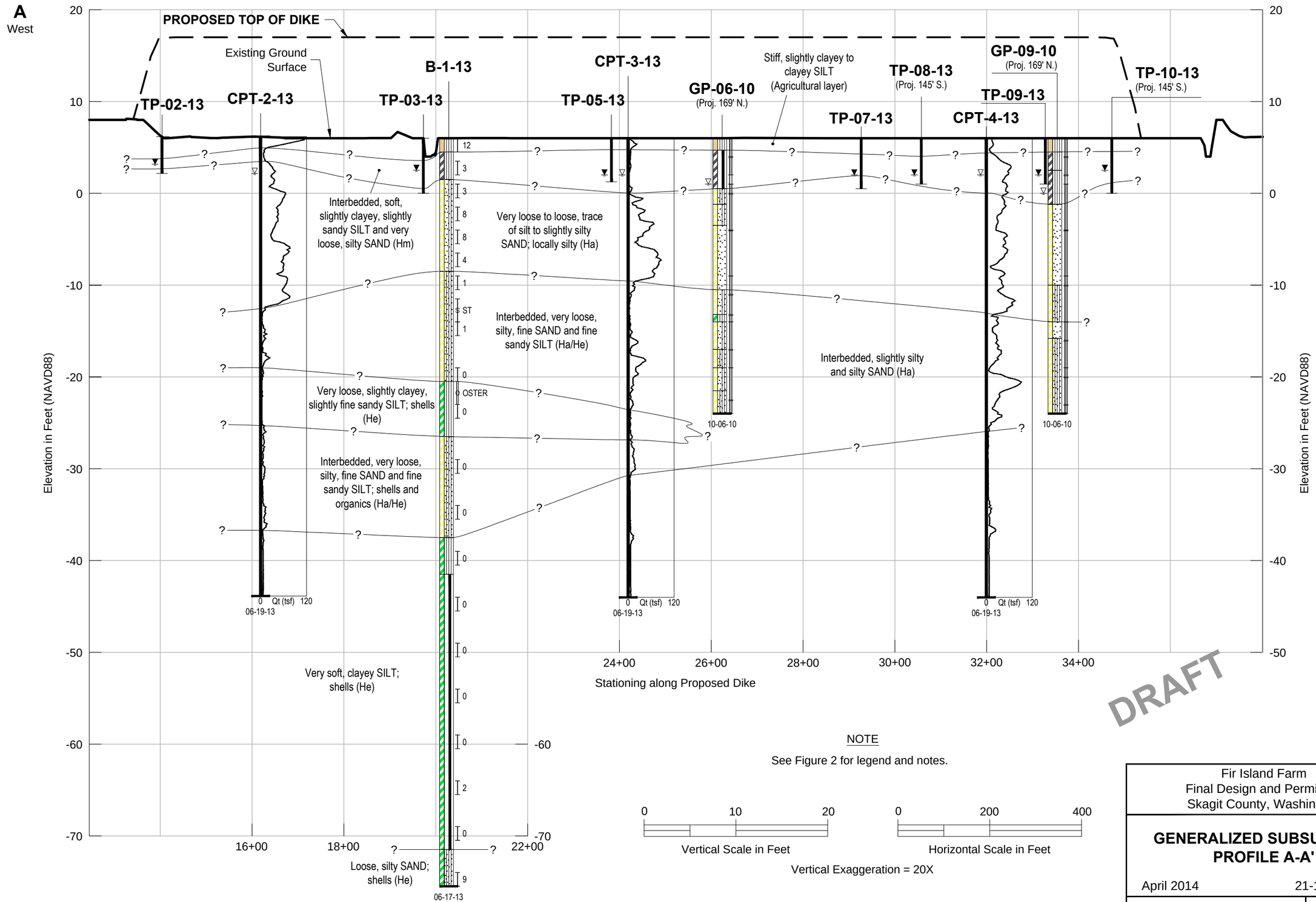
GEOLOGIC PROFILE LEGEND AND NOTES

April 2014 21-1-12318-224

SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 2

Filename: I:\WIP\21-1\12318 Fir Island\CAD\FIGURES\-224\21-1-12318-224 Legend.dwg Date: 04-30-2014 Login: cnt



NOTE

See Figure 2 for legend and notes.

0 10 20
Vertical Scale in Feet

0 200 400
Horizontal Scale in Feet

Vertical Exaggeration = 20X

Fir Island Farm
Final Design and Permitting
Skagit County, Washington

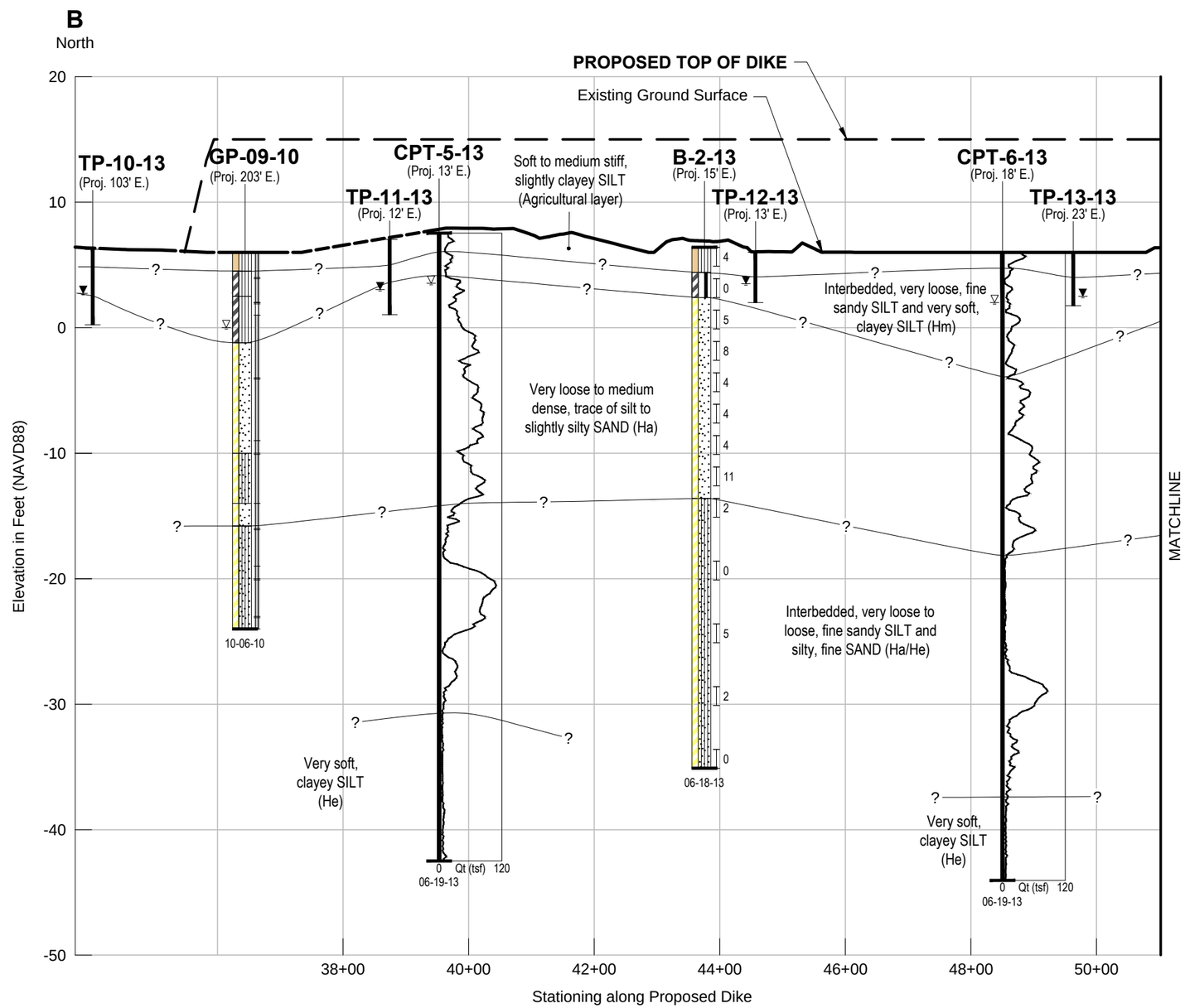
GENERALIZED SUBSURFACE PROFILE A-A'

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 3

Filename: I:\WIP\21-1\12318 Fir Island\CAD\FIGURES\-224\21-1-12318-224 Fig 3 - Profiles.dwg Date: 04-30-2014 Login: cnt



DRAFT

NOTE

See Figure 2 for legend and notes.

0 10 20 — Vertical Scale in Feet

0 200 400 — Horizontal Scale in Feet

Vertical Exaggeration = 20X

Fir Island Farm
Final Design and Permitting
Skagit County, Washington

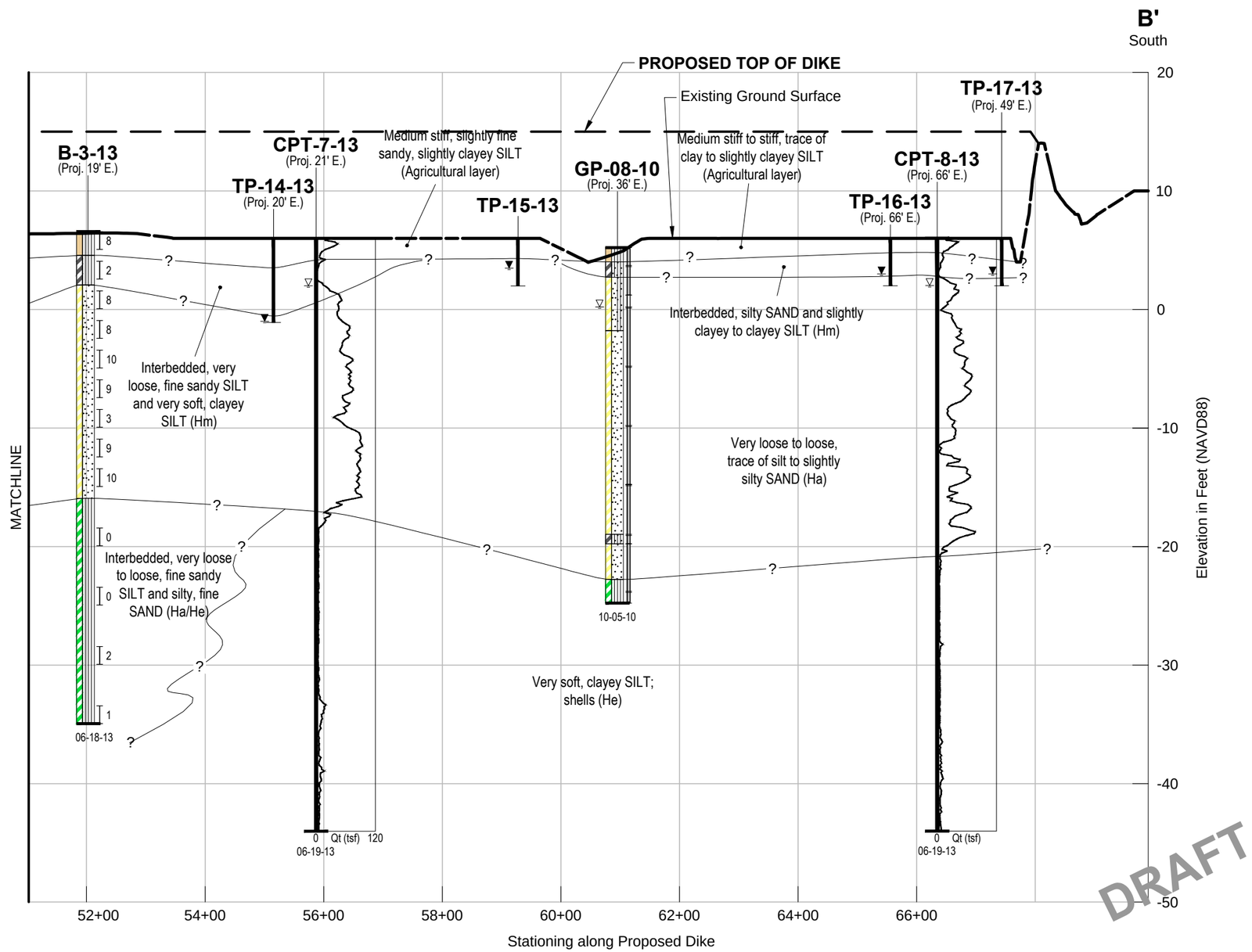
GENERALIZED SUBSURFACE PROFILE B-B'

April 2014 | 21-1-12318-224

SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 4
Sheet 1 of 2

Filename: I:\WIP\21-1\12318 Fir Island\CAD\FIGURES\-224\21-1-12318-224 Fig 3 - Profiles.dwg Date: 04-30-2014 Login: cnt



NOTE

See Figure 2 for legend and notes.

0 10 20 — Vertical Scale in Feet

0 200 400 — Horizontal Scale in Feet

Vertical Exaggeration = 20X

Fir Island Farm
Final Design and Permitting
Skagit County, Washington

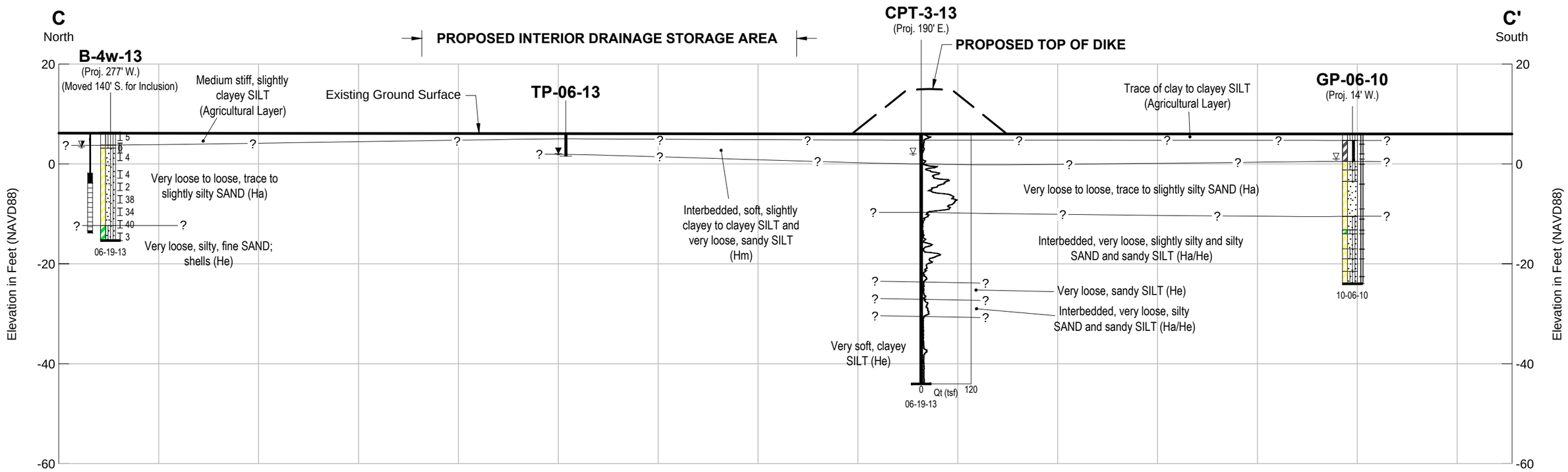
GENERALIZED SUBSURFACE PROFILE B-B'

April 2014 | 21-1-12318-224

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 4
Sheet 2 of 2

Filename: I:\WIP\21-1\12318 Fir Island\CAD\FIGURES\-224\21-1-12318-224 Fig 3 - Profiles.dwg Date: 04-30-2014 Login: cnt



NOTE

See Figure 2 for legend and notes.

0 20 40

Vertical Scale in Feet

0 40 80

Horizontal Scale in Feet

Vertical Exaggeration = 2X

DRAFT

Fir Island Farm
Final Design and Permitting
Skagit County, Washington

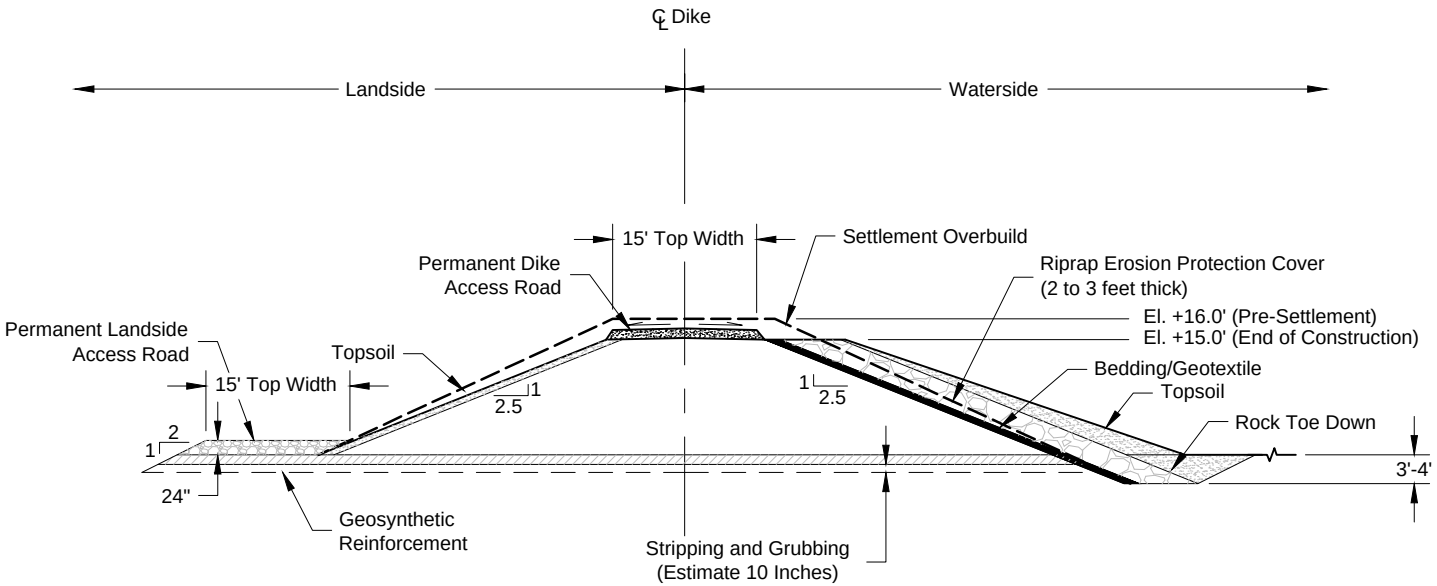
GENERALIZED SUBSURFACE PROFILE C-C'

April 2014 21-1-12318-224

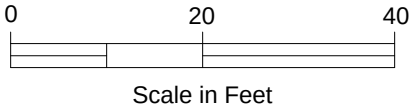
SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 5

Filename: I:\WIP\21-1\12318 Fir Island\CAD\FIGURES\-224\21-1-12318-224 Fig 3 - Profiles.dwg Date: 04-30-2014 Login: cnt



Notes:
1. Backfill excavated stripping depth with Dike Select Fill



DRAFT

Fir Island Farm
Final Design and Permitting
Skagit County, Washington

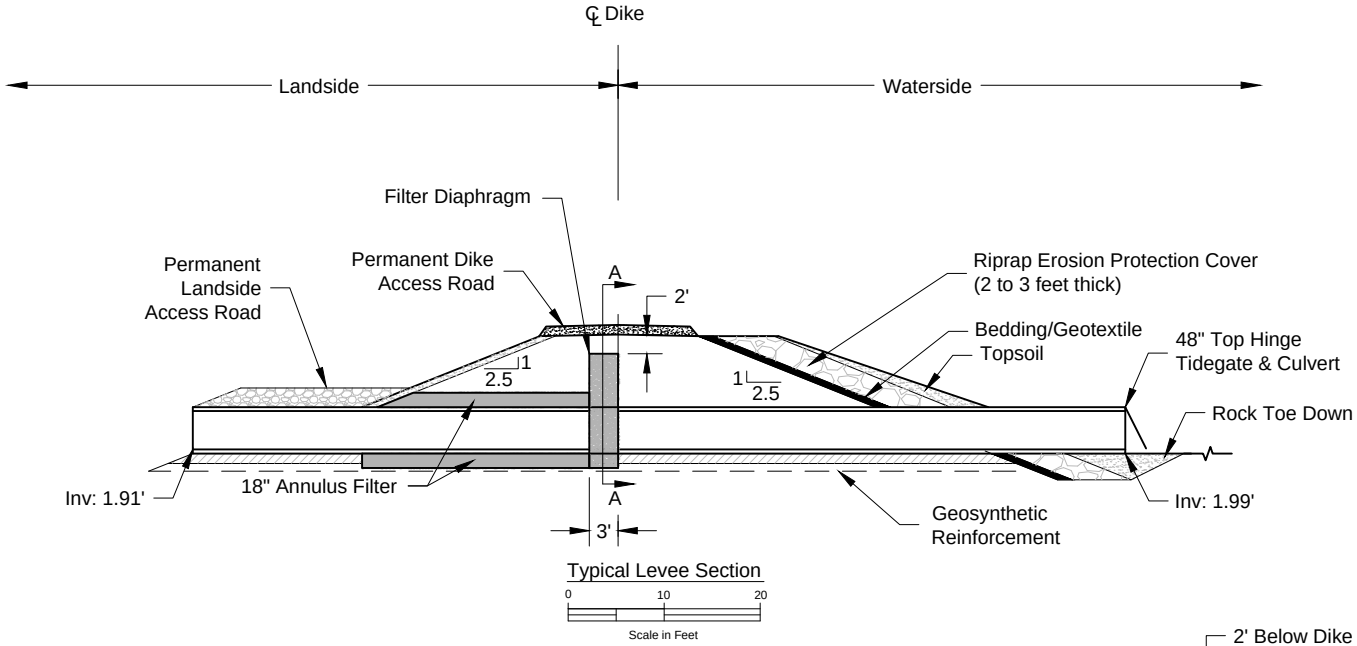
TYPICAL DIKE SECTION

April 2014 21-1-12318-224

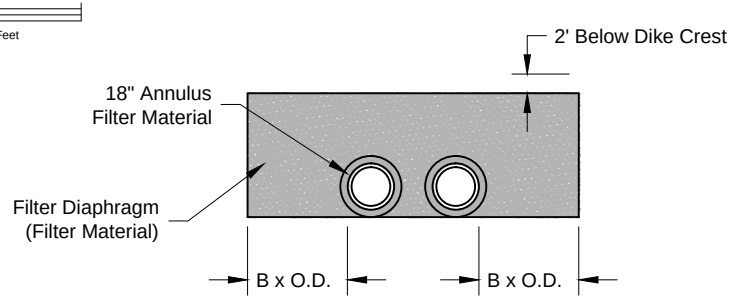
SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 6

FIG. 6



Typical Levee Section



Section A-A
Not to Scale

Notes:
1. Backfill excavated stripping depth with Dike Select Fill
2. Backfill filter trench with Filter Material.
3. Vertical 2' clearance measured to end of construction height (EL.+15.0')
4. Annulus filter to end at landside toe of dike.
5. If the culvert pipe is rigid, the horizontal extension of the filter diaphragm should be the lesser of 3 x O.D. or 5 feet past any excavation slope made to install the culvert

Legend
B = 2 (flexible pipe)
B = 3 (rigid pipe)
O.D. = outside diameter

DRAFT

Fir Island Farm
Final Design and Permitting
Skagit County, Washington

FILTER DIAPHRAGM

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 7

FIG. 7

APPENDIX A

SLOPE STABILITY AND SEEPAGE ANALYSES AND RESULTS

Draft

APPENDIX A

SLOPE STABILITY AND SEEPAGE ANALYSES AND RESULTS

TABLE OF CONTENTS

Page

A-1 INTRODUCTION A-1

A-2 SUBSURFACE CONDITIONS A-1

A-3 SOIL PARAMETERS A-2

A-4 DESIGN CASES A-2

A-5 SLOPE STABILITY ANALYSIS METHODOLOGY A-3

A-6 SEEPAGE AND GLOBAL STABILITY ANALYSIS RESULTS A-4

A-7 REFERENCES A-4

TABLE

A-1 Interpreted Range of Engineering Soil Parameters

FIGURES

A-1 Tide Gate Section (Sta. 21+00) Case 1, End-of-Construction; Land Side
A-2 Tide Gate Section (Sta. 21+00) Case 1, End-of-Construction; Water Side
A-3 Tide Gate Section (Sta. 21+00) Case 2, Storm Drawdown; Land Side
A-4 Tide Gate Section (Sta. 21+00) Case 2, Storm Drawdown; Water Side
A-5 Tide Gate Section (Sta. 21+00) Case 3, Daily Tidal; Land Side
A-6 Tide Gate Section (Sta. 21+00) Case 3, Daily Tidal; Water Side
A-7 Tide Gate Section (Sta. 21+00) Case 4, Interior Flooding; Land Side
A-8 Tide Gate Section (Sta. 21+00) Case 4, Interior Flooding; Water Side
A-9 Pond Section (Sta. 34+15) Case 1, End-of-Construction; Land Side
A-10 Pond Section (Sta. 34+15) Case 1, End-of-Construction; Water Side
A-11 Pond Section (Sta. 34+15) Case 2, Storm Drawdown; Land Side
A-12 Pond Section (Sta. 34+15) Case 2, Storm Drawdown; Water Side
A-13 Pond Section (Sta. 34+15) Case 3, Daily Tidal; Land Side
A-14 Pond Section (Sta. 34+15) Case 3, Daily Tidal; Water Side

Draft

TABLE OF CONTENTS (cont.)

FIGURES (cont.)

A-15 Pond Section (Sta. 34+15) Case 4, Interior Flooding; Land Side
A-16 Pond Section (Sta. 34+15) Case 4, Interior Flooding; Water Side
A-17 Dry Slough Section (Sta. 48+90) Case 1, End-of-Construction; Land Side
A-18 Dry Slough Section (Sta. 48+90) Case 1, End-of-Construction; Water Side
A-19 Dry Slough Section (Sta. 48+90) Case 2, Storm Drawdown; Land Side
A-20 Dry Slough Section (Sta. 48+90) Case 2, Storm Drawdown; Water Side
A-21 Dry Slough Section (Sta. 48+90) Case 3, Daily Tidal; Land Side
A-22 Dry Slough Section (Sta. 48+90) Case 3, Daily Tidal; Water Side
A-23 Dry Slough Section (Sta. 48+90) Case 4, Interior Drainage; Land Side
A-24 Dry Slough Section (Sta. 48+90) Case 4, Interior Drainage; Water Side

Draft

APPENDIX A

SLOPE STABILITY AND SEEPAGE ANALYSES AND RESULTS

A-1 INTRODUCTION

Three dike cross sections were selected for seepage and global stability analyses. One cross section, denoted "Tide Gate," was selected at the proposed tide gate locations at No Name Slough, approximate Station 21+00. The remaining two cross sections, denoted "Pond" and "Dry Slough," were selected to represent typical soil conditions along the dike alignment, differing dike geometries, anticipated scour, and proximity to tidal channels or interior drainage features. Cross sections "Pond" and "Dry Slough" are located at approximate Stations 34+15 and 48+90, respectively.

For each dike cross section, we prepared a coupled global stability and seepage computer model using the software suite Geostudio Version 8 (Geo-Slope International Ltd. [Geo-Slope], 2012a). The seepage module of Geostudio, SEEP/W, is a two-dimensional, finite-element seepage analysis program that simulates fluid flow and pressure distribution in saturated and unsaturated materials such as soil and rock. The global stability module of Geostudio, SLOPE/W, uses limit equilibrium analysis methods to calculate a factor of safety (FS) against global instability. Geostudio allows porewater pressures calculated by SEEP/W to be imported into SLOPE/W analyses.

A-2 SUBSURFACE CONDITIONS

Our interpretation of the subsurface conditions along the proposed dike alignment is presented in Figures 3 and 4 of the report. The profiles show the subsurface soil layering and contact elevations we interpreted from our subsurface explorations and laboratory test results. The following generalized geologic soil layers were categorized beneath the proposed setback dike:

- Slightly clayey to clayey silt and fine sandy silt (Agricultural Layer / Mudflat Deposits, Hm)
- Clean to slightly silty sand (Alluvial Deposits, Ha)
- Silt, sandy silt, silty sand, and clayey silt (Estuarine Deposits, He)

Subsurface layering for the individual analyses are based on Figures 3 and 4, and shown in Figures A-1 through A-24. The subsurface layering was modeled with a constant elevation for each analysis cross section.

Draft

A-3 SOIL PARAMETERS

Soil parameters used in our slope stability and seepage analyses are summarized in Table A-1. Strength parameters for slope stability analyses were selected using available:

- Geotechnical boring logs,
- Geotechnical test pit logs,
- Geotechnical laboratory test results,
- Cone penetration test (CPT) results, and
- Published correlations and parameters.

Effective stress internal friction angles were used for the Ha layer and the Hm and He layers for long-term loading conditions. For the End of Construction condition (Case 1), undrained shear strengths (S_U) were used for the Hm and He layers.

Hydraulic conductivity values used in seepage analyses were estimated using:

- CPT dissipation tests,
- Correlations with grain size distribution,
- Consolidation test data, and
- CPT correlations.

A-4 DESIGN CASES

Following guidelines presented in the U.S. Army Corps of Engineers (USACE) Engineering Manuals 1110-2-1902 and 1110-2-1913, the following conditions were evaluated for each of the three dike cross sections:

- Case 1 – End of Construction
- Case 2 – Storm Drawdown
- Case 3 – Daily Tidal Drawdown
- Case 4 – Interior Flooding

The Case 1 model geometry was based on the existing ground surface with the proposed dike and an additional over-build height, ΔH, of 1 foot to account for anticipated settlement (see report text). The dike was modeled with a 15-foot-wide crest and embankment side slopes inclined at 2.5H horizontal to H+ΔH vertical. The resulting pre-settlement slope angles would vary from 2.2H:1V (2.2 Horizontal to 1 Vertical) to 2.4H:1V. The groundwater level was assumed to be at the ground surface. No water was assumed in the proposed storage pond. A surface water elevation of 3.23 feet was assumed in Dry Slough and No Name Slough.

Draft

For Cases 2, 3, and 4, we assumed that the long-term settlement had occurred (i.e., the additional over-build height is not present) and that the dike crest is at elevation +15.0 feet (North American Vertical Datum of 1988). The tidal and flooding hydrodynamic effects were interpreted from our Draft Coastal Engineering Report (Shannon & Wilson, Inc., 2014) and modeled using transient SEEP/W analyses.

For Case 2, the floodwater was modeled to rise and fall with the tide as indicated below:

- Water rises from the mean lower low water (MLLW) elevation of 3.1 feet to the flood elevation of 15 feet in 3 hours and 15 minutes
- Water falls from the flood elevation to the mean low water (MLW) elevation of 3.2 feet in 8 hours.
- Water in the pond and slough remains constant at elevation 3.23 feet.

For Case 3, a 24-hour tidal cycle was modeled to rise and fall as indicated below:

- Water rises from the MLLW elevation to the mean higher high water (MHHW) elevation of 9.3 feet in 3 hours and 15 minutes.
- Water falls from the MHHW elevation to the MLW elevation in 8 hours.
- Water rises from the MLW elevation to the mean high water (MHW) elevation of 6.7 feet in 3 hours and 45 minutes.
- Water falls from the MHW elevation to the MLW elevation in 5 hours and 15 minutes.
- Water continues to fall from the MLW elevation to the MLLW elevation in 3 hours and 45 minutes.
- Water in the pond and slough remains constant at elevation 3.23 feet.

Case 4 consists of a condition in which the interior (landside) of the dike floods concurrently with the waterside and remains as the tide falls. The floodwater on the waterside was modeled identical to Case 2, Storm Drawdown. The floodwater on the landside was modeled to rise from the MLLW elevation to elevation 10 feet in 3 hours and 15 minutes and remain constant.

A-5 SLOPE STABILITY ANALYSIS METHODOLOGY

Slope stability analyses were performed in accordance with the USACE *Levee Design and Construction Manual EM 1110-2-1913* (USACE, 2000) and the *Slope Stability Manual EM 1110-2-1902* (USACE, 2003). The analyses used limit equilibrium slope stability analysis methods in the computer program SLOPE/W (Geo-Slope, 2012b). Circular failure surfaces were analyzed at three dike cross sections to calculate the FS. An automated search routine was used

Draft

to identify the failure surface with the lowest FS (critical failure surface). The critical failure surface was then modified using the optimization feature in SLOPE/W as a non-circular surface and a revised FS calculated. The SLOPE/W optimization technique was employed for all static analyses cases. The Morgenstern and Price (1965) and Spencer (1967) methods of analysis, which satisfy both moment and force equilibrium, were used to search for the location of the most critical slip surfaces and their corresponding FSs.

Due to the soft foundation soil (Agricultural layer and Mudflat deposits), a basal reinforcing geosynthetic was needed to meet the global stability FS for Case 1. The minimum tensile strengths required to achieve the minimum FS are summarized in Table 2 of the main text.

A-6 SEEPAGE AND GLOBAL STABILITY ANALYSIS RESULTS

Results of our seepage and global stability analyses are discussed in the main text of this report. Seepage results for the upward exit hydraulic gradient and seepage flow rates for steady state flow conditions during a design flood are summarized in Table 1 in the report. Global stability results for the three cases evaluated are summarized in Table 2 in the report. Global stability results are presented graphically as Figures A-1 through A-24.

A-7 REFERENCES

Geo-Slope International Ltd. (Geo-Slope), 2012a, Seepage modeling with SEEP/W 2012 version 8.12.

Geo-Slope International Ltd. (Geo-Slope), 2012b, Stability modeling with SLOPE/W 2012 version 8.12.

Morgenstern, N.R., and Price, V.E., 1965, The analysis of the stability of general slip surfaces, Geotechnique, v. 15, p. 79-93.

Shannon & Wilson, Inc., 2014, Fir island farm final design and permitting, draft coastal engineering report, Skagit County, Washington: Report prepared by Shannon & Wilson, Inc., Seattle, Wash., 21-1-12318-217, for Washington Department of Fish and Wildlife (WDFW), January 10.

Spencer, E., 1967, A method of analysis of the stability of embankments assuming parallel inter-slice forces: Geotechnique, v. 17, no. 1, p. 11-26.

U.S. Army Corps of Engineers (USACE), Engineering and Design, 2000, Design and Construction of Levees, Engineer Manual 1110-2-1913: Washington, D.C., April.

U.S. Army Corps of Engineers (USACE), Engineering and Design, 2003, Slope Stability, Engineering Manual 1110-2-1902: Washington, D.C., October.

Draft

U.S. Army Corps of Engineers (USACE), Engineering and Design, 2005, Design Guidance for Levee Underseepage, Technical Letter ETL 1110-2-569: Washington, D.C., May.

Draft

TABLE A-1
INTERPRETED RANGE OF ENGINEERING SOIL PARAMETERS

Geologic Unit	USCS [3]	Saturated Unit Weight γ_{sat} *(pcf)*	Shear Strength: Effective Stress (Drained) c' *(psf)*	Effective Stress (Drained) ϕ' *(deg)*	Total Stress (Undrained) c *(psf)*	Total Stress (Undrained) ϕ *(deg)*	Undrained Shear Strength S_u *(psf)*	Saturated Hydraulic Conductivity k *(ft/day)*	
								Tide Gate and Pond	*Dry Slough*
Dike Select Fill	MH, ML, SM	120 *(80 - 140)*	*0* *(n/a)*	*32* *(25 - 34)*	*0* *(n/a)*	*32* *(25 - 34)*	*n/a*	1.7	1.7
Agricultural Layer / Mudflat Deposit (Hm)	OH, MH, ML	108 *(80 - 135)*	0 *(n/a)*	28 *(24 - 44)*	n/a	n/a	200 *(100 - 630)*	0.04	0.08
Alluvium (Ha)	ML, SM, SP-SM, SP	120 *(80 - 140)*	0 *(n/a)*	30 *(27 - 51)*	0 *(n/a)*	30 *(27 - 51)*	*n/a*	85	64
Estruarine (He)	MH, ML, SM	108 *(80 - 140)*	0 *(n/a)*	27 *(15 - 38)*	350 *(340 - 370)*	15 *(n/a)*	7·depth +125 (CPT Data)	0.82	0.08

Notes:

[1] The parameters above were based on statistical averages of index properties, laboratory tests, published correlations, and engineering judgment.

[2] Single values shown above represents the selected design value. The ranges shown in parentheses represent the potential variability of the property.

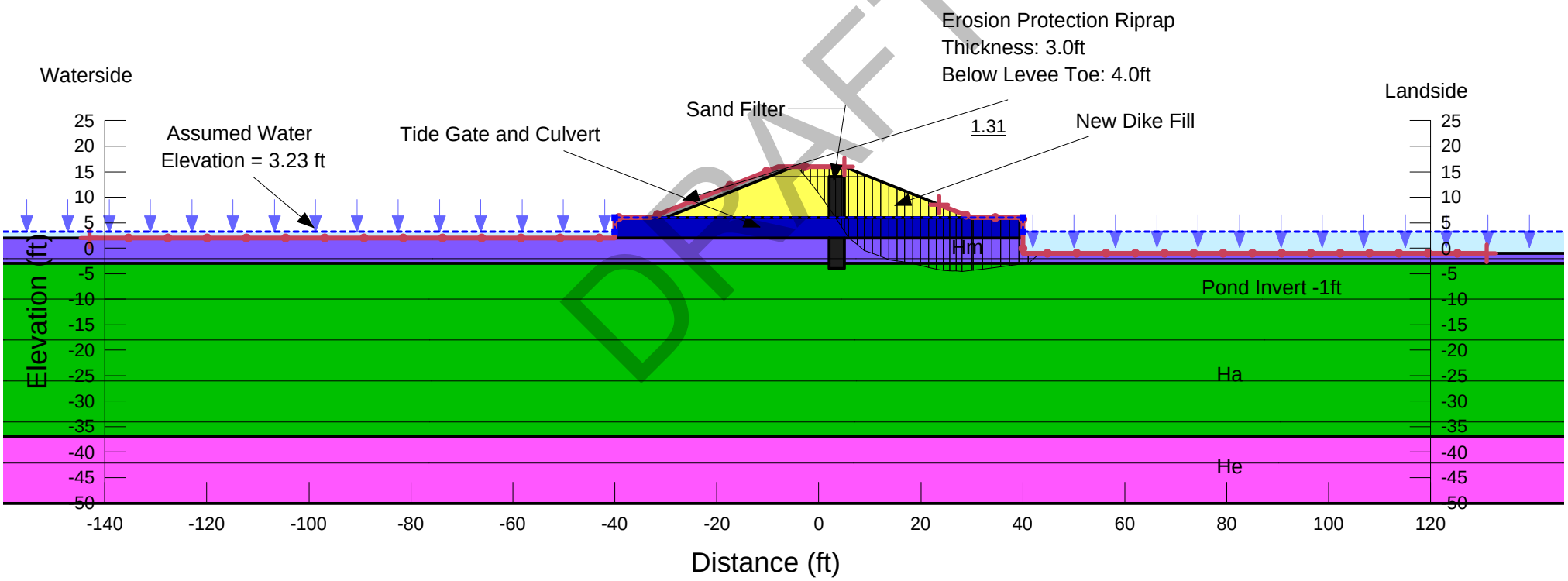
[3] The Unified Soil Classification System (USCS) definitions are as follows: ML = silt; MH = elastic silt; OH = organic soil; SM = silty sand; SP = poorly graded sand; SP-SM = poorly graded sand with 5 to 15 percent silt.

deg = degrees
n/a = "not applicable"
pcf = pounds per cubic foot
psf = pounds per square foot
S_U = undrained shear strength

Fir Island 60% Design
21-1-12318-224
Tide Gate Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 1: End of Construction
F of S: 1.31

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 ° Piezometric Line: 1
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Tidegate Model: Mohr-Coulomb Unit Weight: 14.5 pcf Cohesion': 1,000 psf Phi': 32 ° Piezometric Line: 1
Name: O-Sand Filter Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 ° Piezometric Line: 1
Name: O-Hm-undrained Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 200 psf Phi': 0 ° Piezometric Line: 1
Name: O-He-undrained Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 350 psf Phi': 15 ° Piezometric Line: 1



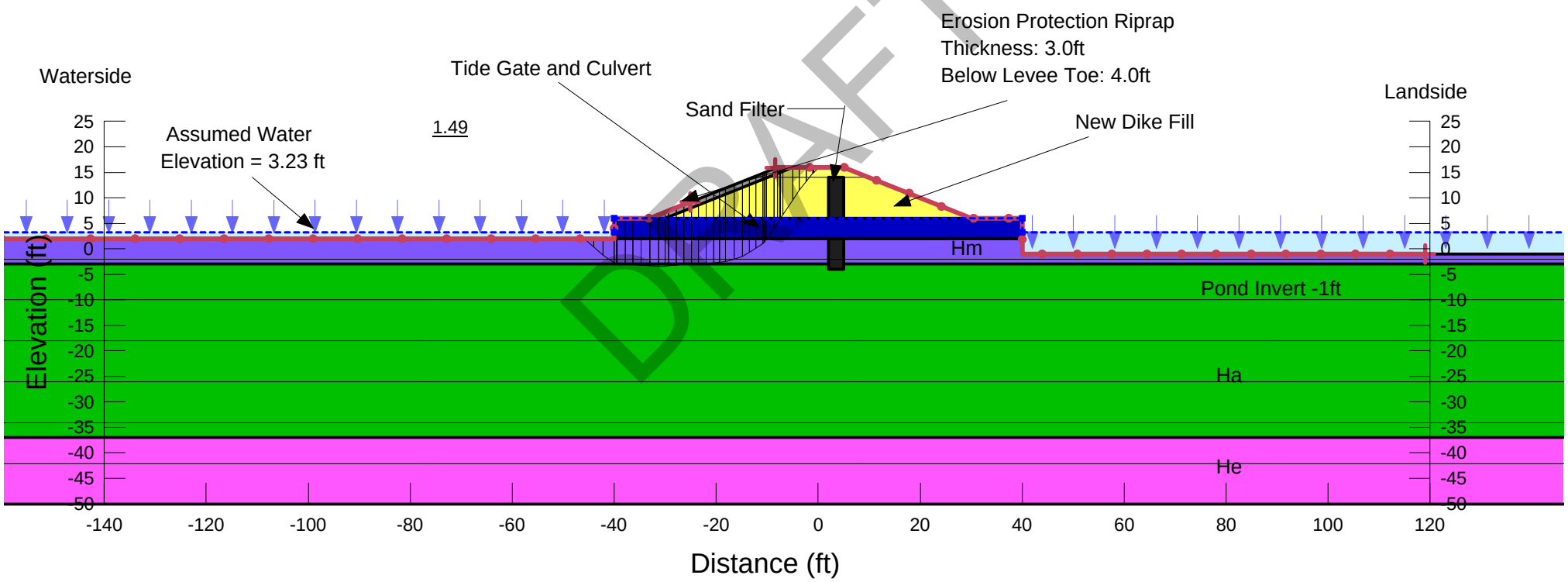
File Name: Tide Gate 2.5to1 - Seeps.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Tide Gate\
Date: 4/29/2014

Figure A-1

Fir Island 60% Design
21-1-12318-224
Tide Gate Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 1: End of Construction
F of S: 1.49

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 ° Piezometric Line: 1
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Tidegate Model: Mohr-Coulomb Unit Weight: 14.5 pcf Cohesion': 1,000 psf Phi': 32 ° Piezometric Line: 1
Name: O-Sand Filter Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 ° Piezometric Line: 1
Name: O-Hm-undrained Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 200 psf Phi': 0 ° Piezometric Line: 1
Name: O-He-undrained Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 350 psf Phi': 15 ° Piezometric Line: 1



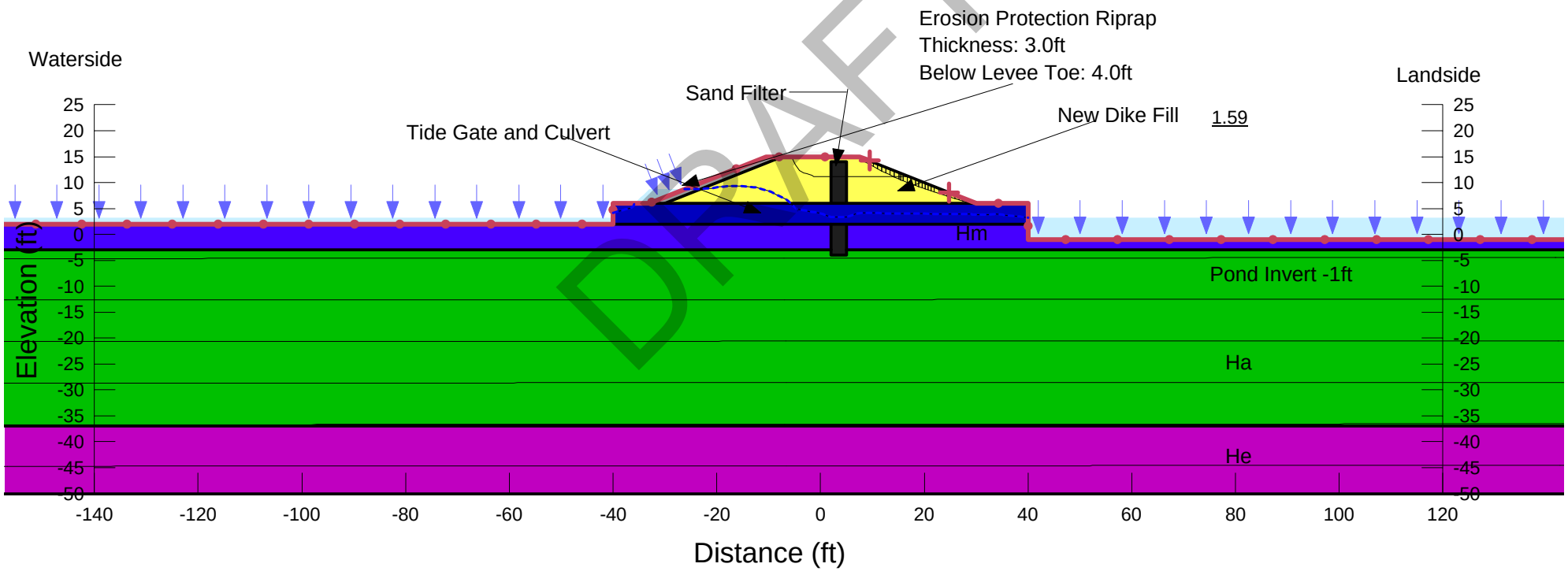
File Name: Tide Gate 2.5to1 - Seeps.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Tide Gate\
Date: 4/29/2014

Figure A-2

Fir Island 60% Design
21-1-12318-224
Tide Gate Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 2: Storm Drawdown
F of S: 1.59

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Tidegate Model: Mohr-Coulomb Unit Weight: 14.5 pcf Cohesion': 1,000 psf Phi': 32 °
Name: O-Sand Filter Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °



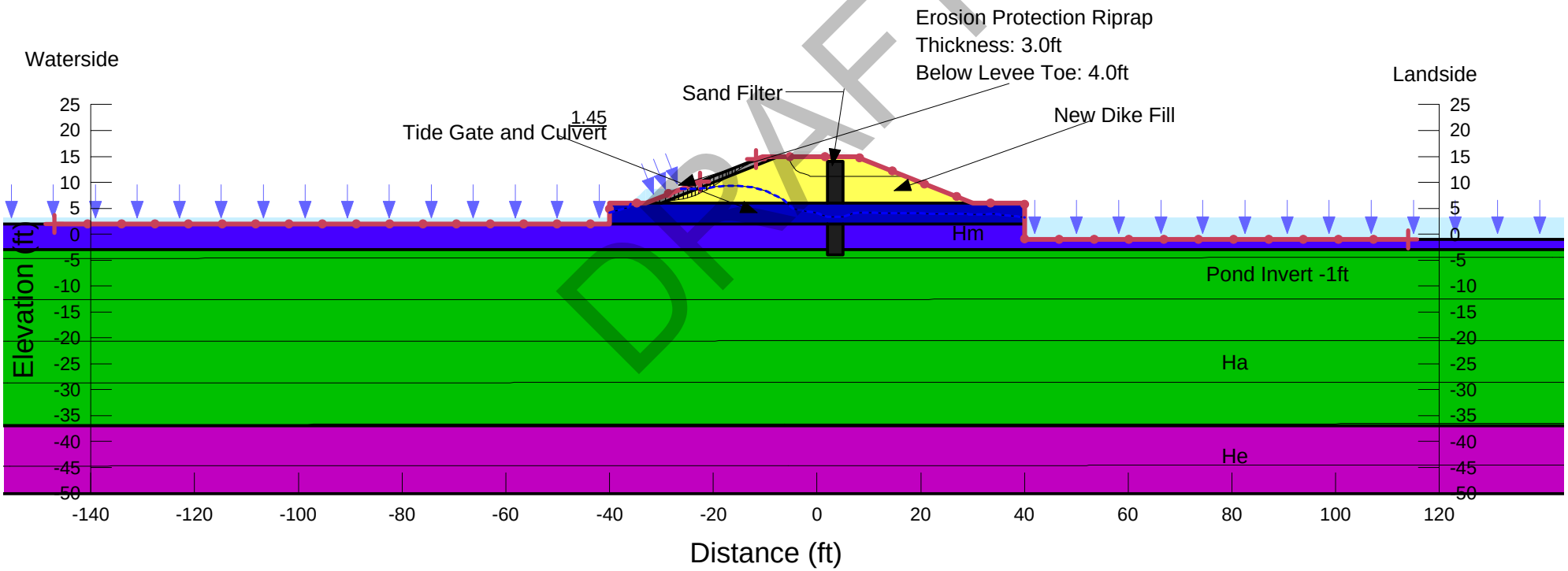
File Name: Tide Gate 2.5to1 - Seeps.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Tide Gate\
Date: 4/4/2014

Figure A-3

Fir Island 60% Design
21-1-12318-224
Tide Gate Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 2: Storm Drawdown
F of S: 1.45

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Tidegate Model: Mohr-Coulomb Unit Weight: 14.5 pcf Cohesion': 1,000 psf Phi': 32 °
Name: O-Sand Filter Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °



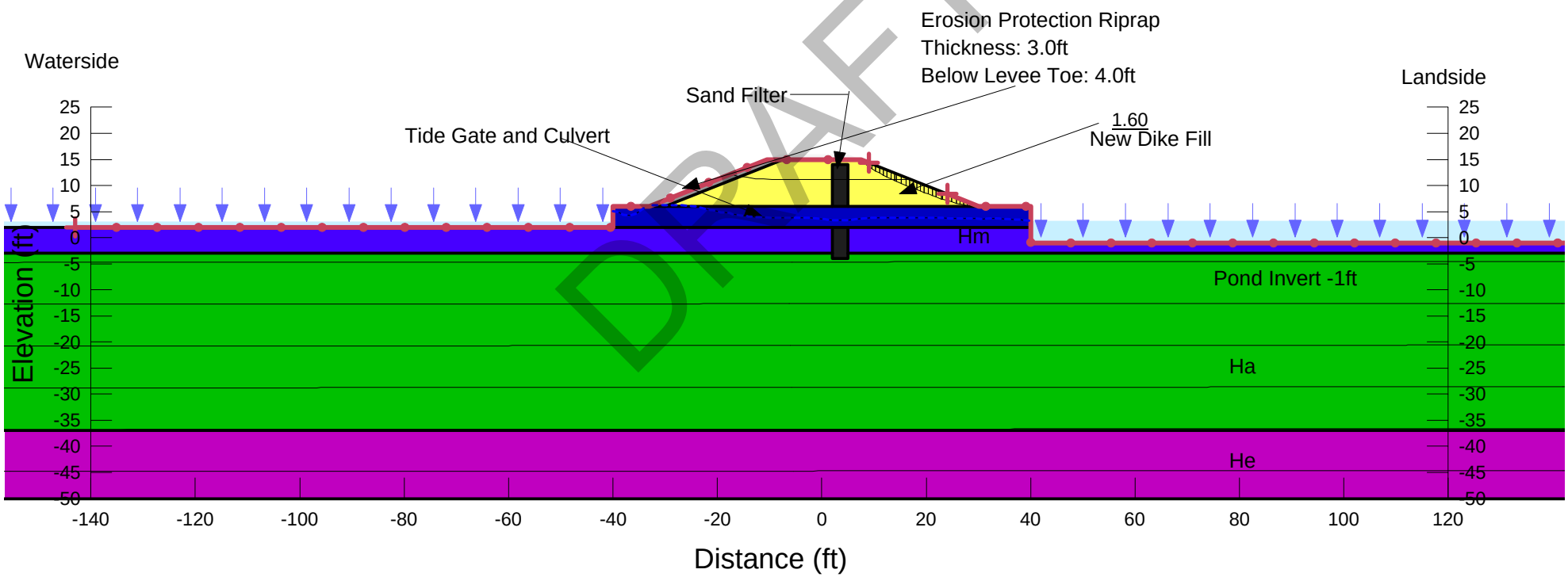
File Name: Tide Gate 2.5to1 - Seeps.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Tide Gate\
Date: 4/4/2014

Figure A-4

Fir Island 60% Design
21-1-12318-224
Tide Gate Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 3: Daily Tidal Drawdown
F of S: 1.60

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Tidegate Model: Mohr-Coulomb Unit Weight: 14.5 pcf Cohesion': 1,000 psf Phi': 32 °
Name: O-Sand Filter Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °



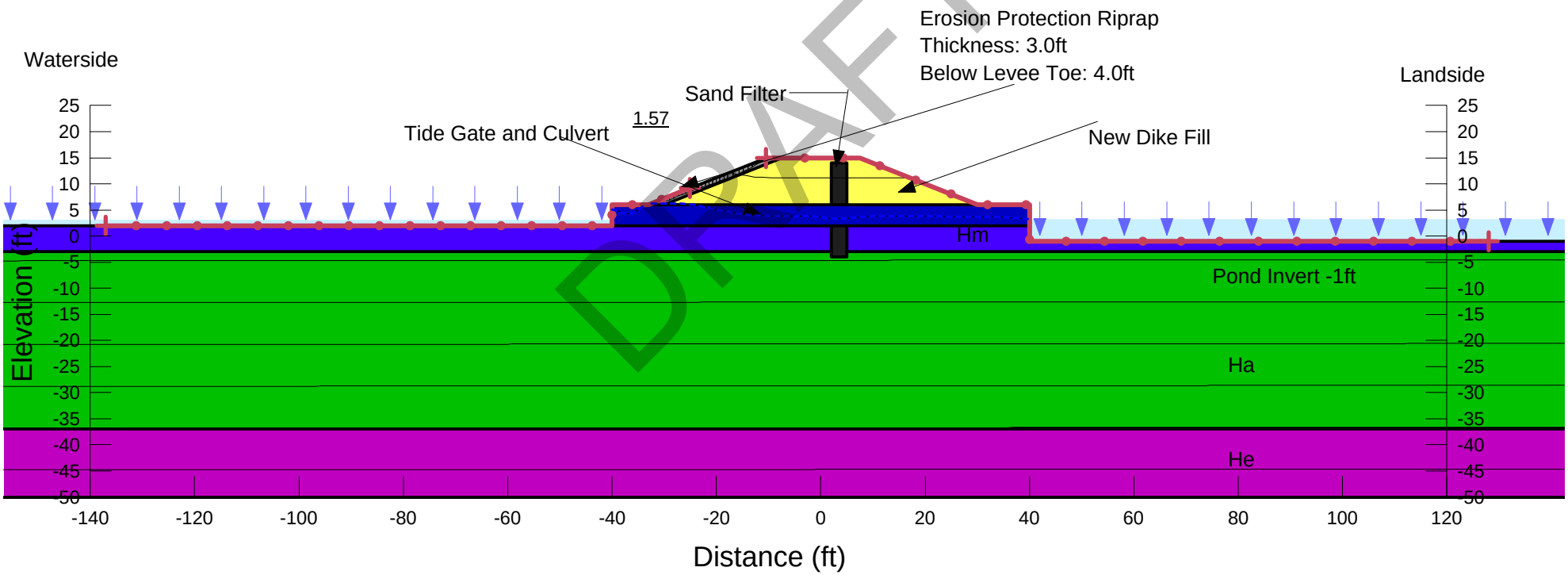
File Name: Tide Gate 2.5to1 - Seeps.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Tide Gate\
Date: 4/4/2014

Figure A-5

Fir Island 60% Design
21-1-12318-224
Tide Gate Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 3: Daily Tidal Drawdown
F of S: 1.57

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Tidegate Model: Mohr-Coulomb Unit Weight: 14.5 pcf Cohesion': 1,000 psf Phi': 32 °
Name: O-Sand Filter Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °



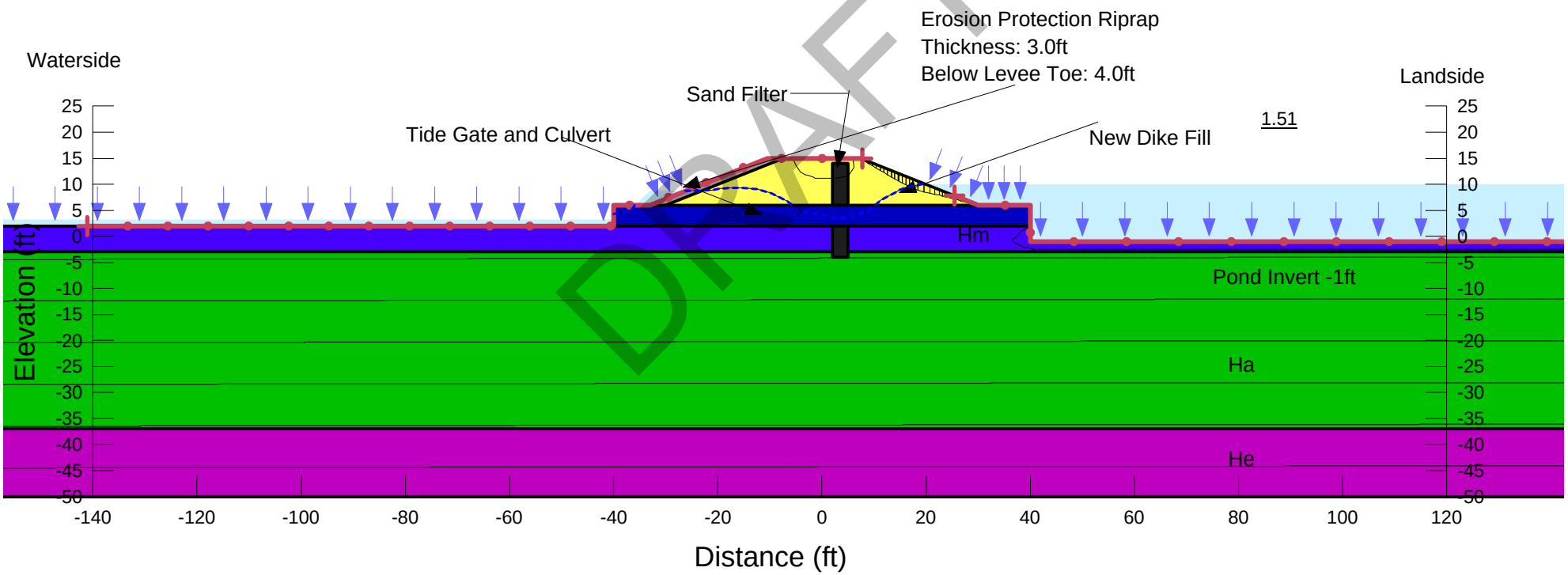
File Name: Tide Gate 2.5to1 - Seeps.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Tide Gate\
Date: 4/4/2014

Figure A-6

Fir Island 60% Design
21-1-12318-224
Tide Gate Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 4: Interior Flooding
F of S: 1.51

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Tidegate Model: Mohr-Coulomb Unit Weight: 14.5 pcf Cohesion': 1,000 psf Phi': 32 °
Name: O-Sand Filter Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °



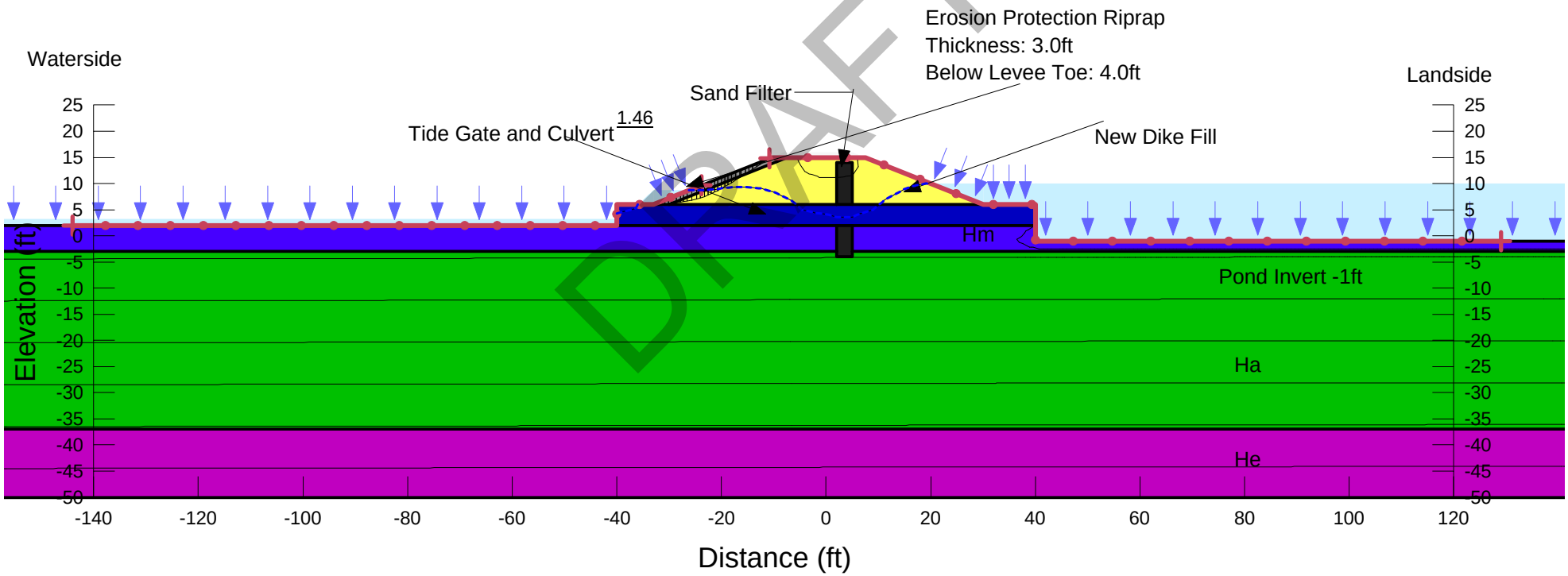
File Name: Tide Gate 2.5to1 - Seeps.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Tide Gate\
Date: 4/4/2014

Figure A-7

Fir Island 60% Design
21-1-12318-224
Tide Gate Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 4: Interior Flooding
F of S: 1.46

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Tidegate Model: Mohr-Coulomb Unit Weight: 14.5 pcf Cohesion': 1,000 psf Phi': 32 °
Name: O-Sand Filter Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °



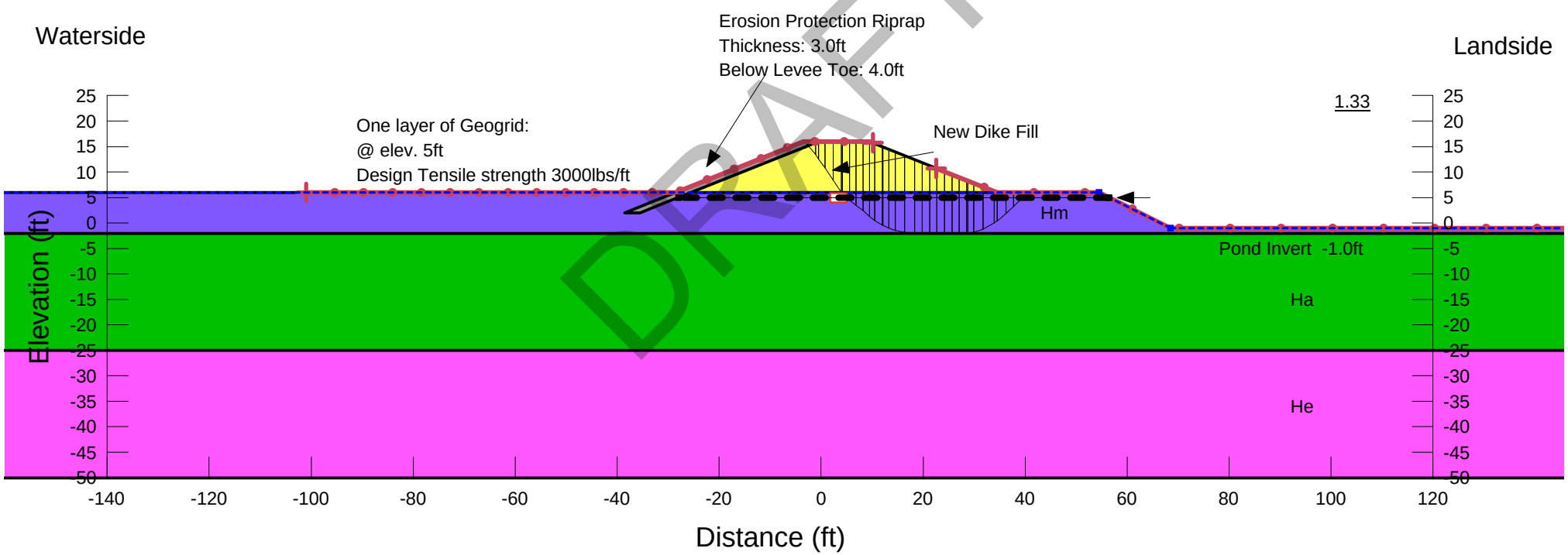
File Name: Tide Gate 2.5to1 - Seeps.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Tide Gate\
Date: 4/4/2014

Figure A-8

Fir Island 60% Design
21-1-12318-224
Pond Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 1: End of Construction
F of S: 1.33

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 ° Piezometric Line: 1
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Hm-undrained Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 200 psf Phi': 0 ° Piezometric Line: 1
Name: O-He-undrained Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 350 psf Phi': 15 ° Piezometric Line: 1



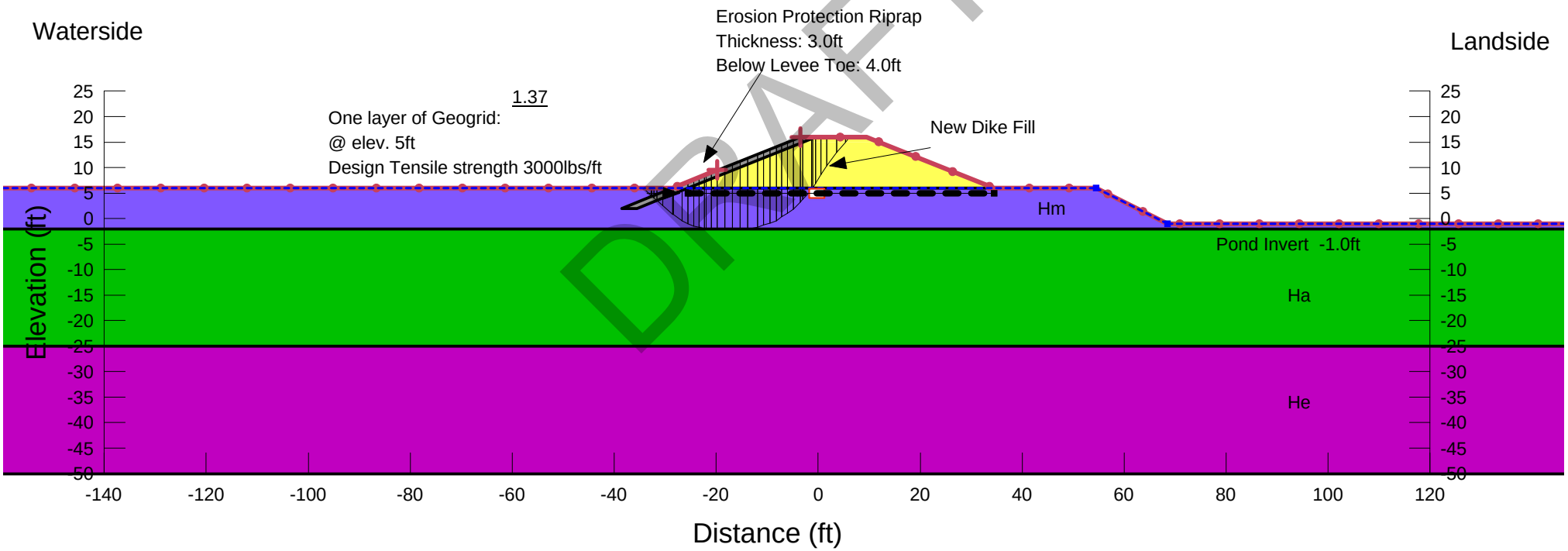
File Name: Pond 2.5to1 - Rev2.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Pond\
Date: 4/29/2014

Figure A-9

Fir Island 60% Design
21-1-12318-224
Pond Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 1: End of Construction
F of S: 1.37

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 ° Piezometric Line: 1
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 ° Piezometric Line: 1
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Hm-undrained Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 200 psf Phi': 0 ° Piezometric Line: 1



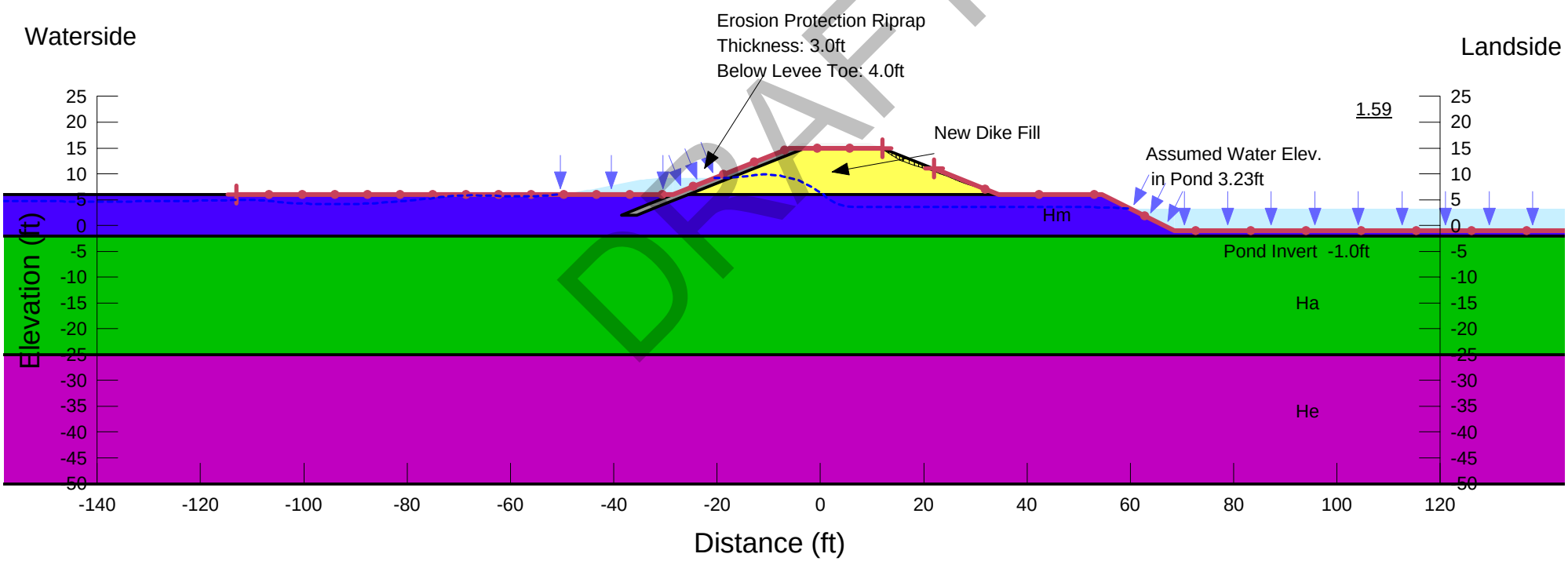
File Name: Pond 2.5to1 - Rev2.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Pond\
Date: 4/29/2014

Figure A-10

Fir Island 60% Design
21-1-12318-224
Pond Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 2: Storm Drawdown
F of S: 1.59

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °



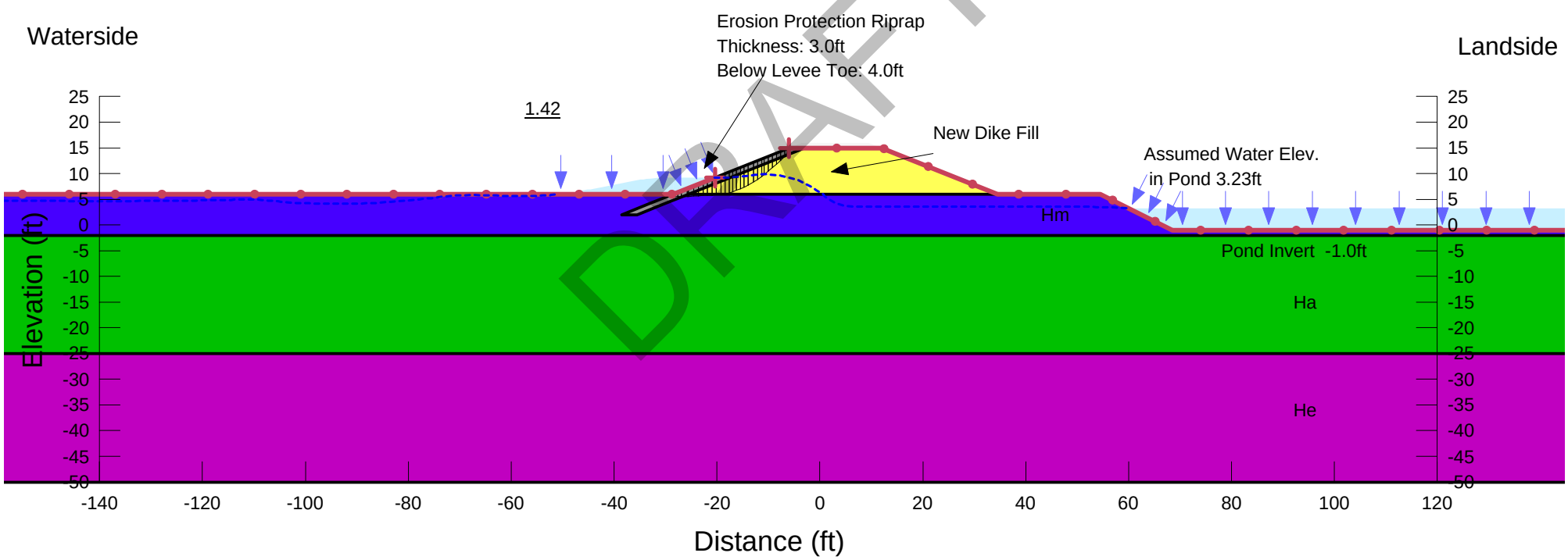
File Name: Pond 2.5to1 - Rev1.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Pond\
Date: 3/28/2014

Figure A-11

Fir Island 60% Design
21-1-12318-224
Pond Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 2: Storm Drawdown
F of S: 1.42

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °



DRAFT

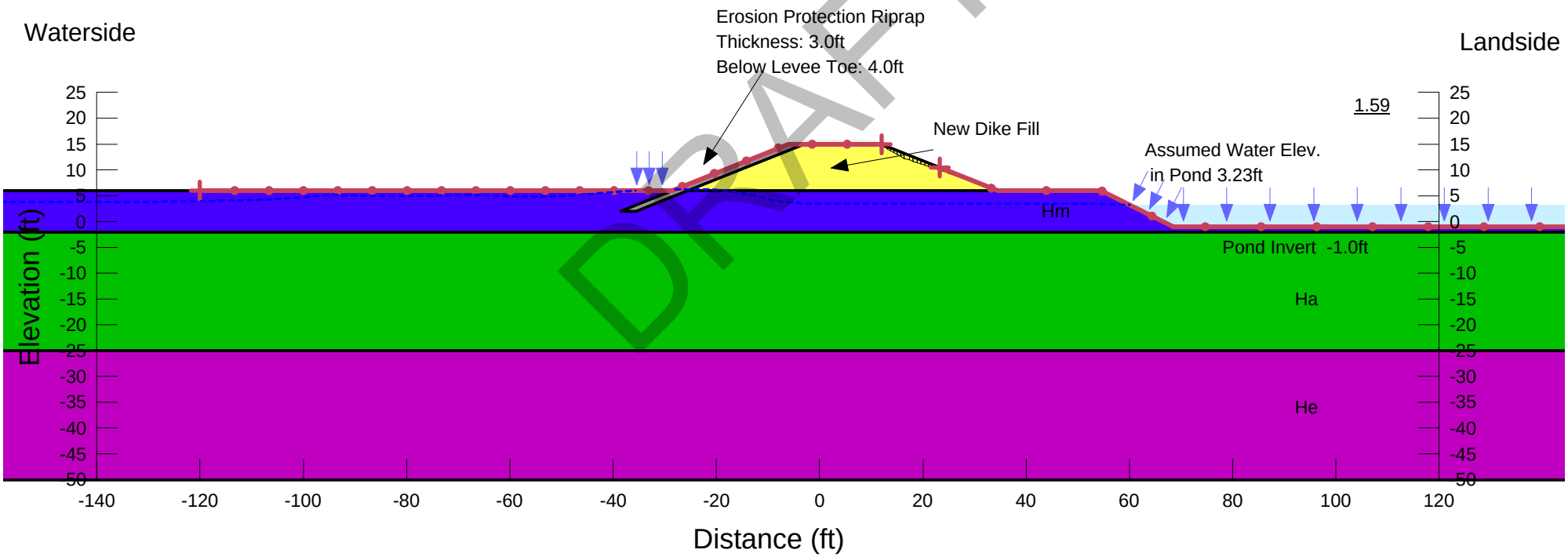
File Name: Pond 2.5to1 - Rev1.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Pond\
Date: 3/28/2014

Figure A-12

Fir Island 60% Design
21-1-12318-224
Pond Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 3: Daily Tidal Drawdown
F of S: 1.59

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °

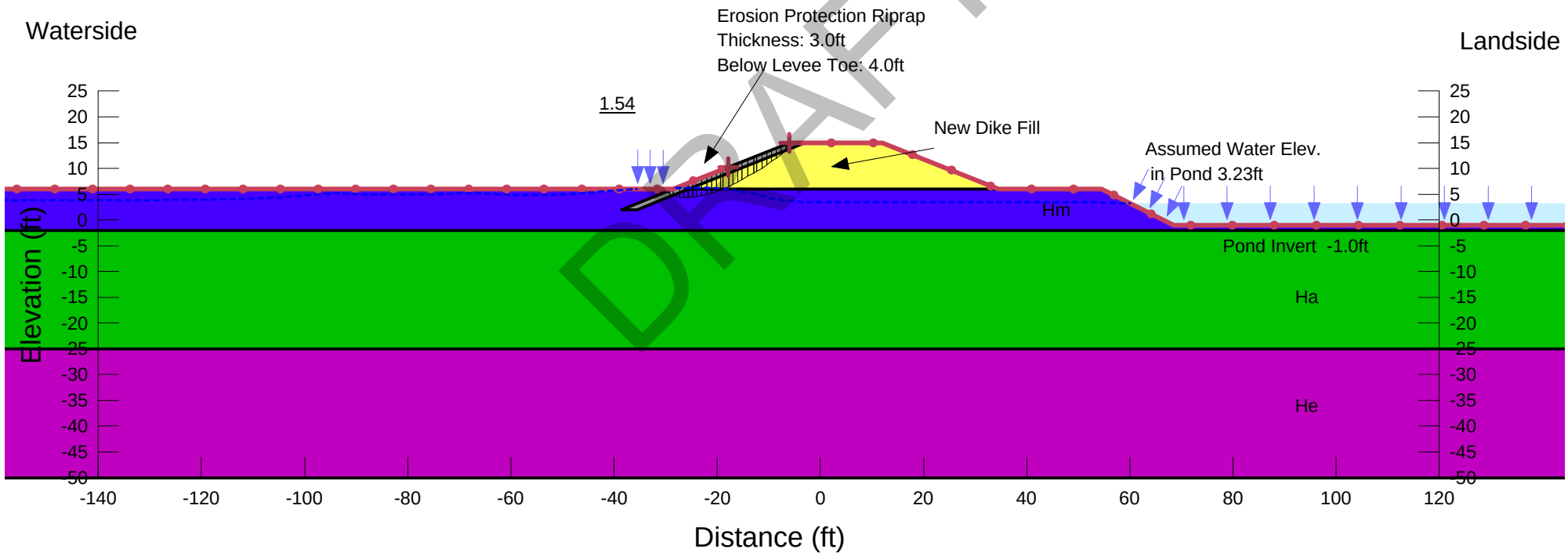


File Name: Pond 2.5to1 - Rev1.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Pond\
Date: 3/28/2014

Figure A-13

Fir Island 60% Design
21-1-12318-224
Pond Section
Levee Slope 2.5H :1V

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °



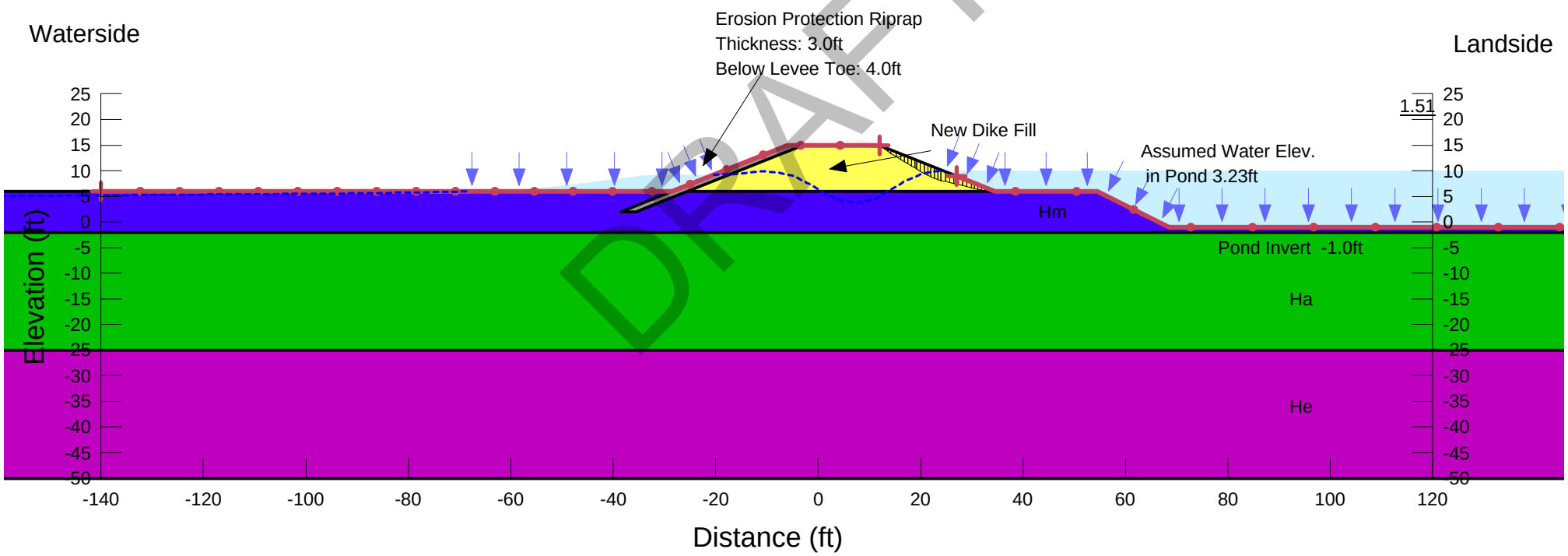
File Name: Pond 2.5to1 - Rev1.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Pond\
Date: 3/28/2014

Figure A-14

Fir Island 60% Design
21-1-12318-224
Pond Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 4: Interior Flooding
F of S: 1.51

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °



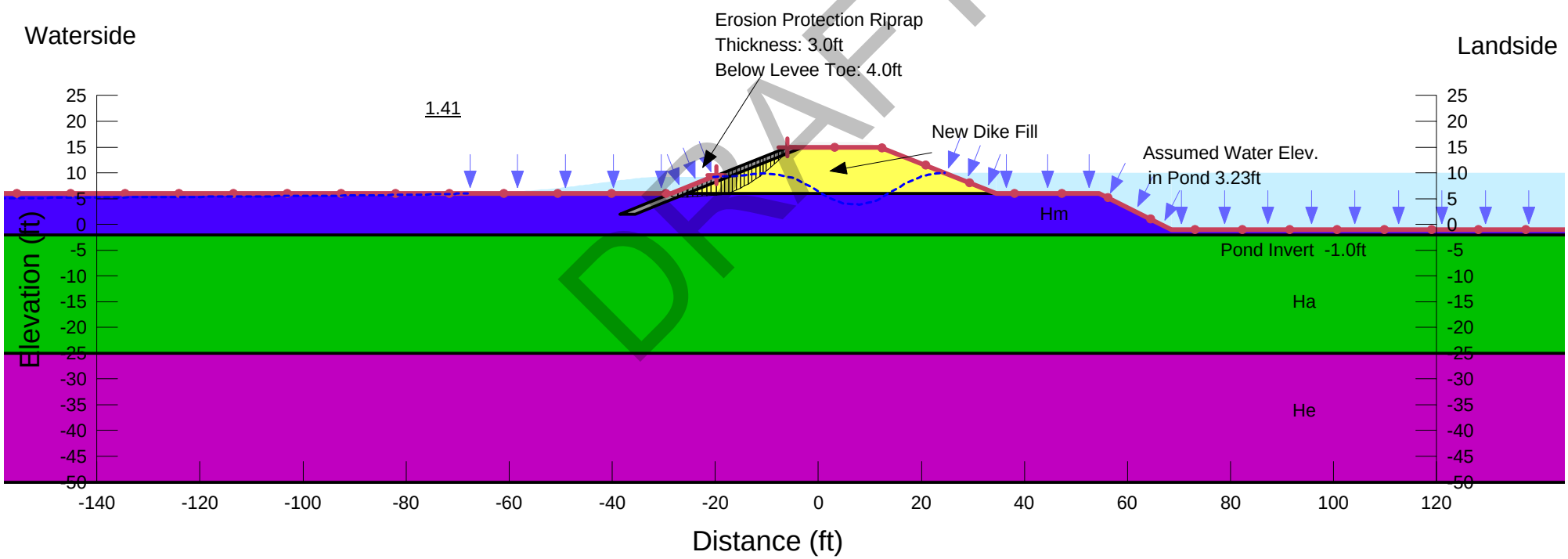
File Name: Pond 2.5to1 - Rev1.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Pond\
Date: 3/28/2014

Figure A-15

Fir Island 60% Design
21-1-12318-224
Pond Section
Levee Slope 2.5H :1V

SlopeW Analysis
Case 4: Interior Flooding
F of S: 1.41

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °



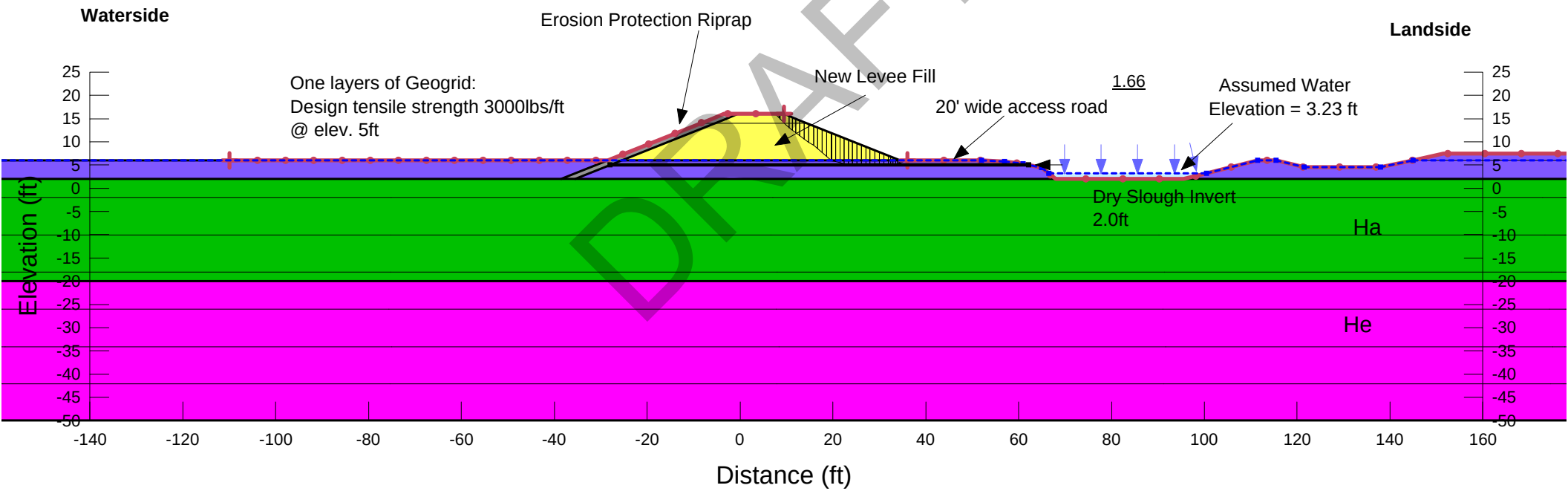
File Name: Pond 2.5to1 - Rev1.gsz,Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Pond\
Date: 3/28/2014

Figure A-16

Fir Island 60% Design
21-1-12318-224
Dry Slough Section
Levee Slope 2.5H:1V

SlopeW Analysis
Case 1: End of Construction
F of S: 1.66

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 ° Piezometric Line: 1
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Hm-undrained Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 200 psf Phi': 0 ° Piezometric Line: 1
Name: O-He-Undrained Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 350 psf Phi': 15 ° Piezometric Line: 1



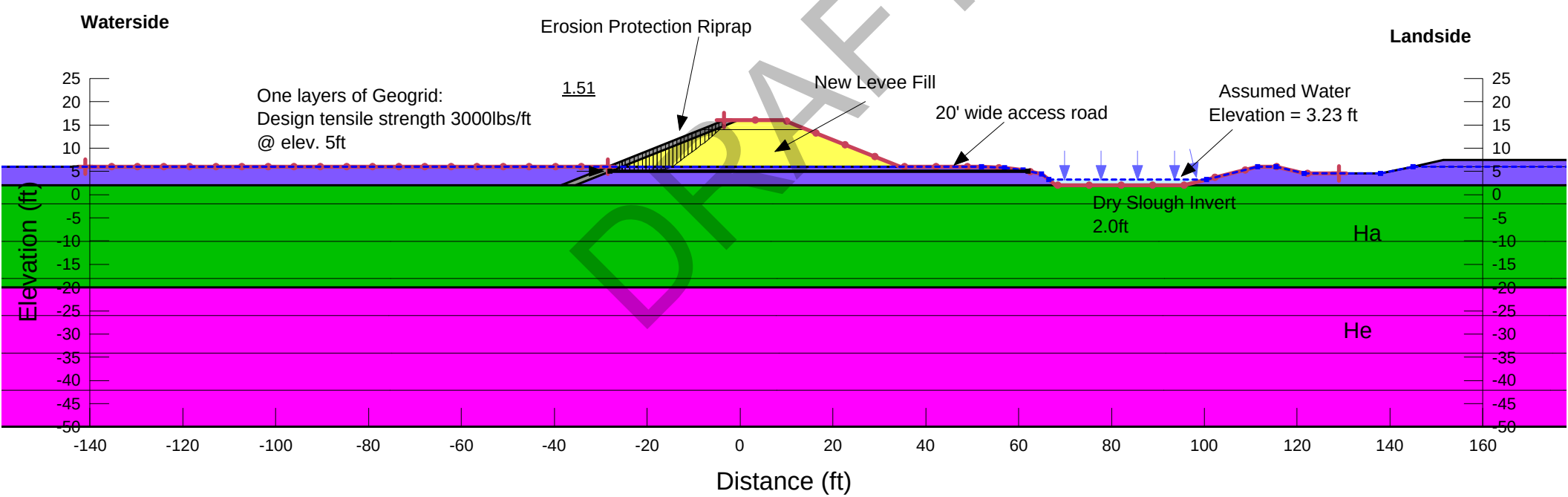
File Name: Dry Slough 2.5to1 - Seeps-Rev2.gsz, Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Dry Slough\
Last Solved Date: 4/29/2014

Figure A-17

Fir Island 60% Design
21-1-12318-224
Dry Slough Section
Levee Slope 2.5H:1V

SlopeW Analysis
Case 1: End of Construction
F of S: 1.51

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 ° Piezometric Line: 1
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 ° Piezometric Line: 1
Name: O-Hm-undrained Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 200 psf Phi': 0 ° Piezometric Line: 1
Name: O-He-Undrained Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 350 psf Phi': 15 ° Piezometric Line: 1



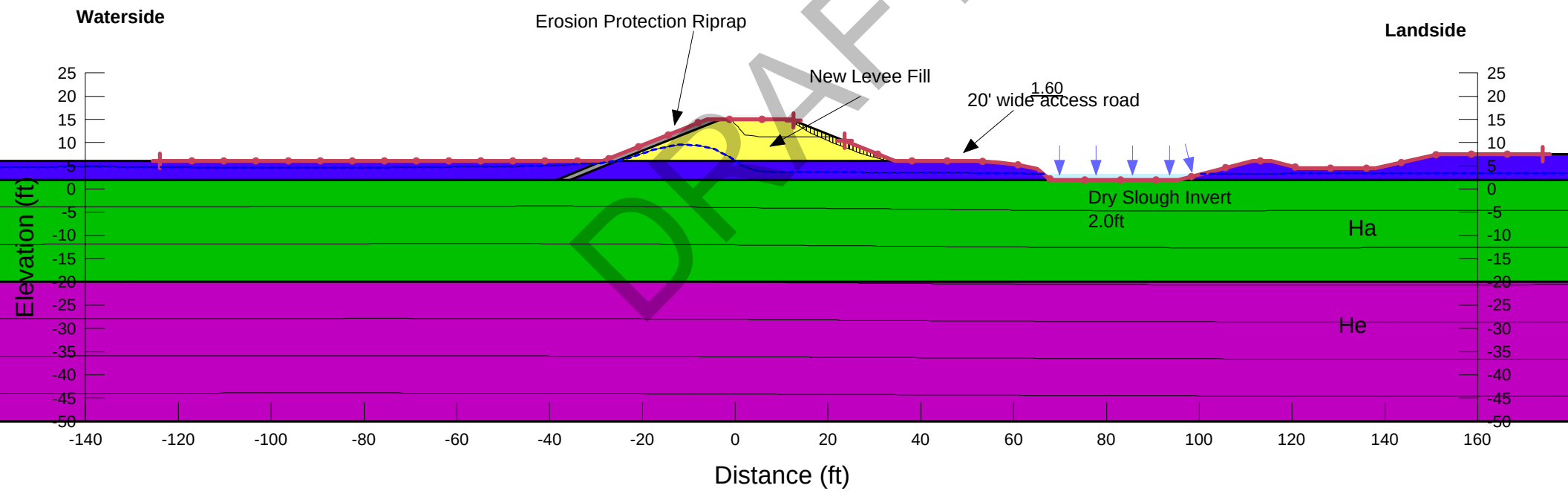
File Name: Dry Slough 2.5to1 - Seeps-Rev2.gsz, Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Dry Slough\
Last Solved Date: 4/29/2014

Figure A-18

Fir Island 60% Design
21-1-12318-224
Dry Slough Section
Levee Slope 2.5H:1V

SlopeW Analysis
Case 2: Storm Drawdown
F of S: 1.60

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °



DRAFT

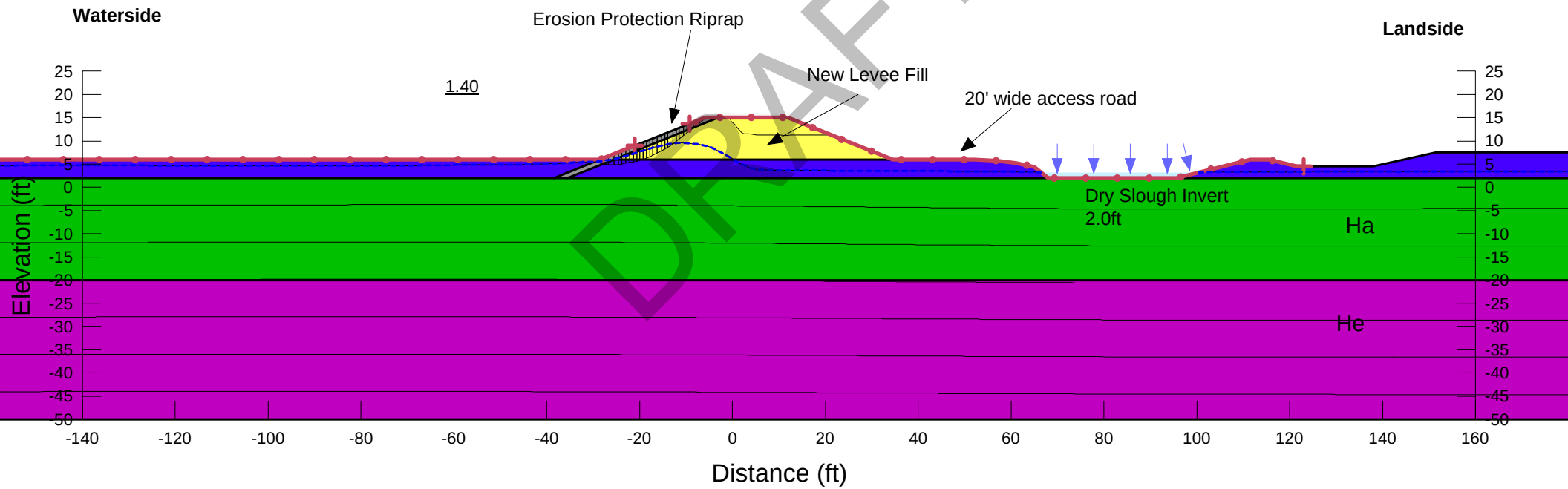
File Name: Dry Slough 2.5to1 - Seeps-Rev1.gsz, Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Dry Slough\
Last Solved Date: 3/31/2014

Figure A-19

Fir Island 60% Design
21-1-12318-224
Dry Slough Section
Levee Slope 2.5H:1V

SlopeW Analysis
Case 2: Storm Drawdown
F of S: 1.40

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °



DRAFT

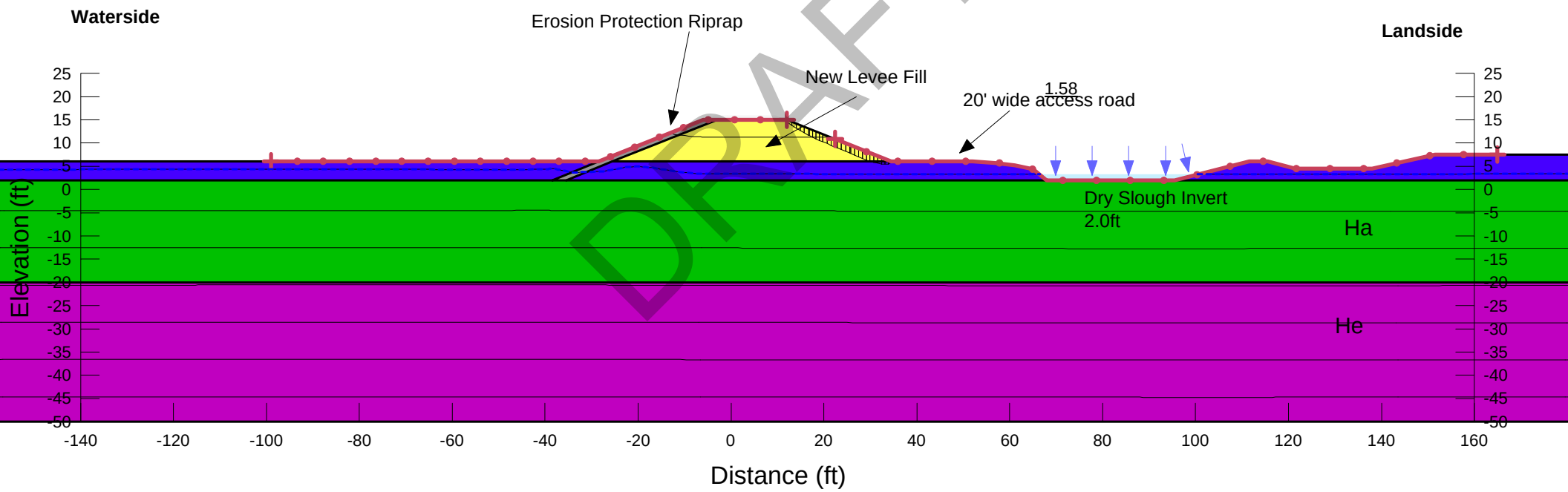
File Name: Dry Slough 2.5to1 - Seeps-Rev1.gsz, Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Dry Slough\
Last Solved Date: 3/31/2014

Figure A-20

Fir Island 60% Design
21-1-12318-224
Dry Slough Section
Levee Slope 2.5H:1V

SlopeW Analysis
Case 3: Daily Tidal Drawdown
F of S: 1.58

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °



DRAFT

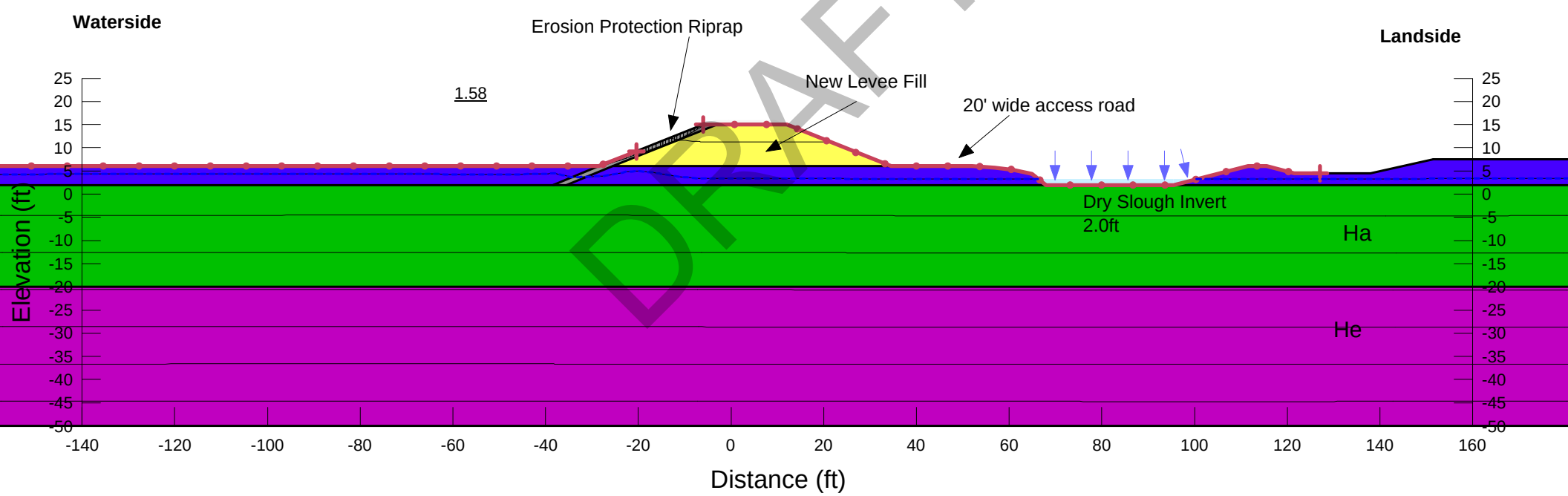
File Name: Dry Slough 2.5to1 - Seeps-Rev1.gsz, Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Dry Slough\
Last Solved Date: 3/31/2014

Figure A-21

Fir Island 60% Design
21-1-12318-224
Dry Slough Section
Levee Slope 2.5H:1V

SlopeW Analysis
Case 3: Daily Tidal Drawdown
F of S: 1.58

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °



DRAFT

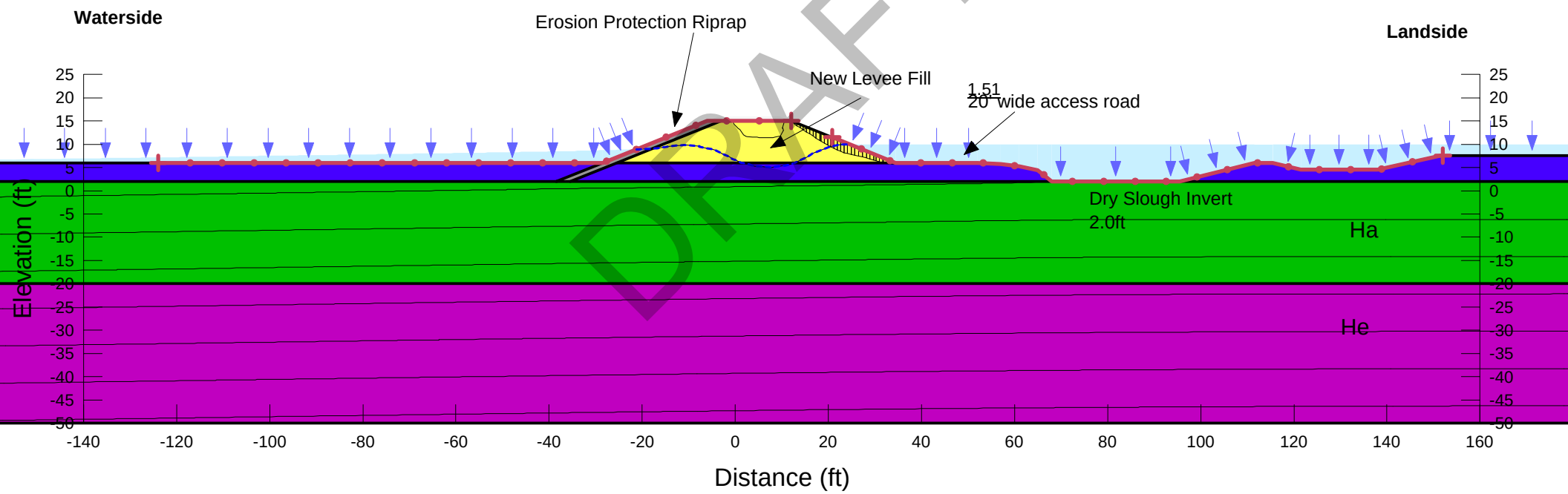
File Name: Dry Slough 2.5to1 - Seeps-Rev1.gsz, Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Dry Slough\
Last Solved Date: 3/31/2014

Figure A-22

Fir Island 60% Design
21-1-12318-224
Dry Slough Section
Levee Slope 2.5H:1V

SlopeW Analysis
Case 4: Interior Flooding
F of S: 1.51

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °



DRAFT

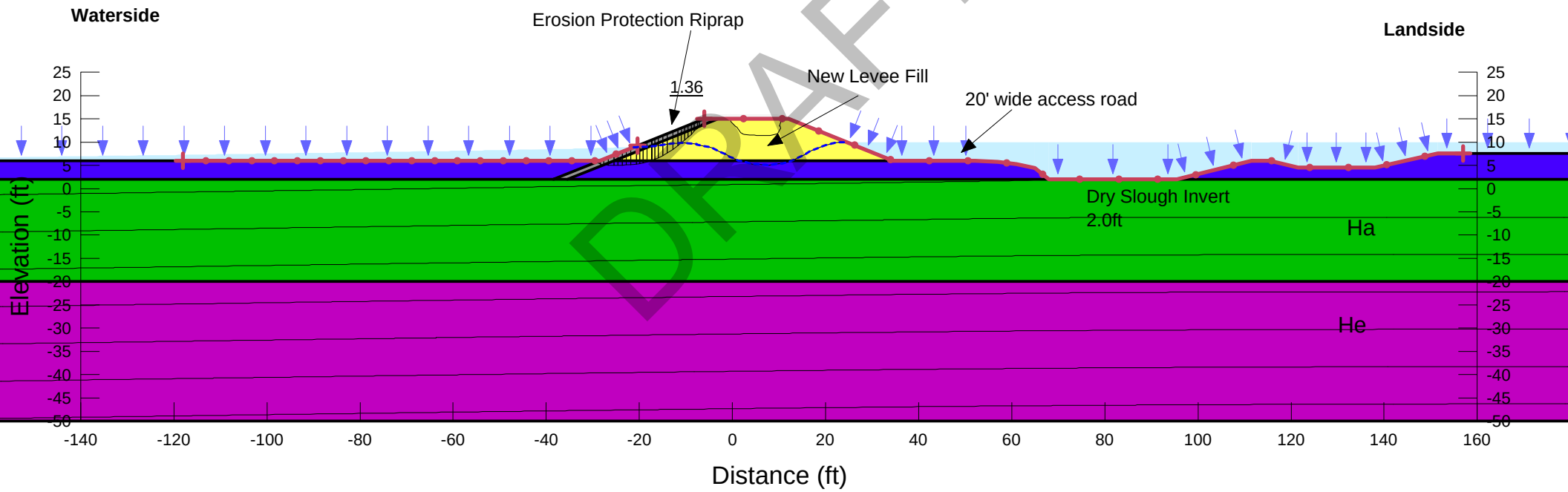
File Name: Dry Slough 2.5to1 - Seeps-Rev1.gsz, Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Dry Slough\
Last Solved Date: 3/31/2014

Figure A-23

Fir Island 60% Design
21-1-12318-224
Dry Slough Section
Levee Slope 2.5H:1V

SlopeW Analysis
Case 4: Interior Flooding
F of S: 1.36

Name: O-New Dike Fill Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 32 °
Name: O-Hm Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 28 °
Name: O-Ha Model: Mohr-Coulomb Unit Weight: 120 pcf Cohesion': 0 psf Phi': 30 °
Name: O-He Model: Mohr-Coulomb Unit Weight: 108 pcf Cohesion': 0 psf Phi': 27 °
Name: O-Riprap Model: Mohr-Coulomb Unit Weight: 130 pcf Cohesion': 0 psf Phi': 32 °



DRAFT

File Name: Dry Slough 2.5to1 - Seeps-Rev1.gsz, Directory: I:\WIP\21-1\12318 Fir Island\224_60% DESIGN\Global Stability\Section - Dry Slough\
Last Solved Date: 3/31/2014

Figure A-24

Draft

APPENDIX B

LIQUEFACTION ANALYSES AND RESULTS

APPENDIX B

LIQUEFACTION ANALYSES AND RESULTS

TABLE OF CONTENTS

Page

B-1 INTRODUCTION B-1

B-2 LIQUEFACTION ANALYSIS APPROACH B-1

B-3 LIQUEFACTION INDUCED SETTLEMENT APPROACH B-2

B-4 REFERENCES B-2

FIGURES

B-1 Results of Liquefaction Analyses, CPT-1-13, Magnitude 6.7
B-2 Results of Liquefaction Analyses, CPT-1-13, Magnitude 9.0
B-3 Results of Liquefaction Analyses, CPT-2-13, Magnitude 6.7
B-4 Results of Liquefaction Analyses, CPT-2-13, Magnitude 9.0
B-5 Results of Liquefaction Analyses, CPT-3-13, Magnitude 6.7
B-6 Results of Liquefaction Analyses, CPT-3-13, Magnitude 9.0
B-7 Results of Liquefaction Analyses, CPT-4-13, Magnitude 6.7
B-8 Results of Liquefaction Analyses, CPT-4-13, Magnitude 9.0
B-9 Results of Liquefaction Analyses, CPT-5-13, Magnitude 6.7
B-10 Results of Liquefaction Analyses, CPT-5-13, Magnitude 9.0
B-11 Results of Liquefaction Analyses, CPT-6-13, Magnitude 6.7
B-12 Results of Liquefaction Analyses, CPT-6-13, Magnitude 9.0
B-13 Results of Liquefaction Analyses, CPT-7-13, Magnitude 6.7
B-14 Results of Liquefaction Analyses, CPT-7-13, Magnitude 9.0
B-15 Results of Liquefaction Analyses, CPT-8-13, Magnitude 6.7
B-16 Results of Liquefaction Analyses, CPT-8-13, Magnitude 9.0
B-17 Results of Liquefaction Analyses, Boring B-1-13, Magnitude 6.7
B-18 Results of Liquefaction Analyses, Boring B-1-13, Magnitude 9.0
B-19 Results of Liquefaction Analyses, Boring B-2-13, Magnitude 6.7
B-20 Results of Liquefaction Analyses, Boring B-2-13, Magnitude 9.0
B-21 Results of Liquefaction Analyses, Boring B-3-13, Magnitude 6.7
B-22 Results of Liquefaction Analyses, Boring B-3-13, Magnitude 9.0
B-23 Results of Liquefaction Analyses, Boring B-4w-13, Magnitude 6.7
B-24 Results of Liquefaction Analyses, Boring B-4w-13, Magnitude 9.0

Draft

TABLE OF CONTENTS (cont.)

FIGURES (cont.)

B-25 Results of Liquefaction Analyses, Boring B-6w-13, Magnitude 6.7
B-26 Results of Liquefaction Analyses, Boring B-6w-13, Magnitude 9.0
B-27 Results of Liquefaction Analyses, Boring B-8w-13, Magnitude 6.7
B-28 Results of Liquefaction Analyses, Boring B-8w-13, Magnitude 9.0

Draft

APPENDIX B

LIQUEFACTION ANALYSES AND RESULTS

B-1 INTRODUCTION

We evaluated the liquefaction susceptibility of the foundation soil along the proposed setback dike using a ground motion level corresponding to the Operating Basis Earthquake (OBE). The U.S. Army Corps of Engineers (USACE) defines the OBE as "an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50-percent probability of exceedence during the service life" (USACE, 1995). Assuming the new dike has a service life of 100 years, this corresponds to about a 144-year return period.

The determination of the site ground motion and the liquefaction analyses results are discussed in the main text of this report. The analytical approach used in our evaluation is discussed in the following section. Plots of the factors of safety (FSs) against liquefaction versus depth are presented as Figures B-1 through B-28.

B-2 LIQUEFACTION ANALYSIS APPROACH

We evaluated the liquefaction potential along the proposed setback dike alignment using an in-house spreadsheet and the software program, LiquefyPro (CivilTech Software, 2007). Our analyses are based on the following empirical procedures:

- Robertson and Wride (1997)
- Youd and others (2001)
- Cetin and others (2004)
- Idriss and Boulanger (2006)

For empirical liquefaction evaluation, the Standard Penetration Test N-value and the cone penetration test cone tip resistance are correlated to the liquefaction resistance of the soil (expressed as cyclic resistance ratio). Other factors affecting the liquefaction resistance include the fines content for a granular soil and the Atterberg Limits plasticity index for a cohesive soil. The soil resistance is compared to the earthquake-induced loading (expressed as cyclic stress ratio), and a corresponding FS against liquefaction is calculated. For our analyses, we assumed the groundwater level to be at the ground surface.

Draft

B-3 LIQUEFACTION INDUCED SETTLEMENT APPROACH

Potential effects of liquefaction include post-earthquake settlement. We evaluated these effects using our in-house spreadsheet and the software program, LiquefyPro (CivilTech Software, 2007).

Several factors influence the post-earthquake settlement of liquefied soils, including soil density, induced shear strains, and maximum excess porewater pressures. Two widely used empirical estimates of post-earthquake settlement were developed by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992) and used in our analyses. Both empirical methods are based on case histories and laboratory testing of clean, saturated sands.

B-4 REFERENCES

Cetin, K.O., Seed, R.B., Der Kiureghian, Armen; and others, 2004, Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential: Journal of Geotechnical and Geoenvironmental Engineering, v. 130, no. 12, p. 1314-1340.

CivilTech Software, 2007, Liquefaction analyses of CPT results with LiquefyPro version 5.5b.

Idriss, I.M., and Boulanger, R.W., 2006, Semi-empirical procedures for evaluating liquefaction potential during earthquakes: Soil Dynamics and Earthquake Engineering, v. 26, no. 2-4, p. 115-130.

Ishihara, K., and Yoshimine, M., 1992, Evaluation of settlements in sand deposits following liquefaction during earthquakes, Soils and Foundations, JSSMFE, v. 32, no. 1, March, p. 173-188.

Robertson, P.K., and Wride, C.E., 1997, Cyclic liquefaction and its evaluation based on the SPT and CPT: Proc. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022, p. 41-88.

Tokimatsu, K., and Seed, H.B., 1987, Evaluation of settlement in sands due to earthquake shaking: ASCE Journal of Geotechnical Engineering, v. 113, no. 89, August.

U.S. Army Corps of Engineers (USACE), Engineering and Design, 1995, Earthquake Design and Evaluation for Civil Works Projects, Engineering Report 1110-2-1806: Washington, D.C., July.

Youd, T.L., Idriss, I.M., Andrus, R.D., and others, 2001, Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils: Journal of Geotechnical and Geoenvironmental Engineering, v. 127, no. 10, p. 817-833.

Draft

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-1-13 Water Depth=0 ft Surface Elev.=6

Magnitude=6.7
Acceleration=0.21g

DRAFT

LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-1-13 Water Depth=0 ft Surface Elev.=6

Magnitude=9.0
Acceleration=0.21g

DRAFT

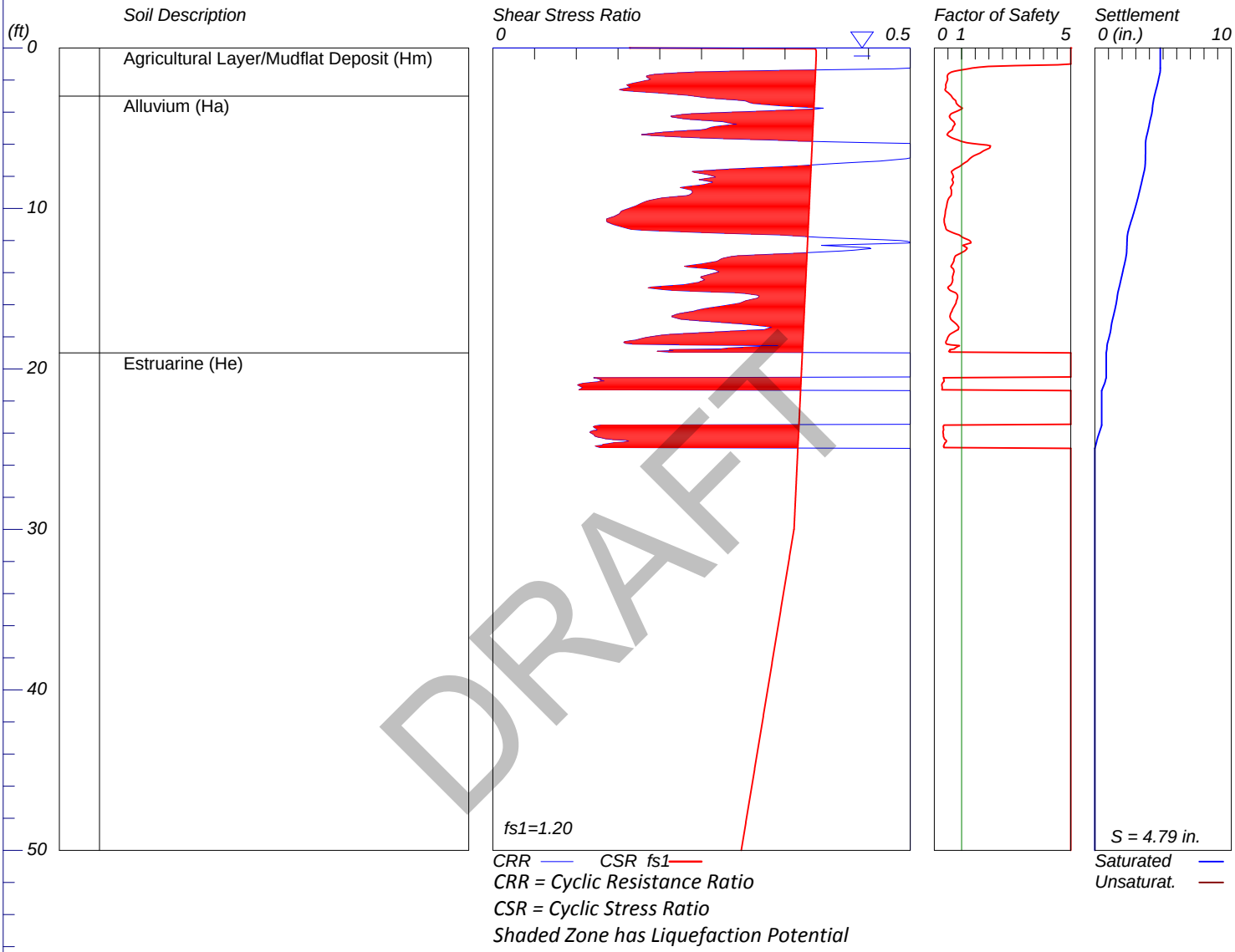
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-2-13 Water Depth=0 ft Surface Elev.=6

Magnitude=6.7
Acceleration=0.21g



DRAFT

LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-2-13 Water Depth=0 ft Surface Elev.=6

Magnitude=9.0
Acceleration=0.21g

DRAFT

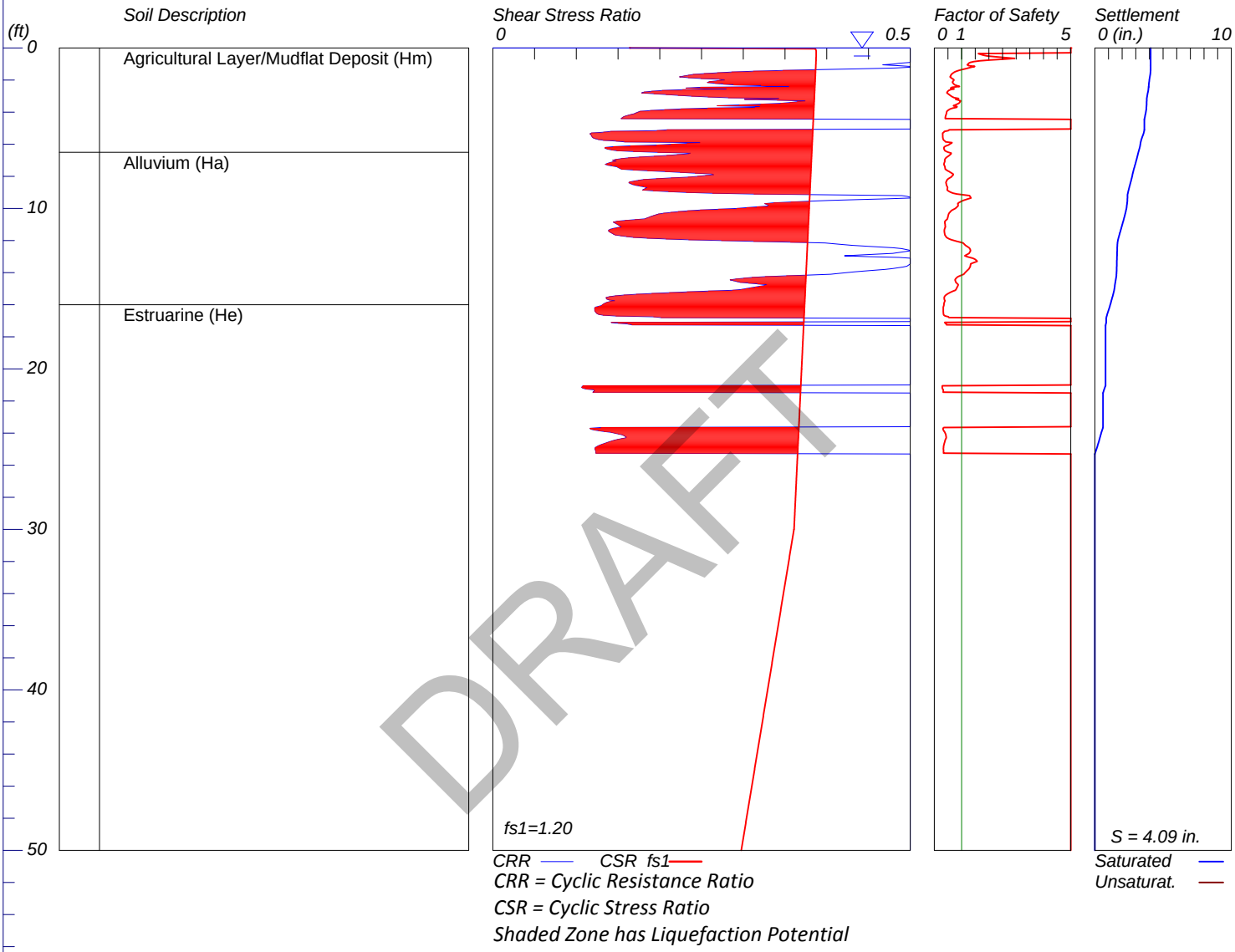
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-3-13 Water Depth=0 ft Surface Elev.=6

Magnitude=6.7
Acceleration=0.21g



DRAFT

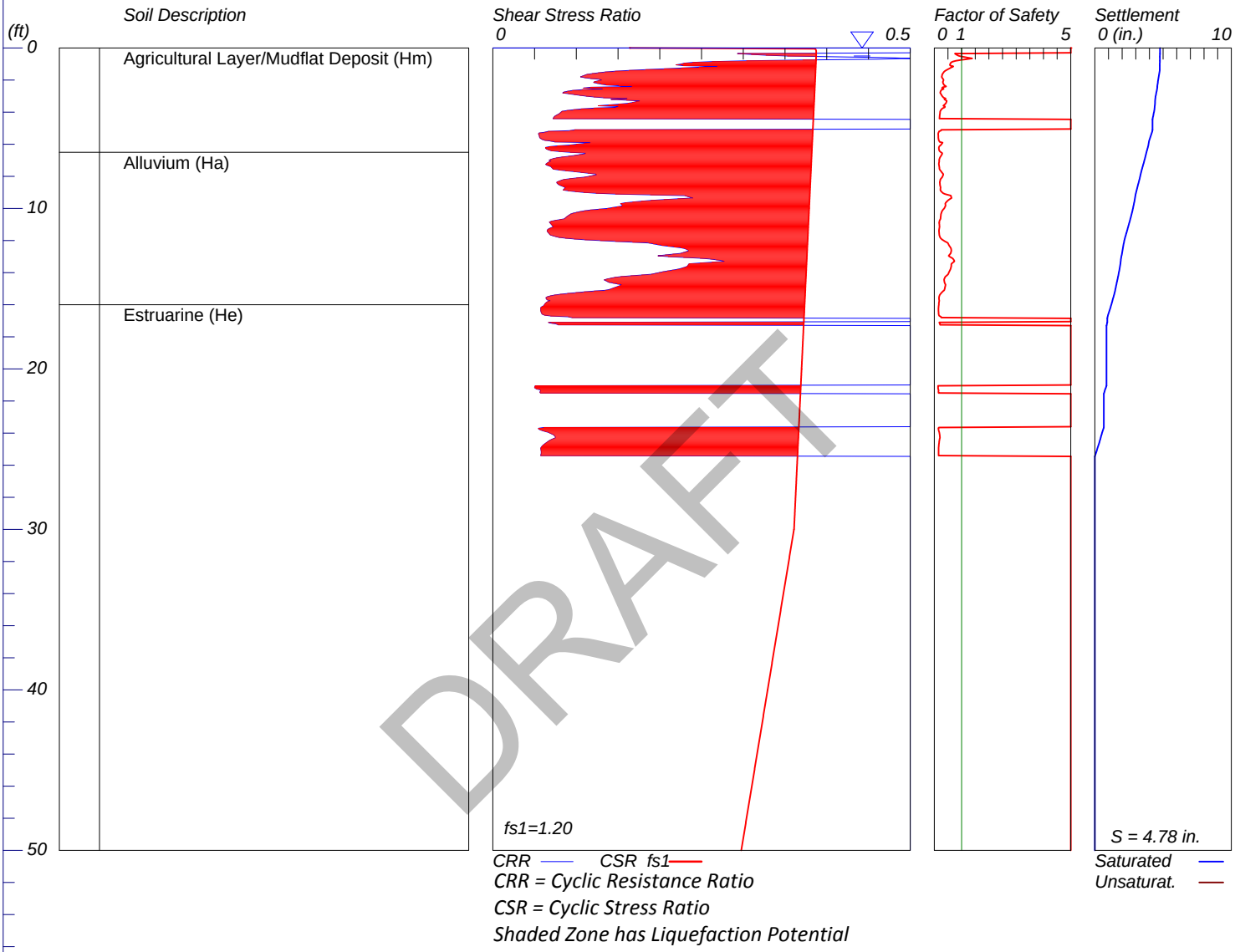
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-3-13 Water Depth=0 ft Surface Elev.=6

Magnitude=9.0
Acceleration=0.21g



DRAFT

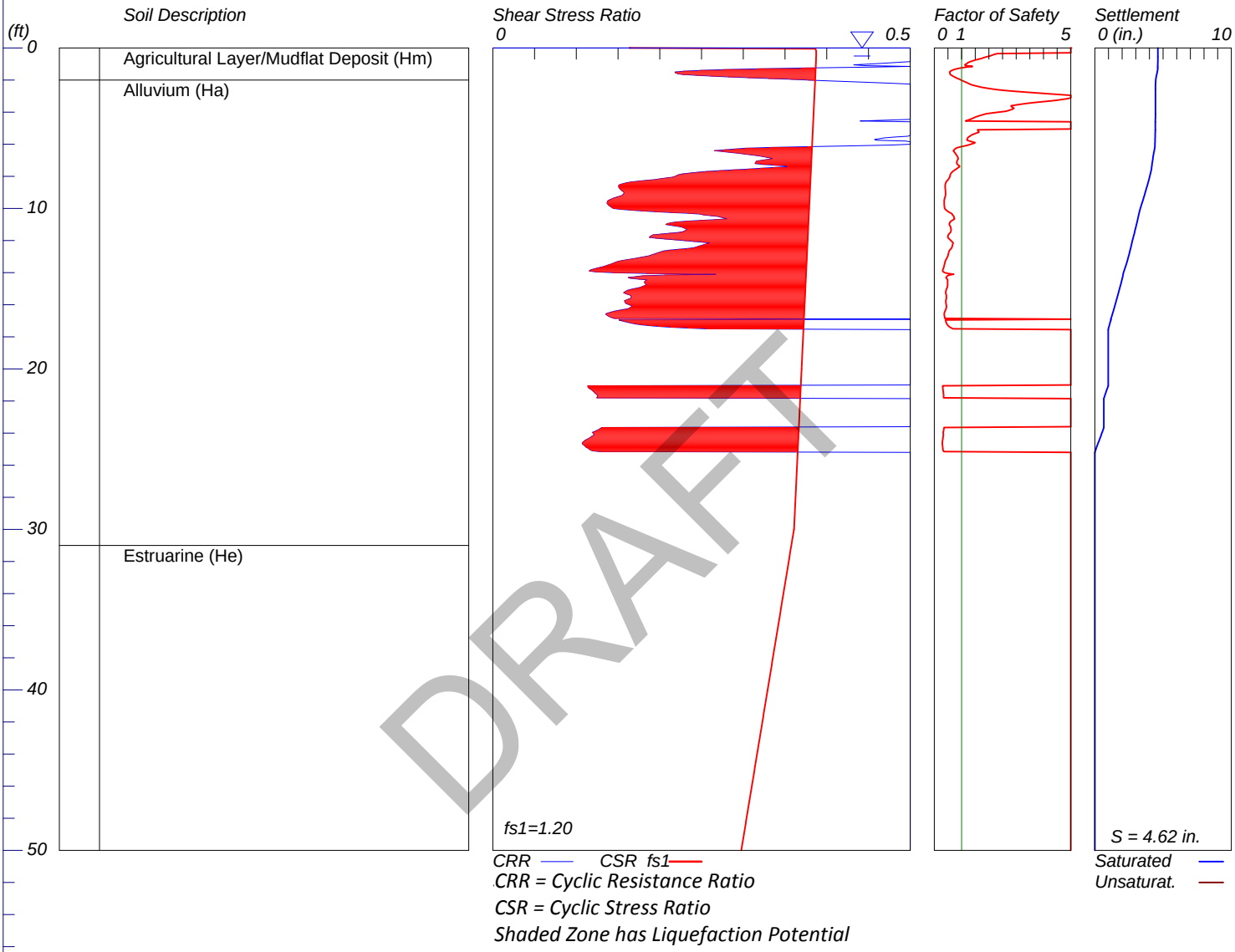
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-4-13 Water Depth=0 ft Surface Elev.=6

Magnitude=6.7
Acceleration=0.21g



DRAFT

LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-4-13 Water Depth=0 ft Surface Elev.=6

Magnitude=9.0
Acceleration=0.21g

DRAFT

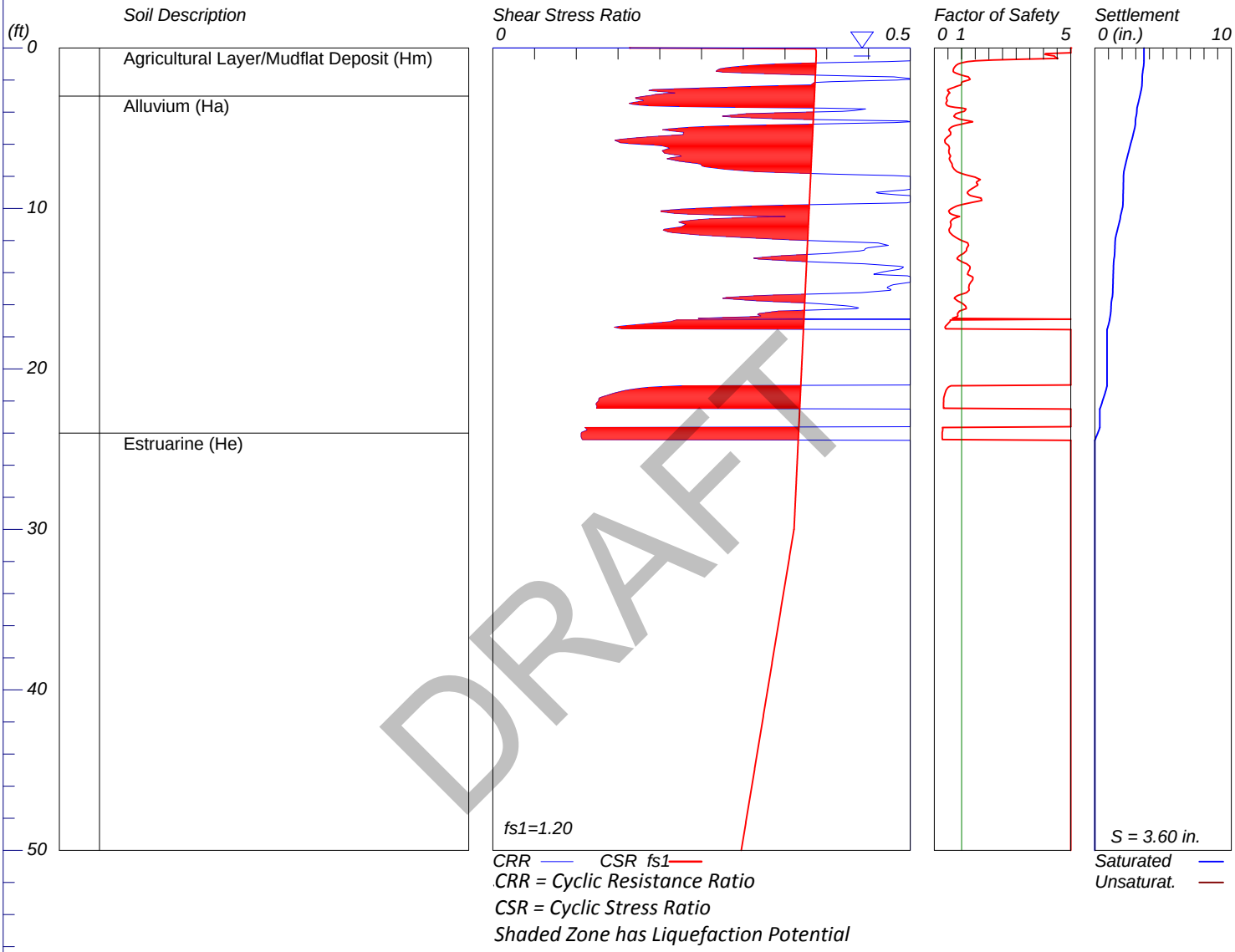
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-5-13 Water Depth=0 ft Surface Elev.=6

Magnitude=6.7
Acceleration=0.21g



DRAFT

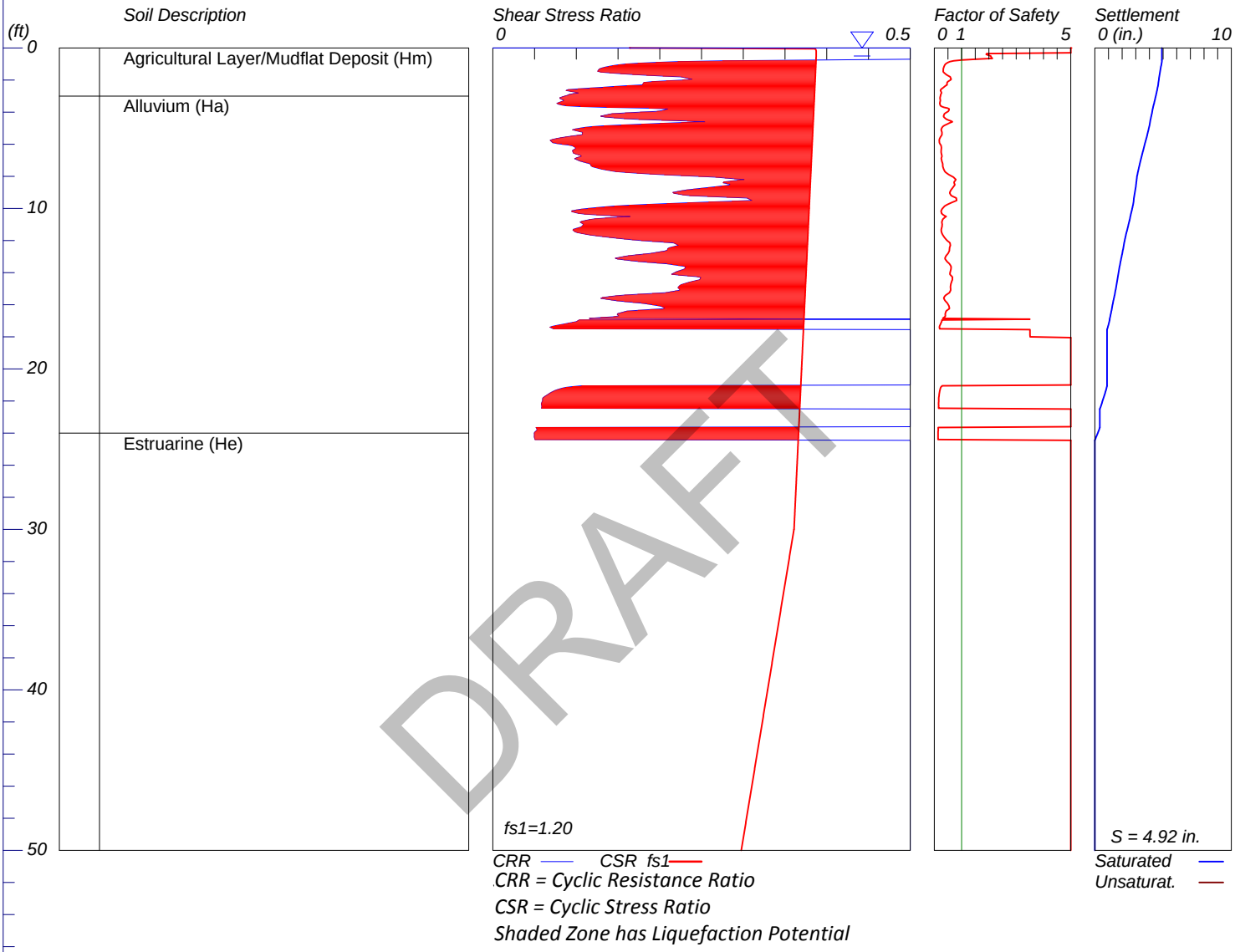
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-5-13 Water Depth=0 ft Surface Elev.=6

Magnitude=9.0
Acceleration=0.21g



DRAFT

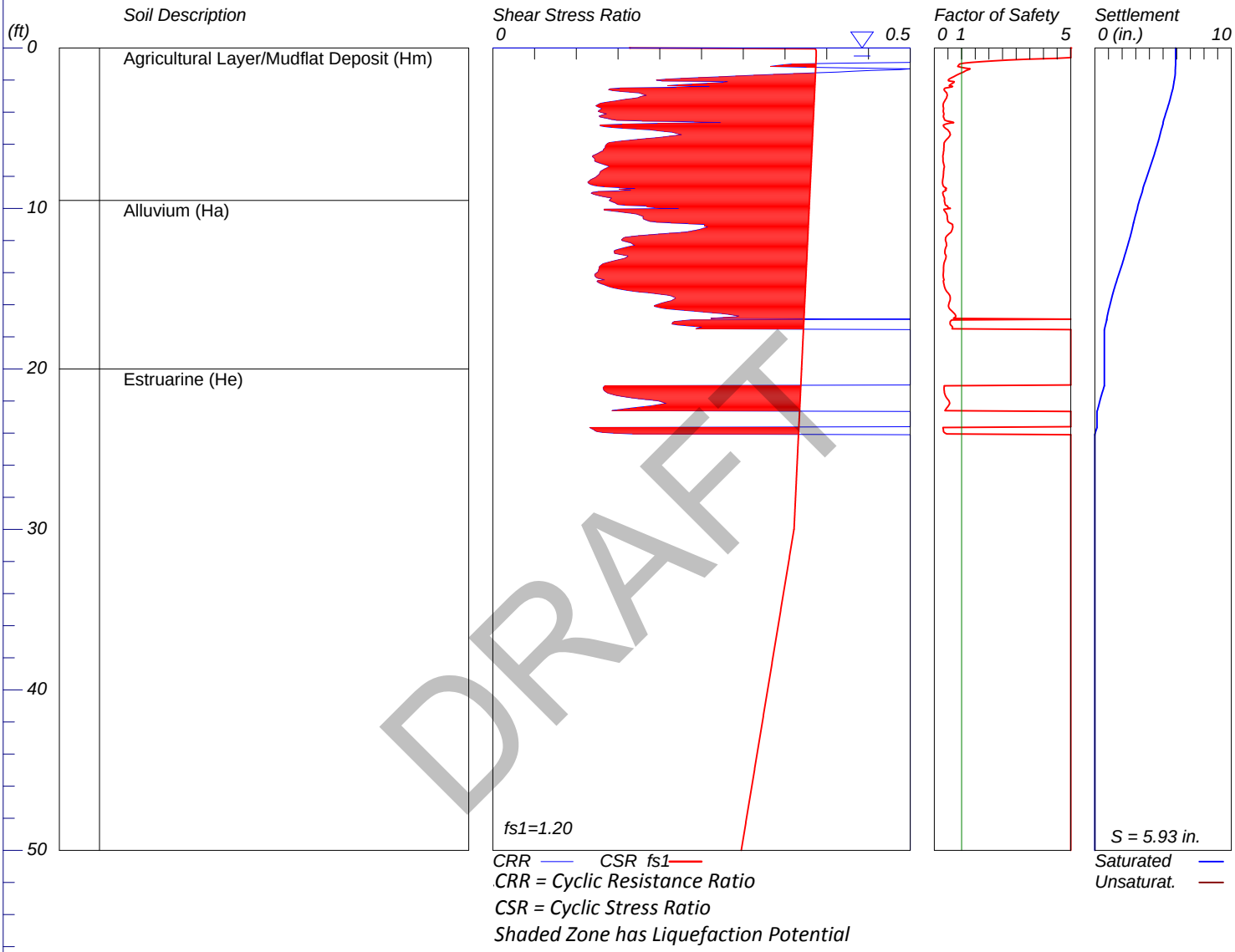
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-6-13 Water Depth=0 ft Surface Elev.=6

Magnitude=6.7
Acceleration=0.21g



DRAFT

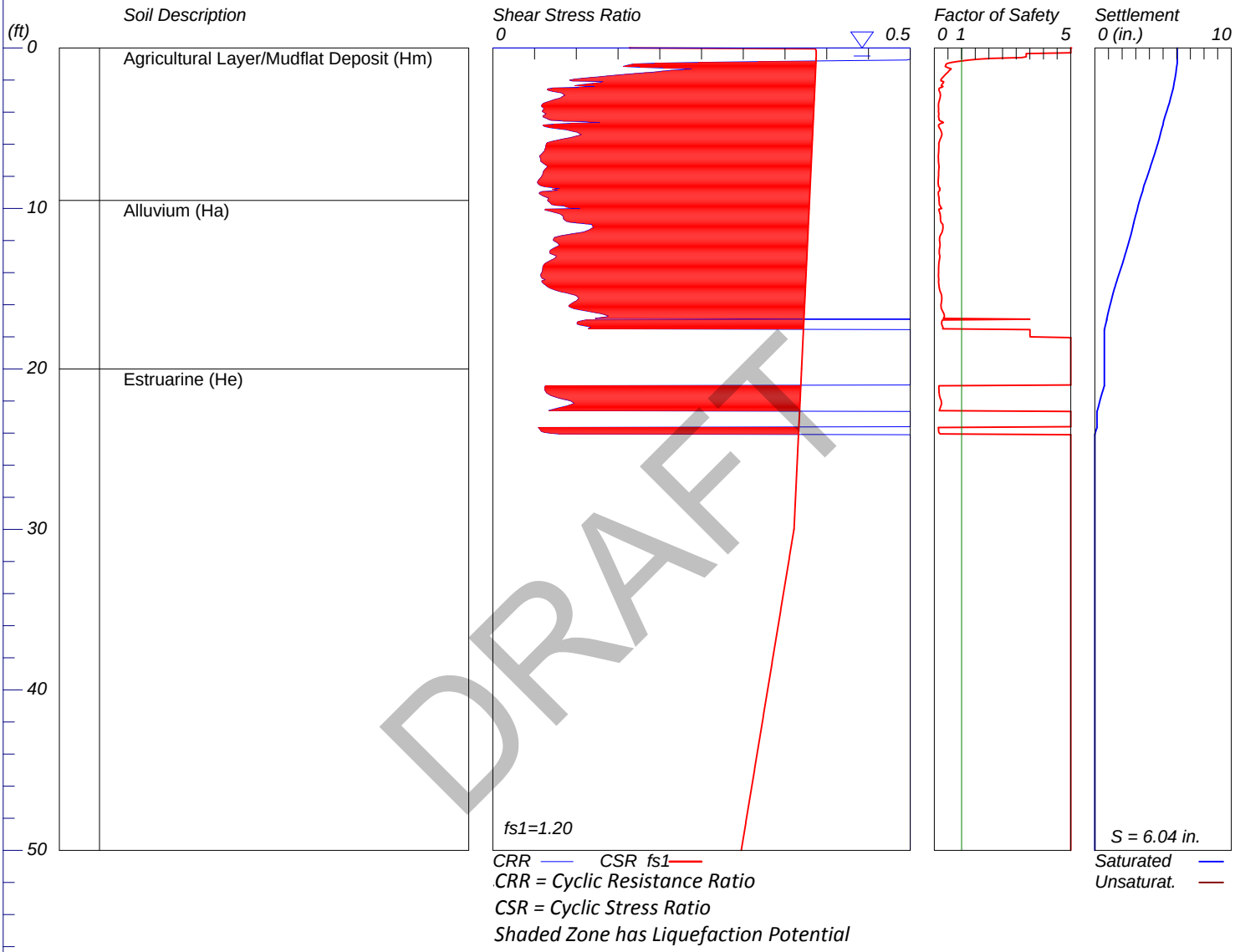
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-6-13 Water Depth=0 ft Surface Elev.=6

Magnitude=9.0
Acceleration=0.21g



DRAFT

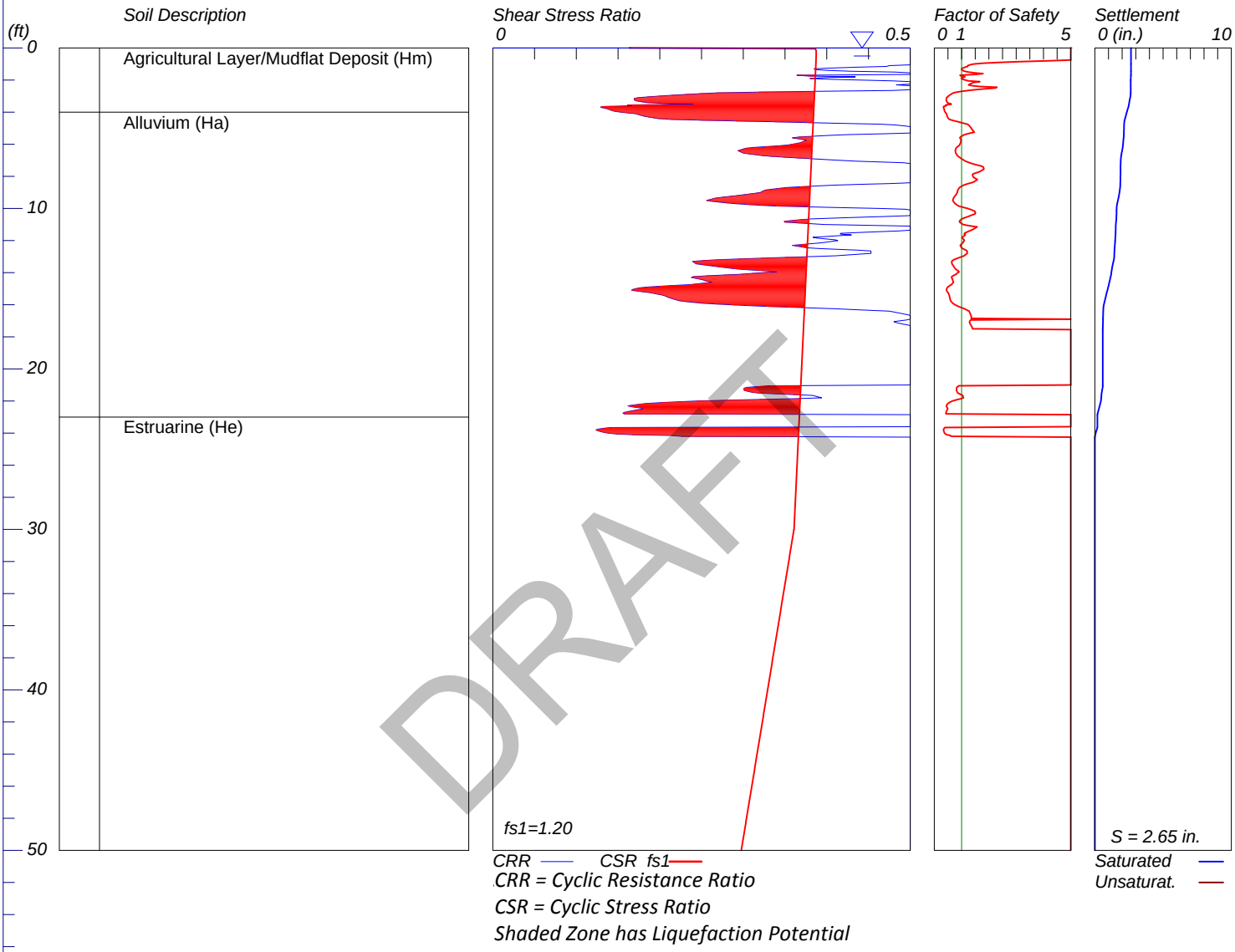
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-7-13 Water Depth=0 ft Surface Elev.=6

Magnitude=6.7
Acceleration=0.21g



DRAFT

LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-7-13 Water Depth=0 ft Surface Elev.=6

Magnitude=9.0
Acceleration=0.21g

CRR = Cyclic Resistance Ratio
CSR = Cyclic Stress Ratio
Shaded Zone has Liquefaction Potential

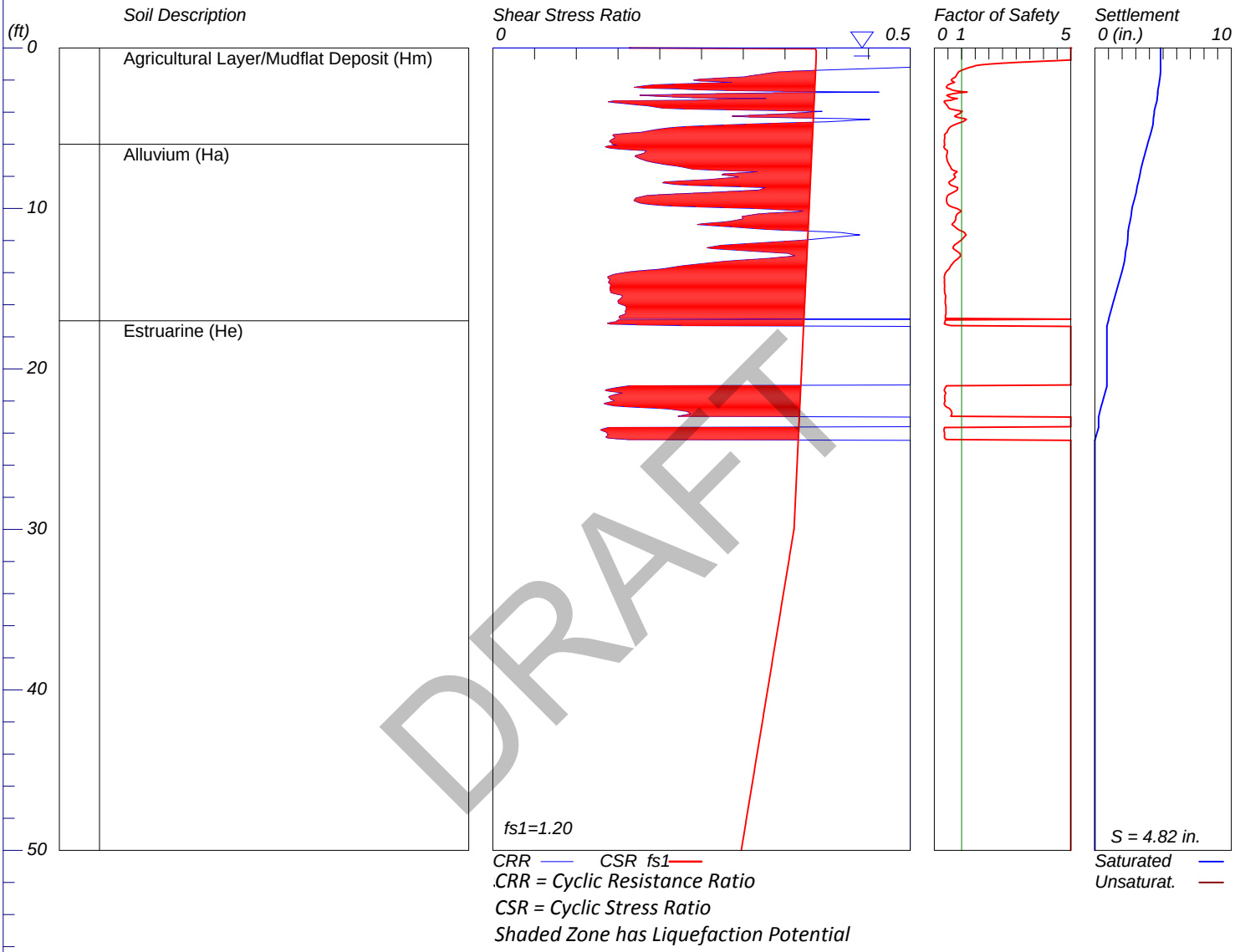
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-8-13 Water Depth=0 ft Surface Elev.=6

Magnitude=6.7
Acceleration=0.21g



CRR = Cyclic Resistance Ratio
CSR = Cyclic Stress Ratio
Shaded Zone has Liquefaction Potential

DRAFT

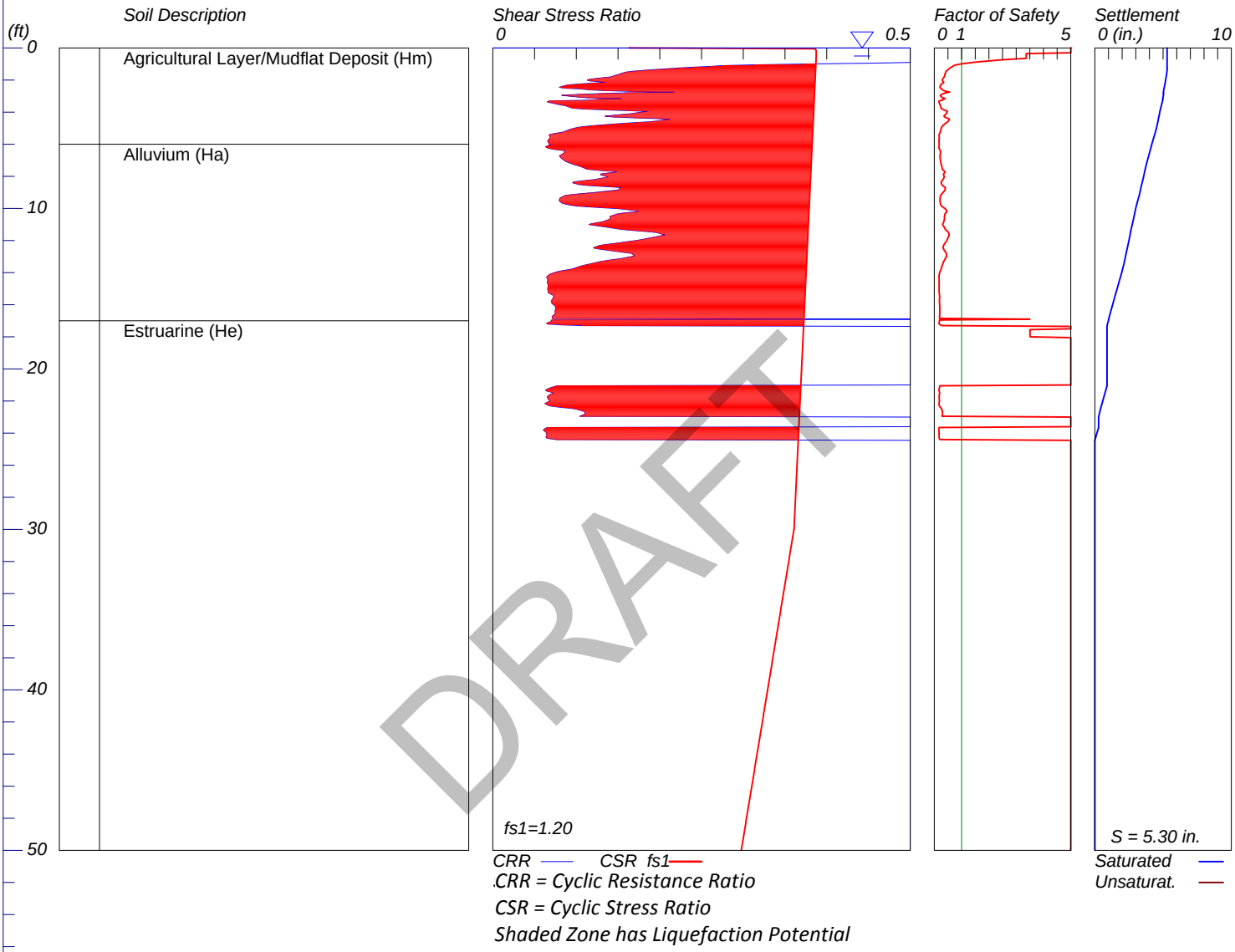
LiquefyPro CivilTech Software USA www.civiltech.com

LIQUEFACTION ANALYSIS

Fir Island

Hole No.=CPT-8-13 Water Depth=0 ft Surface Elev.=6

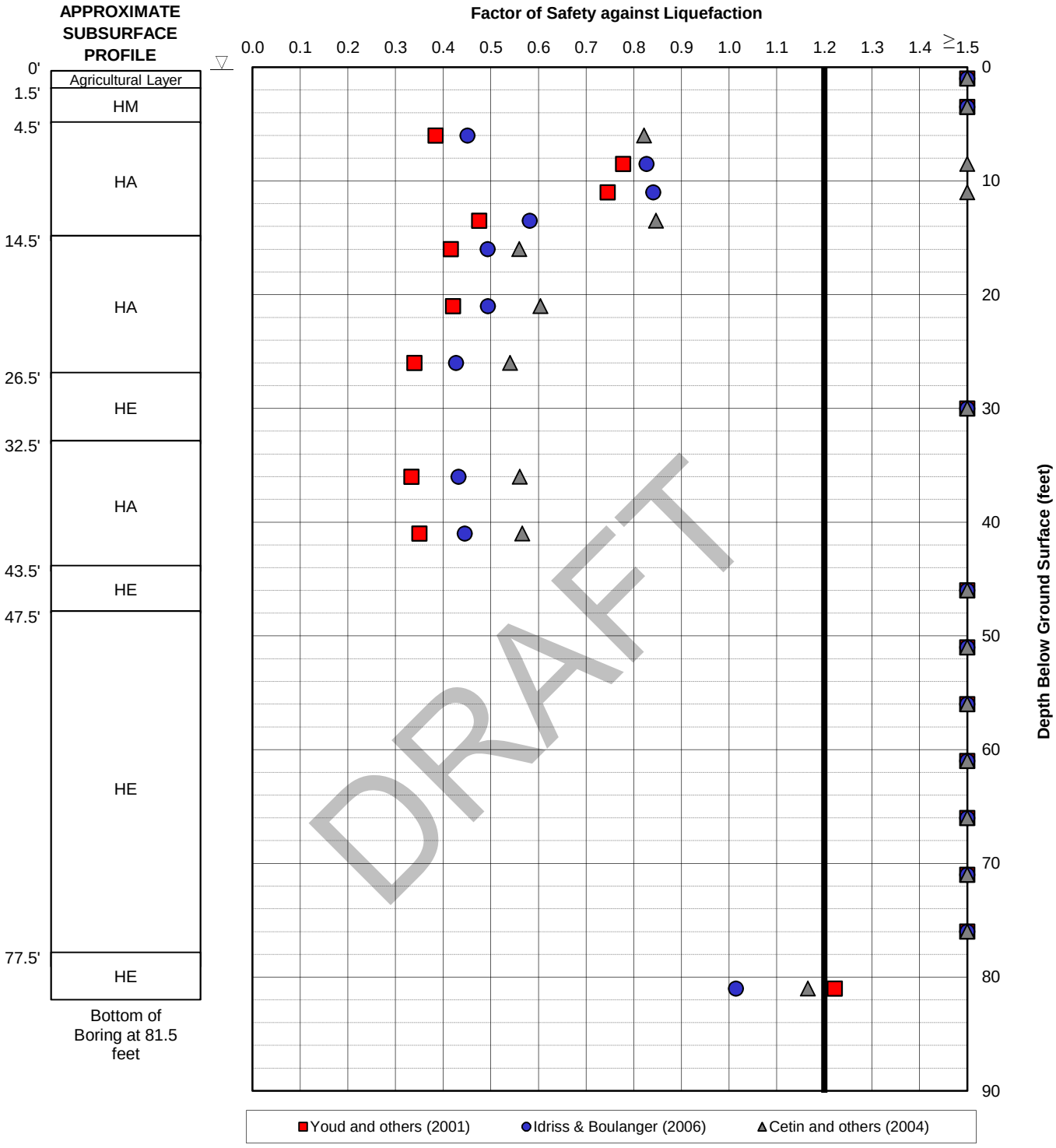
Magnitude=9.0
Acceleration=0.21g



CRR = Cyclic Resistance Ratio
CSR = Cyclic Stress Ratio
Shaded Zone has Liquefaction Potential

DRAFT

LiquefyPro CivilTech Software USA www.civiltech.com



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

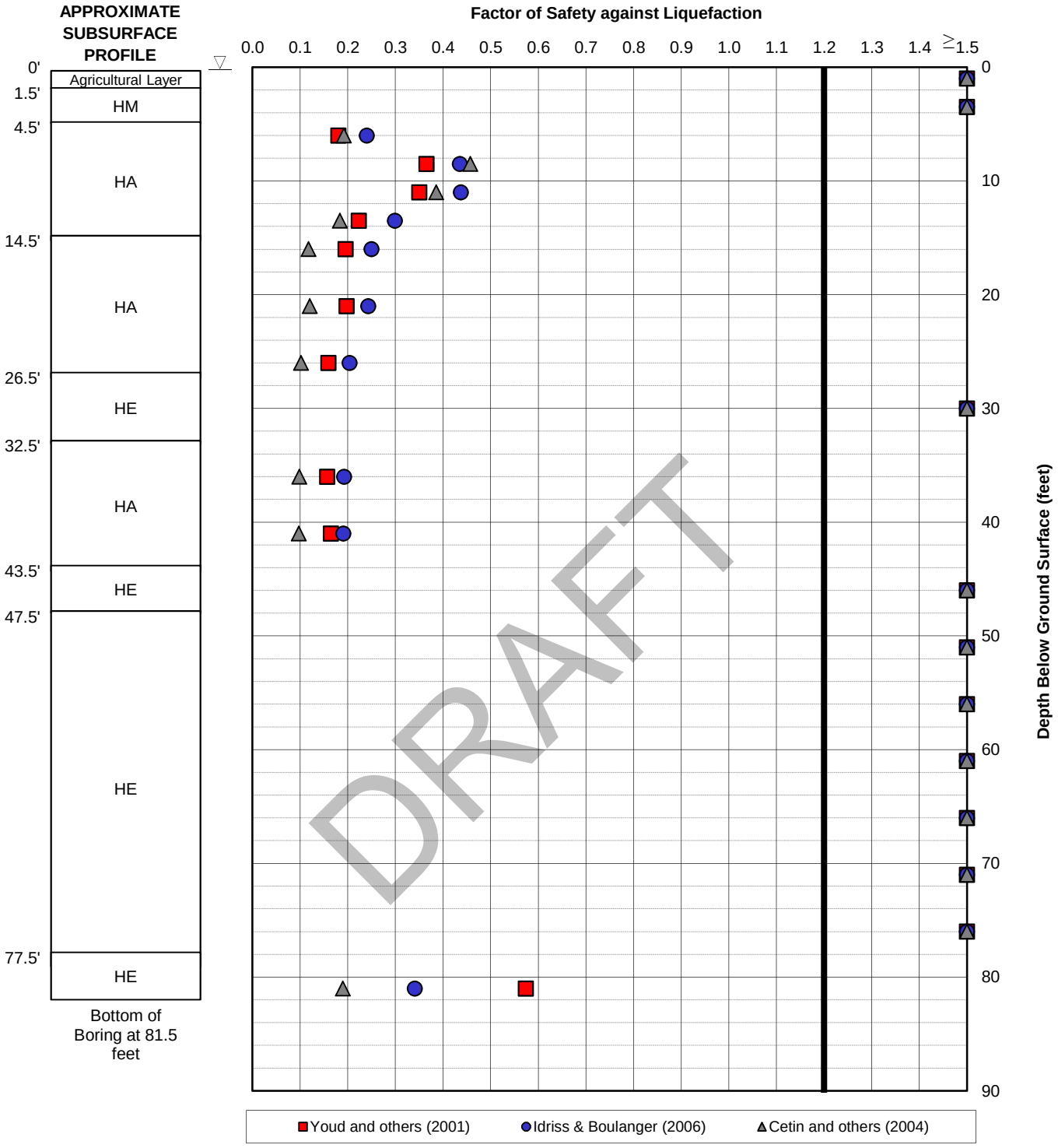
Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-1-13
M = 6.7, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-17



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

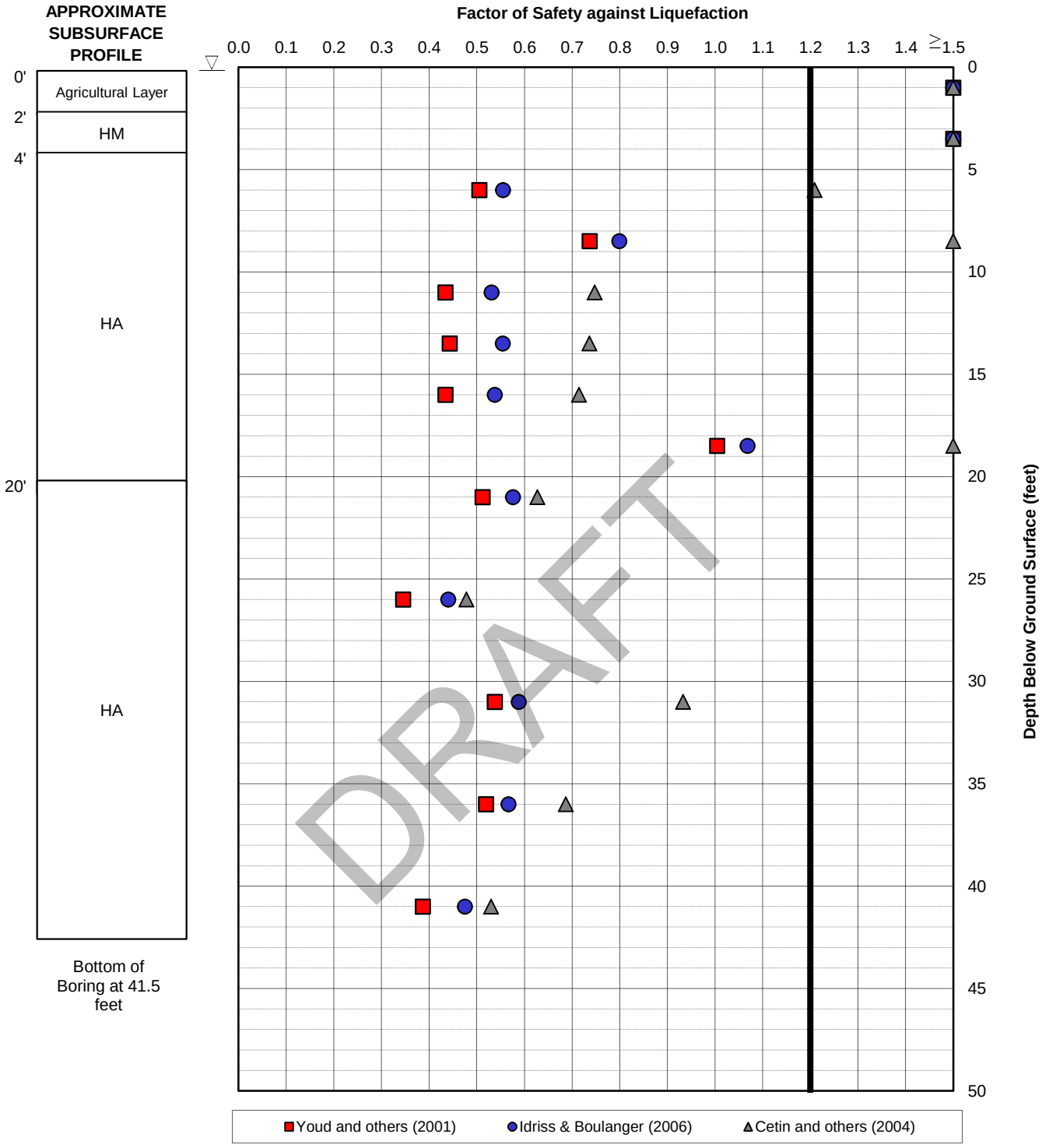
Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-1-13
M = 9.0, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-18



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

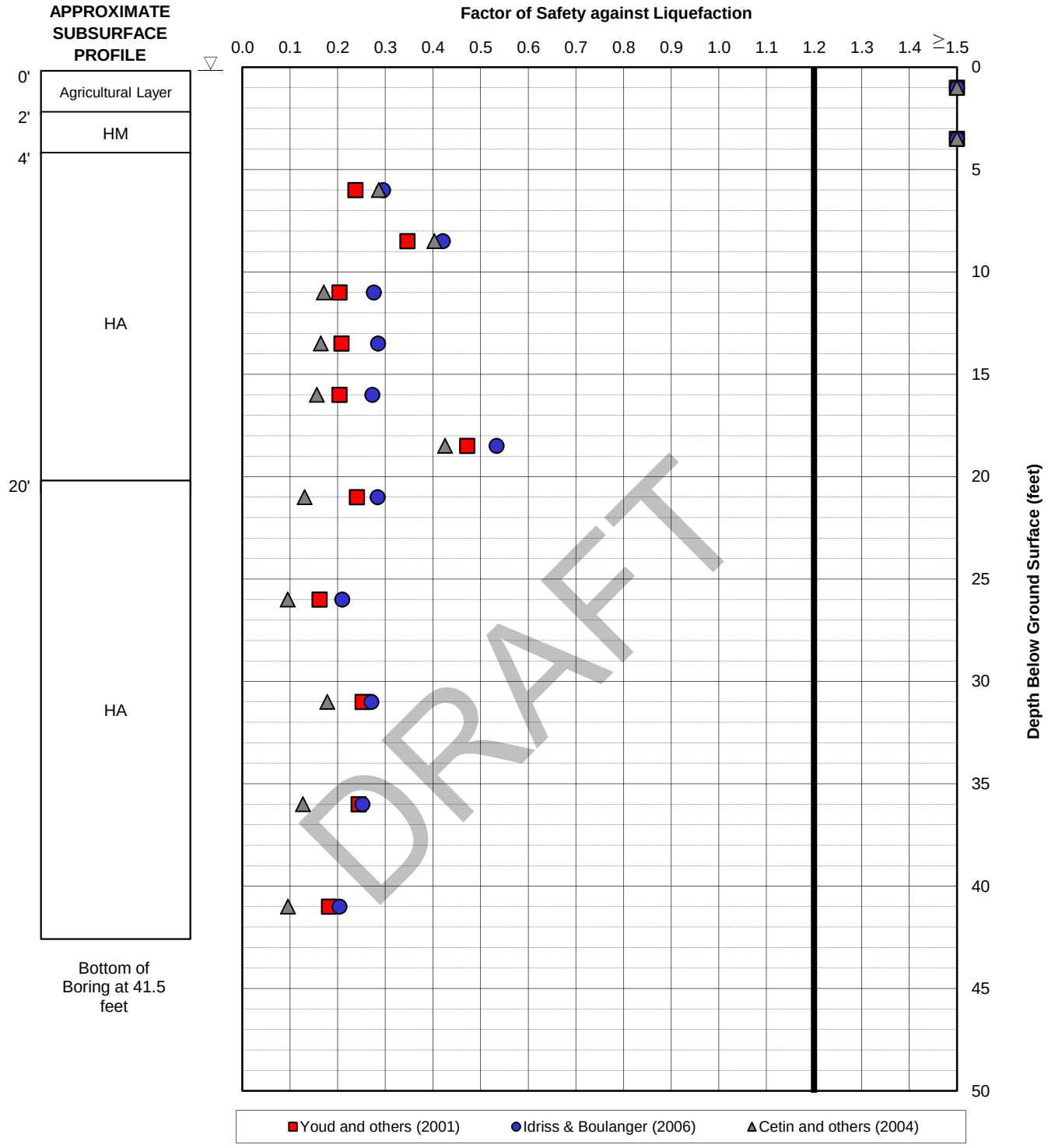
Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-2-13
M = 6.7, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-19



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

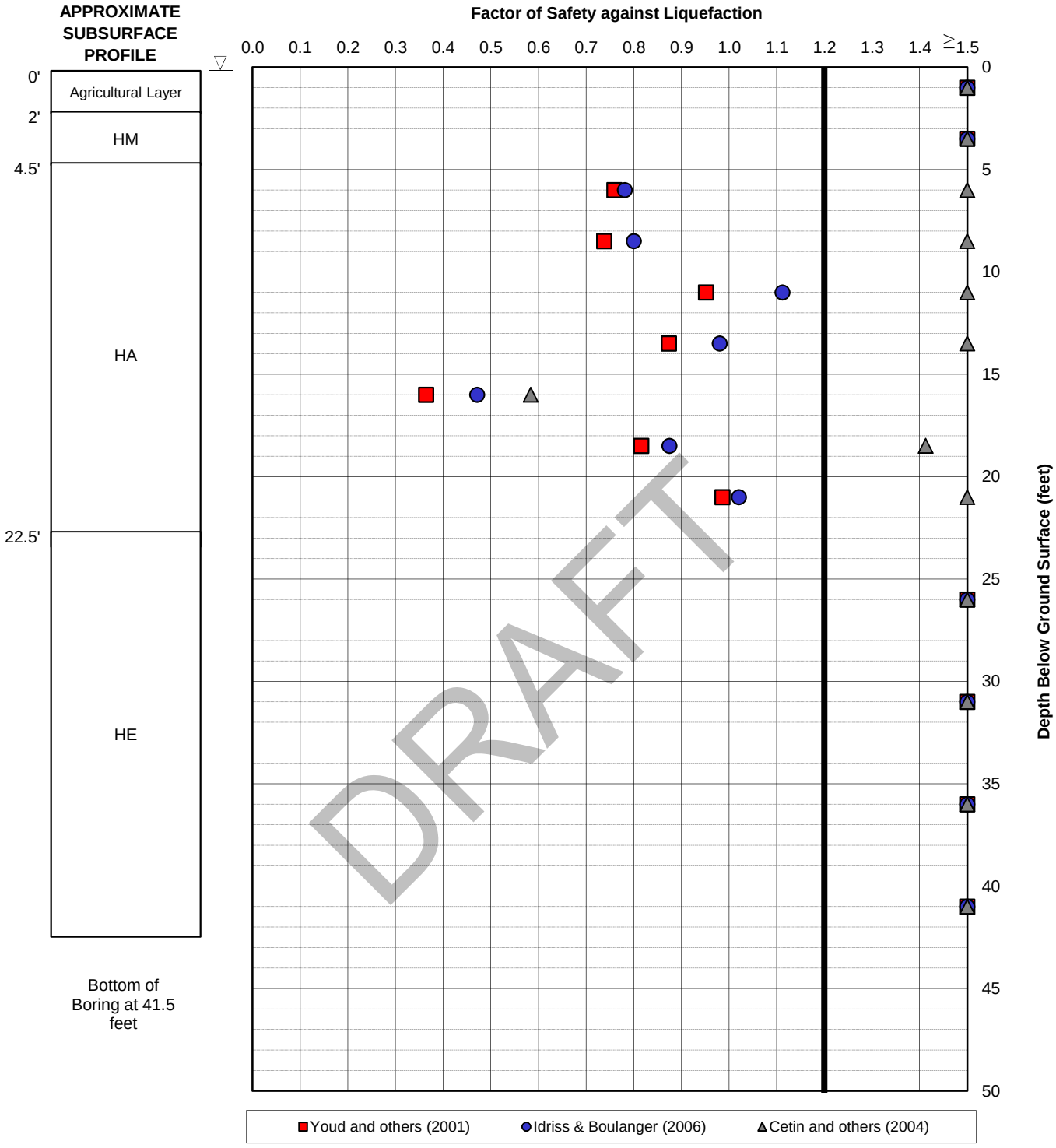
Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-2-13
M = 9.0, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-20



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

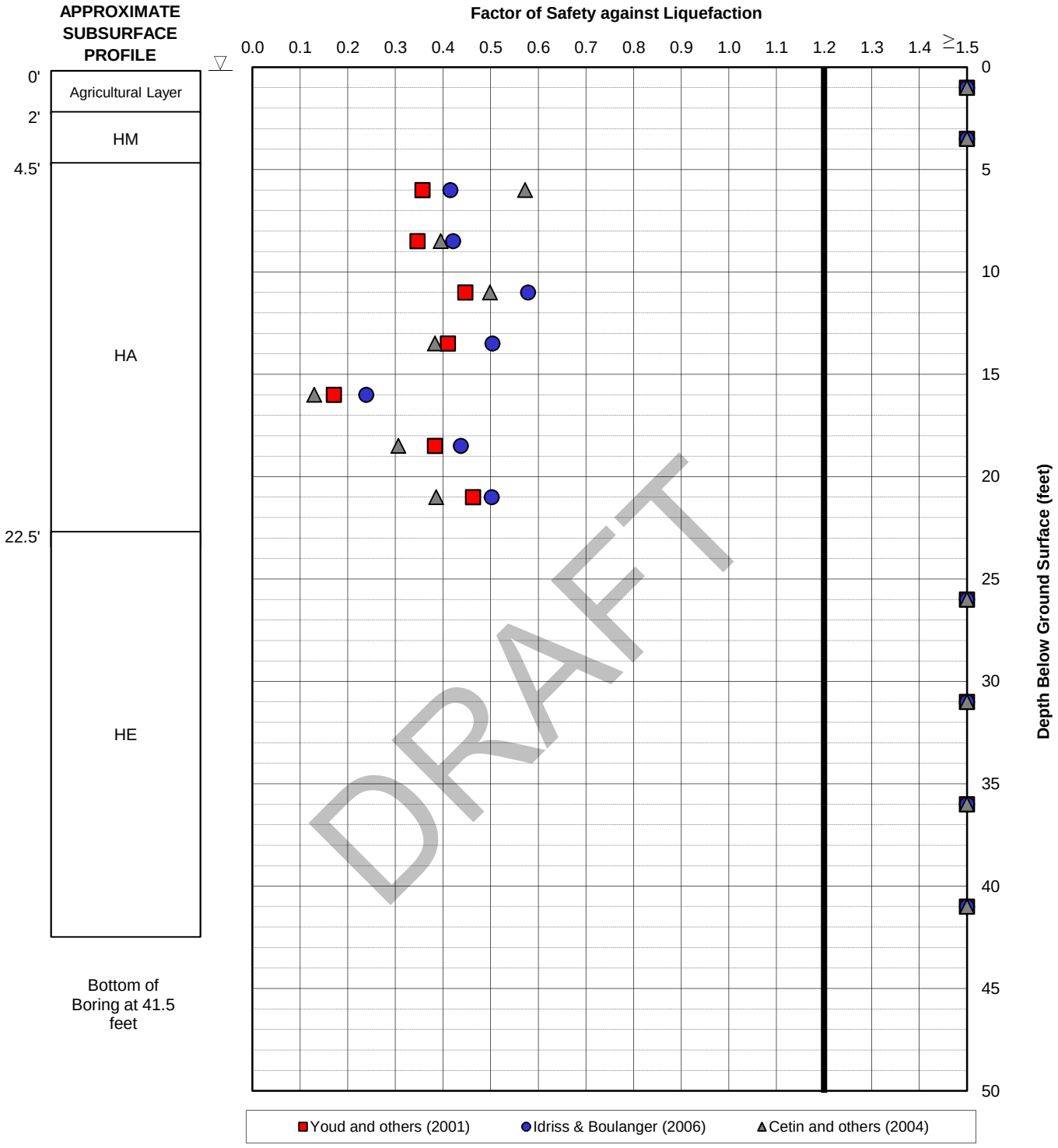
Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-3-13
M = 6.7, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-21



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

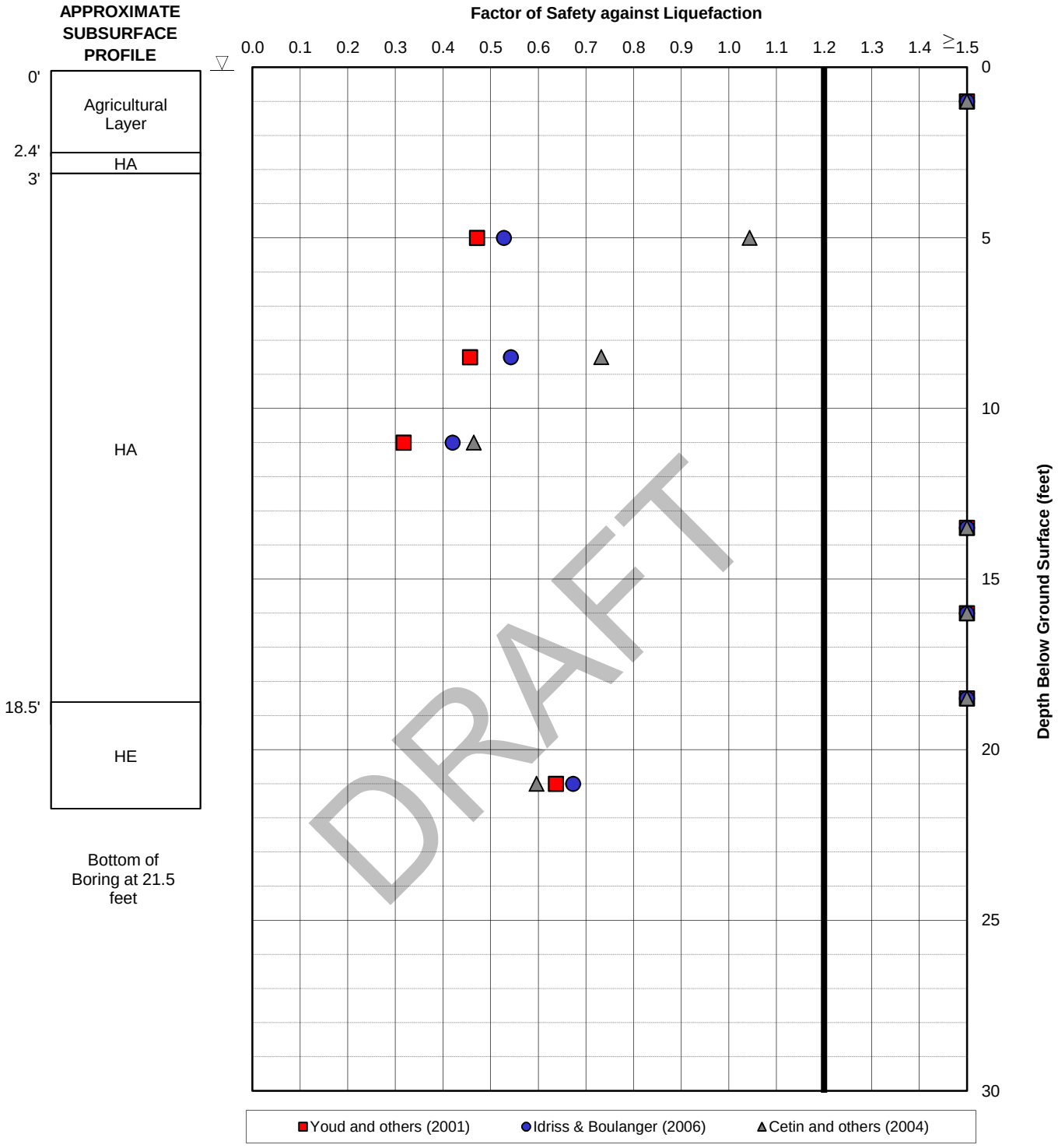
Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-3-13
M = 9.0, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-22



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

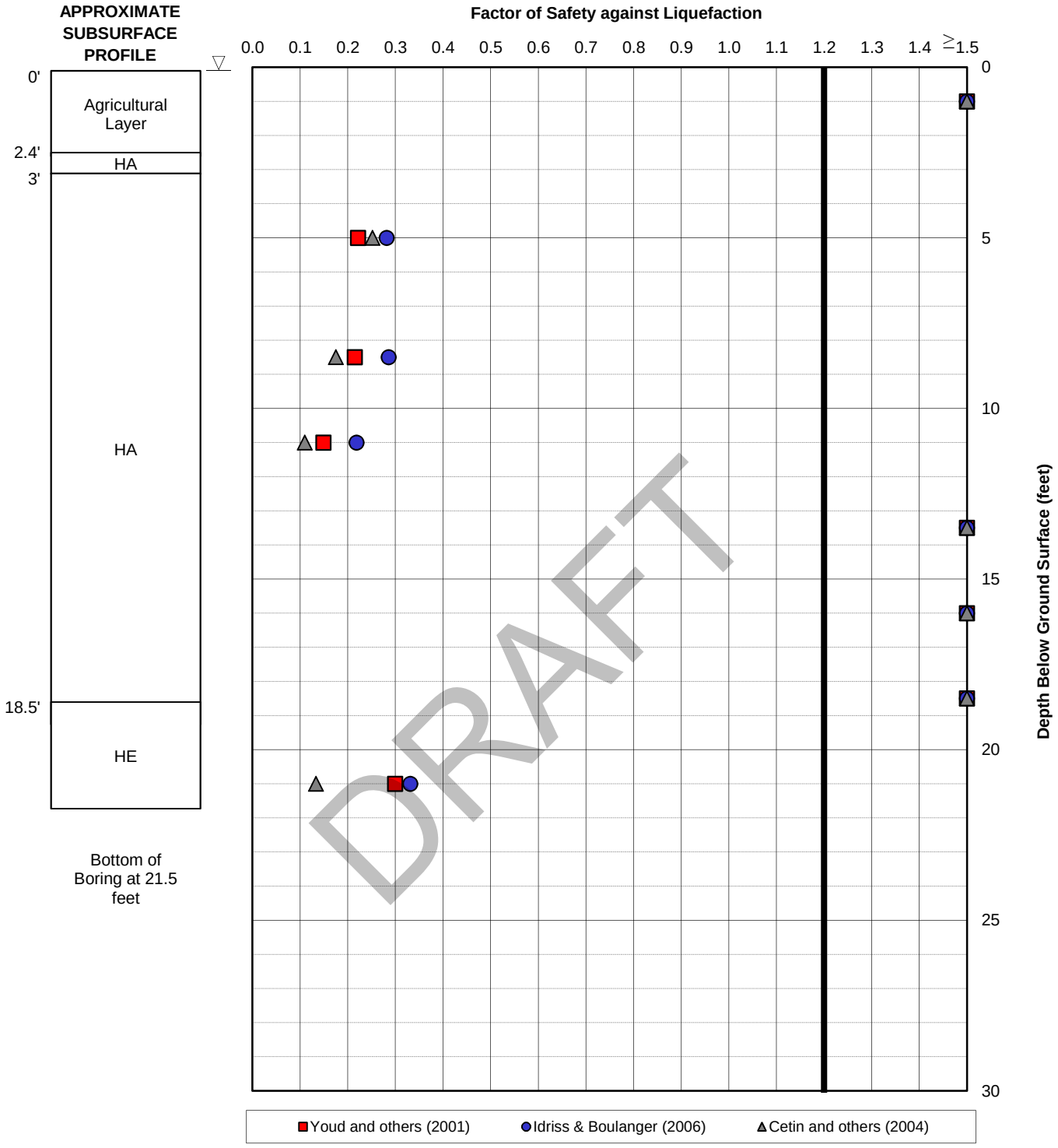
Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-4W-13
M = 6.7, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-23



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

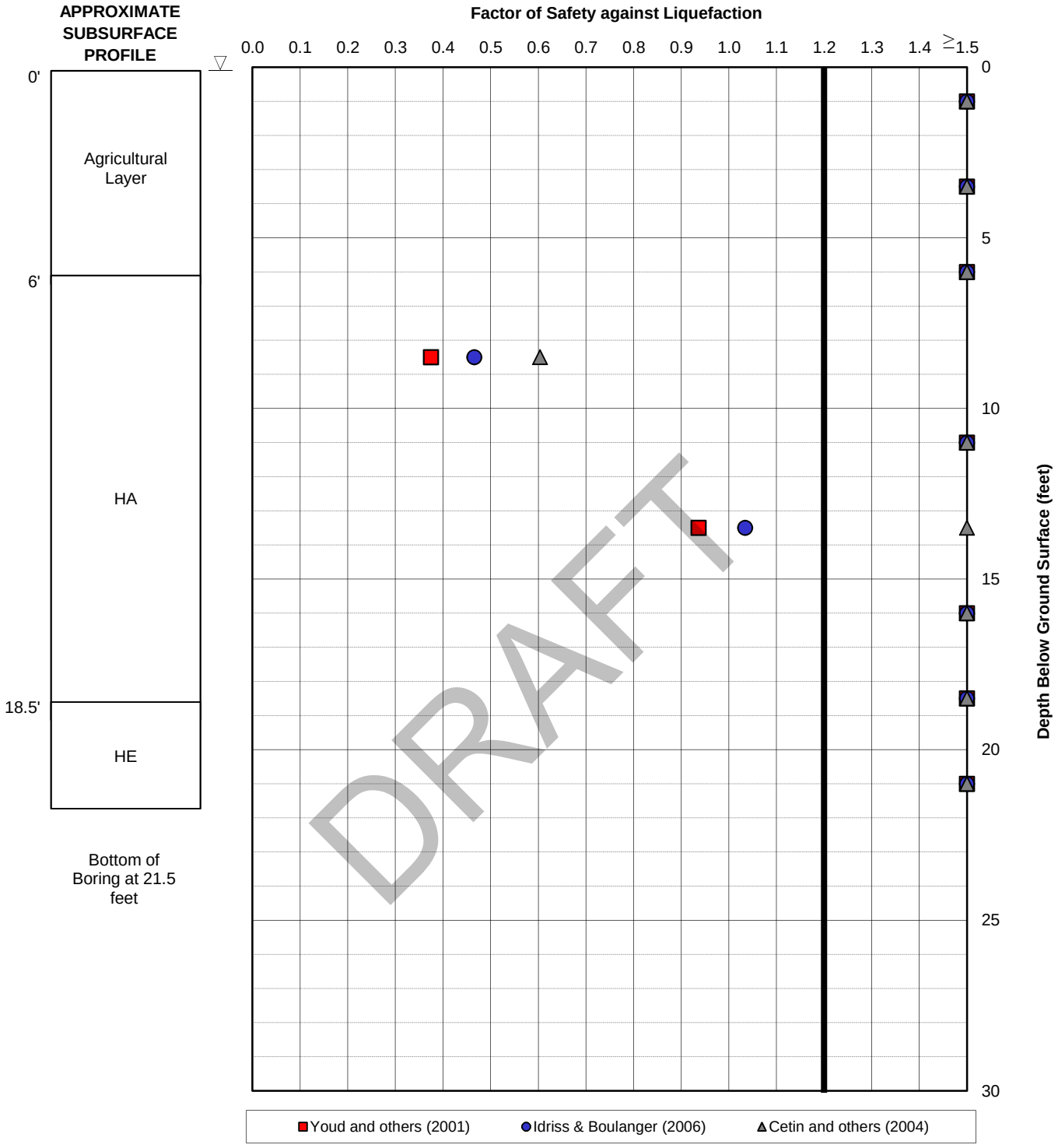
Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-4W-13
M = 9.0, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-24



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

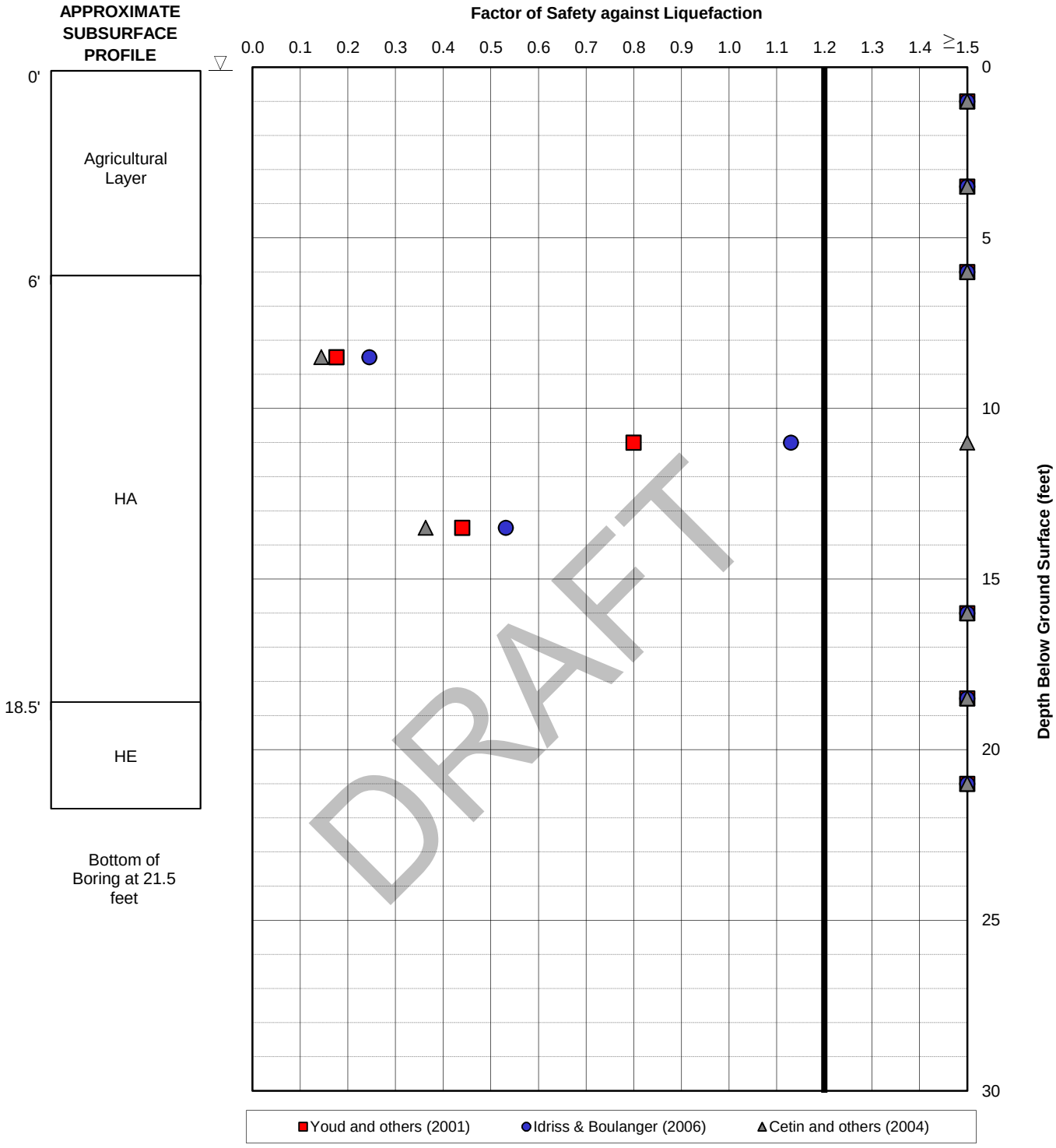
Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-6W-13
M = 6.7, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-25



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

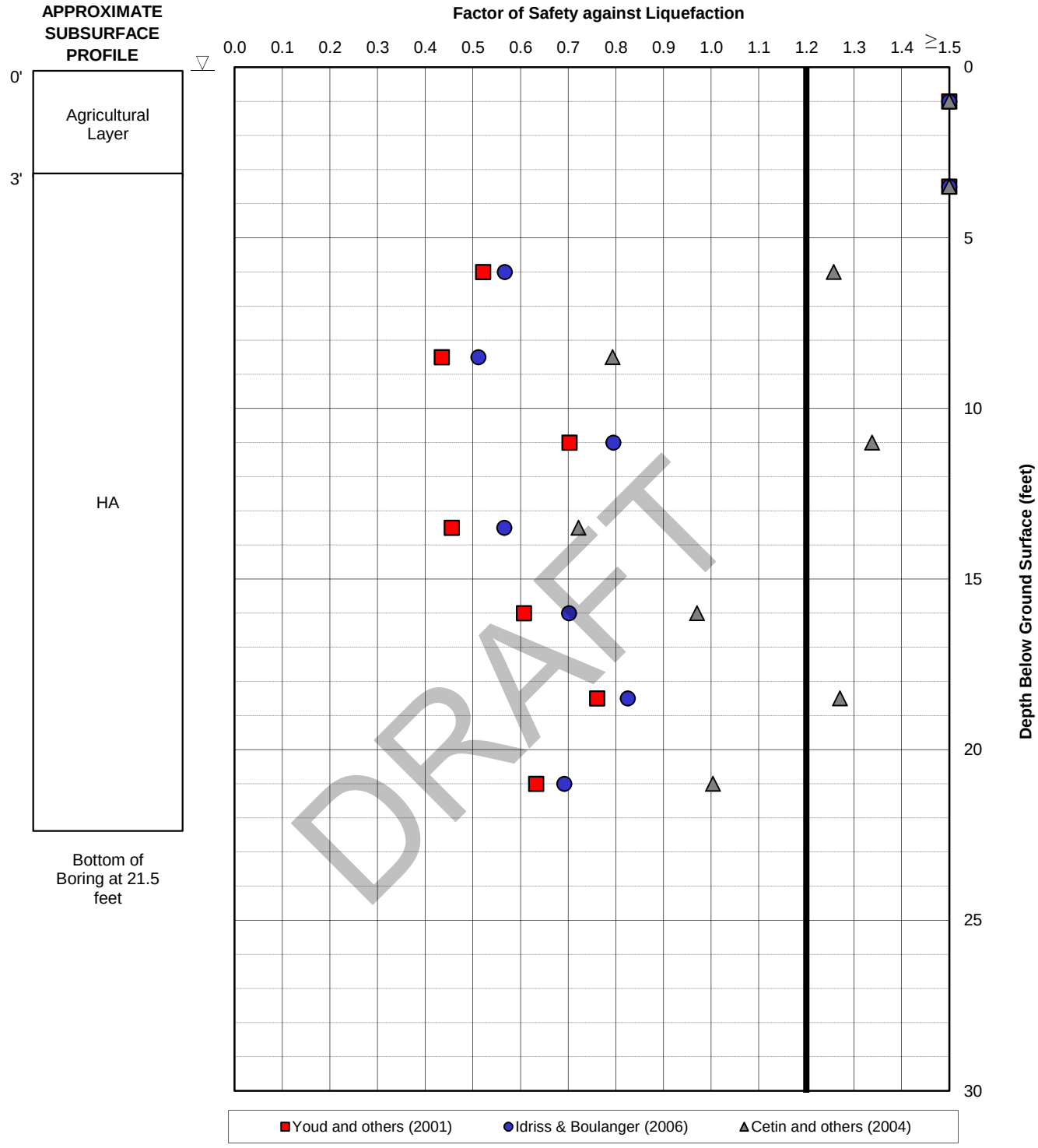
Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-6W-13
M = 9.0, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-26



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

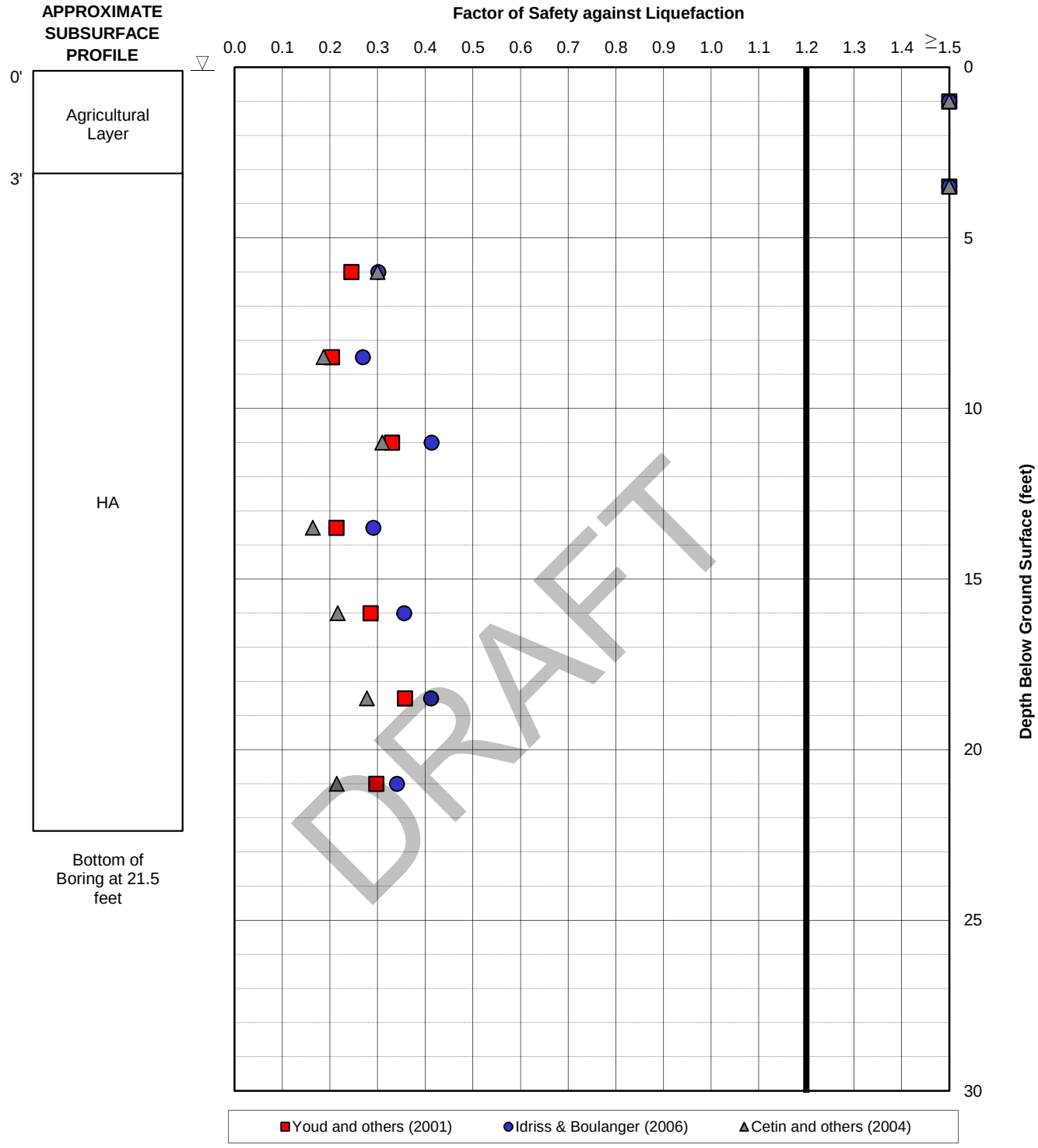
Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-8W-13
M = 6.7, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-27



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Fir Island Farm
Final Design and Permitting
Skagit County, Washington

RESULTS OF LIQUEFACTION ANALYSES
BORING B-8W-13
M = 9.0, PGA = 0.21

April 2014 21-1-12318-224

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-28

Draft

APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHHNICAL/ENVIRONMENTAL REPORT

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland