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GEOTECHNICAL INVESTIGATION: REVISION 1

Cache Slough Mitigation Bank

SOLANO COUNTY, CALIFORNIA

SHANNON & WILSON

April 11, 2023

Shannon & Wilson No: 110926

Submitted To: Westervelt Ecological Services, LLC
3636 American River Drive, Suite 120
Sacramento, California 95864
Attn: Angela Lagneaux

Subject: GEOTECHNICAL INVESTIGATION: REVISION 1, CACHE SLOUGH MITIGATION BANK, SOLANO COUNTY, CALIFORNIA

We performed a geotechnical investigation for the Cache Slough Mitigation Bank in Solano County, California in accordance with the proposal dated January 11, 2023. The results of the investigation are presented in the attached report.

It was a pleasure working with you on this project and we look forward to working with you during construction. If you have any questions, please call.

Sincerely,

SHANNON & WILSON

Gregory R. Olsen, GE, PE
Senior Engineer

R. Kevin Tillis, GE, PE
Vice President

GRO:RKT/kxb

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ACRONYMS

CPT	Cone Penetration Test
CVFPB	Central Valley Flood Protection Board
DWR	California Department of Water Resources
GDR	Geotechnical Data Report
GWT	Groundwater Table
H:V	Horizontal to Vertical
ID	inside diameter
MHHW	Mean Higher High Water
MTL	Mean Tide Level
MLW	Mean Low Water
MLLW	Mean Lower Low Water
NAVD88	North American Vertical Datum of 1988
NRCS	Natural Resources Conservation Service
OD	outside diameter
pcf	pounds per cubic foot
PFR	Preliminary Foundation Report
PHA	peak horizontal acceleration
psf	pounds per square foot
SR-84	State Route 84
USACE	United States Army Corps of Engineers
USDA	United States Department of Agriculture
USGS	United States Geological Survey

1 INTRODUCTION

This report presents the results of our geotechnical investigation for the Cache Slough Mitigation Bank in Solano County, California. A vicinity map showing the approximate location of the site is presented on Figure 1. The site layout is shown on the Site Plan, Figure 2. The project consists of converting agricultural land to tidal waters. The existing embankment on State Route 84 (SR-84) will be breached, allowing tidal flow onto the property. The parcel interior will be graded to provide a variety of habitat types with internal channels and berms. The existing SR-84 embankment is constructed to a restricted height that is above the daily tide level but below the design flood levels for neighboring properties. A new berm will be constructed around the perimeter of the parcel to approximately match the height of the existing SR-84 embankment.

A new water crossing structure for SR-84 will be constructed to allow traffic to cross over the breach. The geotechnical considerations related to the water crossing structure are presented in a separate report.

Our scope of services was outlined in our proposal dated January 11, 2023. Our scope of services consisted of conducting a geotechnical investigation that included reviewing existing data collected at and near the site, conducting Cone Penetration Tests (CPT), excavating hand auger borings, performing laboratory testing, and developing conclusions and recommendations regarding geotechnical aspects of the project. Our scope includes preparing three deliverables. We issued a draft Preliminary Foundation Report (PFR) dated September 15, 2023 which provided data and recommendations related to the water crossing structure. We also published a Geotechnical Data Report (GDR) on December 20, 2023, which included logs of borings, test pits, CPTs, and hand auger borings that were conducted for this investigation and previous investigations near the site. The GDR includes the laboratory test results from the exploration. We used data from the December 20, 2023 GDR to produce this report. This report is the third deliverable and comprises our discussion and recommendations regarding the geotechnical considerations for design and construction of the project. We previously published this report on December 22, 2023. After we published this report on December 22, 2023, the project stationing was modified. We have revised this report to reflect the stationing shown on the 60 percent design plans.

2 EXISTING DATA, FIELD EXPLORATION AND LABORATORY TESTING

We reviewed logs of previous subsurface explorations within the proposed project footprint and surrounding vicinity. We explored subsurface conditions by excavating hand auger borings and performing CPTs. The approximate exploration locations are presented on the Site Plan, Figure 2. A description of the data review and field exploration is presented in the December 20, 2023 GDR along with exploration logs and the laboratory test results.

Elevations referred to in this report are referenced to the North American Vertical Datum of 1988 (NAVD88) unless otherwise stated.

3 SITE CONDITIONS

3.1 Regional Geology

The site is located near the foot of the Montezuma Hills, at the fringe of the Sacramento-San Joaquin Delta. The United States Geological Survey (USGS) has published maps for the Sacramento-San Joaquin Delta (Atwater 1982).

The Atwater geologic map that includes the site and the geologic descriptions of the map units are presented on Figure 3. The map shows the landward margin of tidal wetland at low river stages circa 1850. The map indicates that much of the parcel interior is located within the margins of former tidal wetland. The margins of former tidal wetland meander along the edges of the parcel. The north and northeast edges of the parcel are situated south of a low, gradual ridge that trends northwest to southeast. Former channels cross the west and southwest boundaries of the parcel.

The geology map indicates that areas mapped within the margins of tidal wetland are generally covered by peat and mud of tidal wetlands and waterways (Qpm). Areas mapped as Qpm are shown where the peat and mud are generally thicker than 5 feet. In areas with less than 5 feet of peat and/or mud, the underlying geologic unit is mapped. The areas outside the margins of the tidal wetland are generally mapped as older alluvium of the Montezuma Hills and vicinity (Qom).

The present configuration of the Sacramento-San Joaquin River Delta began to form after the last ice age, about 10,000 to 13,000 years ago. During the ice age, sea levels were 200 to 300 feet below present levels. Sea levels rose rapidly for several thousand years then the rate of sea level rise slowed. As sea levels rose, the Delta was inundated. The rise in sea level was slow enough to allow for the accretion of marsh vegetation and sediments and formation of a widespread inland delta covered by marsh deposits (mapped as Qpm). Marsh deposits continued to accumulate as sea levels rose. The marsh formation was halted upon reclamation of land in the late 1800's and early 1900's within the Delta.

The Montezuma Hills are located southwest of the site and predate the present configuration of the Sacramento-San Joaquin River Delta. Storm runoff from the hills formed streams and eroded the soil, carrying it downstream as alluvium. The older alluvial soils (Qom) were present prior to the sea level rise that occurred after the last ice age.

The United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) develops soil survey maps of the upper five feet of soil across the United States. The portion of the NRCS soil survey map that includes Little Egbert Tract is presented on Figure 4. The NRCS map shows that most of the site is blanketed by Valdez Silt Loam with small portions mapped as Pescadero Silty Clay. Valdez Silt Loam is generally described as lean clay with sand to sandy lean clay with low to moderate plasticity. Pescadero Clay is generally described as sandy lean clay to sandy fat clay with moderate to high plasticity.

3.2 Surface Conditions

The site is bordered by levees, berms, and highways. The Mellin Levee and Mellin Levee Extension lie outside of the property boundary for this project adjacent to the southwestern and western boundaries of the site, respectively. The northern and northeastern boundaries of the site are bordered by an unnamed berm which separates the parcel from the Watson Hollow Diversion Canal (Watson Hollow). The southeastern boundary of the site is bordered by the embankment that carries SR-84 and separates the parcel from Cache Slough and the Sacramento River. The Mellin Levee and Mellin Levee Extension, north and northeastern berm, SR-84 Embankment, Watson Hollow, and parcel interior are described in further detail below.

3.2.1 Mellin Levee and Mellin Levee Extension

The elevation of the Mellin Levee and Mellin Levee Extension crests range from 10.5 to 19 feet and 10 to 21 feet, respectively. The inclination of the landside slope is between 2H:1V and 8H:1V and the inclination of the waterside slope is between 2H:1V and 8H:1V for both levees. The levee crest width generally varies from 15 to 30 feet for both levees. An approximately 800-foot-long segment at the north end of the Mellin Levee Extension is narrower, with a crest width of less than 7 feet. The elevation of the waterside toe varies from approximately 5 to 10 feet. The elevation of the landside toe is irregular, as the land west of the levee has been extensively regraded while in use as a soil and rock stockpile and borrow site. The new perimeter berm will be located east of the Mellin Levee and Mellin Levee Extension toe and be oriented parallel to the Mellin Levee and Mellin Levee Extension. The existing ground surface along the centerline of the new perimeter berm varies from approximately Elevation 4 to 9 with relatively gradual transitions (flatter than 10H:1V) from high areas to low areas. The footprint for the new berm is currently vegetated with grasses.

3.2.2 North and Northeastern Berm

The elevation of the existing berm crest, adjacent to Watson Hollow, along the north and northeastern sides of the project varies from 6 to 12 feet. The berm crest width generally varies from 11 to 25 feet. The berm crest and slopes are vegetated with grasses. The inclination of the slope facing Watson Hollow varies between 1H:1V and 3H:1V above the water level in Watson Hollow and flattens below the water level in Watson Hollow. The berm toe is generally underwater in Watson Hollow and ranges from approximately Elevation -3 to 3 feet. The inclination of the slope facing the project site is between 2.5H:1V and 5H:1V. The elevation of the toe, within the project site, varies from approximately 1 to 6 feet. At approximately Station 110+80, a pipe conduit passes through the berm to convey water between Watson Hollow and the parcel interior.

3.2.3 State Route 84 Embankment

The elevation of the SR-84 road surface varies from approximately 10 to 12 feet. The top of the embankment is approximately 40 feet wide, relatively flat and carries the asphalt-paved roadway of State Route 84. The two traveled lanes are approximately 25 feet wide total. The southwest-bound side contains a shoulder approximately 12 feet wide and surfaced with gravel. The northeast-bound side contains an asphalt-paved shoulder approximately 3 feet wide. The inclination of the current landside slope (adjacent to the project interior) varies from approximately 5H:1V to 10H:1V and the inclination of the current waterside slope (adjacent to the Sacramento River and Cache Slough) varies from approximately 3H:1V to 8H:1V. The waterside slope is armored with riprap and vegetated with tules, grasses, shrubs, and trees. The landside slope is vegetated with grasses, shrubs, and trees.

3.2.4 Watson Hollow

Watson Hollow is a man-made channel with the upstream end of the channel north of the project area. Water levels in Watson Hollow upstream of the project are controlled by a gate structure near the northwestern tip of the project parcel. Downstream of the gate structure, adjacent to the north and northeast boundaries of the site, water levels in Watson Hollow fluctuate tidally. Watson Hollow connects to Cache Slough near the confluence of Cache Slough and the Sacramento River through multiple culvert pipes that flow beneath SR-84.

We understand that the pipes are open to tidal flow in Cache Slough. The bottom of Watson Hollow generally ranges from Elevation -5 to Elevation 1 feet. The channel width generally ranges from 85 to 200 feet at the waterline.

3.2.5 Parcel Interior

The interior of the parcel is relatively flat, with gentle slopes. Elevations within the interior range from approximately 2 to 7 feet. A natural gas well is located approximately 1000 feet southwest of the northwest tip of the project area. The parcel is traversed by multiple unpaved roads and irrigation canals. The site is currently used for bird hunting in fall and winter months and grazing during other times of the year. Vegetation consists of a variety of tules, grasses, shrubs, and trees.

3.2.6 Tidal Levels

Water Levels in the Sacramento – San Joaquin River Delta are subject to tidal variation. Tidal levels for the project site are presented in Exhibit 3-1 below. The 100-year flood level along the Mellin Levee varies from Elevation 14.7 feet at the south end of the project to Elevation 16.1 feet at the north end of the project. The 10-year flood level is Elevation 10.2 feet.

Exhibit 3-1: Tidal Datums at Project Site, from MBK (2022)

Tide Level	Elevation, feet (NAVD88)
Mean Higher High Water (MHHW)	6.5
Mean High Water (MHW)	5.9
Mean Tide Level (MTL)	4.4
Mean Low Water (MLW)	2.6
Mean Lower Low Water (MLLW)	2.1

3.3 Subsurface Conditions

We have grouped the subsurface soils encountered at the proposed project site into four geologic units which include embankment fill, hydraulic dredge fill, marsh deposits, and older alluvium. The units are described below. The subsurface conditions beneath the proposed perimeter berm alignment are represented graphically on the idealized subsurface profiles presented in Appendix A and the idealized subsurface cross sections shown in Appendix B. We did not collect data between Stations 10+00 and 20+32 and have not included them in the idealized subsurface profile. Based on our review of the geologic conditions at the site, we infer that the subsurface conditions from Station 10+00 to 20+32 are similar to the conditions shown between Stations 20+32 and 25+00. Our interpretations of the surface geology based on our exploration are presented on Figure 5. The main difference between Figure 5 and the Atwater Geologic Map on Figure 3 is that we interpreted the marsh soils to have a greater extent within the parcel than the extent shown on the Atwater Geologic Map.

3.3.1 Soil Units

3.3.1.1 Embankment Fill

Fill for the existing north and northeastern berm mainly consists of moderate to high plasticity lean clay and fat clay. The upper 1 to 2 feet are generally lean clay, while the underlying fills are fat clay. The fill is generally dry to moist at the ground surface and becomes wetter with depth. The consistency ranges from medium-stiff to stiff. The existing fills do not appear well-compacted. The fill likely derived from excavation within Watson Hollow.

3.3.1.2 Hydraulic Dredge Fill

Hydraulic dredge fill is mapped by Atwater at the ground surface in the property on the landside of the Mellin Levee Extension. We encountered hydraulic dredge fill in the parcel interior consisting of poorly graded sand, silty sand, silt, lean clay, and fat clay. Generally, the fine-grained hydraulic dredge fills were dry to moist, medium stiff to stiff, and ranged in plasticity from low to high. The sand hydraulic dredge fills were dry to wet and loose to medium dense. In some of the test pits, “clay balls” that ranged in size from gravel- to cobble-sized were observed within the silty sand. We did not observe a clear pattern of where the various soil types from the hydraulic dredge fill are located. We encountered hydraulic dredge fill beneath the embankment fill in some of the exploration locations in the north and northeast berm.

3.3.1.3 Marsh Deposits

Marsh deposits were encountered beneath the embankment fills and hydraulic dredge fills beneath most of the perimeter berm alignment and parcel interior. The marsh deposits typically consist of peat, organic clay, and organic silt. Marsh deposit soils were deposited in tidal waters and are typically relatively weak and compressible. Near the ground surface, the marsh deposits are generally dry to moist and stiff to very stiff. Generally, the marsh deposits become wet and soft to medium-stiff between 3 and 8 feet below the top of the marsh deposit layer.

3.3.1.4 Older Alluvium

The older alluvium soils consist of sand, silt, and clay. Throughout most of the site, the older alluvium is buried below the marsh deposits. Near the southern and western corners of the site, the older alluvium is present at the ground surface. Fine-grained alluvium (clay and silt) was consistently encountered at the top of the older alluvium deposit and in layers at greater depths. In between the layers of fine-grained alluvium, sand alluvium was encountered.

The fine-grained older alluvium soils generally consist of low to medium plasticity lean clay, with occasional zones of lower plasticity silt or higher plasticity fat clay. The consistency of the fine-grained older alluvium typically ranges from stiff to very stiff, with occasional zones of medium-stiff soil.

The sand alluvium is generally wet, dense to very dense, and of variable fines content. Sieve analysis tests in the sand alluvium resulted in fines contents ranging from 6 to 49 percent. Generally, the fines content decreases with depth. The fines typically consist of silt. The deeper sand layers contain gravel.

3.3.2 Perimeter Berm

The subsurface conditions beneath the proposed perimeter berm are depicted in Appendix A. Idealized subsurface cross sections for seven stations along the alignment are presented in Appendix B.

From Station 10+00 to approximately Station 74+00, the perimeter berm will be offset east of the Mellin Levee and Mellin Levee Extension in an area where there is currently no existing embankment fill. Marsh deposits are present at the ground surface between approximately Stations 28+00 to 52+00 and 65+00 to 74+00, which are up to 30 feet thick. Clay alluvium was typically encountered beneath the marsh deposits within these station ranges. At Stations 10+00 to 28+00 and 52+00 to 65+00, clay alluvium was generally encountered at the ground surface. The thickness of clay alluvium ranges from 5 to 18 feet. The bottom of the clay alluvium ranges from 6 to 41 feet below the existing ground surface. The clay alluvium is underlain by sand alluvium. The bottom of the sand alluvium ranges in depth from 80 to 92 feet. The sand alluvium is underlain by another layer of clay alluvium to the maximum depth encountered in our borings and CPTs, up to 115 feet below existing grade.

From Station 74+00 to 127+54, the new berm alignment follows the existing berm alignment adjacent to Watson Hollow. The existing berm ranges in thickness from 1 to 5 feet. The existing berm is typically underlain by hydraulic dredge fill. The thickness of the hydraulic dredge fill varies from 0 to 7 feet. The bottom of the fill ranges in depth from 4 to 10 feet below the existing berm crest. The fill is underlain by marsh deposits ranging in thickness from 5 to 29 feet. The bottom of the marsh deposits ranges from 6 to 35 feet below the existing berm crest. The marsh deposits are typically underlain by clay alluvium ranging in thickness from 7 to 10 feet from approximately Station 74+00 to 87+00 and Station 106+00 to

127+54. The base of the clay alluvium ranges from 18 to 35 feet below the existing berm crest. Sand alluvium is located beneath the clay alluvium from approximately Stations 74+00 to 87+00 and 106+00 to 127+54 and beneath the marsh deposits from approximately Stations 87+00 to 106+00. The sand alluvium is at least 20 feet thick and extends below the maximum depth explored in some locations. The base of the sand alluvium ranges from 51 to more than 86 feet below the crest of the berm. Where the bottom of the alluvial sand was encountered, it was underlain by another layer of clay alluvium.

3.3.3 Parcel Interior

The near-surface soil conditions are depicted on Figure 5. In the areas denoted with marsh deposits, high plasticity and often organic-rich soil was encountered at the ground surface or buried beneath a relatively thin layer of hydraulic dredge fill. We did not drill borings in the parcel interior and therefore have limited data regarding the depth of the marsh deposits in the parcel interior. Based on the borings and CPTs along the perimeter of the parcel, we estimate that the base of marsh deposits is at least 30 feet at the deepest locations within the parcel interior. Based on our perimeter borings and the geologic mapping, we estimate that the marsh deposits may be deepest in the southeast corner of the site. The marsh deposits are generally underlain by stiff older alluvium that consists of clay, silt, and sand.

Near the west side of the parcel interior, older alluvium is present near the ground surface and marsh soils are largely absent. The alluvial soils near the ground surface in this area are described below.

3.3.3.1 Perimeter Berm Borrow Area

The west side of the parcel interior near the west corner had little to no marsh deposit soils at the ground surface. Older alluvium was present at the ground surface and continued to the maximum depth explored. The area of the parcel interior with minimal marsh deposits will be referred to throughout the remainder of this report as the Perimeter Berm Borrow Area because this area contains soils at relatively shallow depths which meet United States Army Corps of Engineers (USACE) requirements for levee fill. The USACE requirements are discussed in the Discussion and Conclusions section of this report. The location of the Perimeter Berm Borrow Area is presented on Figure 6.

The soil in the Perimeter Berm Borrow Area generally consists of fat clay, underlain by lean clay, underlain by sand. The top layer of the older alluvium is generally high plasticity fat clay which is dry to moist and stiff to hard. The top layer typically ranges in thickness from 2 to 3 feet, with some exploration locations containing about 8 feet of surface fat clay and others containing no fat clay. The depth below existing grade to the bottom of the fat clay is indicated at various exploration locations next to the exploration number on Figure 6 as the first number in parenthesis. Note that the locations drilled in the crest of the Mellin Levee show the bottom of fat clay at deeper depths than discussed above. These deeper depths were due to the approximate 5- to 10-foot thickness of levee fill placed over the native soils. The elevation of the bottom of fat clay is similar between the borings drilled in the levee crest and the borings and test pits completed at the toe. The liquid limits of the fat clay soils ranged from 56 to 68 and the plasticity indices ranged from 37 to 47.

The second layer of the older alluvium is generally moist, medium-stiff to very stiff, low to medium plasticity, lean clay and silt. Generally, the lean clay soils were encountered below the fat clay soils described above, but in some locations, the lean clay soils were encountered at the ground surface. The lean clay soils typically range in thickness from 3 to 10 feet. Not all test pits encountered the bottom of the lean clay soils. The depth to the bottom of the lean clay is indicated on Figure 6 by the second number in parenthesis. The thickness of lean clay can be calculated by subtracting the first number in parenthesis from the second. The liquid limits of the lean clay soils ranged from 33 to 41 and the plasticity indices ranged from 8 to 25.

The third layer of the older alluvium was generally moist, medium-dense, poorly graded sand with silt and silty sand or stiff to very stiff non-plastic silt. The fines content in the sands tested within this layer ranged from 12 to 14 percent.

3.4 Groundwater

Groundwater measurements were obtained during exploration for some of the borings and test pits. Most of the borings were drilled using rotary wash methods that obscured the groundwater level. Groundwater measurements from the borings and test pits are listed in Exhibit 3-2 below. The borings and test pits were backfilled immediately after drilling and stabilized water levels were not obtained.

Exhibit 3-2: Groundwater Elevations from Borings and Test Pits

Boring or Test Pit #	Depth to Groundwater Table (GWT) (ft)	Surface Elevation (ft)	GWT Elevation (ft)	Date
19P-26	7	6.6	-0.4	7/12/2019
19P-27	10	6.8	-3.2	7/12/2019
19P-28	6	6.3	0.3	7/12/2019
19P-29	5	5.7	0.7	7/12/2019
19P-31	7	6.5	-0.5	7/12/2019
21P-9	11	6	-5	10/12/2021

Boring or Test Pit #	Depth to Groundwater Table (GWT) (ft)	Surface Elevation (ft)	GWT Elevation (ft)	Date
21P-10	9	10	1	10/12/2021
21P-16	8	5	-3	10/13/2021
21P-18	8	4	-4	10/13/2021
21P-19	6	5	-1	10/13/2021
21P-20	11	4	-7	10/13/2021
21P-21	9	7	-2	10/13/2021
21P-22	11	6	-5	10/13/2021
21B-11	9	6	-3	10/1/2021
22B-9	15	10	-5	6/8/2022
23P-1	6	5	-1	10/23/2023
23P-3	7	6	-1	10/24/2023
23P-4	7	6	-1	10/23/2023
23P-5	7	6	-1	10/23/2023
23P-6	7	6	-1	10/23/2023
23P-7	7	6	-1	10/23/2023
23P-8	8.6	6	-2.6	10/23/2023
23P-10	5.7	7	1.3	10/23/2023
23P-11	8	6	-2	10/23/2023
23P-12	9	5	-4	10/23/2023
23P-16	9	4	-5	10/24/2023

Pore pressure dissipation tests were performed during select CPTs. Exhibit 3-3 below summarizes the interpreted groundwater levels from the pore pressure dissipation tests. The interpreted water levels are based on the assumption that hydrostatic water levels are present and that aquifers are unconfined.

Exhibit 3-3: CPT Groundwater Elevation Interpretations

CPT #	Interpreted Depth to GWT (ft)	Surface Elevation (ft)	GWT Elevation (ft)	Date
21C-9	5.9	7	1.1	10/06/2021
21C-15	8	5	-3	10/08/2021
21C-16	3.5	4	0.5	10/08/2021
21C-17	12.7	8.5	-4.2	10/08/2021

CPT #	Interpreted Depth to GWT (ft)	Surface Elevation (ft)	GWT Elevation (ft)	Date
22C-14	9.4	7	-2.4	05/19/2022
22C-18	17.1	15	-2.1	05/23/2022
22C-21	19.2	18	-1.2	08/17/2022
22C-22	15.2	15	-0.2	08/17/2022
22C-23	13.3	15	1.7	08/17/2022
22C-24	8.7	10	1.3	08/17/2022
22C-25	12.1	14.5	2.4	08/18/2022
23C-1	9.3	7.5	-1.8	05/30/2023
23C-2	2.2	5.5	3.3	05/30/2023
23C-3	5.9	6.5	0.6	05/30/2023
23C-4	7.4	8	0.6	05/30/2023
23C-5	9	11	2	05/30/2023
23C-6	7.6	9	1.4	05/31/2023
23C-7	8.4	10	1.6	05/31/2023
23C-8	9.7	9	-0.7	05/31/2023
23C-9	8.2	8	-0.2	05/31/2023
23C-10	8.5	8	-0.5	05/31/2023

The above descriptions of soil and groundwater conditions summarize observations at the time of the investigations. Conditions are expected to vary across the site, with time, and depend on several factors including changes in moisture content resulting from seasonal precipitation and land use changes.

4 DISCUSSION AND CONCLUSIONS

The project will include extensive grading within the parcel to transform the ground surface to elevations better suited to developing tidal and upland habitat. The grading will be completed using on site soil. Fill for the core of the perimeter berm will be taken from a designated area on site. Fill for habitat berms built adjacent to the core of the perimeter berm and other interior fills will be derived from the channel excavations in the parcel interior. Most of the site is underlain by marsh deposit soils, which have relatively low strength and high compressibility. The primary geotechnical engineering considerations for the project include poor compaction of the existing north and northeast berms, settlement of fills placed on the marsh soils, and lack of berm freeboard due to height restrictions for the perimeter berm.

4.1 Perimeter Berms

4.1.1 Existing Berms

The existing perimeter berm along the north and northeast edges of the site (approximately Station 74+00 to 127+54) was likely constructed by casting up soil from Watson Hollow or other nearby borrow sources. The berm fill consists primarily of highly plastic clays that appear poorly compacted. The existing berms also have steep landside slopes, a relatively narrow crest width, and a low crest elevation. Because of these characteristics, the existing berms do not meet generally accepted reliability criteria for compacted fill embankments. We conclude that the existing north and northeast berm fills should be removed within the footprint of the new berm before the new berm is constructed and the existing berm soil should not be reused for fill in the core of the new perimeter berm. If portions of the existing berm are to remain in place, the new berm may be offset from the existing berm. The existing fills that are removed can be used for habitat berms or other non-structural fills in the project interior.

4.1.2 New Berm Configuration

The new berm will be constructed with a restricted height similar to the existing berms. The crest elevation is intended to be limited in order to avoid impacting flood conveyance of the Yolo Bypass upstream of the project. The project design team selected a design crest elevation of 9.5 feet plus an allowance for settlement. The design crest width for the new berm is 20 feet for the reach parallel to the Mellin Levee and Mellin Levee Extension (approximately Station 10+00 to 74+00) and 15 feet for the reach adjacent to the existing north and northeast berm (approximately Station 74+00 to 127+54). The new berm crest will be near or below the 10-year flood level. For design, we used the typical sections shown on Figures 7 and 8. In addition to the design crest width and elevation, the berm slopes will be 3H:1V or flatter. The design includes a settlement allowance and keyway.

The berm geometries and layout described above do not include the planned habitat berms. The configuration of the habitat berms is being developed by others. We understand that the habitat berms are anticipated to be relatively flat slopes located adjacent to the waterside of the berm within the project parcel. The habitat berms are expected to provide benefits for seepage and stability of the berms but are not included in our evaluation of the berms.

4.1.3 New Berm Analysis

We performed analysis to evaluate settlement, seepage through and below the new berms, and slope stability. We analyzed seven (7) cross sections.

Exhibit 4-1: Cross Sections for Analyses

Station
21+32
36+32
56+32
90+32
97+32
103+32
122+32

The selected cross sections are presented above in Exhibit 4-1 and are intended to represent the range of existing levee geometry and geologic conditions for the site. The idealized subsurface cross-sections at each station are presented in Appendix B.

4.1.3.1 Settlement

We performed analysis to estimate settlement based on the theory of consolidation. We used Terzaghi's theory of one-dimensional consolidation to estimate the magnitude of settlement due to the weight of new fill. We used the data observed from the borings, test pits, and CPTs to develop material properties. To estimate the magnitude of settlement, we used the parameters in Exhibit 4-2, below.

Exhibit 4-2: Soil Properties Used for Settlement Analyses

New Fill Unit Weight	125 pcf*
Existing Fill Unit Weight	95 pcf
Marsh Soils Unit Weight	95 pcf
Marsh Soils Compression Ratio, Cc / (1+ e_0)	0.25 to 0.45

*pcf is pounds per cubic foot

4.1.3.2 Seepage and Slope Stability

Our analysis was based on guidelines set forth by the USACE and Central Valley Flood Protection Board (CVFPB).

The main documents used for our analysis include but are not limited to:

- Guidance Document for Geotechnical Analyses (DWR, 2015)
- EM 1110-2-1913 Design and Construction of Levees (USACE, 2000).

A discussion of the design criteria, analysis, and results are presented in Appendix C for seepage and Appendix D for slope stability.

4.1.4 New Berm Considerations

4.1.4.1 Settlement

Fill is needed to construct the perimeter berm, create habitat berms, and for other non-structural features. The marsh deposits will consolidate from the weight of new fills, resulting in settlement of fills placed over the marsh soils. Primary consolidation occurs from compression of the marsh soils, beginning when weight is placed on the soil. The initial weight is transferred to the water within the soil. The water builds up pressure, causing flow to occur. As the water flows out of the soil, the soil structure compresses and continues to compress until the water flow is complete and the water pressure returns to hydrostatic levels.

Secondary compression is deformation without flow of water. With most soils, the amount of secondary compression is small relative to the primary consolidation and is not a concern. With peat, and to a lesser extent, organic soil, secondary compression is a significant phenomenon and will cause continued settlement of the berm and the loss of freeboard. The secondary compression will continue for many years at a diminishing rate with time.

The settlement analysis indicates that the new berm will need to be designed to accommodate settlement. We have provided a summary of the marsh soil thickness and recommended allowance by berm station in Exhibit 4-3 below.

Exhibit 4-3: Recommended Settlement Allowance by Station

Station	Approximate Marsh Thickness (feet)	Settlement Allowance (feet)
10+00 to 27+32	0	0.5
27+32 to 50+32	30	1.5
50+32 to 65+32	0	0.5
65+32 to 96+26	20	1.0
96+26 to 127+54	30	1.5

The overbuild may consist of constructing the berm above the design crest elevation to accommodate settlement or constructing the berm wide enough to accommodate future raising. If the berm is constructed to allow future raising, it should allow for 3:1 side slopes for additional phases of fill. The amount of settlement will depend on where the new fill is placed in relation to existing berms, the thickness of the new fill, and the thickness of the compressible soil layer.

The rate of settlement is dependent on several factors including the permeability, compressibility, and thickness of the marsh deposit soils. In areas without marsh soil, we recommend a settlement allowance to accommodate uncertainty in subsurface conditions. In areas with marsh soil thickness of 20 feet, we estimate that about half of the settlement will occur within 2 years of fill placement and about 90% of the settlement will occur within 8 years of fill placement. In areas with marsh soil thickness of 30 feet, we estimate that about half of the settlement will occur within 5 years of fill placement and about 90% of the settlement will occur within 20 years.

4.1.4.2 Seepage

The perimeter berm will be subject to daily wetting from tidal inundation of the project parcel. Because of the height restriction for the berm crest, the berm will have less freeboard under daily conditions than most levees in the delta. With the reduced freeboard, the berm width is narrower at the tide and flood levels than embankments with more freeboard. Through seepage is undesirable and can cause internal erosion (piping) within the berm that can lead to slumping and subsequent overtopping. The berm geometry creates an elevated risk for through seepage when compared with embankments with greater freeboard.

The new berm will be constructed on existing soil with variable composition at the ground surface. The surficial soils are potentially loose from being disturbed by farming, desiccation, or being placed with poor compaction methods. We conclude that a keyway should be constructed of low permeability berm fill beneath the new berm to disrupt potential seepage paths near the interface of the new fill and the existing foundation soil. Recommendations for the keyway material type and dimensions are provided later in this report.

Where the perimeter berm is adjacent to Watson Hollow, prior to breaching, underseepage may be a concern where the marsh soils are relatively thin, and the more permeable sand alluvium is present at shallow depths in Watson Hollow. The berm along Watson Hollow will have water adjacent to both sides of the berm after breaching. Seepage will be a minor concern for the berm after the site becomes tidal. Our seepage analysis indicates that when the water level in Watson Hollow is maintained near the average tide level, the berm meets criteria set forth by USACE for underseepage.

4.1.4.3 Erosion

Erosion can occur from a variety of factors. Berms are subject to erosion from rainwater running down their slopes and from overtopping when floods pass through the site. Channels and breaches are subject to erosion from the flow of tidal water through the openings and around bends. Wind generated waves are another cause of erosion. Erosion protection features, such as armoring, sacrificial fill, planting, and slope flattening may be needed to reduce the negative impacts of erosion. Depending on the method of erosion protection selected, maintenance may be required to restore areas which are damaged by erosion.

Wind generated waves can cause erosion from the repeated pounding of waves on berm slopes. The magnitude of the erosion depends on the wave height and frequency. Wave height and frequency depend on water depth, wave fetch, wind direction, and wind velocity. Exposing the parcel interior to daily tidal inundation may subject perimeter and interior berms to frequent wind generated waves. The design of the erosion protection is not within our scope of work but we should review the design relative to berm safety.

4.1.4.4 Seismic Considerations

The predominant seismic hazard for this site is strong ground shaking resulting from earthquakes. Structures for the project should be designed to accommodate such ground shaking in accordance with existing codes. No known active faults pass through the site, and we conclude that the risk of fault rupture is low. We provided seismic design criteria for the water crossing structure using the Caltrans Acceleration Response Spectrum and present that data in the separate report for the water crossing structure.

Soil liquefaction is a phenomenon in which a loose to medium dense saturated granular soil undergoes reduction of internal strength as a result of increased pore water pressure generated by shear strains within the soil mass. This behavior is most commonly induced by strong ground shaking associated with earthquakes. Soil conditions consist primarily of organic clay marsh deposits. Due to their high plasticity, the risk of liquefaction in the marsh deposits is low. The marsh deposits are underlain by clay and sand alluvium. The alluvium soils are older deposits and generally dense to very dense. Both older deposits and dense deposits have low risk of liquefaction. We conclude that the overall risk of liquefaction is low.

Another consideration for berms is the seismic deformation of the slope due to earthquake loading. We analyzed seismic deformation of the perimeter berm using the simplified procedure presented in the Guidance Document (2015). We describe our analysis procedure in Appendix D. The Ky values range from 0.26 to 0.7. The results of our analysis indicate that the slopes will experience minor slope deformation of likely less than 0.1 ft of deformation under the design level earthquake.

4.2 Interior Grading

4.2.1 General Grading

The proposed project requires extensive cuts and fills to create the various marsh levels and channels within the parcels and to construct habitat berms. Interior channels should have side slopes that are 2H:1V or flatter. Much of the grading will create shallow, nonstructural berms and wetlands. Fill for habitat berms and other non-structural fills may consist of existing fills, marsh deposit, and older alluvium soils.

Some of the material excavated from the site will be significantly wet of optimum moisture content depending on the depth of excavation. The grading contractor should have experience with moving soft, saturated soils. The bottom of the channel excavations will likely be below the groundwater level. The contractor should be prepared to manage water that collects in the channel excavations and process wet soil.

4.2.2 Berm Fill Materials

The core of the perimeter berm will require inorganic, low to moderate plasticity, fine-grained soils to function as designed. Organic marsh deposit soils are not suitable for structural fills. We evaluated areas within the site that might have material suitable for constructing the core for the perimeter berms. We identified one area with suitable fill material for the perimeter berm core. The location where suitable borrow material for the perimeter berm core and other structural fills is present near the ground surface is depicted Figure 6.

Fill for the perimeter berm core may be required by permit to meet the requirements for levee fill set forth by USACE and the CVFPB. The USACE typically specifies that levee material should have a liquid limit less than 45, plasticity index between 8 and 40, and at least 30 percent fines (material passing the No. 200 sieve). The CVFPB typically requires that levee material should have a liquid limit less than 50, a plasticity index greater than 8, and at least 20 percent fines. The lean clay soils indicated within the depth ranges shown in parenthesis on Figure 6 will generally meet USACE and CVFPB requirements.

Most of the area within the borrow zone will require some removal of surface soil to obtain the soil that meets USACE and CVFPB criteria. The surface layer consists of a mixture of lean clay and fat clay soils. The perimeter berm is not a designated flood control levee and may not be required to be constructed from fill that meets this criteria. If permit conditions do not require fill for the perimeter berm core to meet the requirements for levee fill set forth by USACE and the CVFPB, we conclude that the surficial soils may be used as fill for the perimeter berm core.

Within the zones depicted on Figure 6, we have identified preferred borrow limits which minimize the amount of surface soil stripping required above the soil that meets USACE and CVFPB criteria. The approximate acreage of the preferred area and estimated quantities of surface soil and lean clay are presented in Exhibit 4-1 below. The quantity estimates are approximate and should be refined by others using existing survey data. For planning, we recommend assuming a shrinkage factor of 15 percent.

Exhibit 4-4: Estimated Borrow Area Quantities

Limits of Preferred Borrow (acres)	Estimated Average Surface Soil Depth (feet)	Approximate Quantity of Surface Soil (cubic yards)	Estimated Average Thickness of Material Meeting USACE / CVFPB Criteria (feet)	Approximate Quantity of Material Meeting USACE / CVFPB Criteria (cubic yards)
20	2	50,000 to 100,000	6	150,000 to 250,000

The depths and quantities shown in Exhibit 4-1 and Figure 6 are based on relatively widely spaced test pits. Some modifications to the limits and depths of the borrow area during construction may be needed if soils are encountered which are different than those described above.

The excavated soil may require drying prior to placement, shaping, and compaction. Drying could occur at or near the borrow site or at the fill site during construction provided there is time allowed for drying between lifts of fill. A disk should be considered for use in breaking up and drying the wet materials. Our experience with similar soils near the proposed project site indicates that even with disking, the soil could take multiple days or longer to dry before reaching a moisture content that will allow for compaction and placement of additional lifts. Drying time will depend heavily on tide, weather conditions during construction, and the talent and efficiency of the contractor. The soils will dry fastest in the summer months.

5 RECOMMENDATIONS

5.1 Berm Configuration

We recommend that the perimeter berm should generally conform to the details shown on Figures 7 and 8. The section includes a 15- to 20-foot-wide crest, 3H:1V or flatter slopes, a keyway below the crest, and 6-inches of aggregate base on the crest. The minimum berm crest elevation, as established by the design team, is 9.5 feet.

A keyway should be placed below the center of the perimeter berm and should be constructed from fill meeting the requirements below. The keyway should be at least 3 feet deep and 10 feet wide at the base. The keyway slopes should extend up to the ground surface at 2H:1V or flatter.

The existing berm from Station 74+00 to 127+45 should be removed and rebuilt to the configuration described above.

5.1.1 Site Preparation

The site berm footprint should be cleared of existing fill and grubbed of surface and subsurface deleterious matter including trees, grasses, other vegetation, and debris designated for removal. The berm footprint should be stripped to sufficient depth to remove vegetation and soil containing roots. Tree roots greater than 1-inch in diameter should be removed. Stripped and grubbed materials should be removed from the site and should not be used as fill.

If loose or soft materials are encountered, they should be excavated to expose firm soil and placed in accordance with the recommendations presented below. Debris and deleterious material encountered during grading should be removed from the site.

The pipe conduit crossing through the levee at approximately Station 110+80 should be completely removed. Other existing structures, including pipes and conduits through the berm, should also be removed. The structure removal should extend at least 5 feet beyond the berm footprint.

5.1.2 Fill Materials

The core of the perimeter berm should be constructed of materials with low permeability and moderate plasticity. The berm fill material should have at least 20 percent fines passing the No. 200 sieve and 100 percent passing the 2-inch sieve. The plasticity index of the fill should be at least 8. Fill meeting the above criteria may be obtained from the borrow area shown on Figure 6. Removal of surface soil may be required to obtain fill meeting the criteria above.

5.1.3 Compaction

Surfaces within the footprint of the perimeter berm should be scarified to a depth of at least 8-inches. The scarified soil should be moisture conditioned to at least optimum moisture content and compacted to at least 90 percent relative compaction. ASTM test method D-1557 should be used to establish the reference values for computing optimum moisture content and relative compaction.

Fill should be placed in lifts 8-inches or less in loose thickness and moisture conditioned to at least optimum moisture content. Moisture conditioning should be performed prior to compaction. Each lift should be methodically compacted to at least 90 percent relative compaction. Material that fails to meet the moisture or compaction criteria should be loosened by ripping or scarifying, moisture conditioned, and then recompacted. Fill should be placed on horizontal surfaces. At connections with adjoining slopes, the fill should be benched into the existing slope to allow recompaction of some of the existing soil. The horizontal bench width into the existing slopes should not exceed 5 feet.

In areas where traffic is anticipated, including the berm crest and ramps, the upper 6-inches of subgrade should be compacted to at least 95 percent relative compaction and rolled to provide a smooth, firm-yielding surface. Subgrade soils should be proof-rolled prior to placing aggregate base. Soft or pumping areas should be aerated or excavated and recompacted.

Aggregate base should be placed in thin lifts no greater than 6-inches in loose thickness and in a manner that avoids segregation, moisture conditioned as necessary, and compacted to at least 95 percent relative compaction.

5.1.4 Slopes

Fill slopes should be constructed fat and trimmed back to expose well-compacted fill. Finished slopes should be trackwalked perpendicular to the slope face with a bulldozer after completion. The slopes should be hydroseeded or otherwise planted to promote vegetation. Vegetation should be limited to grasses or other vegetation that can be mowed or disked to allow inspection of the landside slope. Trees, bushes, and brush should not be allowed within the footprint of the berm slopes.

5.2 Geotechnical Services During Construction

Before construction, we should review project foundation and grading plans and specifications for conformance with the intent of our recommendations. During construction we should observe and/or test the geotechnical aspects of grading and foundation construction including but not limited to subgrade preparation, placement and compaction of fill, and foundation excavations. If conditions are encountered during construction that are not consistent with those described herein, we should be contacted to review our recommendations and provide alternatives, if appropriate.

6 REFERENCES

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Cache Slough Mitigation Bank
Solano County, California

VICINITY MAP

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FIG. 1

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SITE PLAN

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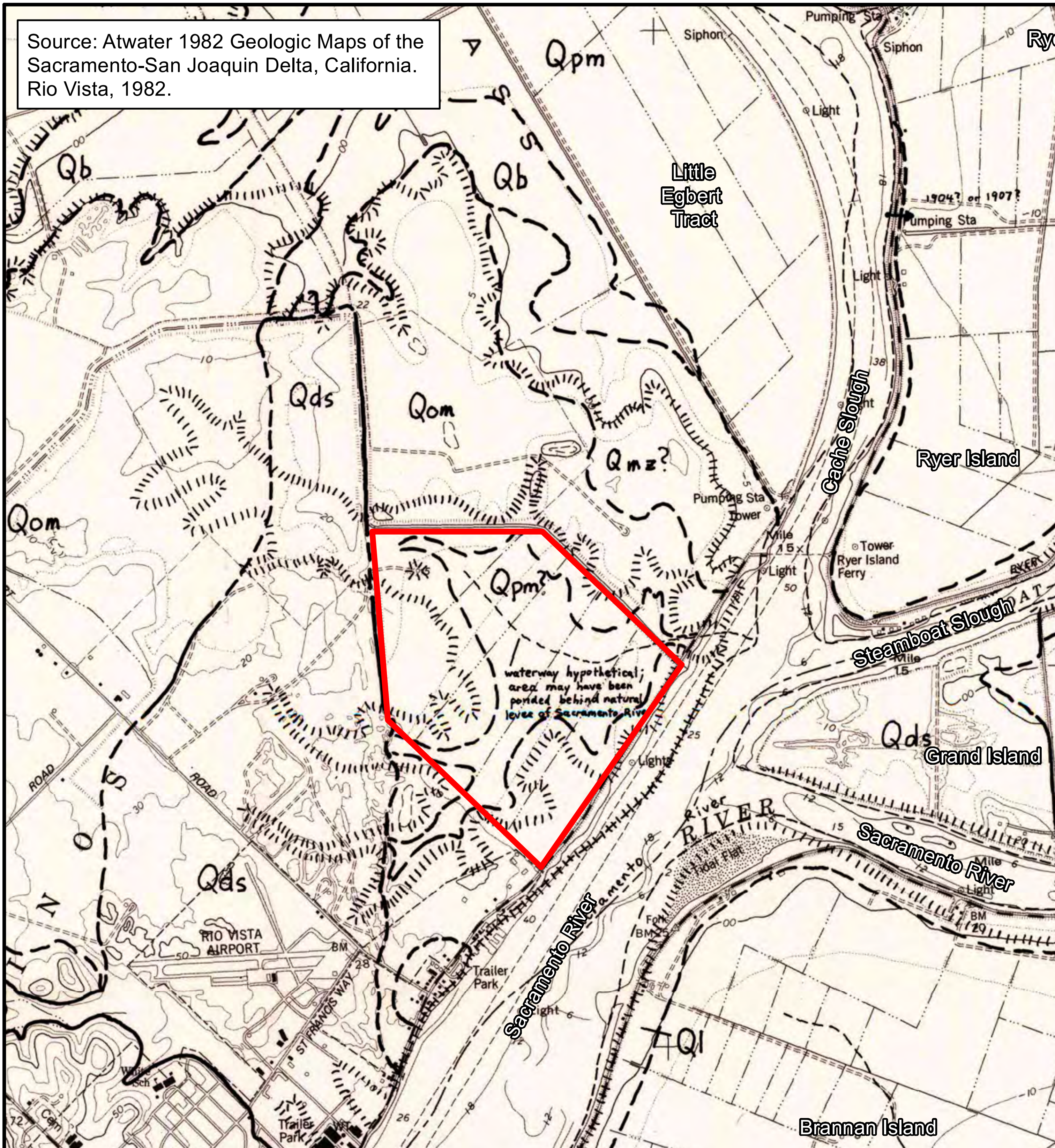
FIG. 2



Project Boundary

"""" - Approximate Landward Margin of Tidal Wetland at Low River Stages circa 1850

Qds - Hydraulic-Dredge Soils
Qpm - Peat & Mud
Qb - Flood-basin deposits
Ql - Natural-levee deposits
Qom - Older Alluvium of Montezuma Hills
Qmz - Montezuma Formation

N

0 2,000
Feet

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ATWATER GEOLOGIC MAP

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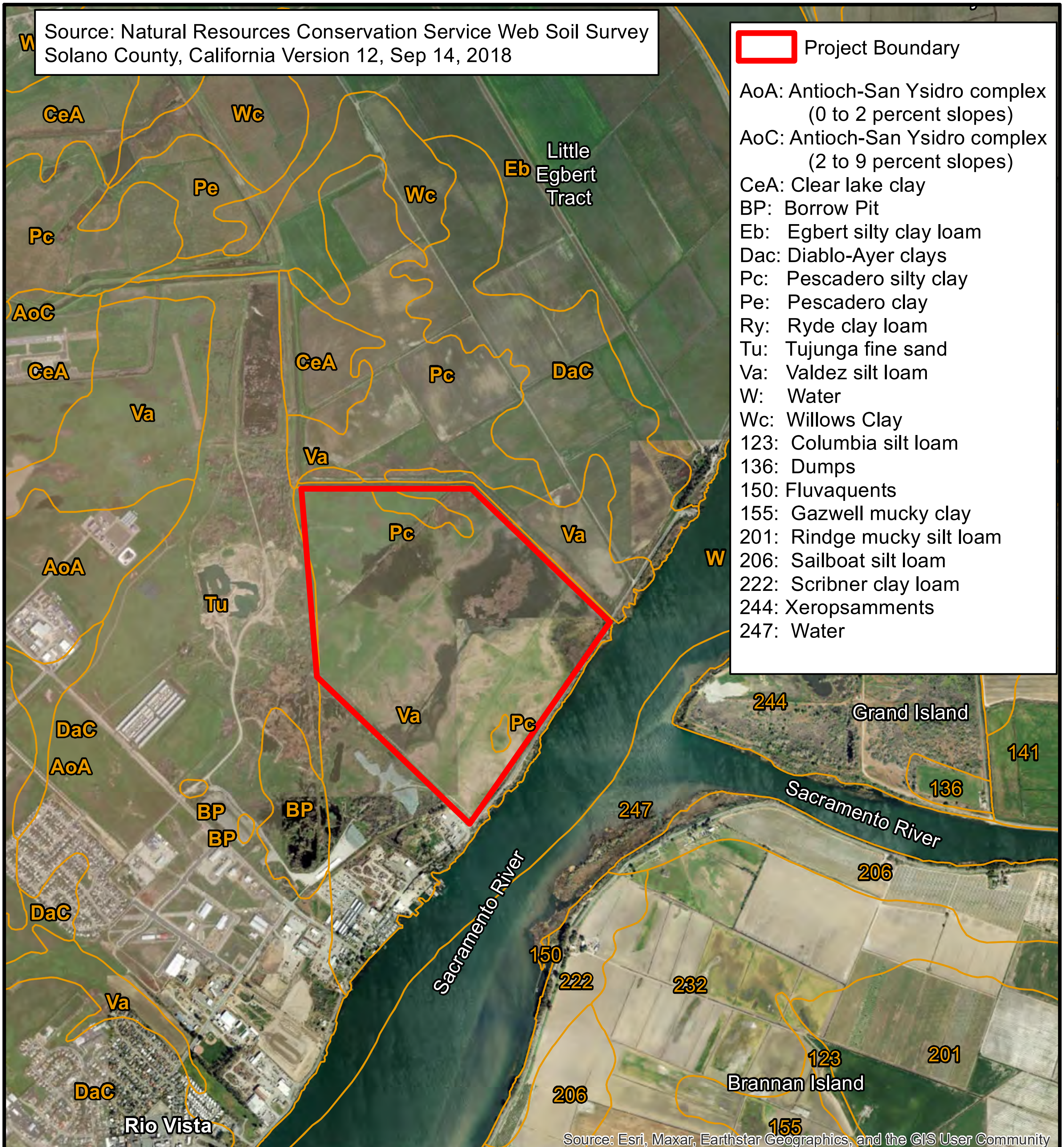
FIG. 3



N

0 2,000
Feet

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SOIL CONSERVATION MAP

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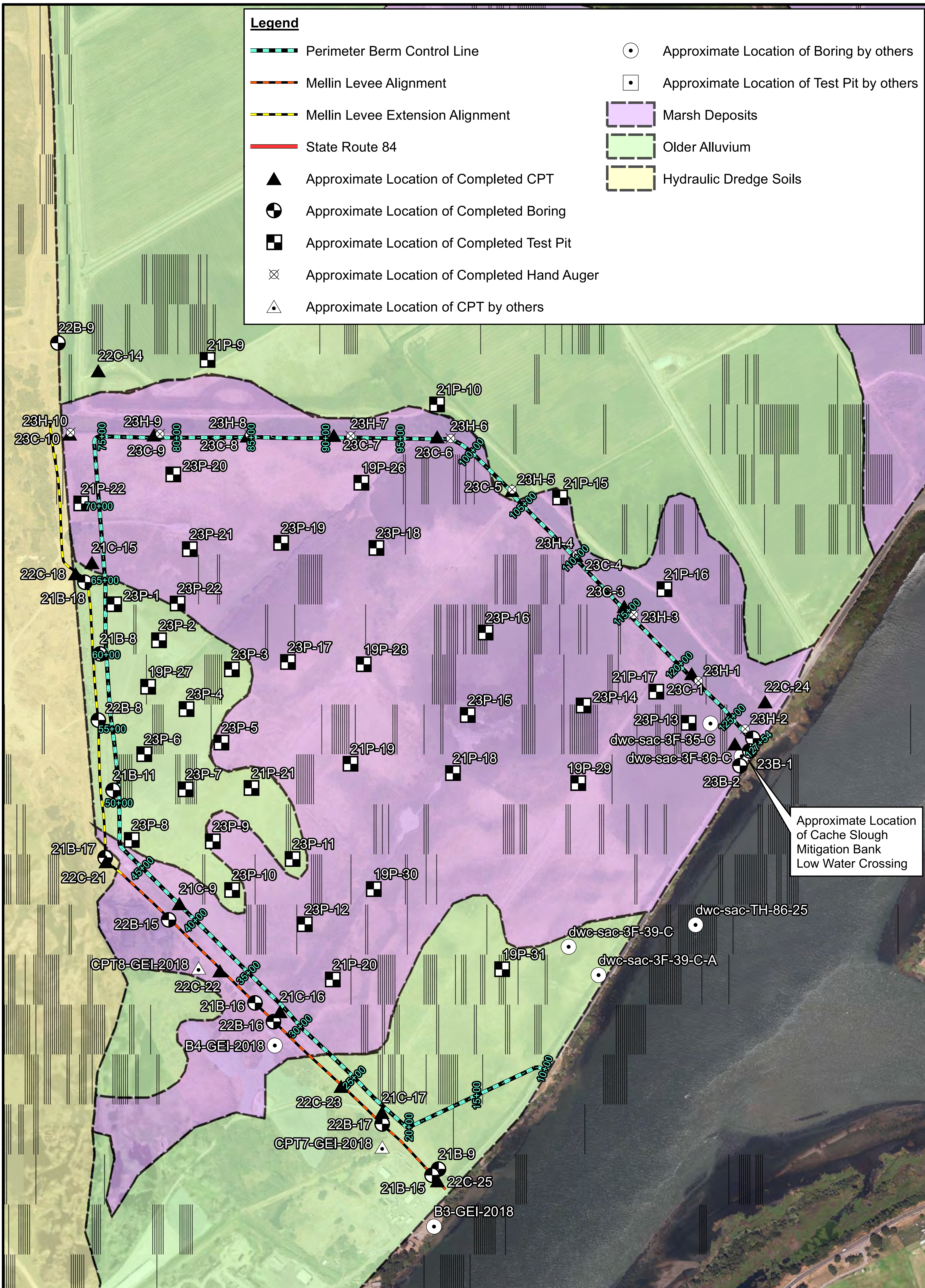
FIG. 4



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SITE GEOLOGY MAP

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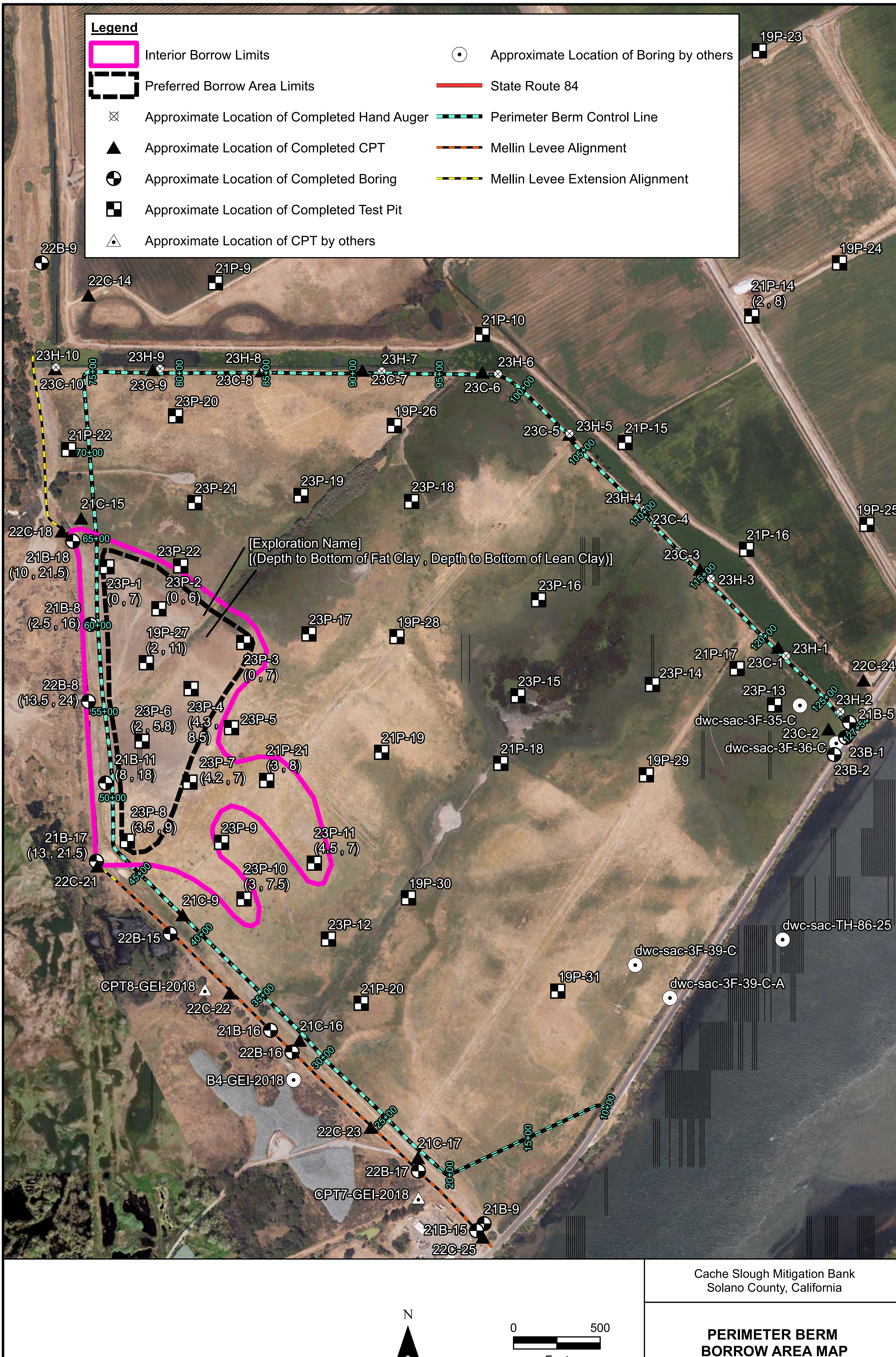
SHANNON & WILSON **FIG. 5**



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PERIMETER BERM BORROW AREA MAP

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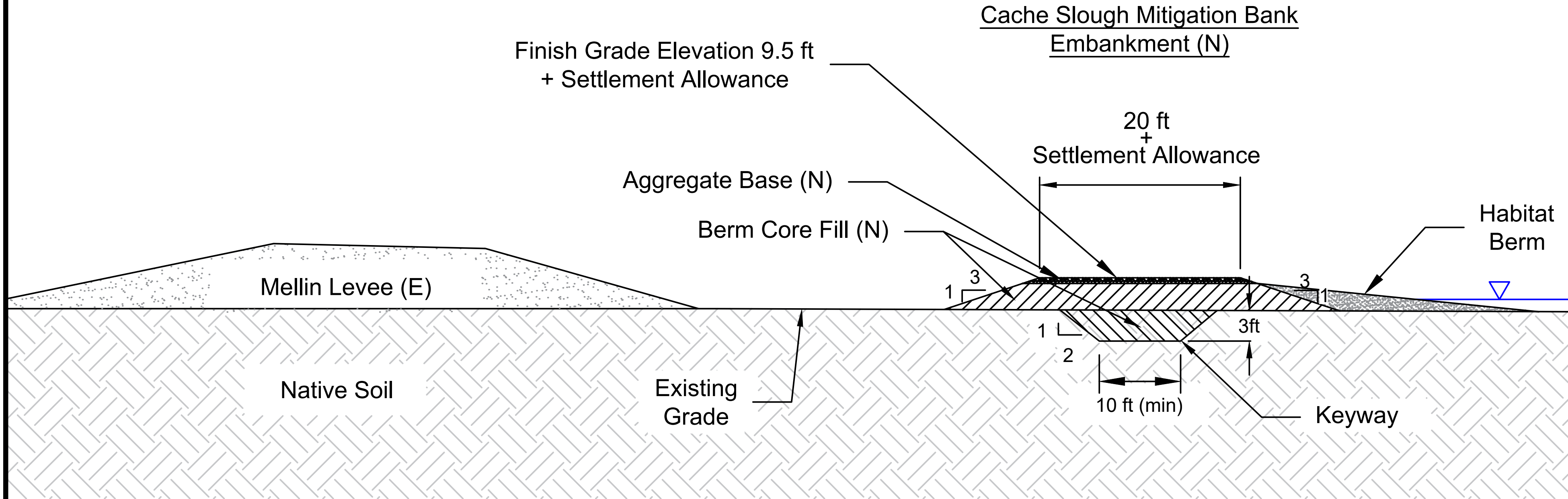
SHANNON & WILSON | FIG. 6



NOT TO SCALE

Notes:

1. Excavate keyway at least 3 feet deep and 10 feet wide with side slopes 2:1 (H:V) or flatter.
2. Scarify and recompact the upper 18-inches of existing soil within berm footprint.
3. Fill for the berm core should be obtained from designated borrow areas.
4. Aggregate base should meet Caltrans Class II Specifications.

Cache Slough Mitigation Bank
Solano County, California

TYPICAL DETAIL
PERIMETER BERM
STATION 10+00 TO 74+00

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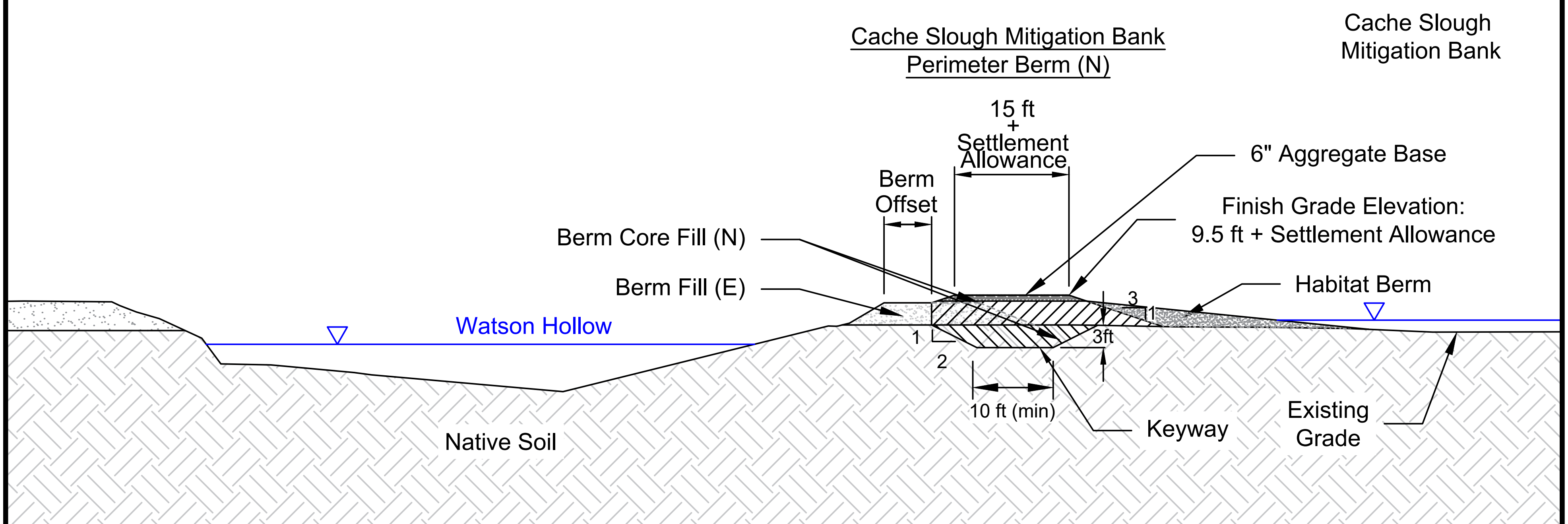
FIG. 7



NOT TO SCALE

Notes:

1. Remove existing berm fill within new berm prism.
2. Excavate keyway at least 3 feet deep and 10 feet wide with side slopes 2:1 (H:V) or flatter.
3. Scarify and recompact the upper 8-inches of existing soil within berm footprint.
4. Fill for the berm core should be obtained from designated borrow areas.
5. Aggregate base should meet Caltrans Class II Specifications.

Cache Slough Mitigation Bank
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TYPICAL DETAIL
PERIMETER BERM
STATION 74+00 TO 127+54

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FIG. 8

APPENDIX A: IDEALIZED GEOLOGIC PROFILES

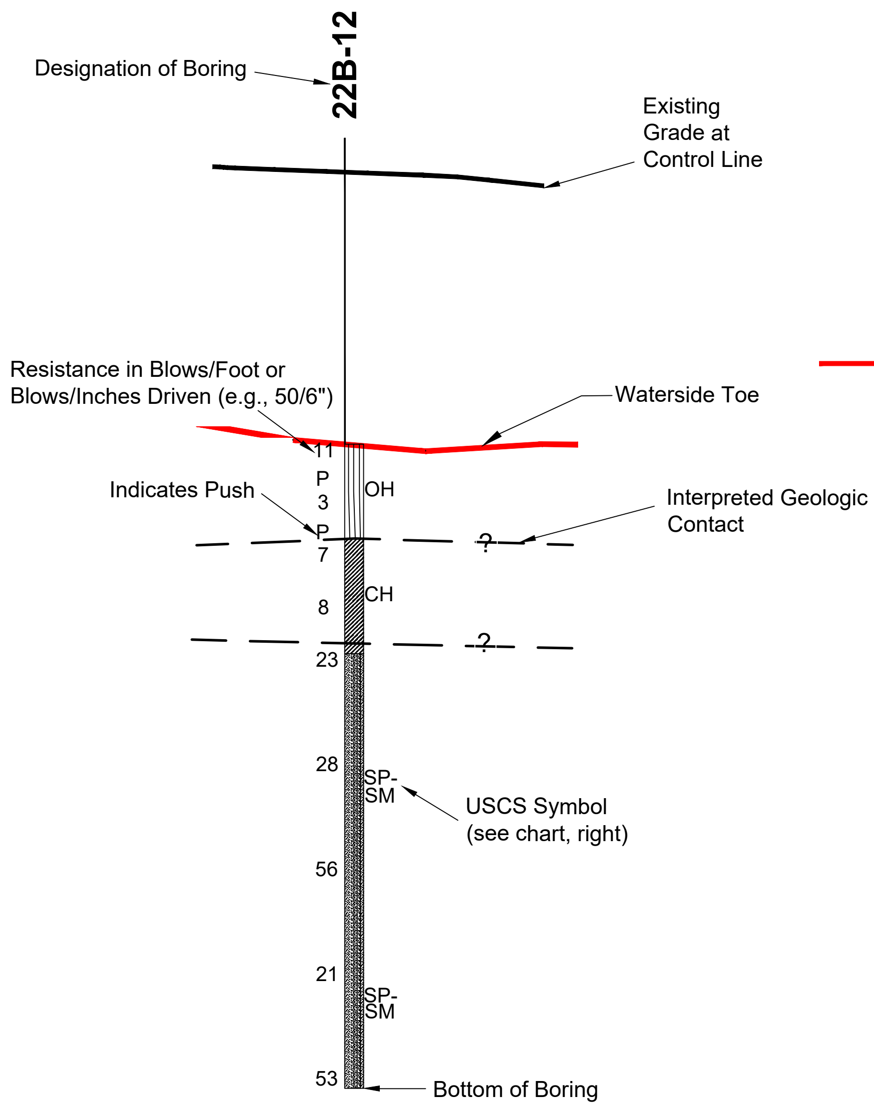
Appendix A

Idealized Geologic Profiles

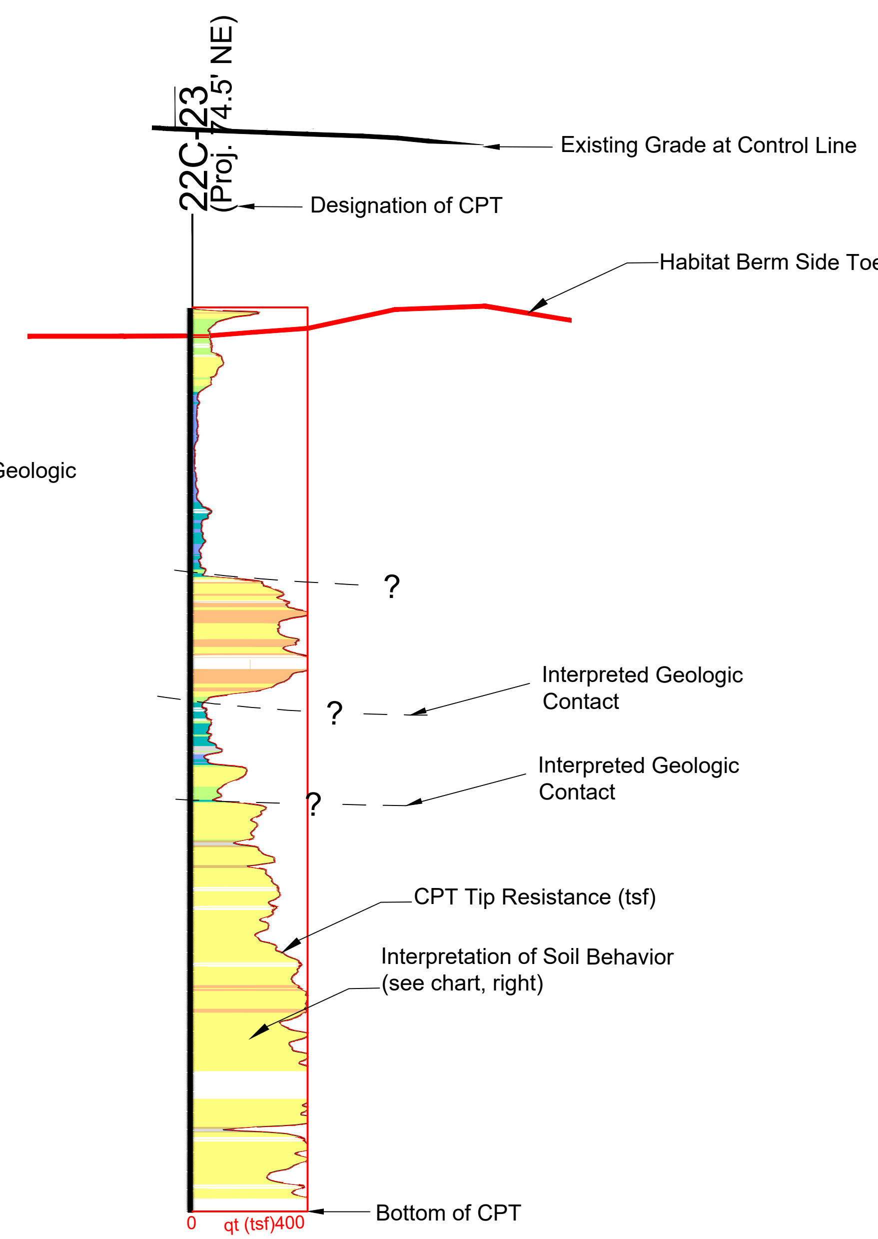
FIGURES

Filename: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Deliverables\Design Report Production\Appendix A\110926 Figure A-1 Geologic Profile Legend restore.dwg Layout: Sheet 1 Landscape Date: 10-

BORING LOG LEGEND



CONE PENETRATION TEST (CPT) LEGEND



MAJOR DIVISIONS				GROUP NAMES
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION IS RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW	WELL GRADED GRAVEL
			GP	POORLY GRADED GRAVEL
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SANDS 50% OR MORE OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SANDS WITH LESS THAN 5% FINES	SW	WELL GRADED SAND
			SP	POORLY GRADED SAND
		SANDS WITH OVER 12% FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE GRAINED SOILS 50% OR MORE PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	SILT
			CL	LEAN CLAY
			OL	ORGANIC CLAY, ORGANIC SILT
	SILTS AND CLAYS LIQUID LIMIT 50 OR MORE		MH	ELASTIC SILT
			CH	FAT CLAY
			OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			Pt	PEAT

UNIFIED SOIL CLASSIFICATION SYSTEM- ASTM D 2487

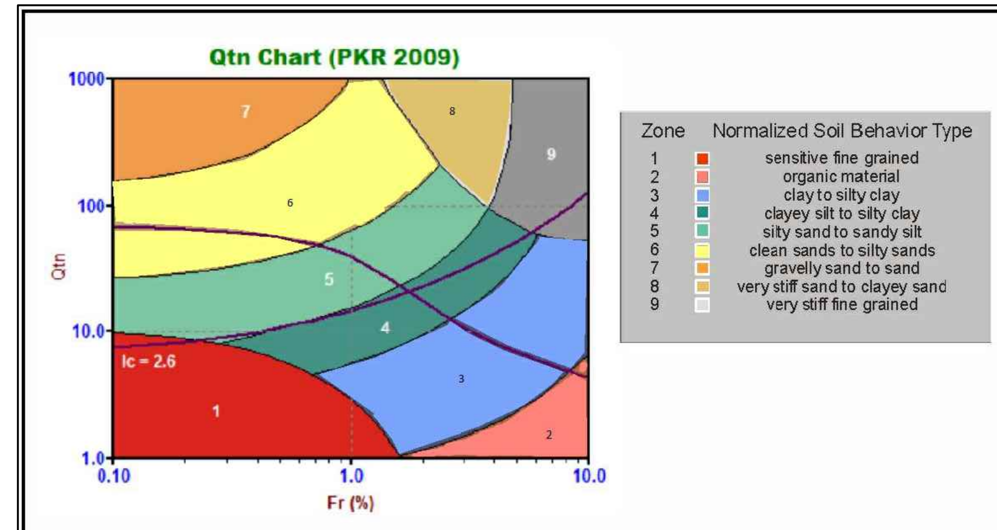


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

Note: Interpretation of Soil Behavior Type is based on the charts described by Robertson et al (2009).

NOTES

1. The profiles are constructed from surface elevations based on the North American Vertical Datum 1988 (NAVD88). The geology shown is derived from borings conducted by Shannon & Wilson, Inc. for this study and from borings conducted by Shannon & Wilson and others for previous studies. Elevations and geologic contacts should be considered approximate. Contacts between borings are based on local geologic experience; however, variations between the profile and actual conditions are likely to exist.

2. Detailed logs of the current project explorations are presented in the data report.

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED SOIL PROFILE LEGEND

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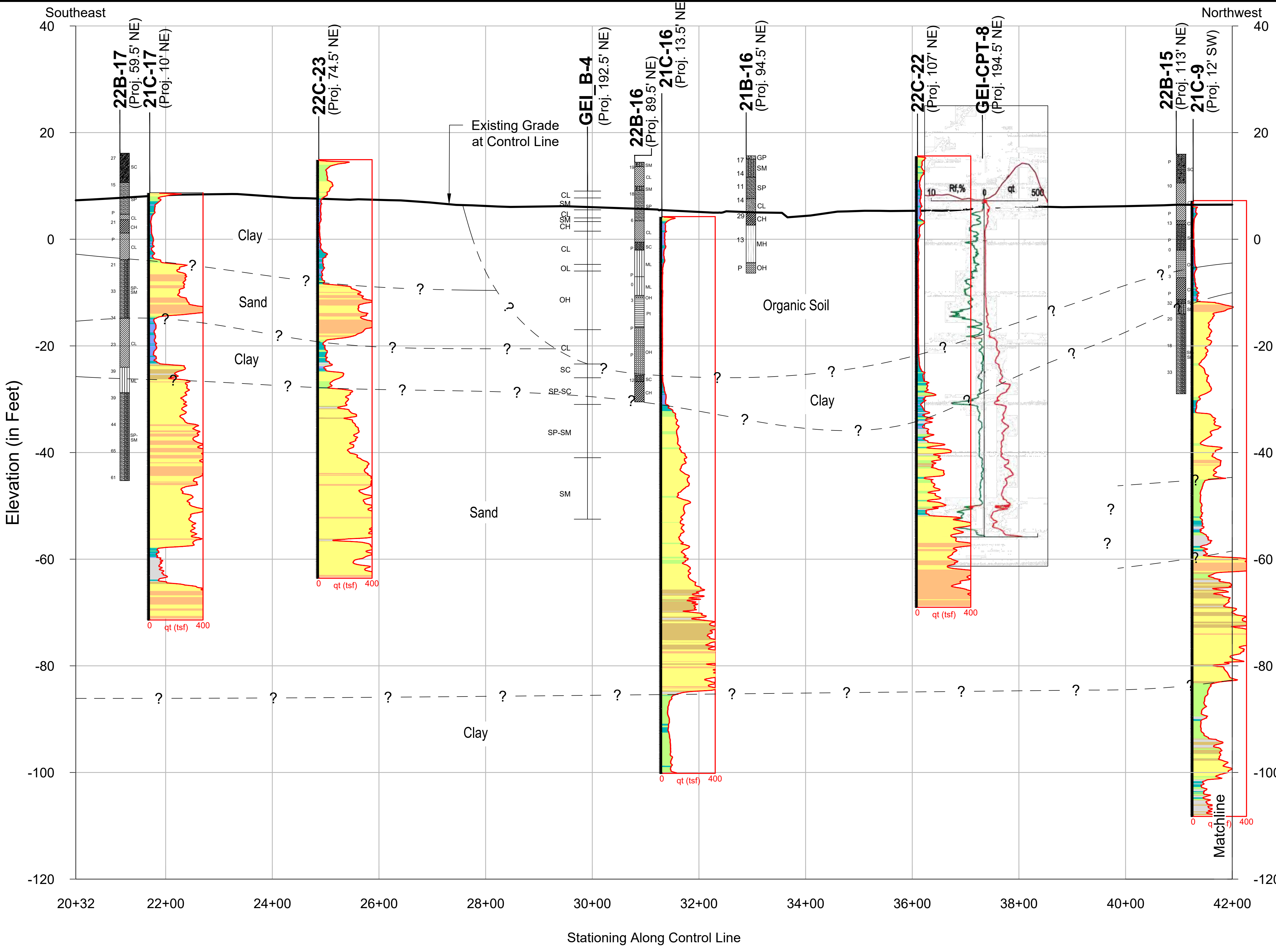
FIG. A-1



NOTE

See Figure A-1 for profile legend and notes.

0 20 40

Vertical Scale in Feet

0 200 400

Horizontal Scale in Feet

Vertical Exaggeration = 10X

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC PROFILE
CACHE SLOUGH MITIGATION BANK
STATION 20+32 to 42+00

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FIG. A-2

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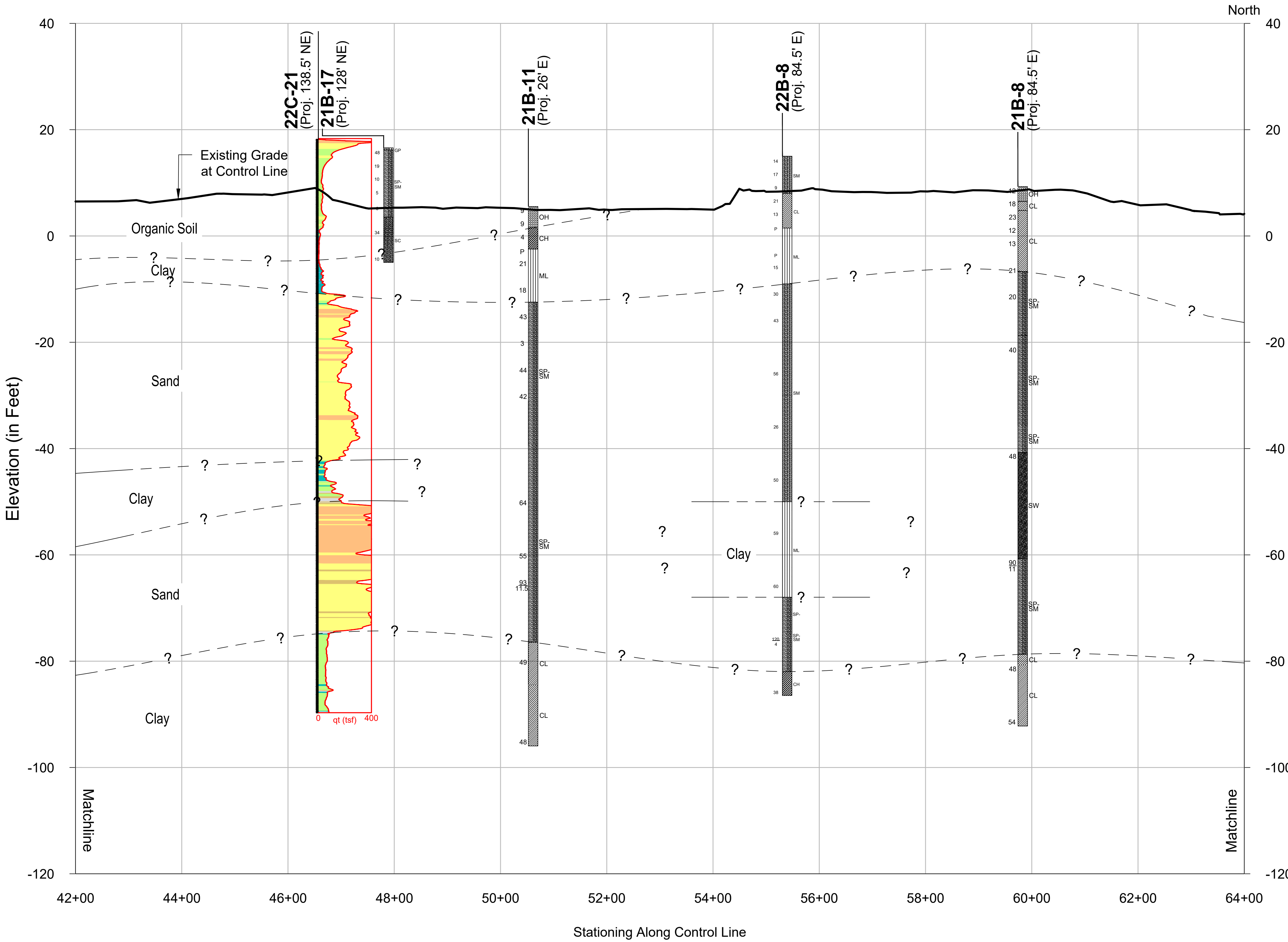

NOTE

See Figure A-1 for profile legend and notes.

Vertical Scale in Feet

Horizontal Scale in Feet

Vertical Exaggeration = 10X

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC PROFILE
CACHE SLOUGH MITIGATION BANK
STATION 42+00 to 64+00

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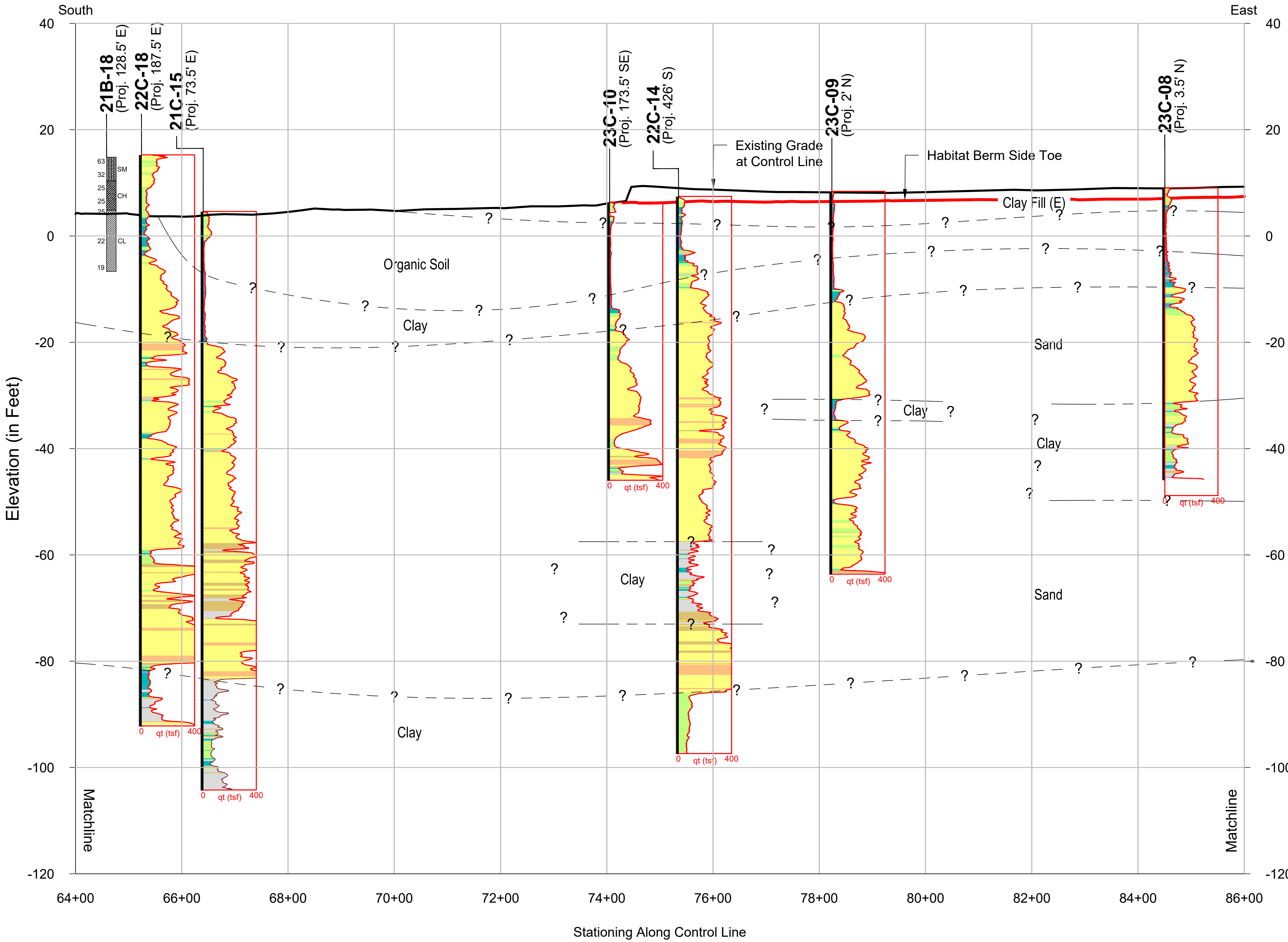
FIG. A-3



NOTE

See Figure A-1 for profile legend and notes.

0 20 40

Vertical Scale in Feet

0 200 400

Horizontal Scale in Feet

Vertical Exaggeration = 10X

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC PROFILE
CACHE SLOUGH MITIGATION BANK
STATION 64+00 to 86+00

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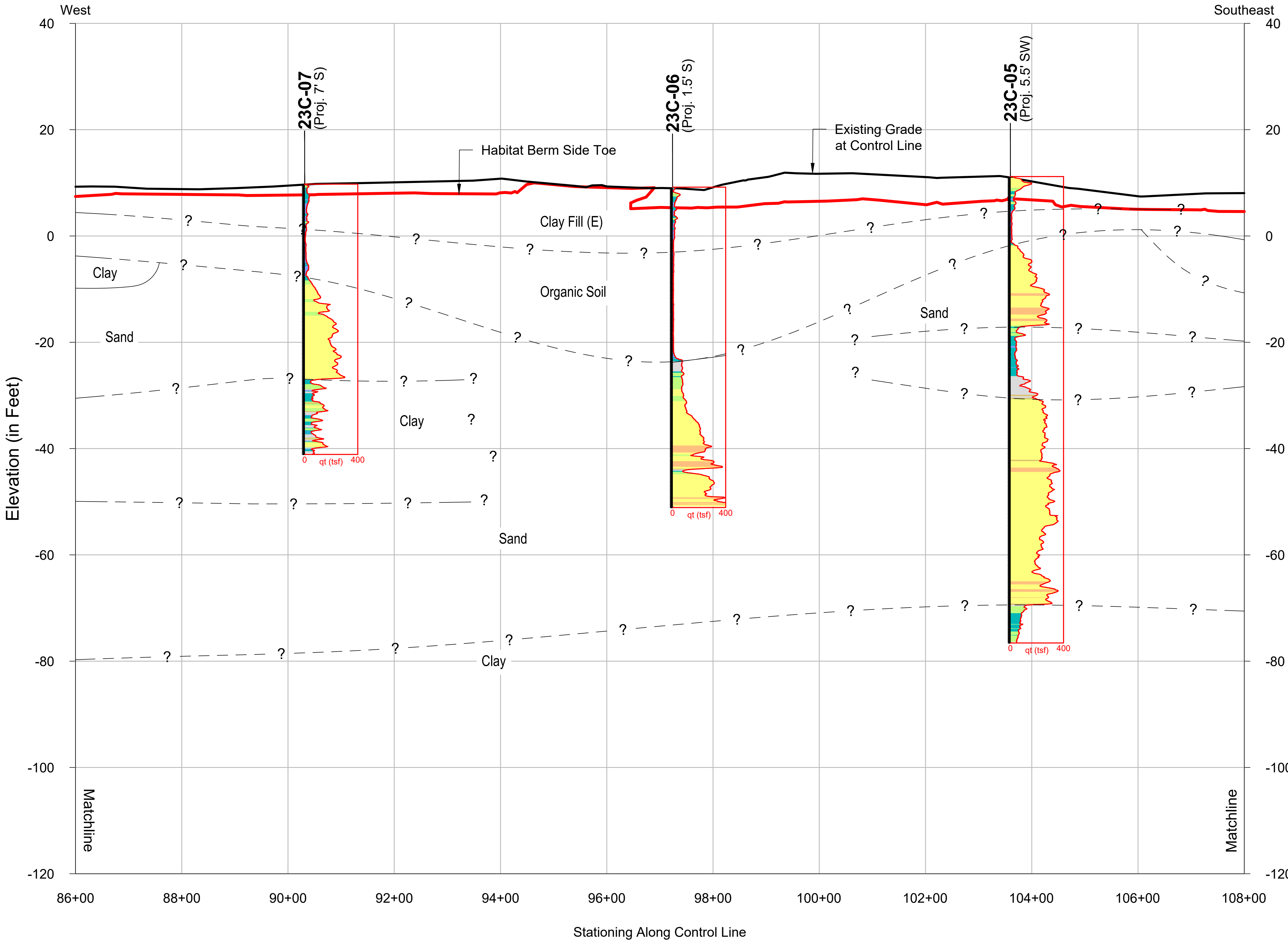
FIG. A-4



NOTE

See Figure A-1 for profile legend and notes.

0 20 40

Vertical Scale in Feet

0 200 400

Horizontal Scale in Feet

Vertical Exaggeration = 10X

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Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC PROFILE
CACHE SLOUGH MITIGATION BANK
STATION 86+00 to 108+00

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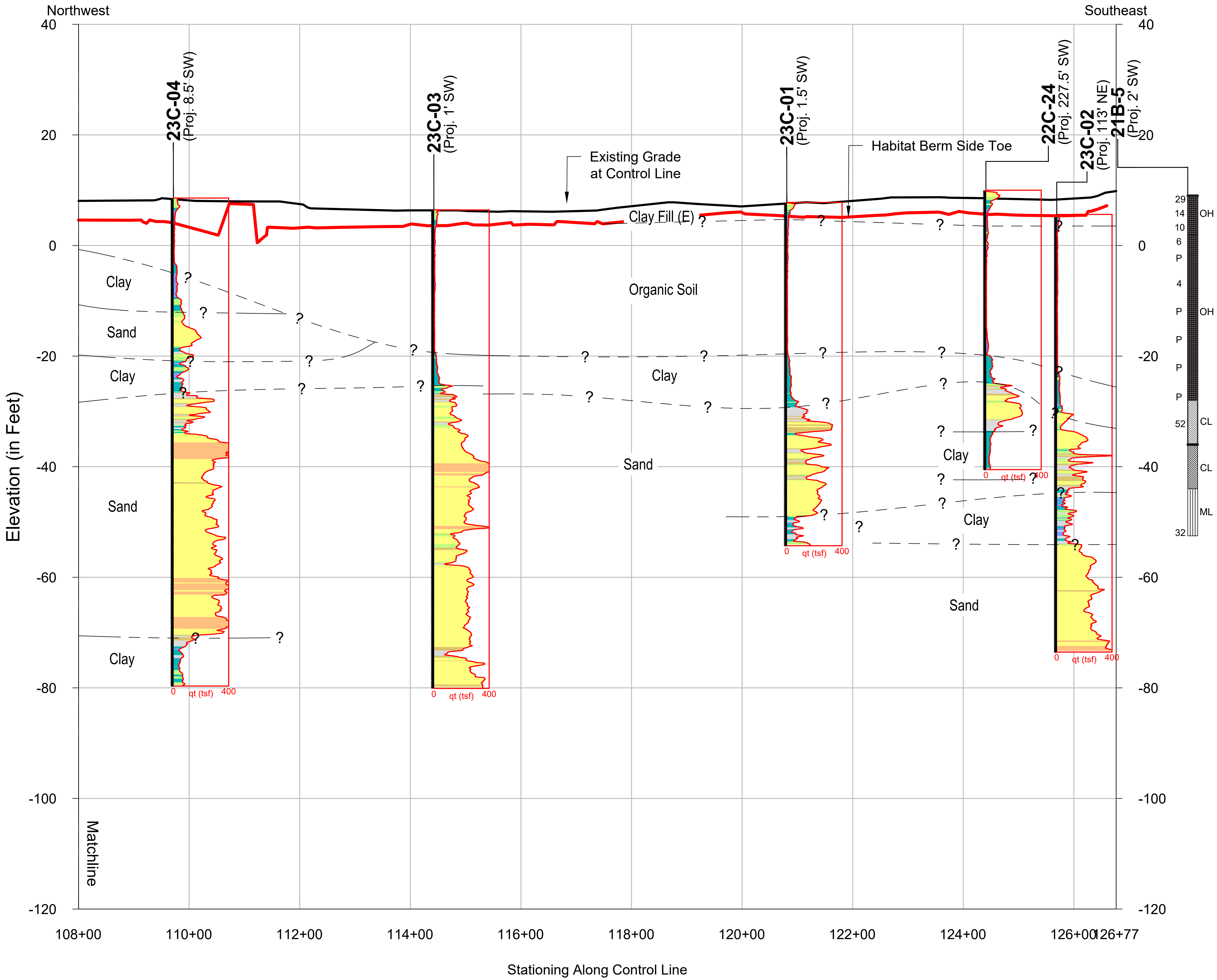
FIG. A-5



NOTE

See Figure A-1 for profile legend and notes.

0 20 40

Vertical Scale in Feet

0 200 400

Horizontal Scale in Feet

Vertical Exaggeration = 10X

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC PROFILE
CACHE SLOUGH MITIGATION BANK
STATION 108+00 to 126+77'

April 2024 110926

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FIG. A-6

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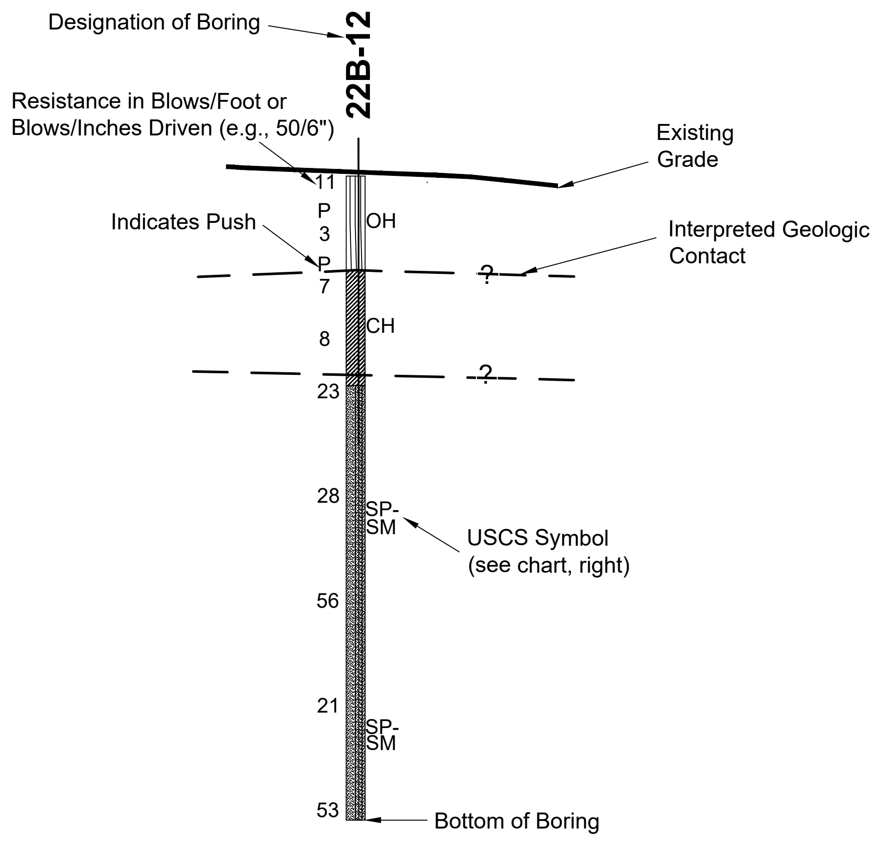
APPENDIX B: IDEALIZED GEOLOGIC CROSS SECTIONS

Appendix B

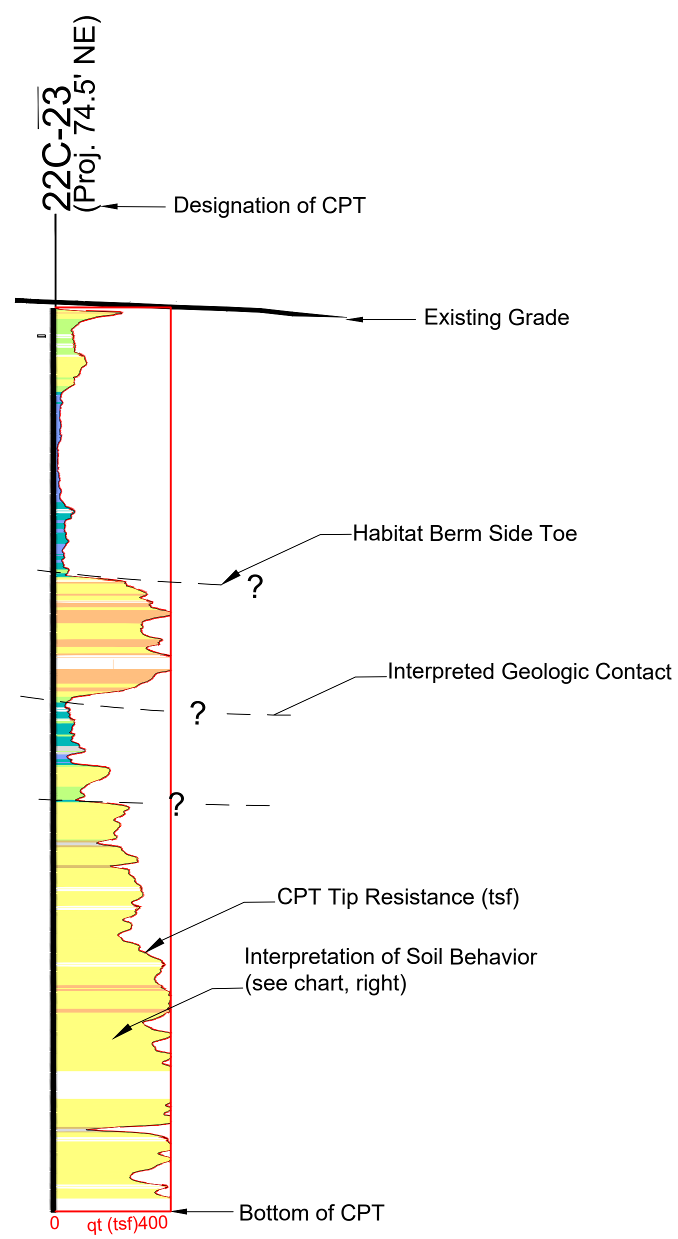
Idealized Geologic Cross Sections

FIGURES

BORING LOG LEGEND



CONE PENETRATION TEST (CPT) LEGEND



MAJOR DIVISIONS				GROUP NAMES
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION IS RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW	WELL GRADED GRAVEL
			GP	POORLY GRADED GRAVEL
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SANDS 50% OR MORE OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SANDS WITH LESS THAN 5% FINES	SW	WELL GRADED SAND
			SP	POORLY GRADED SAND
		SANDS WITH OVER 12% FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE GRAINED SOILS 50% OR MORE PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	SILT
			CL	LEAN CLAY
			OL	ORGANIC CLAY, ORGANIC SILT
	SILTS AND CLAYS LIQUID LIMIT 50 OR MORE		MH	ELASTIC SILT
			CH	FAT CLAY
			OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			Pt	PEAT

UNIFIED SOIL CLASSIFICATION SYSTEM- ASTM D 2487

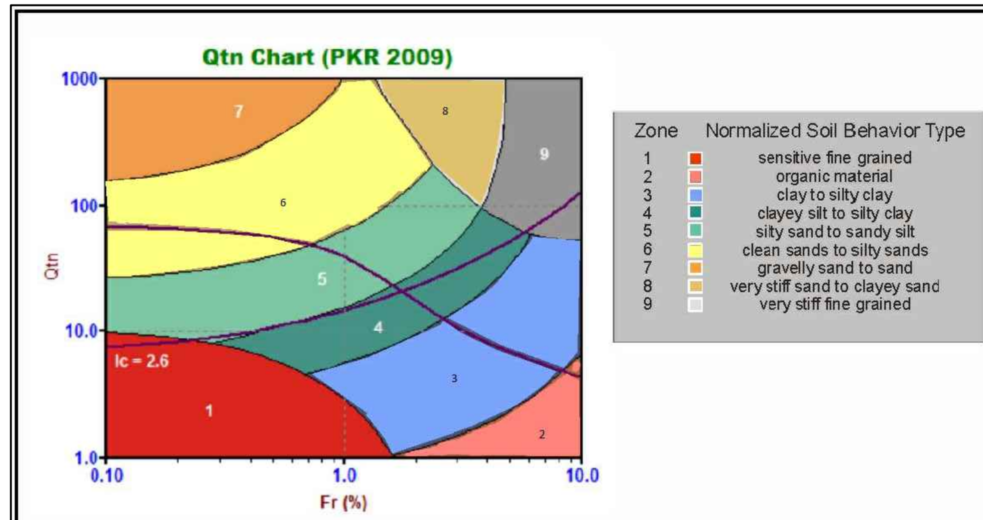


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

Note: Interpretation of Soil Behavior Type is based on the charts described by Robertson et al (2009).

NOTES

1. The profiles are constructed from surface elevations based on the North American Vertical Datum 1988 (NAVD88). The geology shown is derived from borings conducted by Shannon & Wilson, Inc. for this study and from borings conducted by Shannon & Wilson and others for previous studies. Elevations and geologic contacts should be considered approximate. Contacts between borings are based on local geologic experience; however, variations between the profile and actual conditions are likely to exist.

2. Detailed logs of the current project explorations are presented in the data report.

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED CROSS SECTION LEGEND

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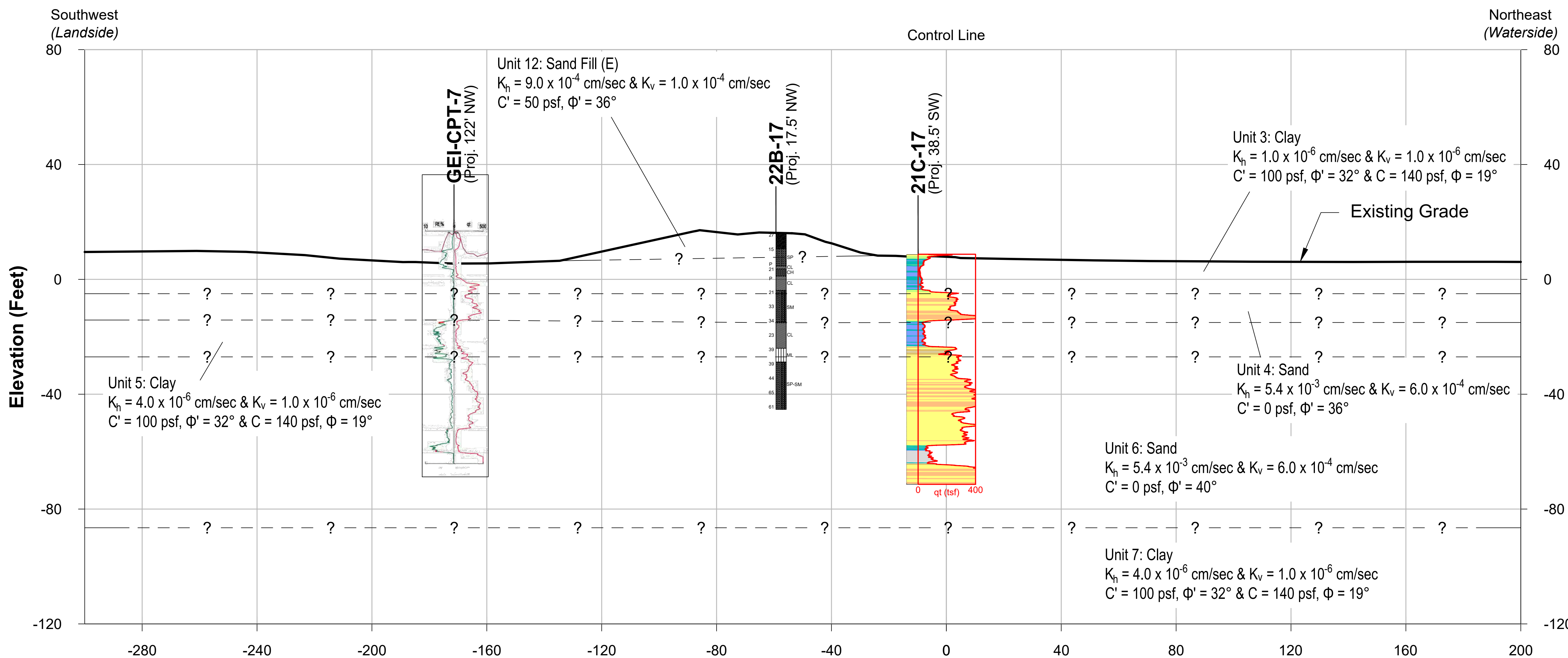
FIG. B-1



NOTE

See Figure No. B-1 for section legend and notes.

0 40 80

Scale in Feet
Vertical = Horizontal

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC CROSS SECTIONS CACHE SLOUGH MITIGATION BANK STATION 21+32

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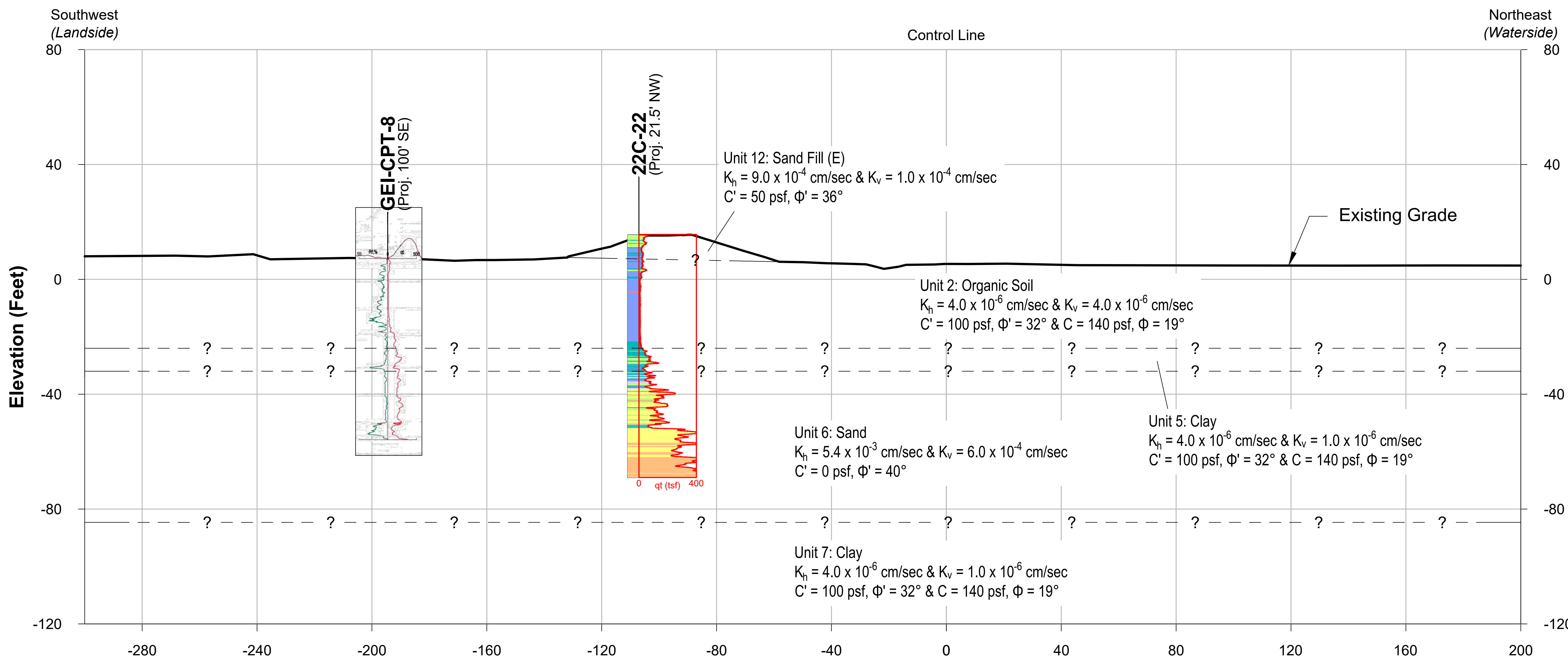
FIG. B-2

Filename: C:\Users\jrs\CAD Group Dropbox\JDrive_SEA\110926\002\110926 Profiles.dwg Layout: Section 6+00 Date: 09-25-2023 Login: JRS



NOTE

See Figure No. B-1 for section legend and notes.

0 40 80

Scale in Feet
Vertical = Horizontal

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC CROSS SECTIONS CACHE SLOUGH MITIGATION BANK STATION 36+32

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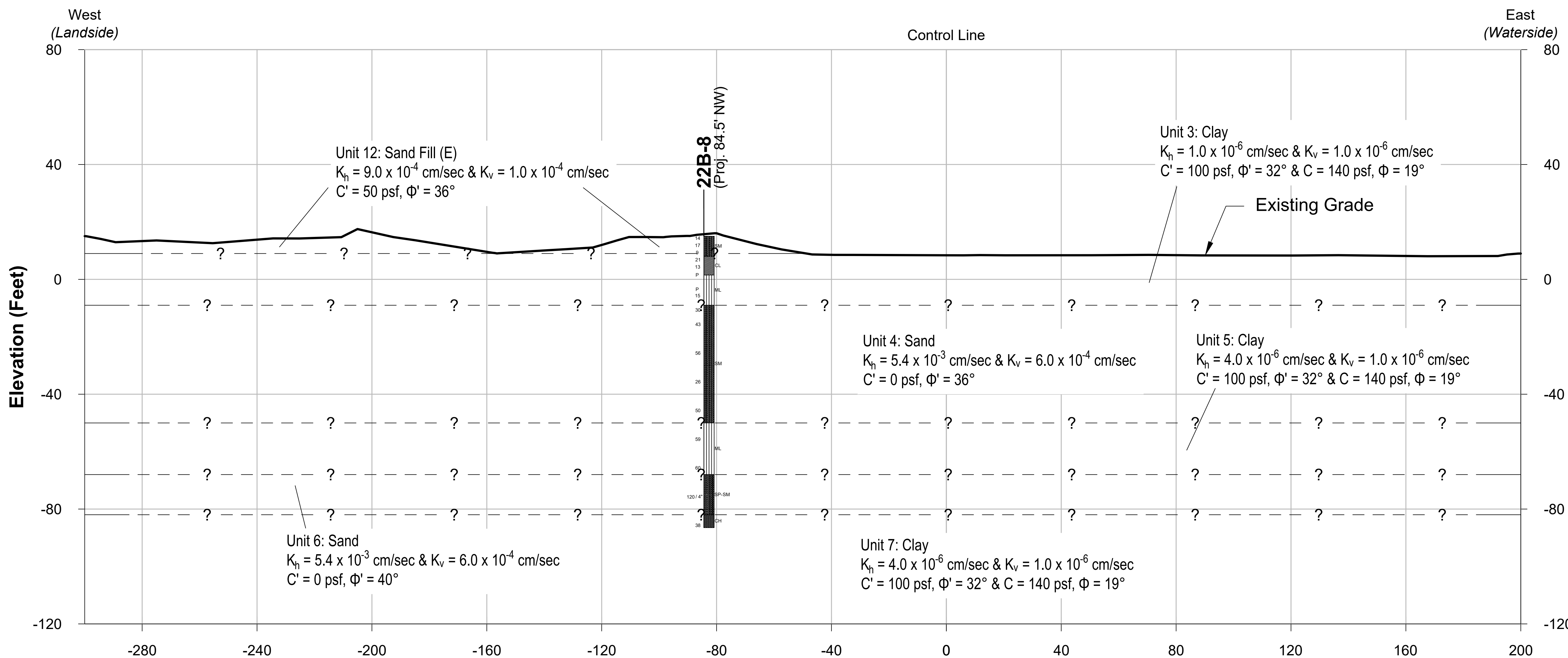
FIG. B-3



NOTE

See Figure No. B-1 for section legend and notes.

0 40 80

Scale in Feet
Vertical = Horizontal

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC CROSS SECTIONS CACHE SLOUGH MITIGATION BANK STATION 56+32

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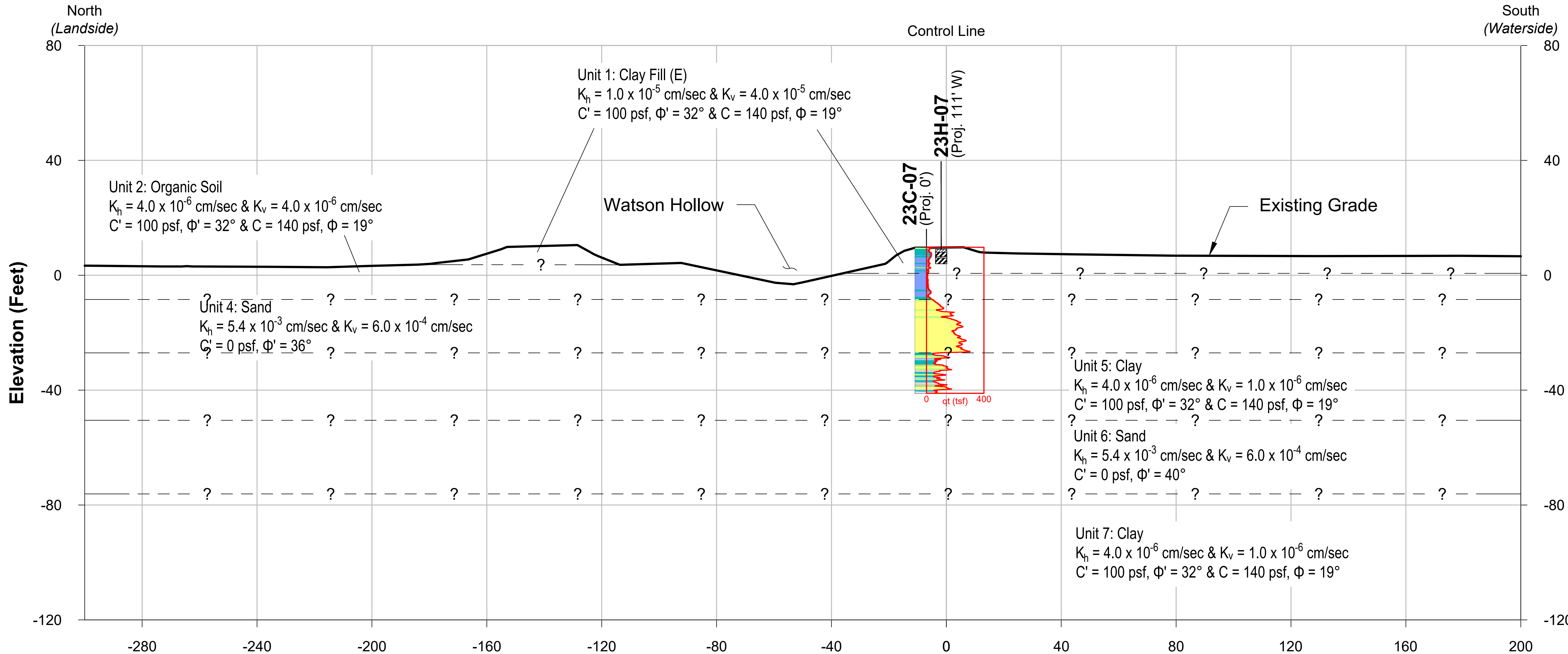
FIG. B-4



NOTE

See Figure No. B-1 for section legend and notes.

0 40 80

Scale in Feet
Vertical = Horizontal

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC CROSS SECTIONS CACHE SLOUGH MITIGATION BANK STATION 90+32

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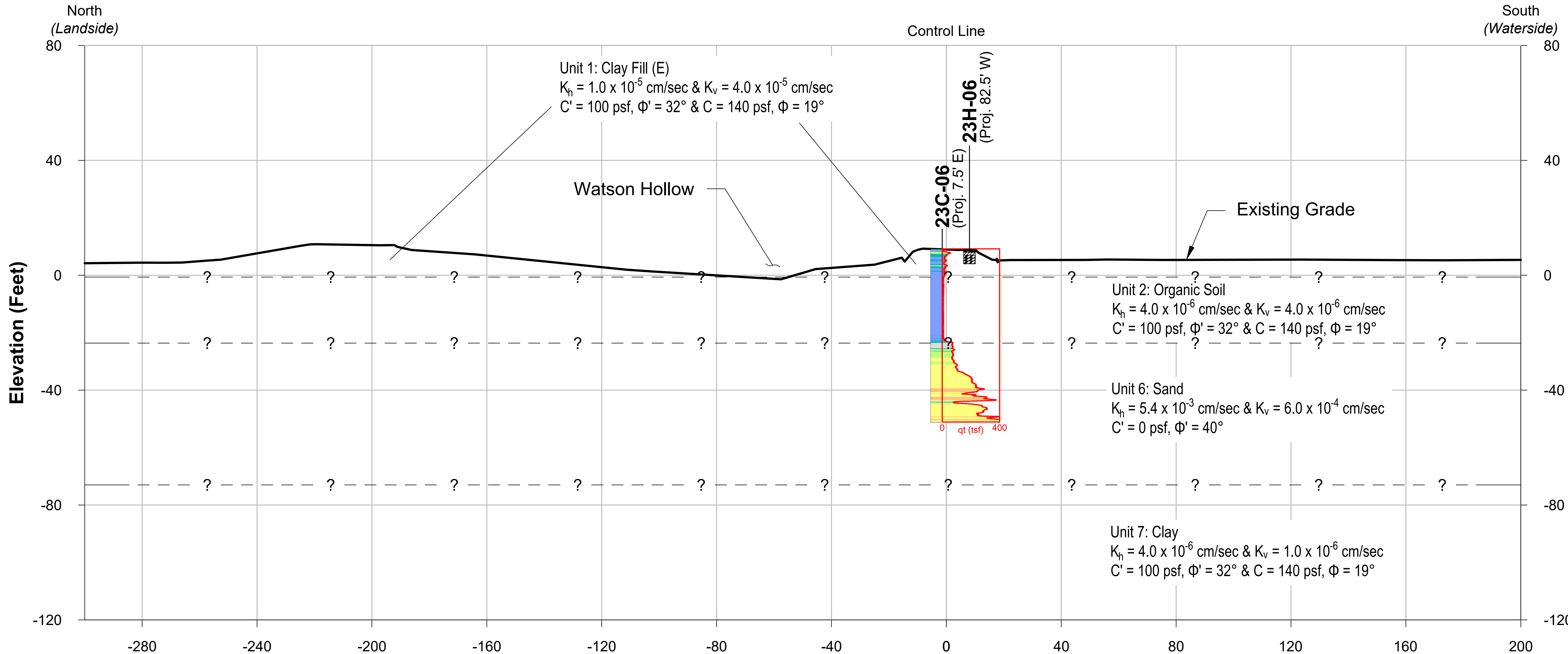
FIG. B-5

Filename: C:\Users\jrs\CAD Group Dropbox\JDrive_SEA\110926\002\110926 Profiles.dwg Layout: Section 75+00 Date: 09-25-2023 Login: JRS



NOTE

See Figure No. B-1 for section legend and notes.

0 40 80

Scale in Feet
Vertical = Horizontal

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC CROSS SECTIONS CACHE SLOUGH MITIGATION BANK STATION 97+32

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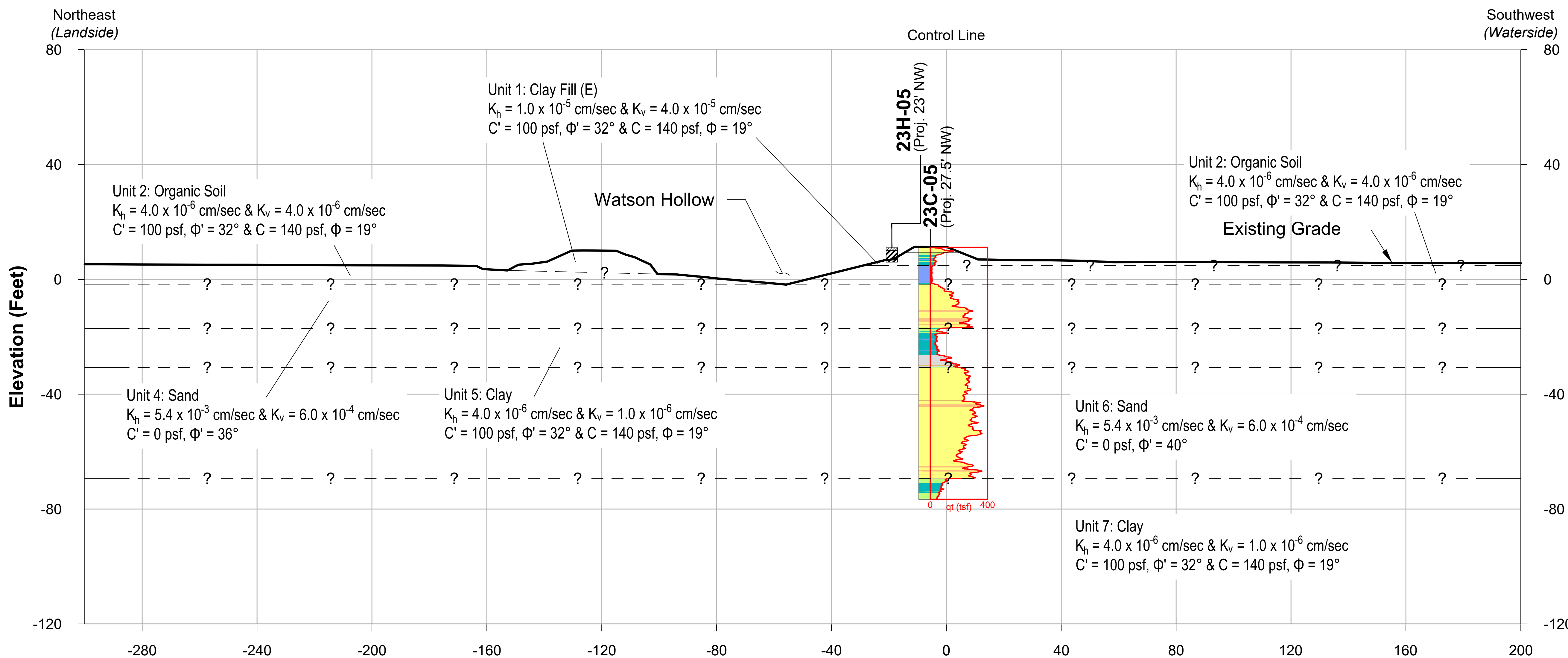
FIG. B-6

Filename: C:\Users\jrs\CAD Group Dropbox\JDrive_SEA\110926\002\110926 Profiles.dwg Layout: Section 82+00 Date: 09-25-2023 Login: JRS



Filename: C:\Users\jrs\CAD Group Dropbox\JDrive_SEA\110926\002\110926 Profiles.dwg Layout: Section 88+00 Date: 09-25-2023 Login: JRS

NOTE

See Figure No. B-1 for section legend and notes.

0 40 80

Scale in Feet
Vertical = Horizontal

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC CROSS SECTIONS CACHE SLOUGH MITIGATION BANK STATION 103+32

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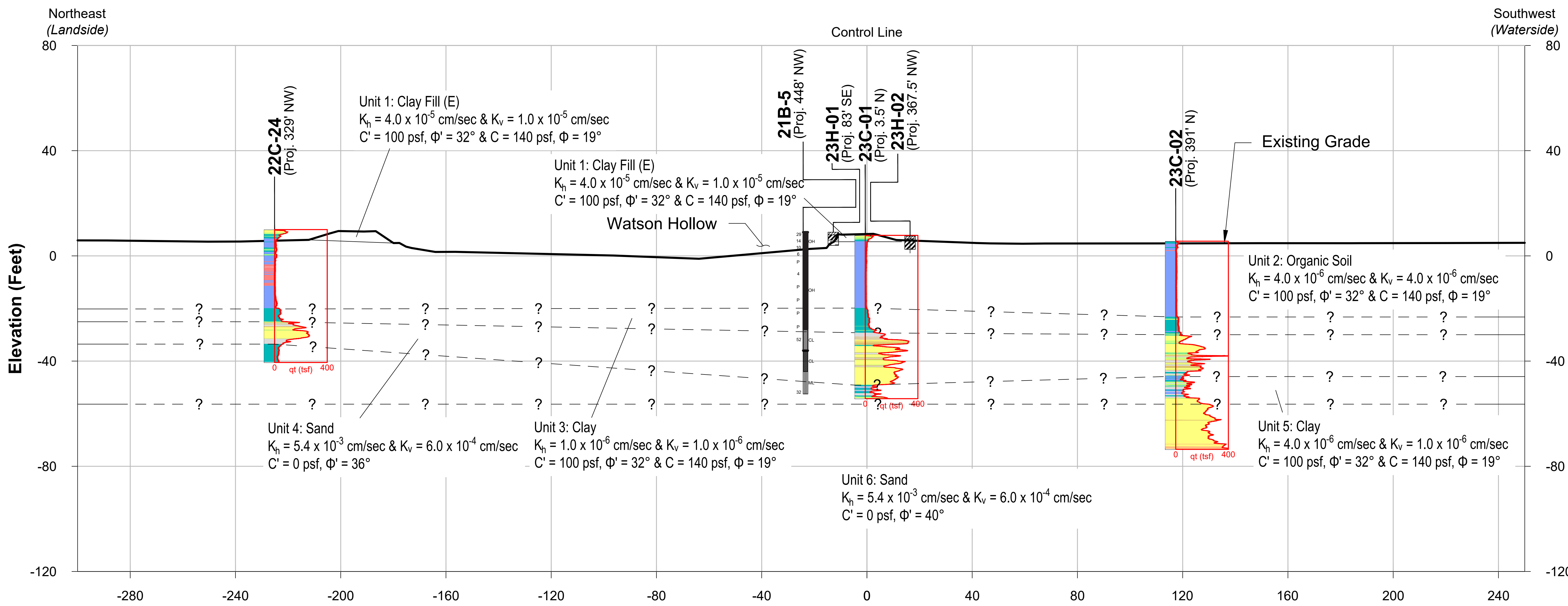
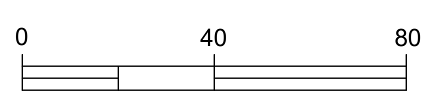
FIG. B-7



NOTE

See Figure No. B-1 for section legend and notes.



Scale in Feet
Vertical = Horizontal

Cache Slough Mitigation Bank
Solano County, California

IDEALIZED GEOLOGIC CROSS SECTIONS CACHE SLOUGH MITIGATION BANK STATION 122+32

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FIG. B-8

Filename: C:\Users\jrs\CAD Group Dropbox\JDrive_SEA\110926\002\110926 Profiles.dwg Layout: Section 107+00 Date: 09-25-2023 Login: JRS

Appendix C

Seepage Analysis

CONTENTS

FIGURES

APPENDIX C
Seepage Analysis

C-1 SEEPAGE ANALYSIS

C.1 General

We performed analysis to evaluate seepage at the seven locations listed in Section 4.1.3 of this report. The subsurface conditions at these locations are described in the main text and depicted in Appendix B. The details of the analysis and results are presented below.

C.2 Soil Parameters

The parameters used in the seepage analysis are presented on Figure C-1. We present a discussion of the selected soil parameters below.

The permeability values were estimated based on laboratory test data to evaluate soil type. We used the Guidance Document's recommended vertical hydraulic conductivity (Figure A2-30) versus fines content plot and Table A2-12 in the Guidance Document to select parameters.

The permeability of the clay fill (Unit 1) was compared to the presumptive values for clay fills placed in an uncontrolled manner.

The permeability of the organic soil (Unit 2) was selected at the high-permeability bound of the range of presumptive values to reflect the effects of seasonal drying. We used an anisotropic ratio of 1. The actual vertical permeability is likely higher than the horizontal permeability where the ground has been desiccated (anisotropic value below 1).

The permeability of Units 3, 5 and 7 were selected near the high-permeability bound of the presumptive values for natural, intact clays. We used an anisotropic ratio of 1 for Unit 3 for the same reason as noted above for Unit 2. We selected an anisotropic ratio of 4 for the deeper clay deposits (Units 5 and 7).

We estimated the permeability of the native sand soils (Units 4 and 6) based on sieve analysis test data and compared them to the presumptive values presented in the Guidance Document (2015) for silty sands with less than 25 percent fines. The fines content of the native sand soils (Units 4 and 6) varied from 6 to 49 percent with an average fines content of 11 percent. We used a permeability ratio of 9 for the sand units because the units are alluvial and stratified.

The new levee fill (Unit 8) will have at least 20 percent fines and 100 percent passing the 2-inch sieve. The fines will have a plasticity index of at least 8. This material would classify as the controlled placement, clayey fines, under the embankments group. We selected an anisotropic ratio of 4 for the new levee fill (Unit 8).

C.3 Analysis Description

We performed seepage analysis using computer program SEEP/W. We analyzed seepage for three water surface elevations. We modeled the water level on the water side of the berm at Elevation 6.5 feet, Elevation 8.5 feet, and Elevation 9.5 feet to represent the MHHW level, 2-year recurrence, and top of berm conditions, respectively. For runs adjacent to Watson Hollow, we modeled the water level in Watson Hollow at Elevation 4.4, the Mean Tide Level.

The seepage analyses assume steady state flow conditions. The models include the new berm constructed to Elevation 9.5 feet. The crest width is 20 feet plus the width required to provide settlement allowance for that station. The side slopes for the new berm are 3H:1V. The model extends 2,000 feet landward and 1,000 feet on the waterside. The model includes a high mesh density (2 feet by 2 feet) in and around the berm within approximately 80 feet from the levee centerline and a lower mesh density (4 feet by 4 feet) outside of approximately 80 feet from the berm centerline.

The model includes a no-flow boundary condition along the vertical face of the waterside boundary and the bottom of each model; the modeled water level as a total head boundary condition applied to the surface of the waterside of the levee slope; and a total head boundary condition along the vertical face of the landside boundary which matched the landside ground surface elevation. Along the levee crown, landside slope and landside ground surface, a no-flow boundary condition, the model includes the "potential seepage face review" option in SEEP/W. We modeled Watson Hollow at the Mean Tide Level by setting the faces of Watson Hollow to a total head boundary condition of 4.4 feet. No other flows into or out of the system were modeled in the analysis, such as infiltration and evapo-transpiration.

A design consideration for underseepage is the average vertical gradient across the landside blanket (clay layer) where a blanket exists. The average vertical gradient is the total head drop in the vertical direction across the landside blanket, divided by the thickness of the blanket. The critical gradient is calculated as the difference between the unit weight of soil and the unit weight of water divided by the unit weight of water.

The average vertical gradient criteria published in the Guidance Document (2015) and used for this study is presented in Exhibit 4-3.

Exhibit C-1: Average Exit Gradient Criteria

Location on Levee	Maximum Average Vertical Gradient
Landside Toe	0.5
150 feet from Landside Toe*	0.8

* For locations between the landside toe and 150 feet from the landside toe, use linear interpolation to determine maximum average vertical gradient criteria.

C.4 Analysis Results

We calculated the average vertical exit gradient through the clay foundation at the landside levee toe for sections not adjacent to Watson Hollow and in the bottom of Watson Hollow for sections adjacent to Watson Hollow. Exhibits C-1 through C-3 present average vertical gradients (y-gradients) for the three water levels analyzed. Figures C- 2 through C-15 present the average vertical gradients for the condition with the water level on the water side at Elevation 9.5 and the water level in Watson Hollow at the Mean Tide Level. The figures provide the graphical output of the program SEEP/W, including total head contours and the resulting phreatic surface.

Exhibit C-2: Average Vertical Gradient with Water Level on Water Side at Elevation 6.5 feet

Station	Average Vertical Gradient at Landside Berm Toe	Average Vertical Gradient at Bottom of Watson Hollow (Watson Hollow at Mean Tide Level)
21+32	0.00	-
36+32	0.10	-
56+32	0.22	-
90+32	-	0.10
97+32	-	0.03
103+32	-	Not Applicable, No Blanket Present at Bottom of Watson Hollow
122+32	-	0.03

Exhibit C-3: Average Vertical Gradient with Water Level on Water Side at Elevation 8.5 feet

Station	Average Vertical Gradient at Landside Berm Toe	Average Vertical Gradient at Bottom of Watson Hollow (Watson Hollow at Mean Tide Level)
21+32	0.01	-
36+32	0.11	-
56+32	0.26	-
90+32	-	0.32
97+32	-	0.06
103+32	-	Not Applicable, No Blanket Present at Bottom of Watson Hollow
122+32	-	0.04

Exhibit C-4: Average Vertical Gradient with Water Level on Water Side at Elevation 9.5 feet

Station	Average Vertical Gradient at Landside Berm Toe	Average Vertical Gradient at Bottom of Watson Hollow (Watson Hollow at Mean Tide Level)
21+32	0.02	-
36+32	0.12	-
56+32	0.27	-
90+32	-	0.42
97+32	-	0.07
103+32	-	Not Applicable, No Blanket Present at Bottom of Watson Hollow
122+32	-	0.05

Our analysis indicates that the berm sections we analyzed meet the criteria set forth by USACE for average vertical exit gradient.

At Station 103+32, the bottom of Watson Hollow penetrates through the clay layer on the landside into the underlying sand layer. With this configuration, Watson Hollow will be susceptible to piping and sand boils. The bottom of Watson Hollow should be monitored closely within 500 feet of Station 103+32 after the project is opened to tidal conditions and during high water events. Modifications to the bottom, such as raising the bottom elevation in this section of Watson Hollow may be required to reduce the risk of boils.

SEEPAGE MODEL MATERIAL PROPERTIES			
LAYER COLOR	MATERIAL TYPE	HORIZONTAL CONDUCTIVITY, k_H (cm/s)	HORIZ./VERT. CONDUCTIVITY RATIO, k_H/k_V
	Unit 1: Clay Fill (E)	$4.0x10^{-5}$	4
	Unit 2: Organic Soil	$4.0x10^{-6}$	1
	Unit 3: Clay	$1.0x10^{-6}$	1
	Unit 4: Sand	$5.4x10^{-3}$	9
	Unit 5: Clay	$4.0x10^{-6}$	4
	Unit 6: Sand	$5.4x10^{-3}$	9
	Unit 7: Clay	$4.0x10^{-6}$	4
	Unit 8: Levee Fill (N)	$4.0x10^{-6}$	4

Cache Slough Mitigation Bank
Solano County, California

Soil Parameters for Seepage Analysis

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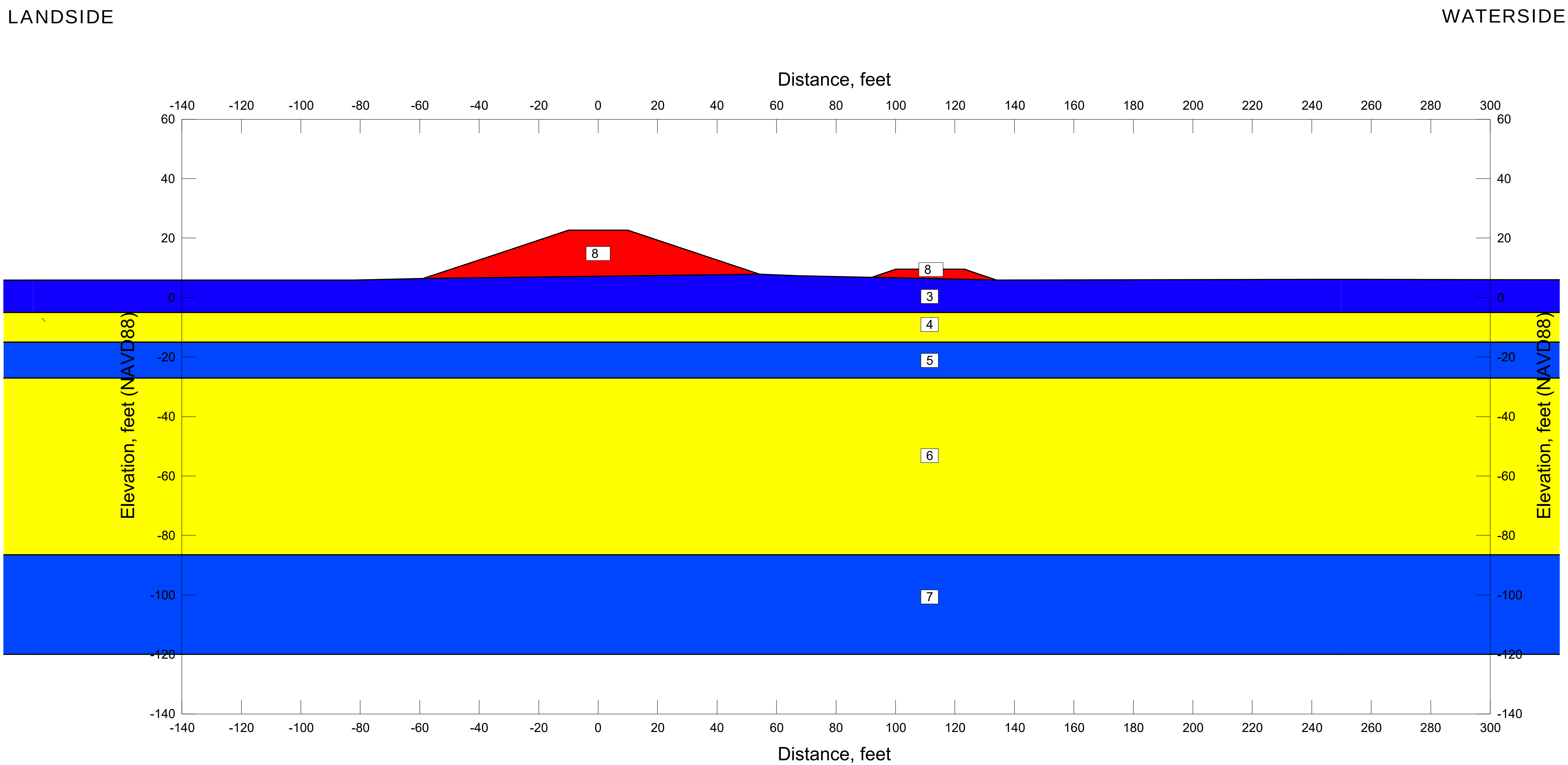
FIG. C-1



Notes:
1. See Figure C-1 for material properties.

Cache Slough Mitigation Bank
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Seepage Analysis Profile
Station 21+32

Steady State Conditions

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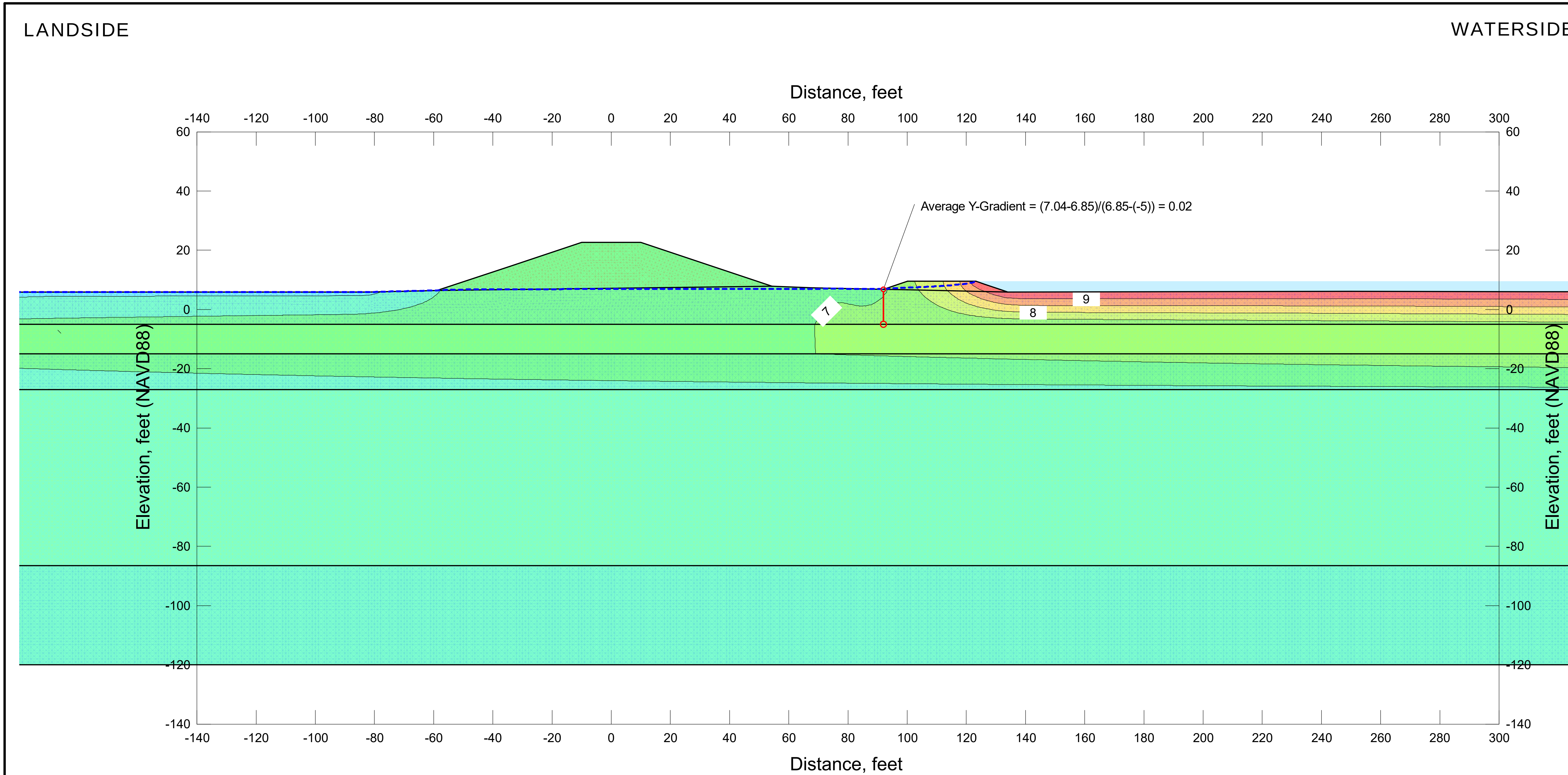
FIG. C-2

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Seepage Model\



Notes:
1. See Figure C-1 for material properties.
2. Contours represent total head in feet of water.

Cache Slough Mitigation Bank
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Seepage Analysis
Station 21+32

Steady State Conditions

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FIG. C-3

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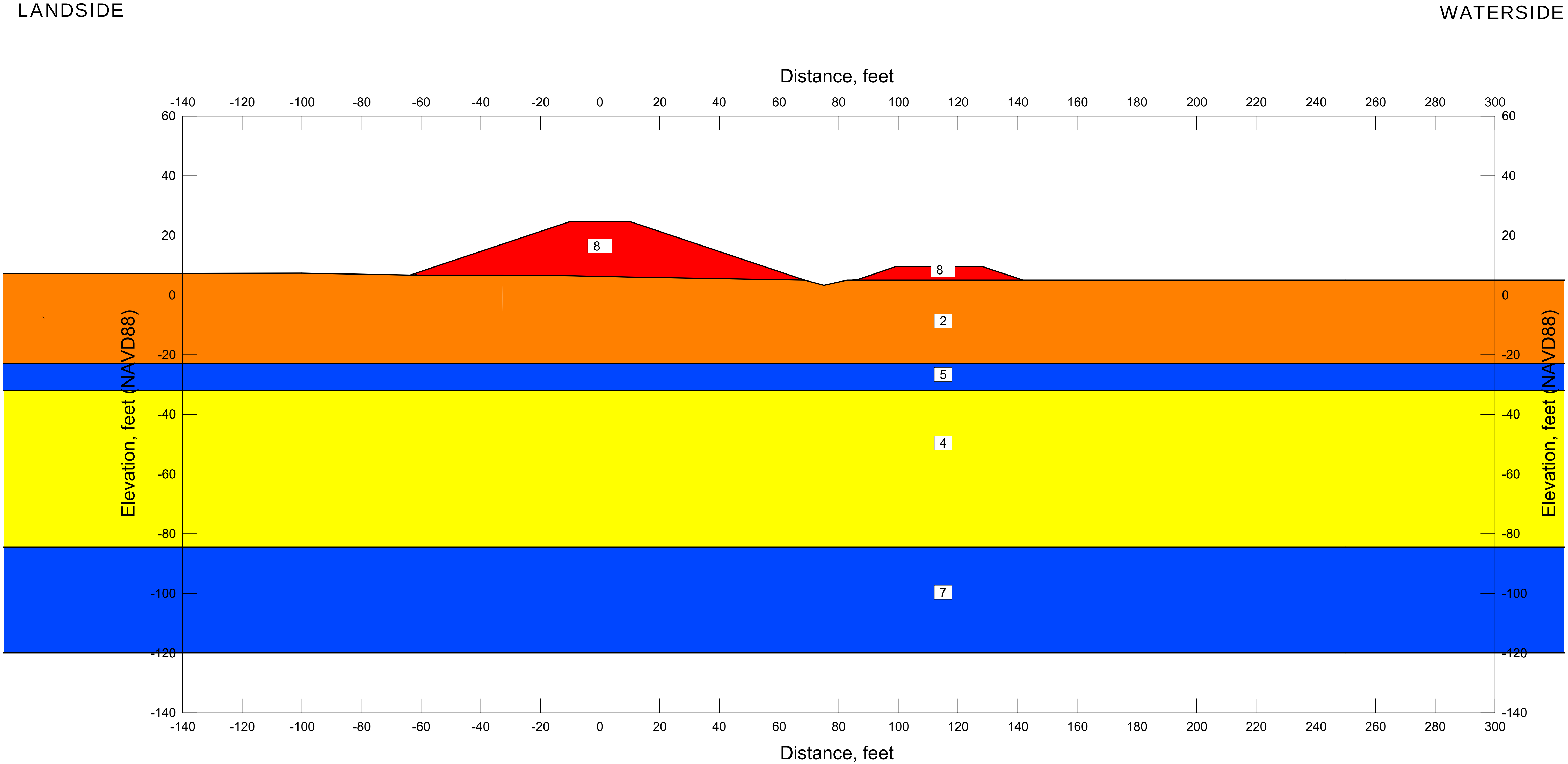

Notes:
1. See Figure C-1 for material properties.

Cache Slough Mitigation Bank
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Seepage Analysis Profile
Station 36+32
Steady State Conditions

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FIG. C-4

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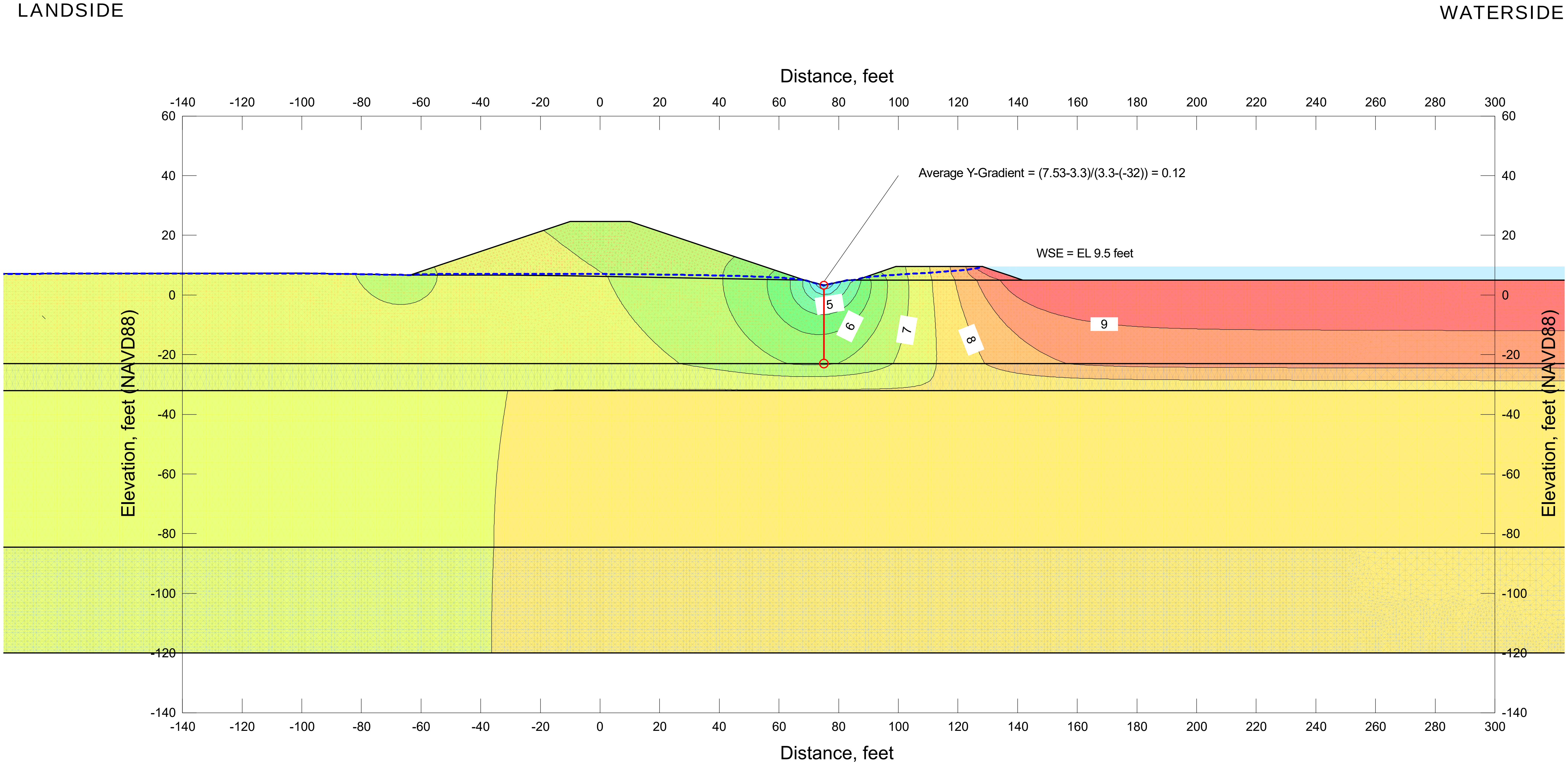

Notes:
1. See Figure C-1 for material properties.
2. Contours represent total head in feet of water.

Cache Slough Mitigation Bank
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Seepage Analysis
Station 36+32
Steady State Conditions

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FIG. C-5

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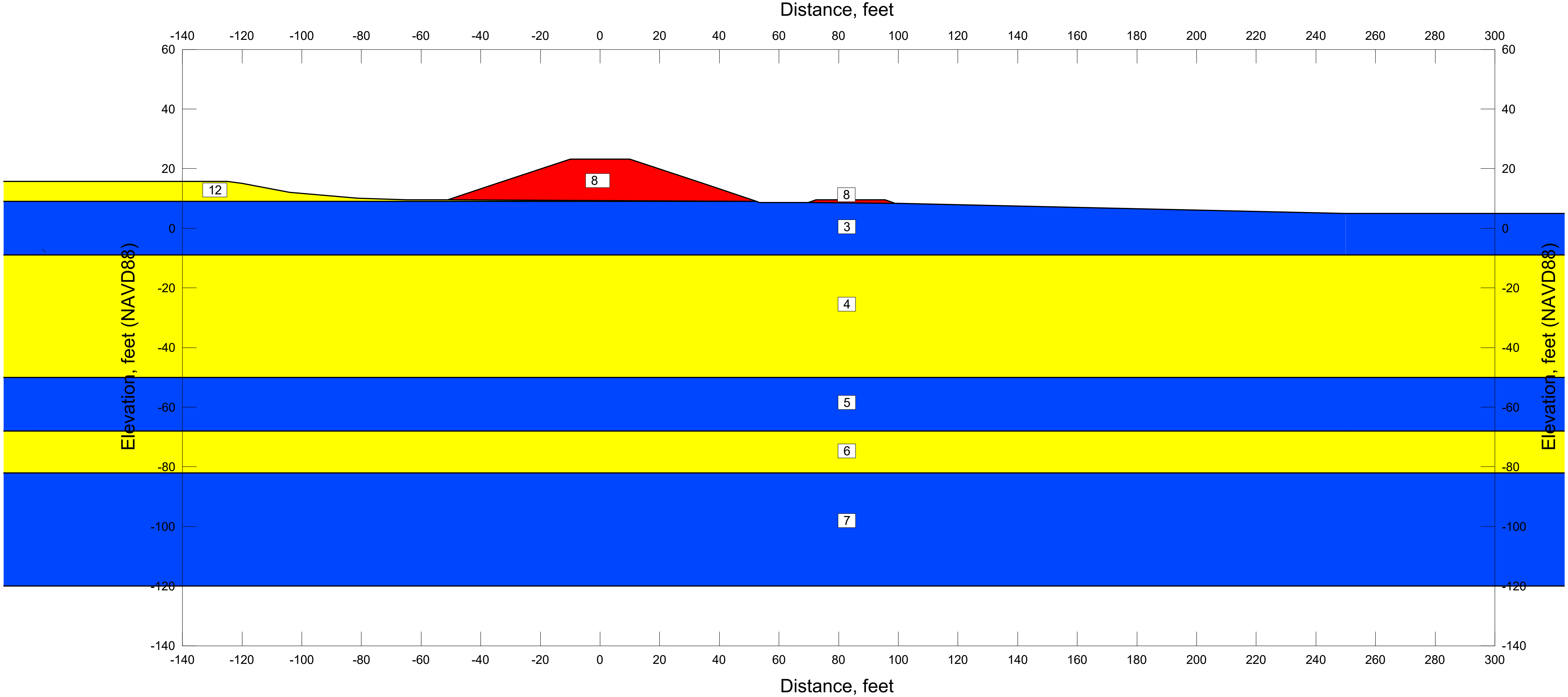
LANDSIDE

WATERSIDE



Notes:
1. See Figure C-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Seepage Analysis Profile
Station 56+32
Steady State Conditions

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FIG. C-6

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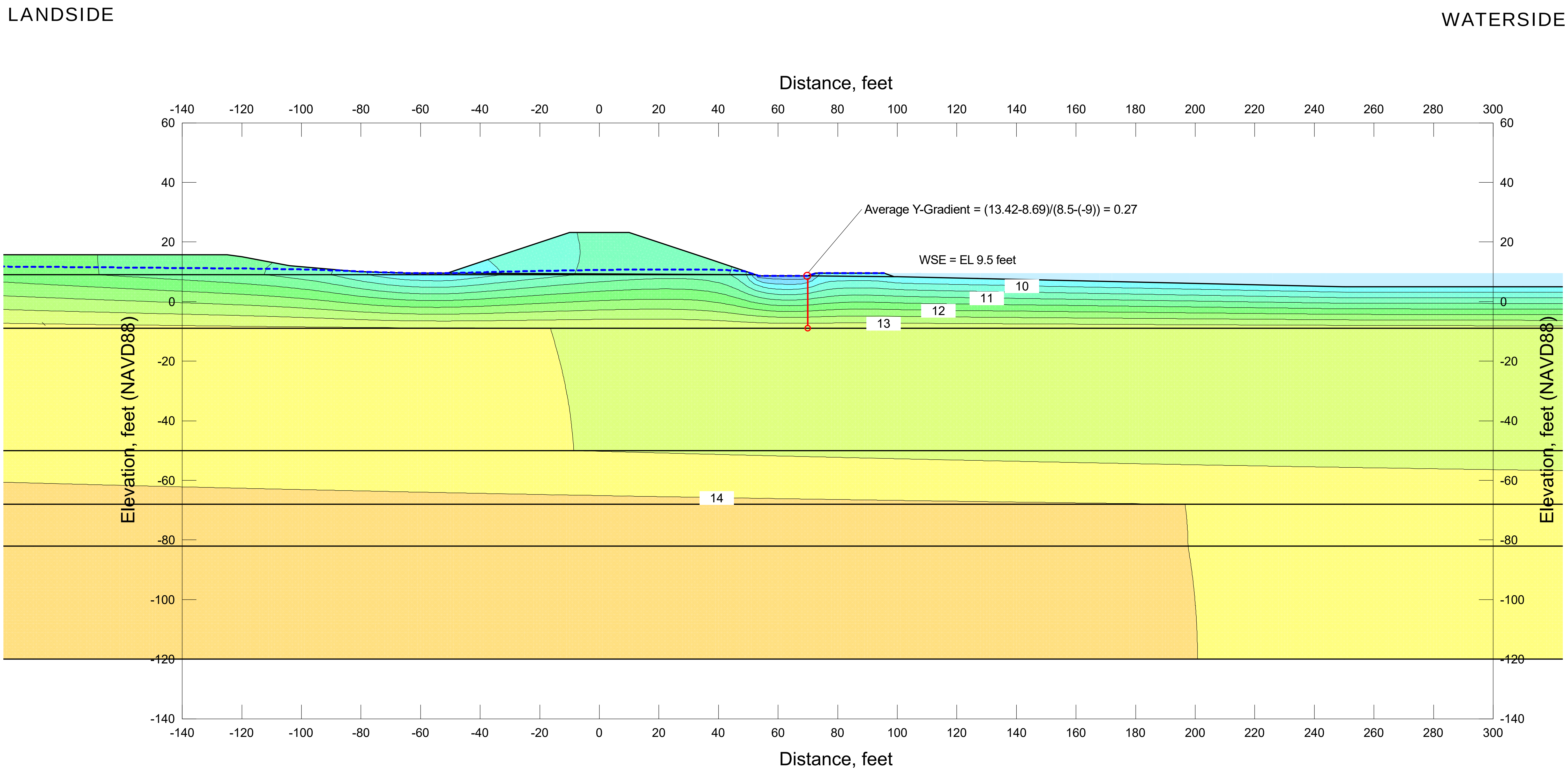

Notes:
1. See Figure C-1 for material properties.
2. Contours represent total head in feet of water.

Cache Slough Mitigation Bank
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Seepage Analysis
Station 56+32
Steady State Conditions

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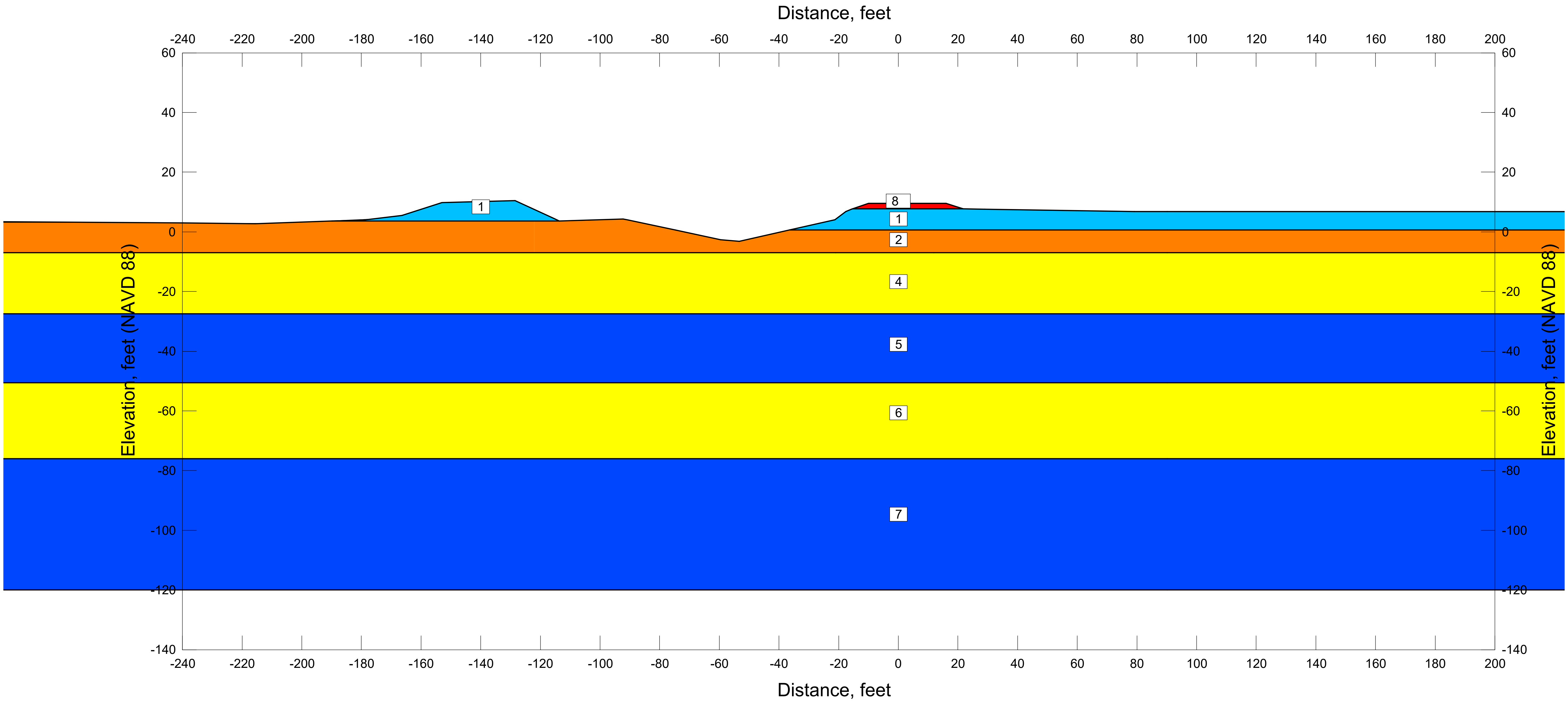
FIG. C-7

LANDSIDE

WATERSIDE



Notes:
1. See Figure C-1 for material properties.

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Solano County, California

Seepage Analysis Profile
Station 90+32
Steady State Conditions

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FIG. C-8

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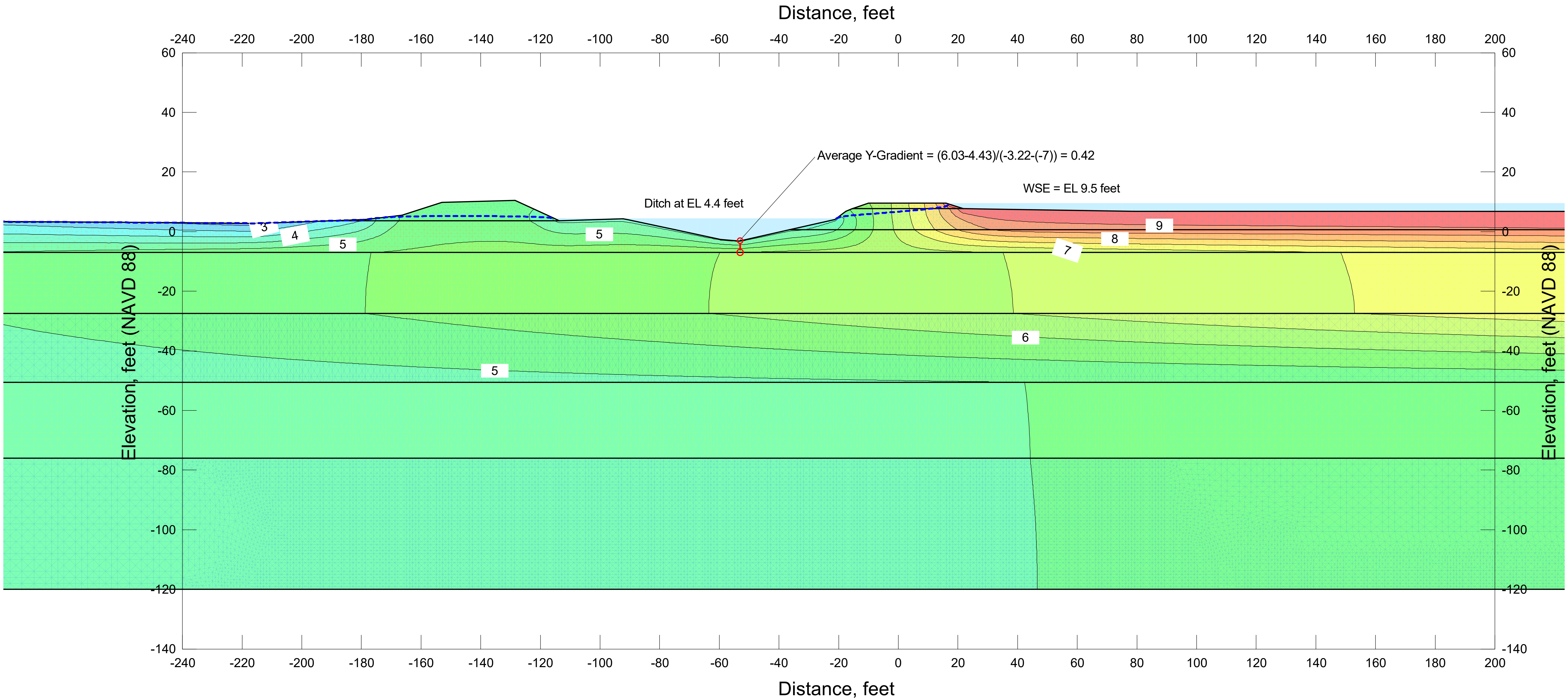

Notes:
1. See Figure C-1 for material properties.
2. Contours represent total head in feet of water.

Cache Slough Mitigation Bank
Solano County, California

Seepage Analysis
Station 90+32
Steady State Conditions

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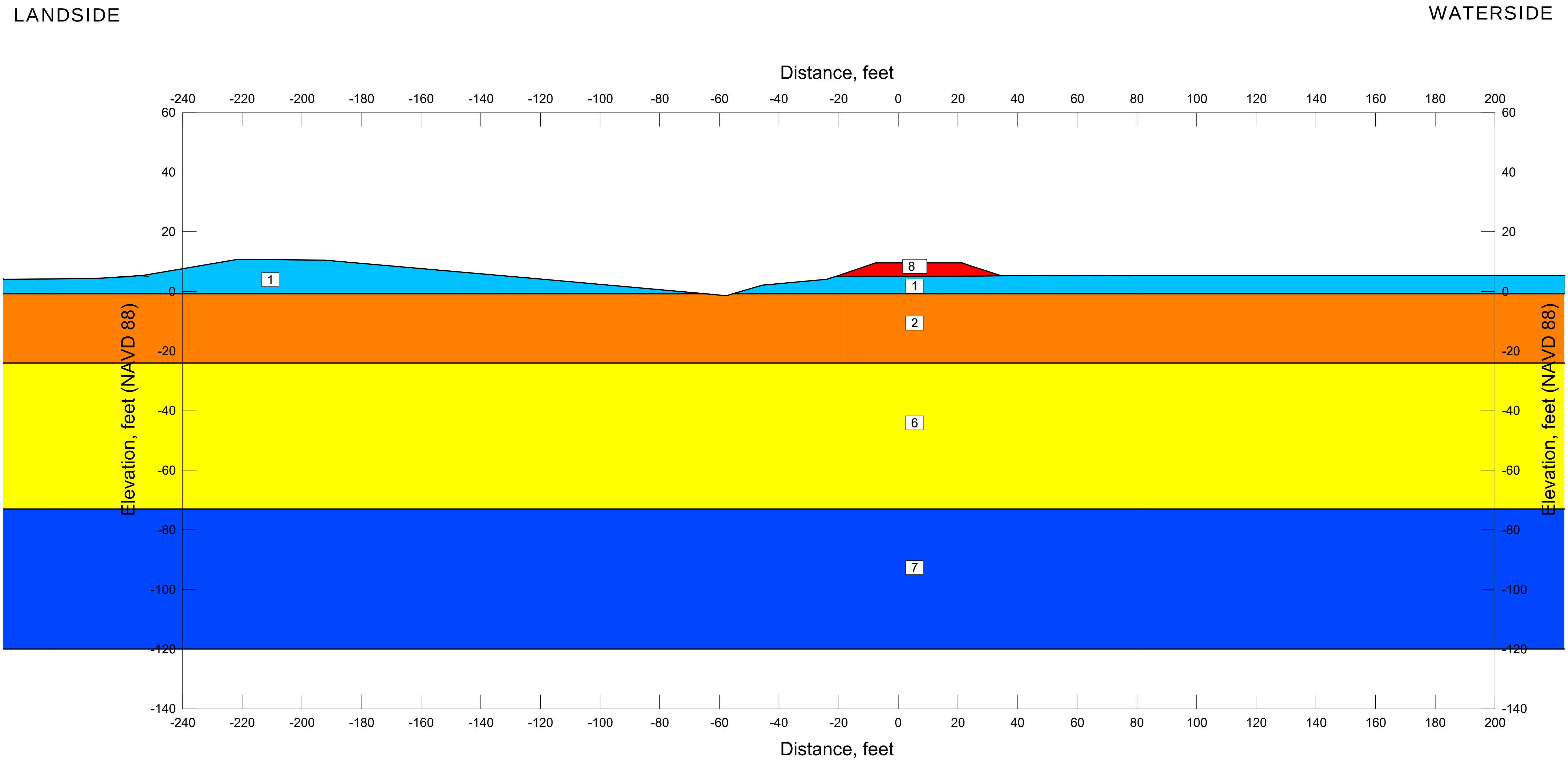
FIG. C-9



Notes:
1. See Figure C-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Seepage Analysis Profile
Station 97+32
Steady State Conditions

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FIG. C-10

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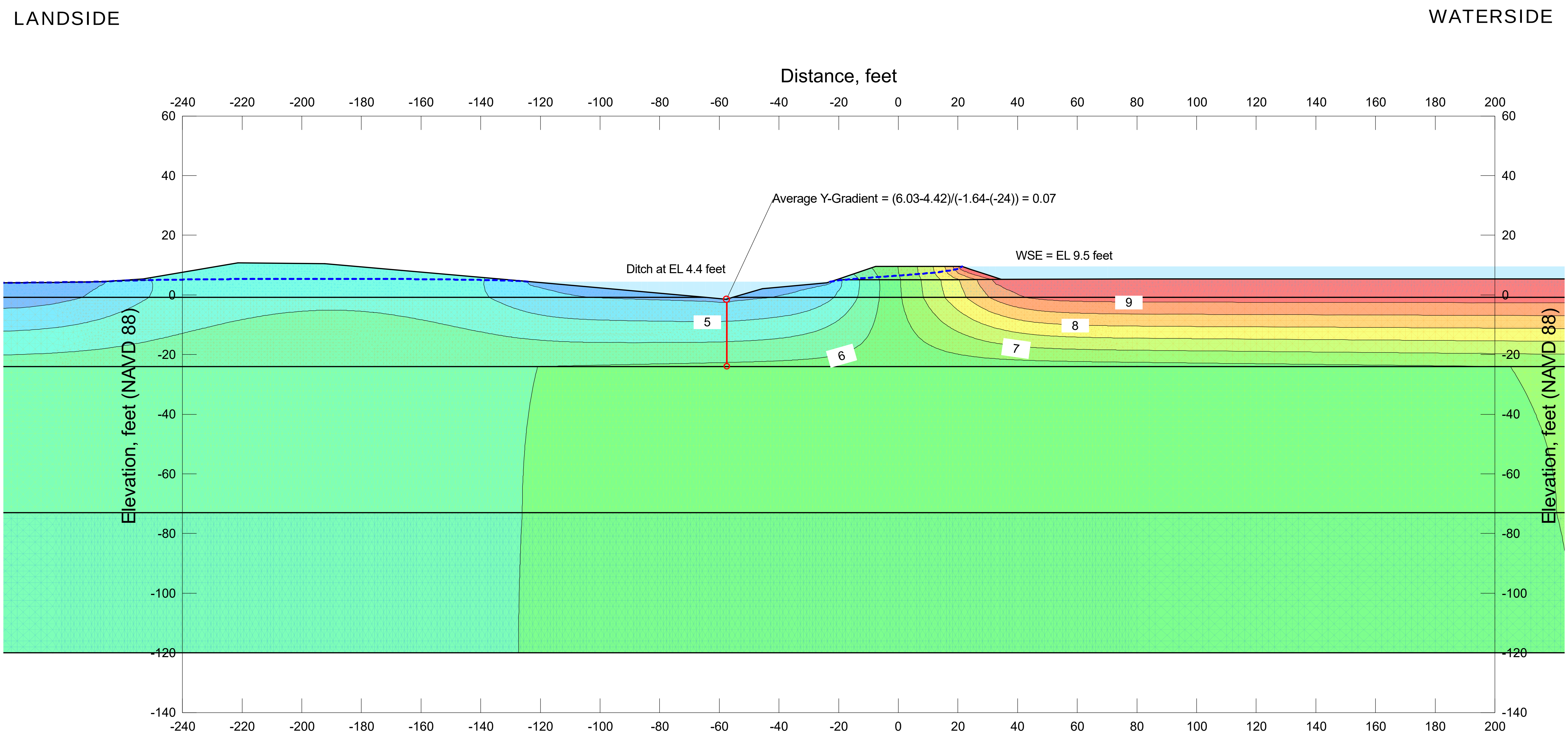

Notes:
1. See Figure C-1 for material properties.
2. Contours represent total head in feet of water.

Cache Slough Mitigation Bank
Solano County, California

Seepage Analysis
Station 97+32
Steady State Conditions

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FIG. C-11

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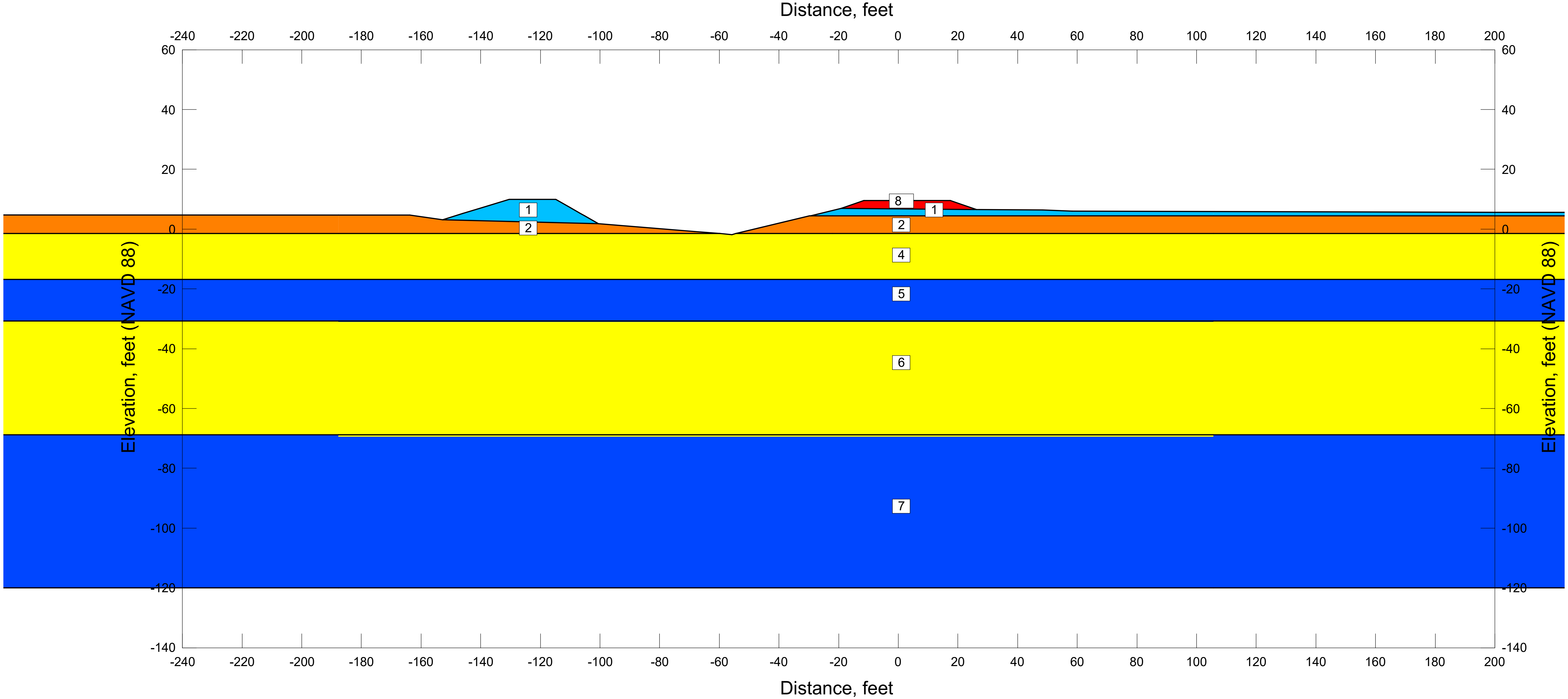
LANDSIDE

WATERSIDE



Notes:
1. See Figure C-1 for material properties.

Cache Slough Mitigation Bank
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Seepage Analysis Profile
Station 103+32
Steady State Conditions

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FIG. C-12

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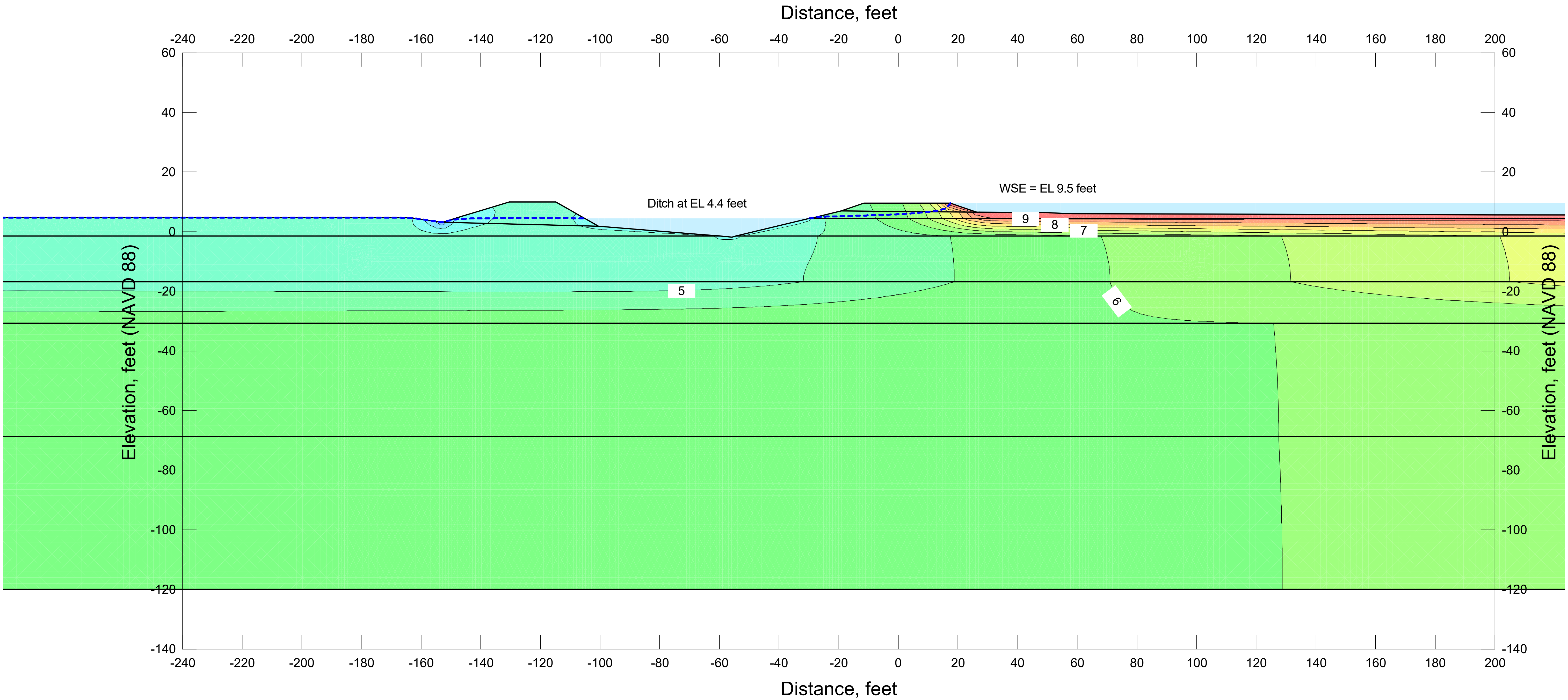
LANDSIDE

WATERSIDE



Notes:
1. See Figure C-1 for material properties.
2. Contours represent total head in feet of water.

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Seepage Analysis
Station 103+32
Steady State Conditions

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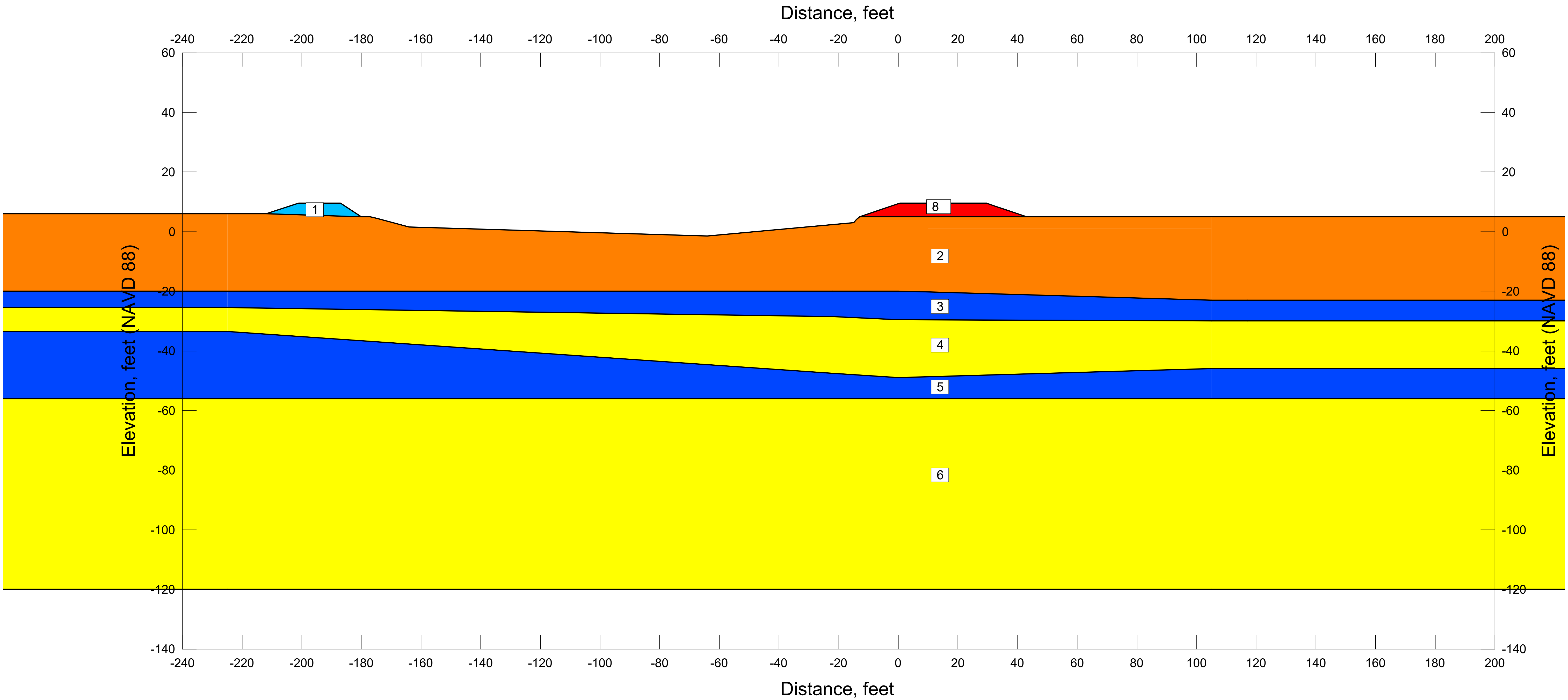
FIG. C-13

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\

LANDSIDE

WATERSIDE



Notes:
1. See Figure C-1 for material properties.

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Seepage Analysis Profile
Station 122+32
Steady State Conditions

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FIG. C-14

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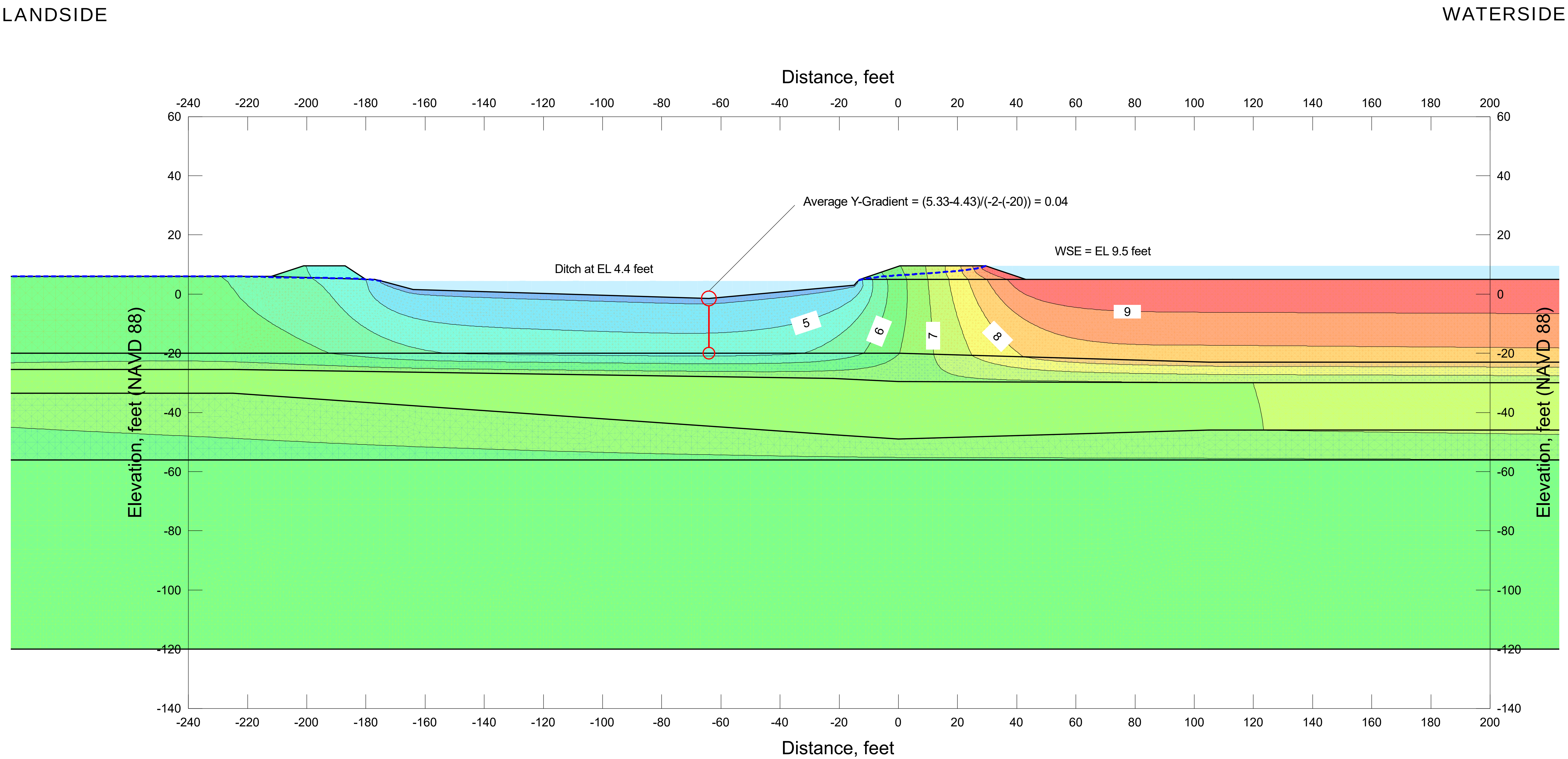

Notes:
1. See Figure C-1 for material properties.
2. Contours represent total head in feet of water.

Cache Slough Mitigation Bank
Solano County, California

Seepage Analysis
Station 122+32
Steady State Conditions

April 2024

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FIG. C-15

APPENDIX D: SLOPE STABILITY ANALYSIS

Appendix D

Slope Stability Analysis

CONTENTS

FIGURES

APPENDIX D: SLOPE STABILITY ANALYSIS

APPENDIX D
Slope Stability Analysis

D.1 General

We performed analysis to evaluate slope stability at the seven locations listed in Section 4.1.3 of this report. The subsurface conditions at these locations are described in the main text and depicted in Appendix B. The details of the analysis and results are presented below.

D.2 Soil Parameters

The parameters used in the stability analysis are presented on Figure D-1. We present a discussion of the selected soil parameters below.

We estimated total unit weights of the units based on laboratory test results.

The shear strength parameters were selected using Figure 5-1 and Tables 5.4 and 5-5 of the Guidance Document.

We estimated the effective stress strengths of the fine-grained, mineral soils (Units 1, 3, 5, and 7) using the boring and CPT data and the laboratory test results. We compared the selected values to the value in Table 5-4 of the Guidance Document (2015).

We estimated the effective stress strengths of the organic soils (Unit 2) using results from Isotropically-Consolidated Undrained Triaxial (TxCU) strength testing from this project site. We compared the values with the limiting effective stress parameters in Table 5-4 of the Guidance Document (2015) for Group 3 foundation soils.

Effective stress strength parameters for the sand units (Units 4, 6, and 12) are based on the $(N_1)_{60}$-values encountered during subsurface exploration. We modified field penetration resistance (blow counts) to $(N_1)_{60}$-values by correcting for sampler size (C_s), hammer energy (C_E), borehole diameter (C_B) rod length (C_R), and overburden pressure (C_N). We used averaged $(N_1)_{60}$-values for each sand unit and estimated the effective friction angle using Hatanaka & Uchida (1996). We assumed that the native sand soil (Units 4 and 6) does not have cohesion.

For the New Levee Fill (Unit 8) we selected strength values based on the recommended strength values presented in Table 5-5 of the Guidance Document (2015).

We used undrained strengths for selecting the strength parameters for the rapid drawdown and end-of-construction analyses. We developed undrained strengths for the fine-grained soil materials (Units 1-3, 5, 7, 8). For the rapid drawdown analysis, we selected undrained strength parameters based on site-specific, TxCU tests for the organic soil (Unit 2) and we used the same values for Units 1, 3, 5, 7, and 8.

For the end-of-construction analysis, we selected strength parameters based on Unconsolidated, Undrained Triaxial (TxUU) laboratory tests as well as pocket penetrometer and torvane strengths measured during exploration. We also used the tip resistances from CPTs to develop strength parameters for the end-of-construction analysis. We modeled the undrained shear strength of the organic soil assuming that the soils were normally consolidated with a ratio of undrained shear strength (S_u) to preconsolidation stress (p) of 0.3 (i.e., S_u/p=0.3). For organic soils outside of existing levee and berm footprints, we modeled a 5-foot-thick soil crust with an undrained shear strength of 1000 psf. Below 5-foot-thick crust, we modeled the organic soils as normally consolidated with the S_u/p ratio described above.

D.3 Analysis Description

We performed analysis to check the factors of safety for the landside and waterside slopes for steady state seepage and rapid drawdown loading conditions before and after the levee rehabilitation. We also analyzed the factor of safety for the landside and waterside slopes for the end-of-construction condition. The stability analysis was performed on the same cross sections that were analyzed for seepage. We used the computer program SLOPE/W and Spencer's method of analysis. We used effective stress strength parameters for analyzing the factors of safety under steady-state seepage conditions. We analyzed the stability of the landslide slope for steady state seepage conditions by importing pore water pressures calculated in the seepage analysis using the waterside water level at Elevation 9.5. We analyzed the stability of the waterside slope for rapid drawdown using the waterside water level at Elevation 9.5. For the rapid drawdown analysis, we used the feature in SLOPE/W that used the Staged Rapid Drawdown Analysis option based on the Duncan et al., 1990 procedure and both the effective stress and undrained strength envelopes. For the end-of-construction analysis we used undrained strength parameters. We modeled a tension crack through the levee fill for the end-of-construction case.

The Guidance Document (2015) provides minimum factors of safety for different loading conditions on the land- and waterside levee slopes, which are presented in Exhibit D-1.

Exhibit D-1: Minimum Factors of Safety for Slope Stability Analysis, from Guidance Document (2015)

Loading Condition	Levee Slope	Minimum Factor of Safety
Steady State Seepage	Landside	1.5
Rapid Drawdown	Waterside	1.2
End of Construction	Landside and Waterside	1.3

D.4 Analysis Results

The results of our slope stability analysis are summarized in Exhibits D-1 and D-2 and presented on Figures D-2 through D-29. The factors of safety meet the required factor of safety criteria described in Guidance Document.

Exhibit D-2: Factors of Safety for Landside Slopes

Station	Steady State Seepage Effective Strength	End of Construction Undrained Strength
21+32	4.49	Greater than 10
36+32	2.46	4.14
56+32	7.63	Greater than 10
90+32	2.15	3.38
97+32	2.71	3.39
103+32	2.58	3.30
122+32	2.39	3.30

Exhibit D-3: Factors of Safety for Waterside Slopes

Station	Rapid Drawdown Undrained and Effective Strength	End of Construction Undrained Strength
21+32	3.55	Greater than 10
36+32	2.72	4.66
56+32	7.33	Greater than 10
90+32	5.35	Greater than 10
97+32	2.92	5.39
103+32	3.82	6.66
122+32	2.83	4.33

D.5 Pseudo-Static Loading and Seismic Deformation

We performed a pseudo-static slope stability analysis for the landside and waterside slopes. We used the feature in SLOPE/W that used the Staged Pseudo-Static Analysis option based on the Duncan et al., 1990 procedure and both the effective stress and undrained strength envelopes. The pseudo-static analysis applies a horizontal force at the center of gravity to model an earthquake force. The yield coefficient is the value of the force resulting in a factor of safety of 1.0. The analysis assumes that materials do not lose strength during earthquake shaking. Exhibit D-4 presents the yield coefficients (K_y). The pseudo-static slope stability analysis results are present on Figures D-30 through D-43.

Exhibit D-4: Yield Coefficient for Pseudo-Static Loading

Station	Landside	Waterside
21+32	0.5	0.45
36+32	0.4	0.26
56+32	0.7	0.37
90+32	0.29	0.39
97+32	0.28	0.27
103+32	0.31	0.32
122+32	0.26	0.27

We analyzed seismic deformation using the simplified procedure presented in the Guidance Document (2015). The analysis is based on an earthquake with a 200-year return period and moment magnitude of 7.0. We estimate that peak horizontal acceleration (PHA) along these levee reaches is 0.23g. Deformations can be estimated based on the ratio of the yield acceleration (K_y) to the maximum seismic coefficient (K_{max}). We estimate that K_{max} is 0.26, based on site location and levee geometry for landside and waterside slopes. The K_y/K_{max} ratio is greater than 0.5 for the landside and waterside slopes for all stations analyzed. If the K_y/K_{max} ratio is greater than 0.5 then "minimal or negligible seismic displacements are anticipated" (URS Guidance Document, 2015). The results of the analysis indicate that the slopes will experience minor slope deformation of likely less than 0.1 ft of deformation under the design level earthquake.

STABILITY MODEL MATERIAL PROPERTIES									
UNIT NO.	LAYER COLOR	MATERIAL TYPE	UNIT WEIGHT (pcf)	EFFECTIVE STRENGTH		UNDRAINED STRENGTH		END OF CONSTRUCTION	
				COHESION (psf)	FRICTION ANGLE (degrees)	COHESION (psf)	FRICTION ANGLE (degrees)	COHESION (psf)	FRICTION ANGLE (degrees)
1		Clay Fill (E)	100	100	32	140	19	1,000	0
2		Organic Soil	95	100	32	140	19	See Note Below	0
3		Clay	125	100	32	140	19	2,500	0
4		Sand	125	0	36	--	--	--	--
5		Clay	125	100	32	140	19	2,500	0
6		Sand	125	0	40	--	--	--	--
7		Clay	125	100	32	140	19	2,500	0
8		Levee Fill (N)	125	100	32	140	19	1,800	--

Note: For the End of Construction case, we modeled the undrained shear strength of the organic soil assuming that the soils were normally consolidated with a ratio of undrained shear strength (S_u) to preconsolidation stress (p) of 0.3 (i.e., S_u/p=0.3). For organic soils outside of the levee footprint, we modeled a 5-foot thick soil crust with an undrained shear strength of 1,000 psf. Below the 5-foot thick crust, we modeled the organic soils as normally consolidate with the S_u/p ratio described above.

Cache Slough Mitigation Bank
Solano County, California

Soil Parameters for Slope Stability Analysis

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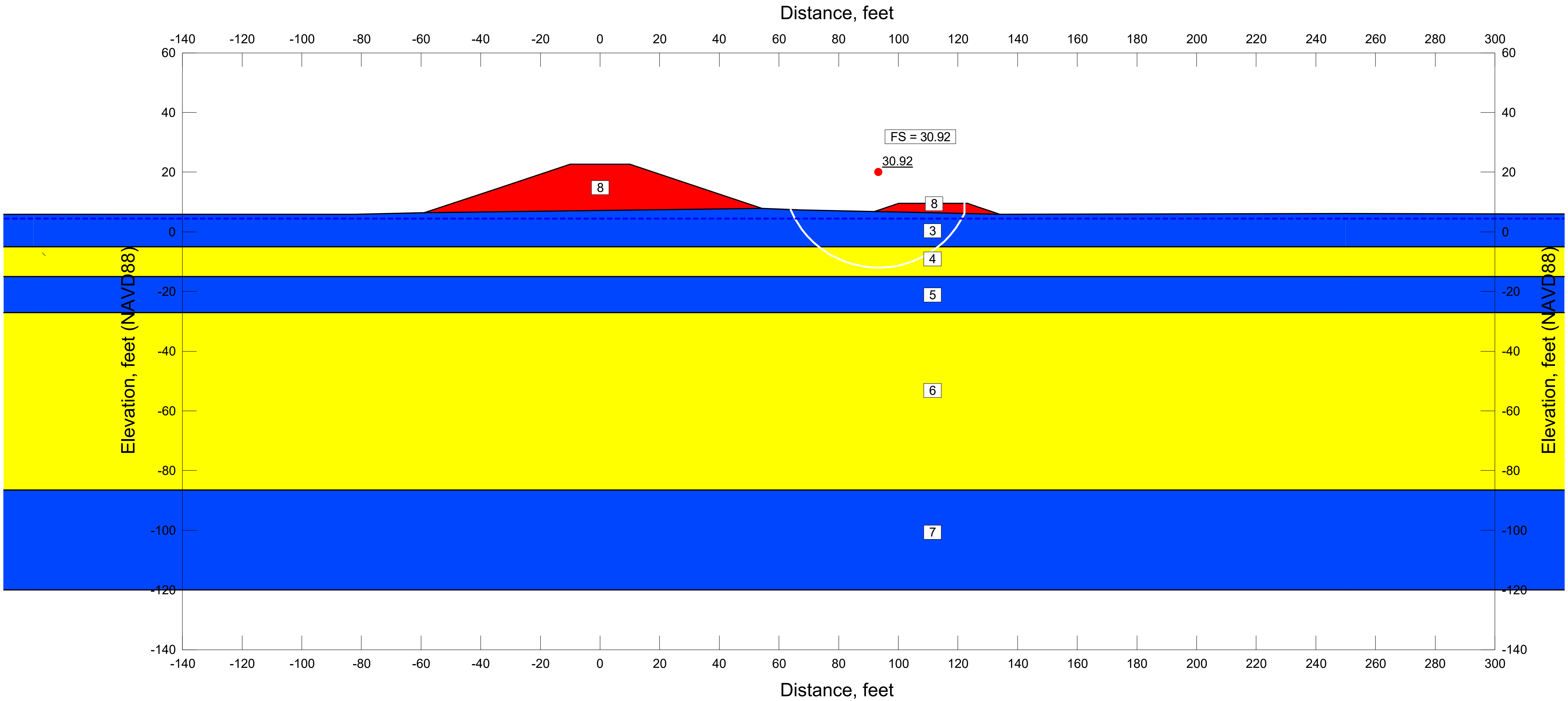
FIG. D-1

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 21+32

End-of-Construction - Landside

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FIG. D-2

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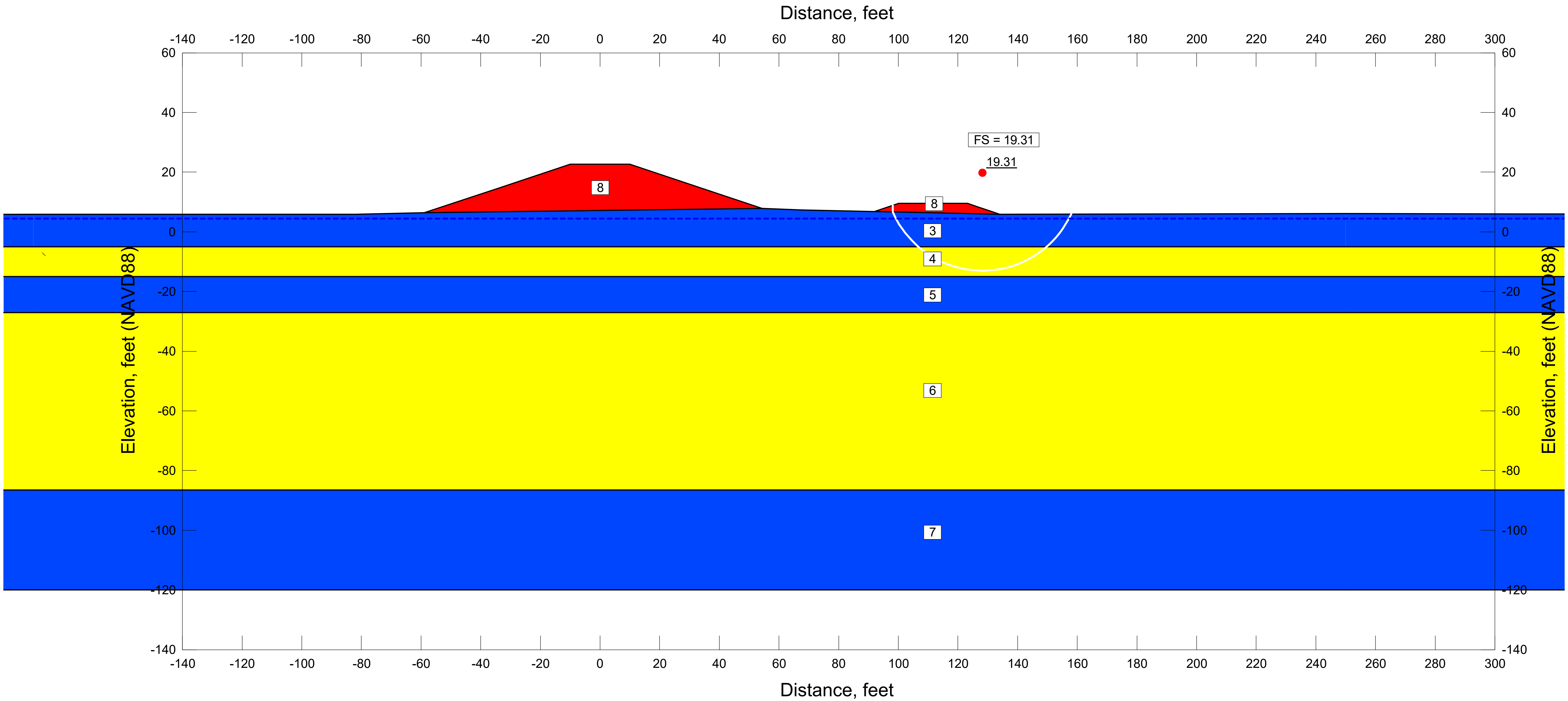
LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 21+32
End-of-Construction - Waterside

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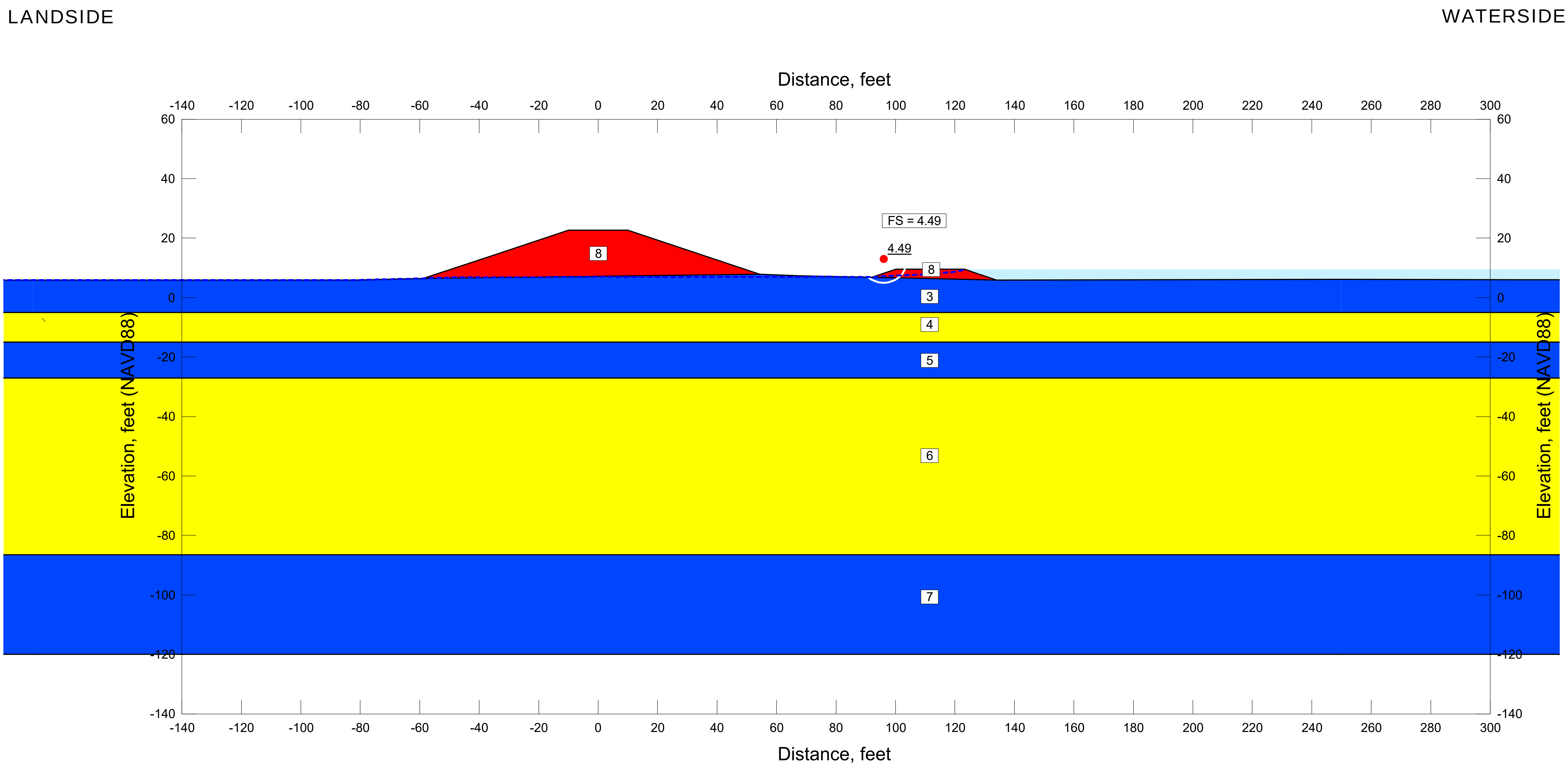
FIG. D-3



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 21+32
Long-Term Steady State

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FIG. D-4

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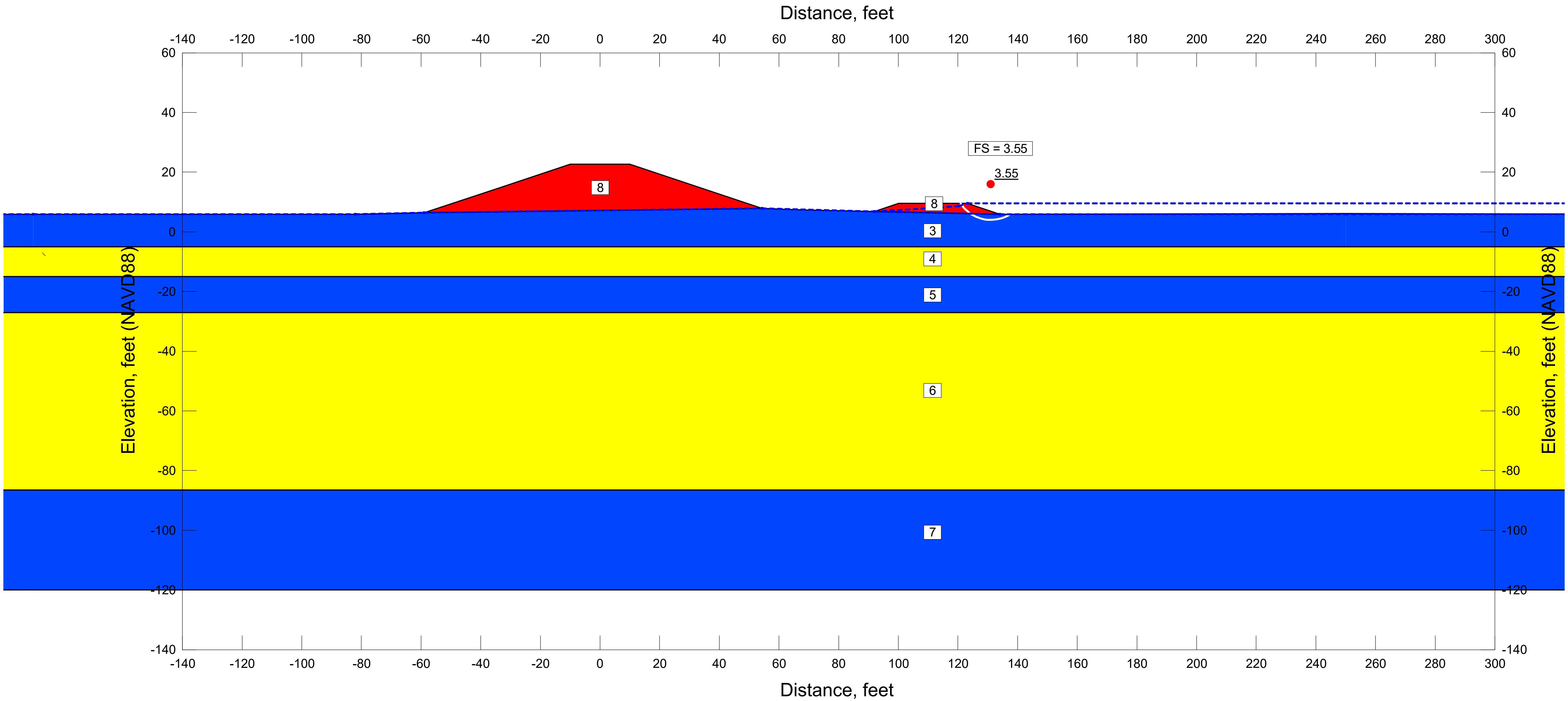

Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 21+32
Rapid Drawdown

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FIG. D-5

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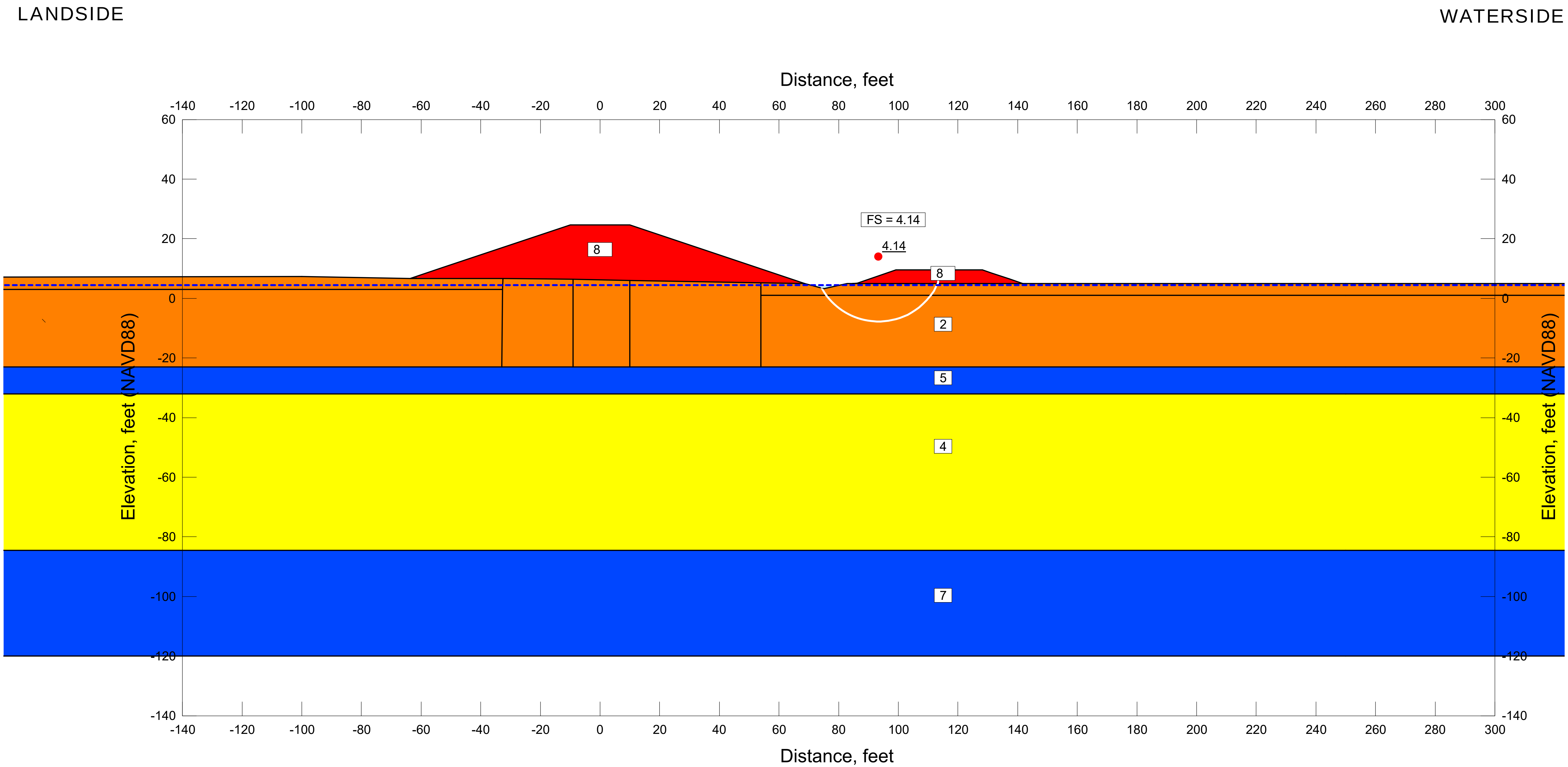

Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 36+32
End-of-Construction - Landside

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FIG. D-6

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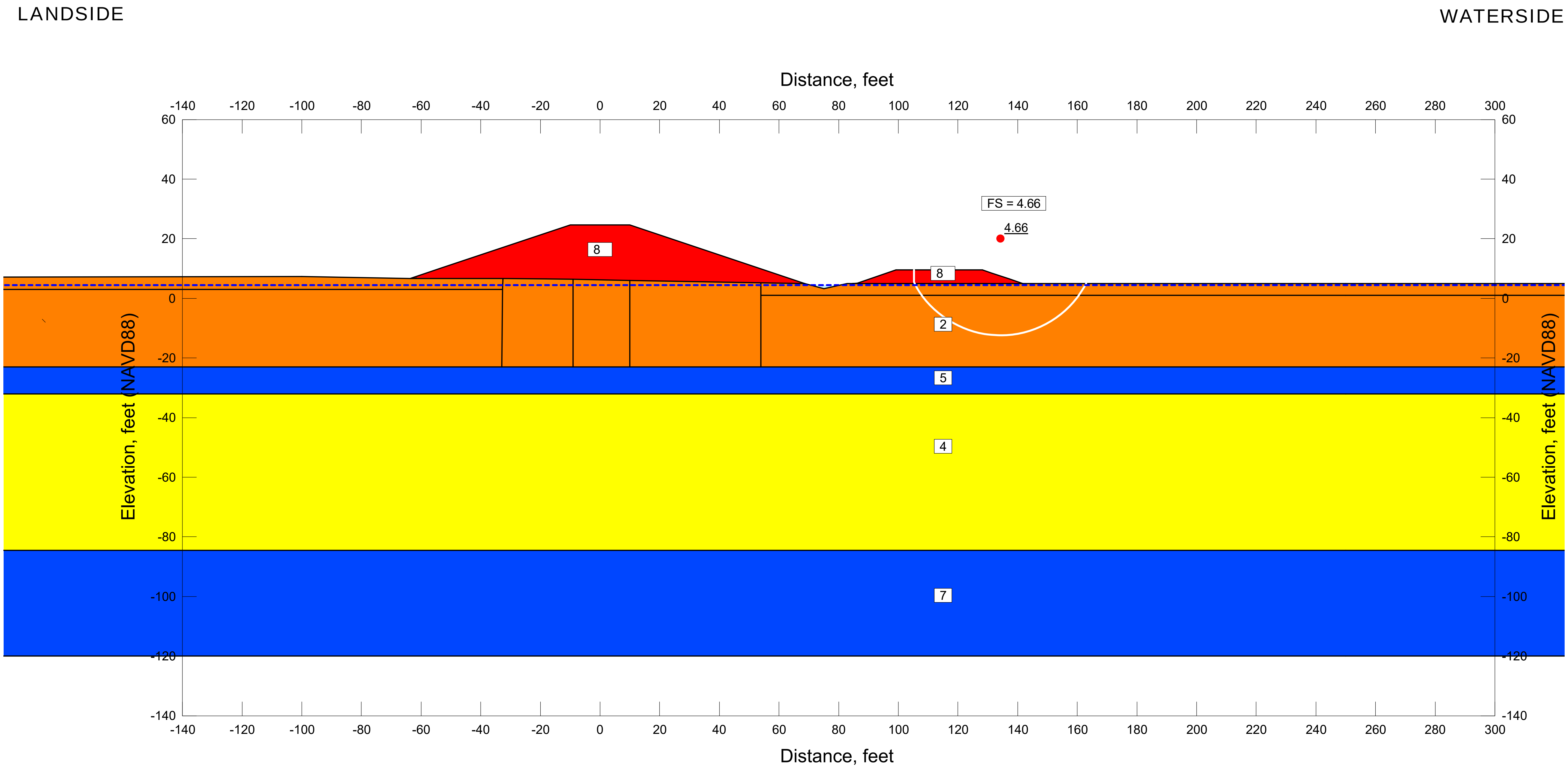

Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 36+32
End-of-Construction - Waterside

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FIG. D-7

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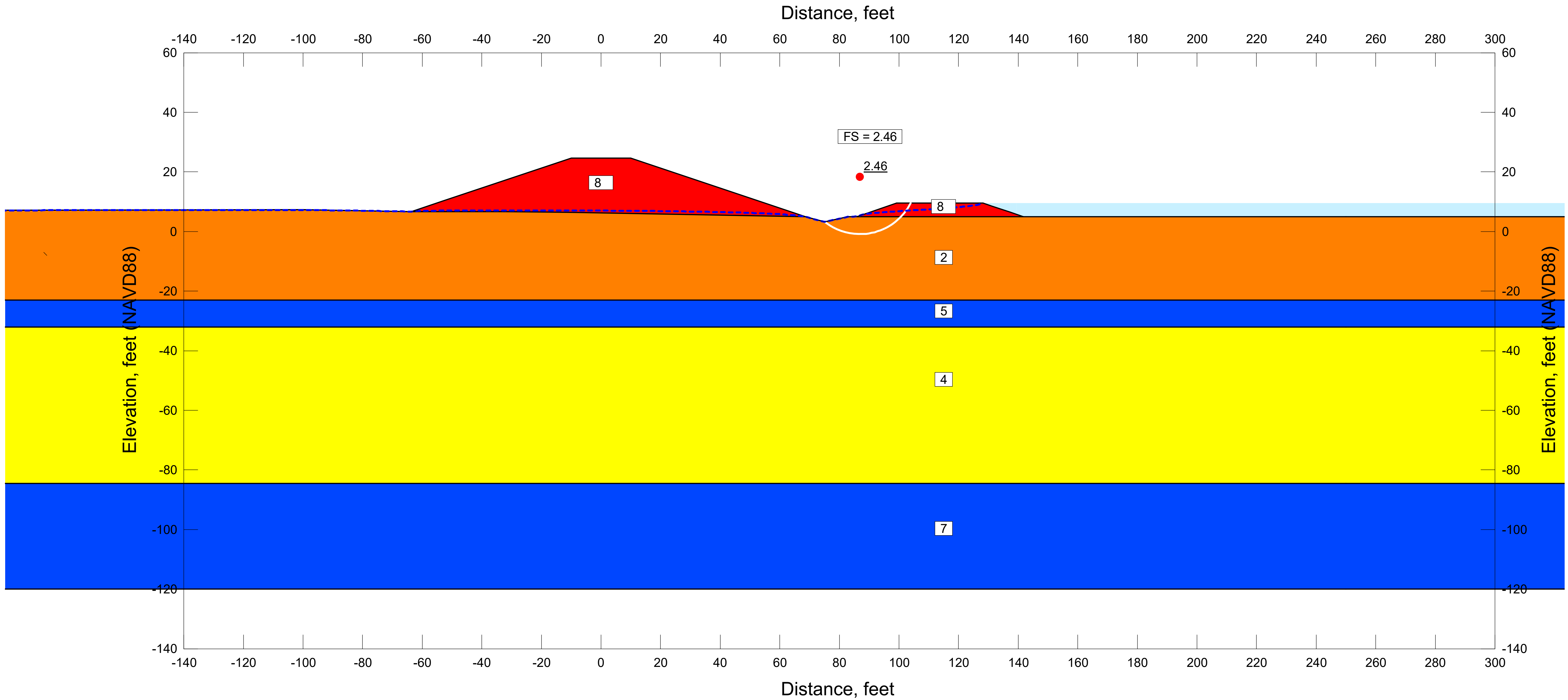
LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 36+32
Long-Term Steady State

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FIG. D-8

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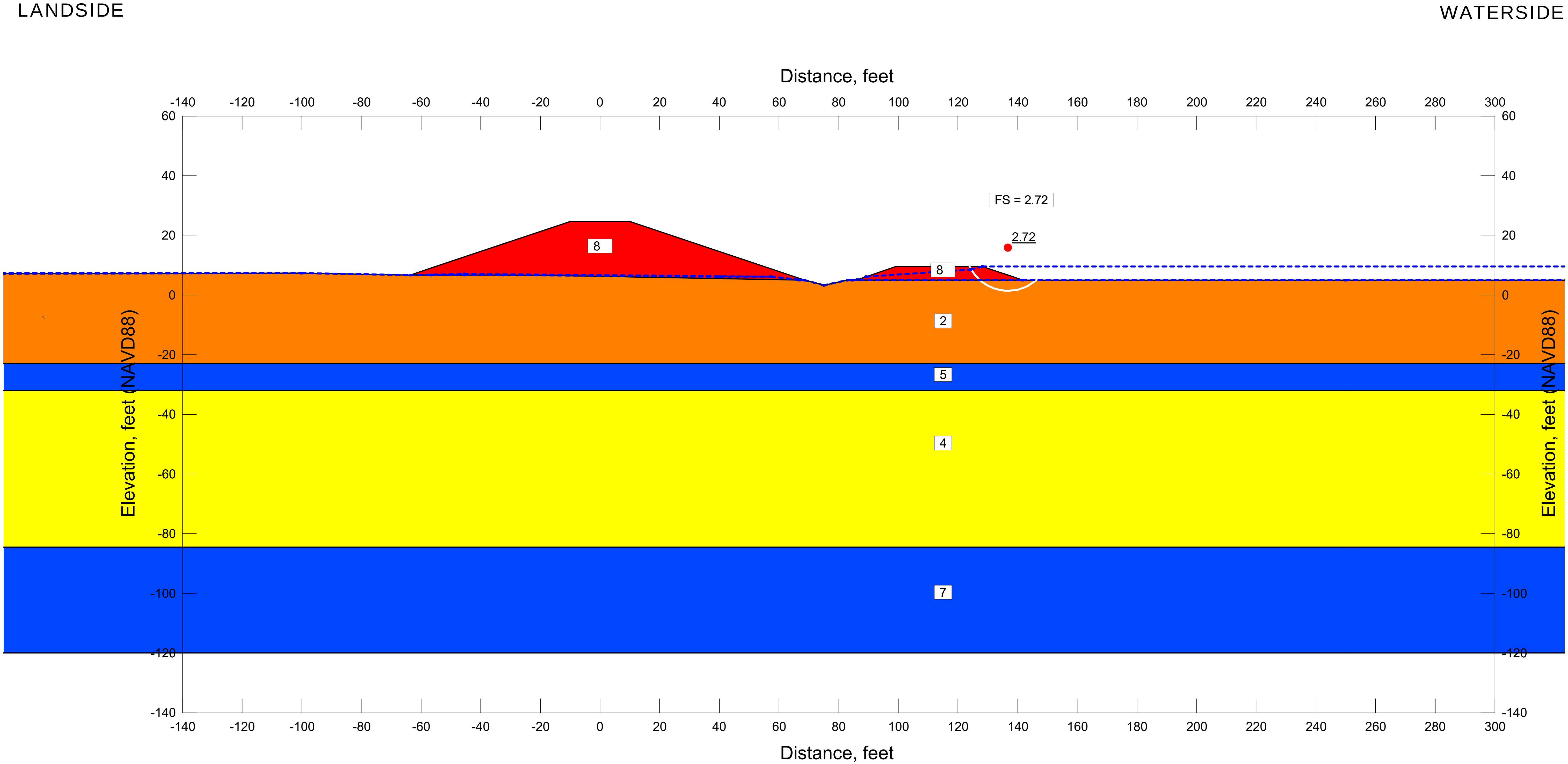

Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 36+32
Rapid Drawdown

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FIG. D-9

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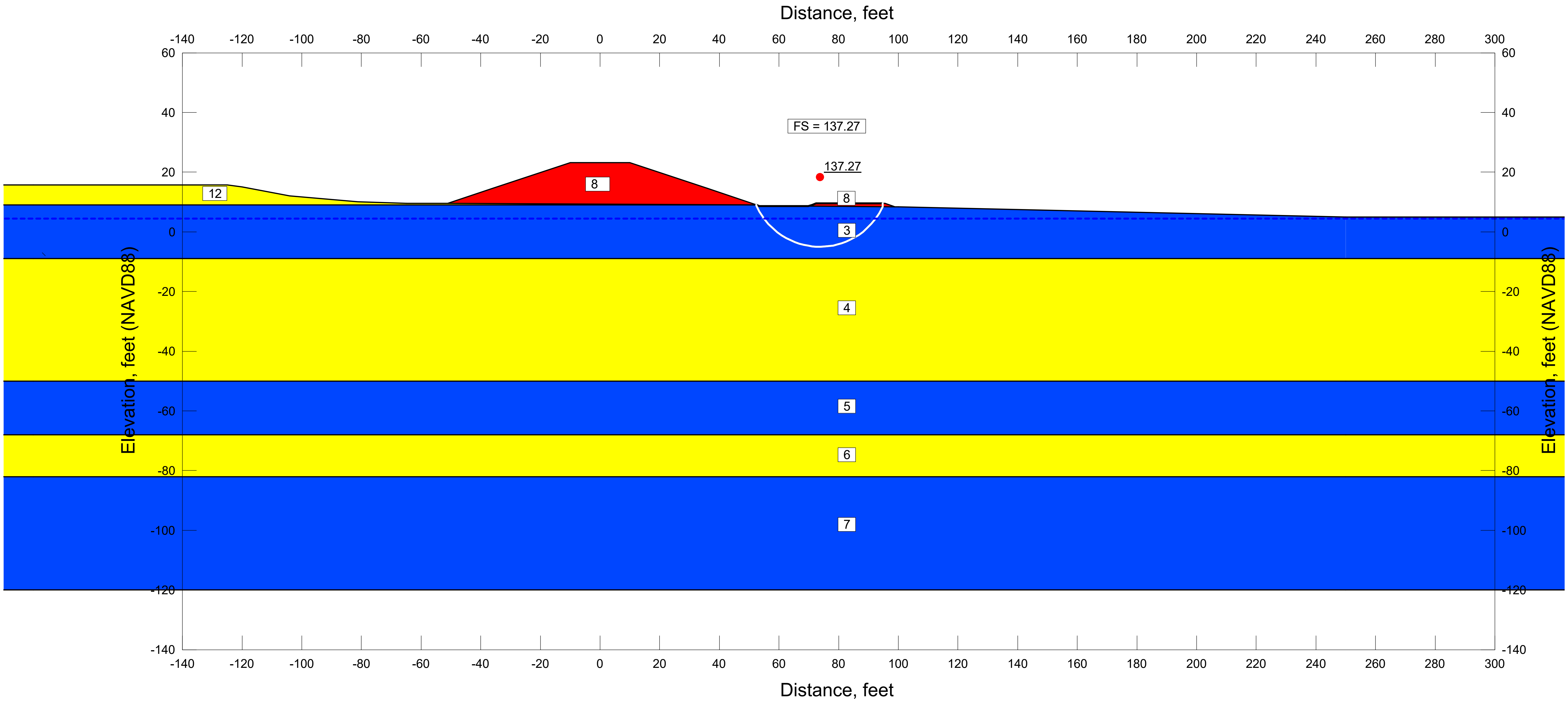
LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 56+32
End-of-Construction - Landside

April 2024

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FIG. D-10

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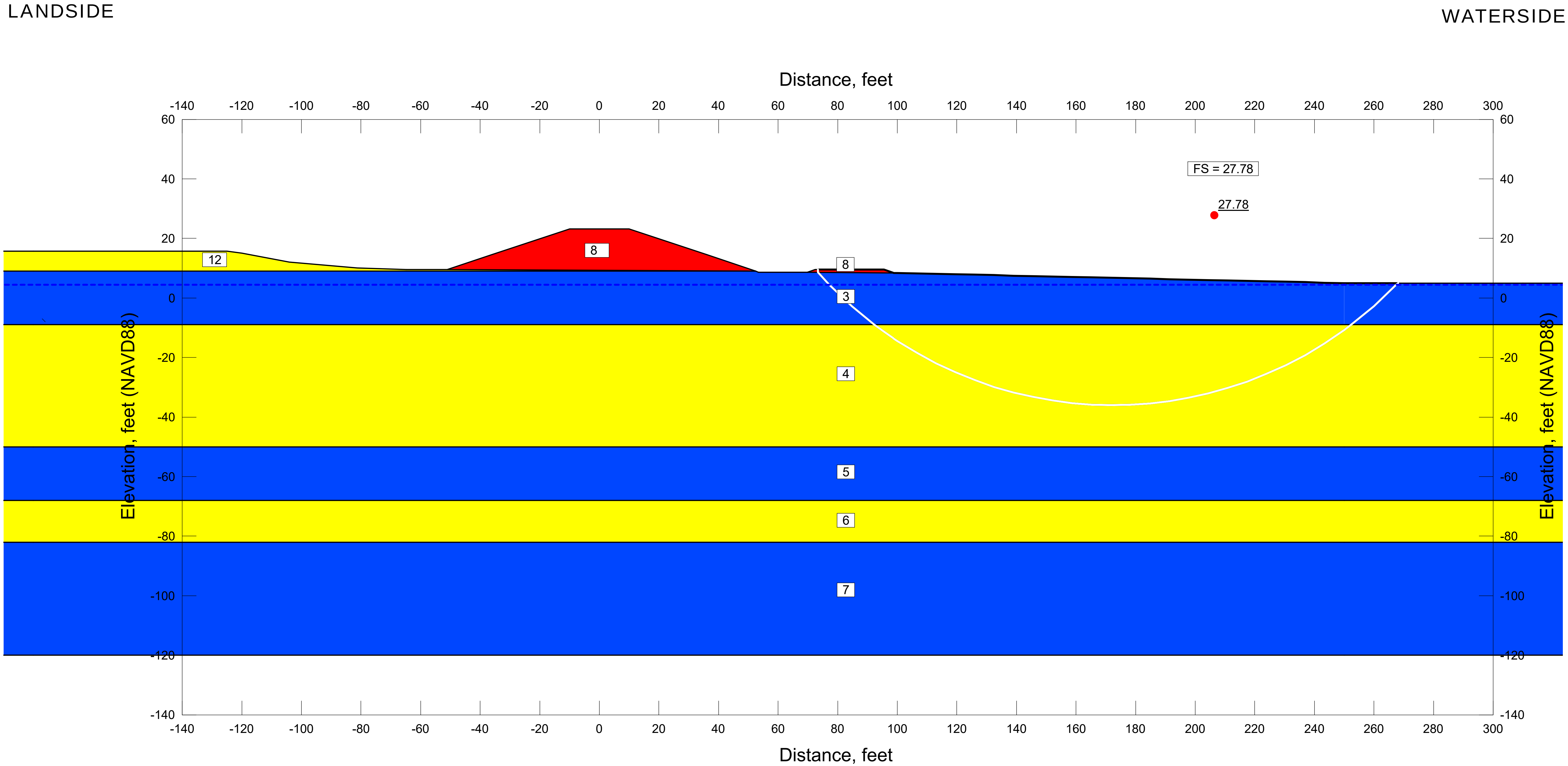

Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 56+32
End-of-Construction - Waterside

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FIG. D-11

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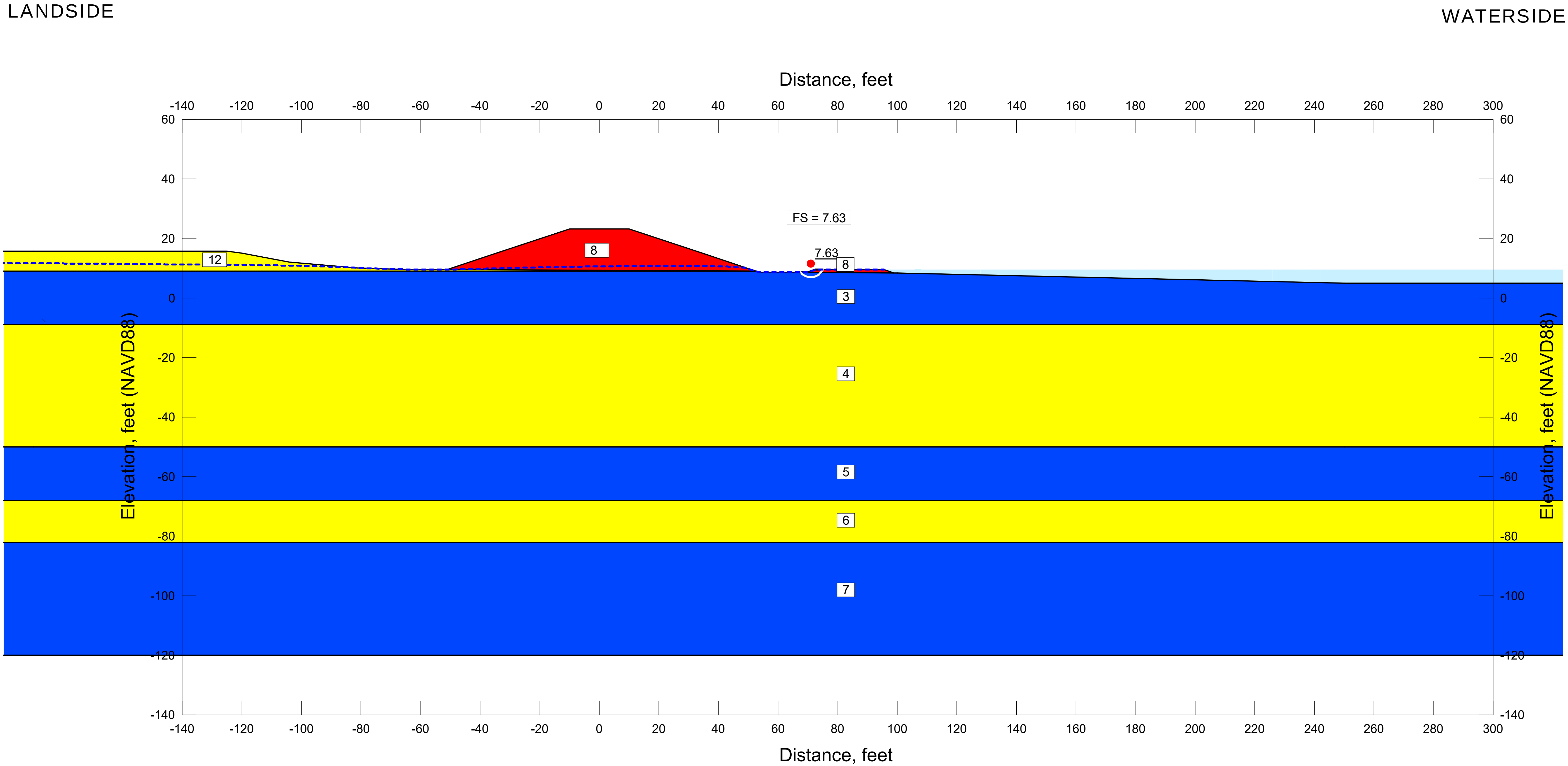

Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 56+32
Long-Term Steady State

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FIG. D-12

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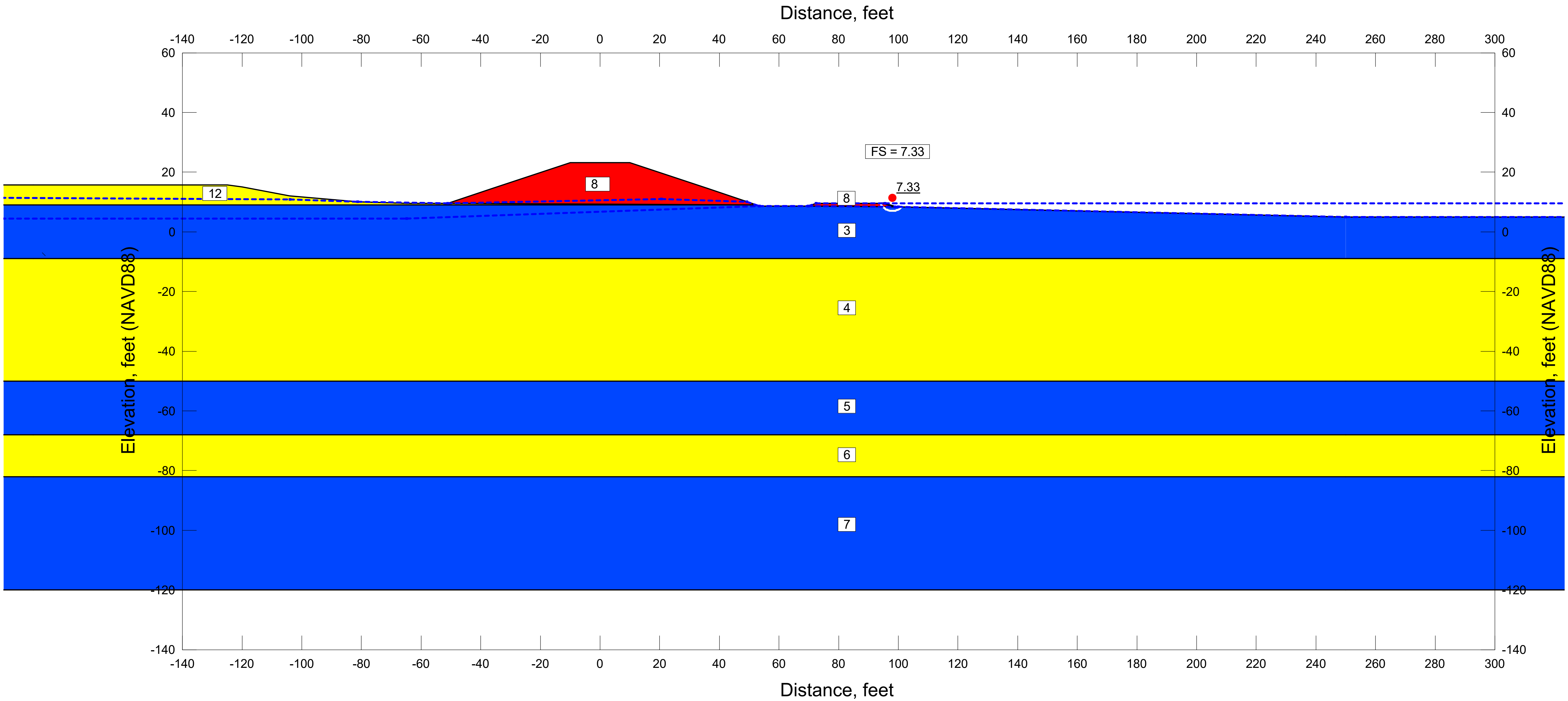
LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 56+32
Rapid Drawdown

April 2024

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FIG. D-13

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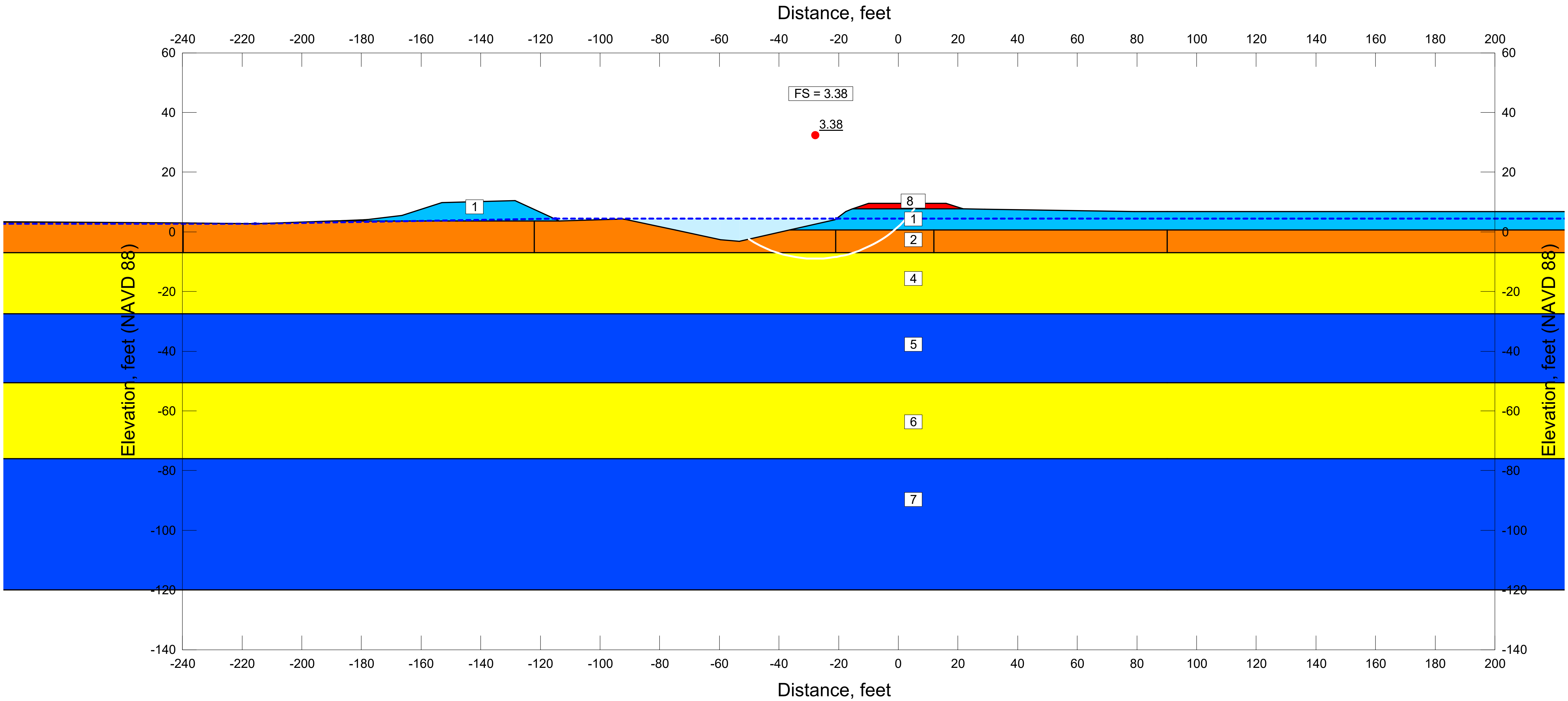
LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 90+32
End-of-Construction - Landside

April 2024

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FIG. D-14

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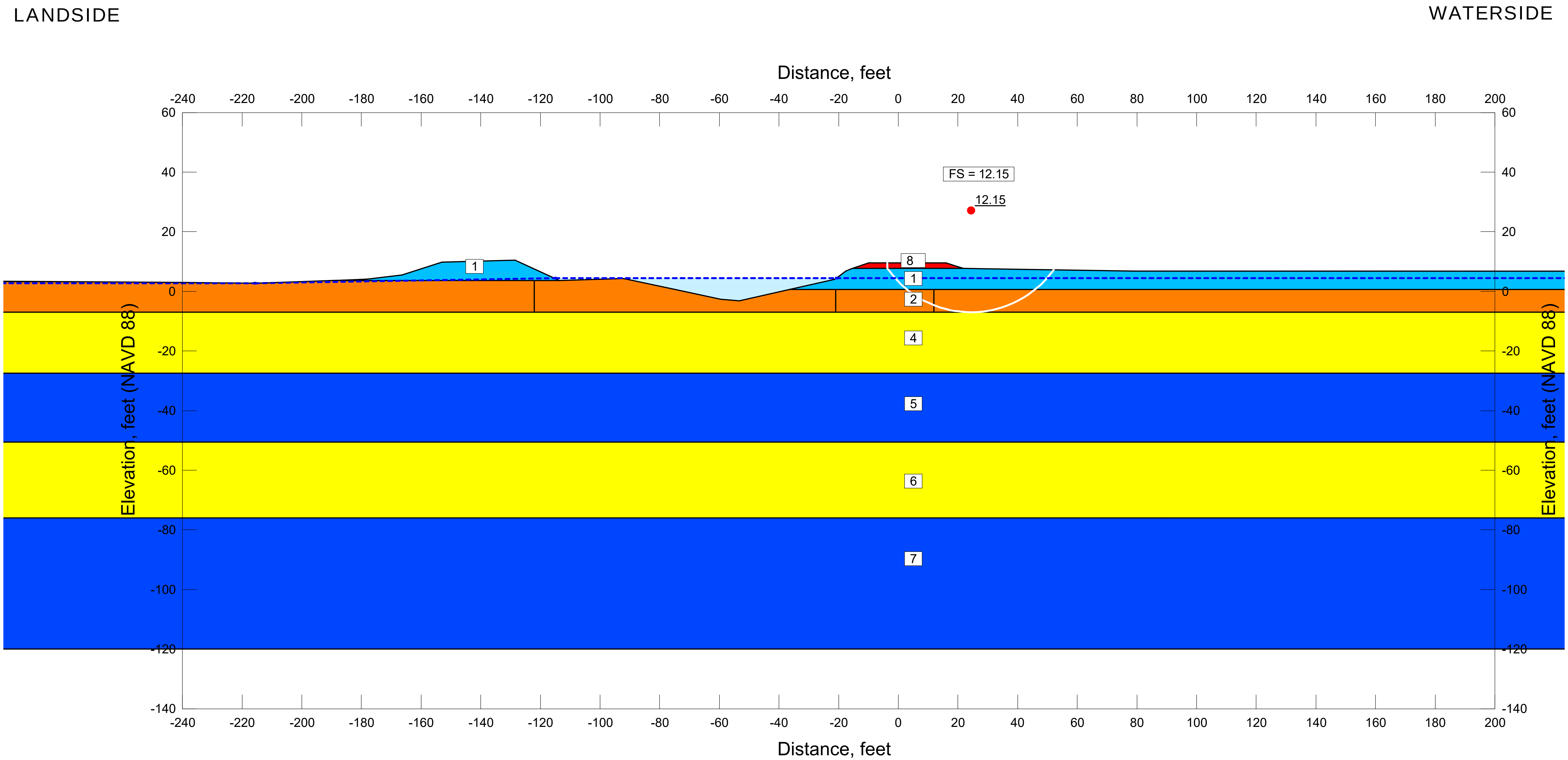

Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 90+32
End-of-Construction - Waterside

April 2024

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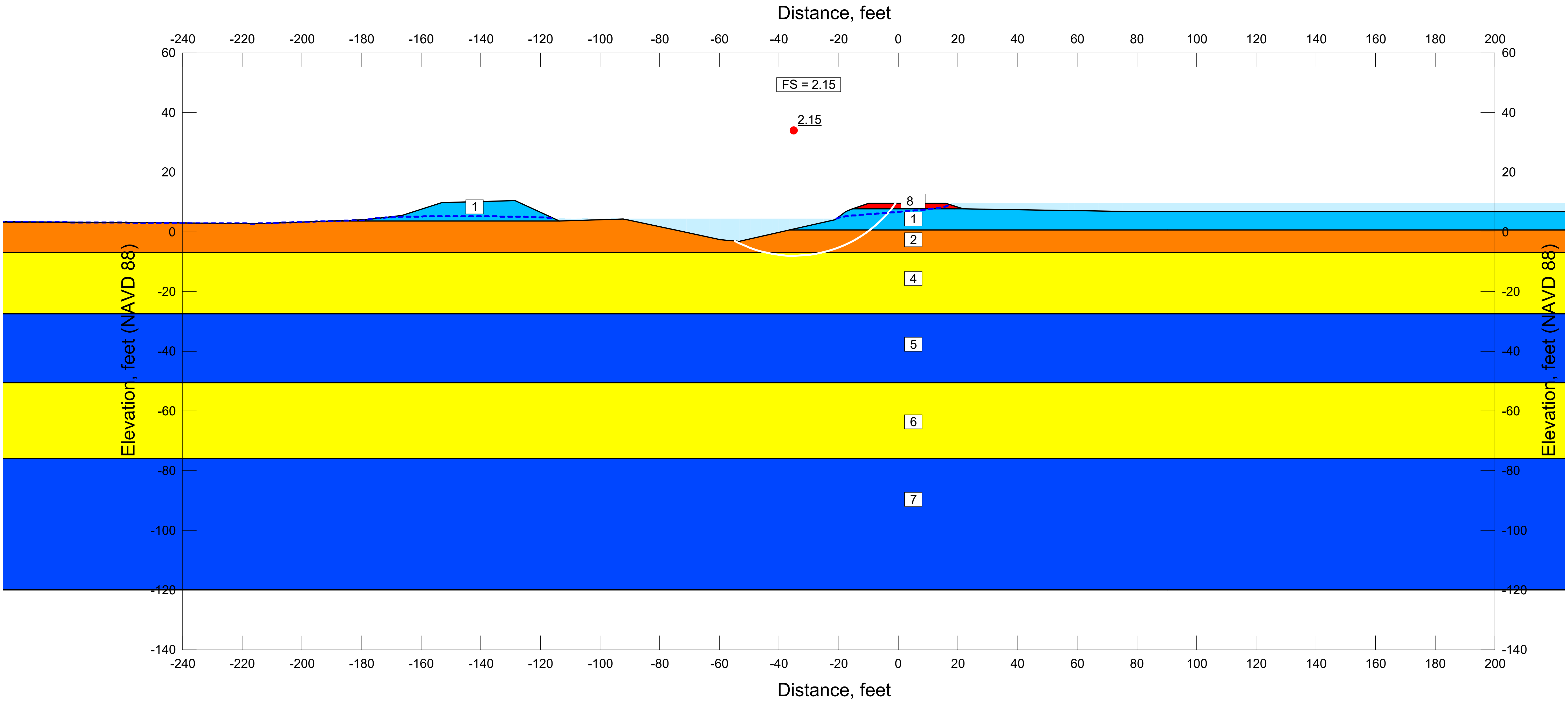
FIG. D-15

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 90+32
Long-Term Steady State

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FIG. D-16

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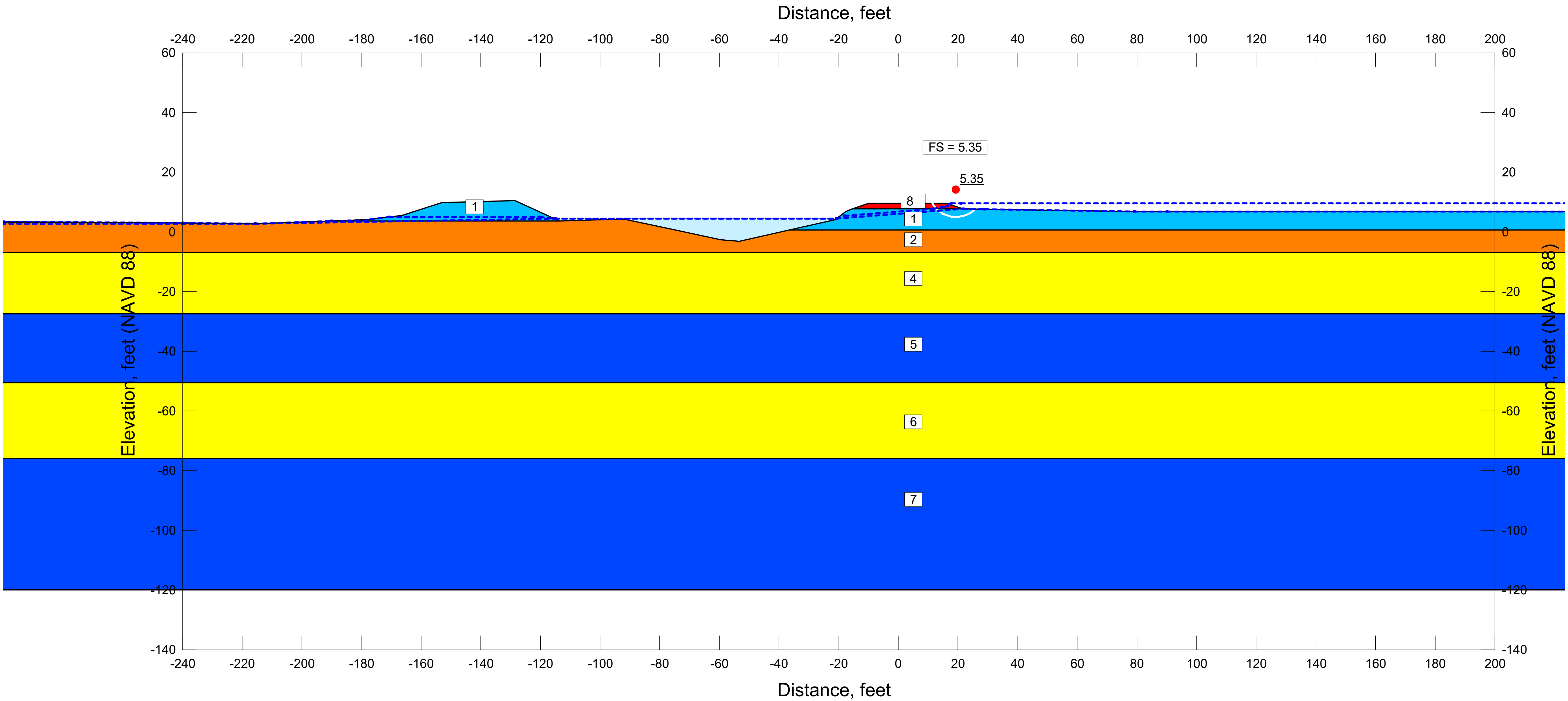
LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 90+32
Rapid Drawdown

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FIG. D-17

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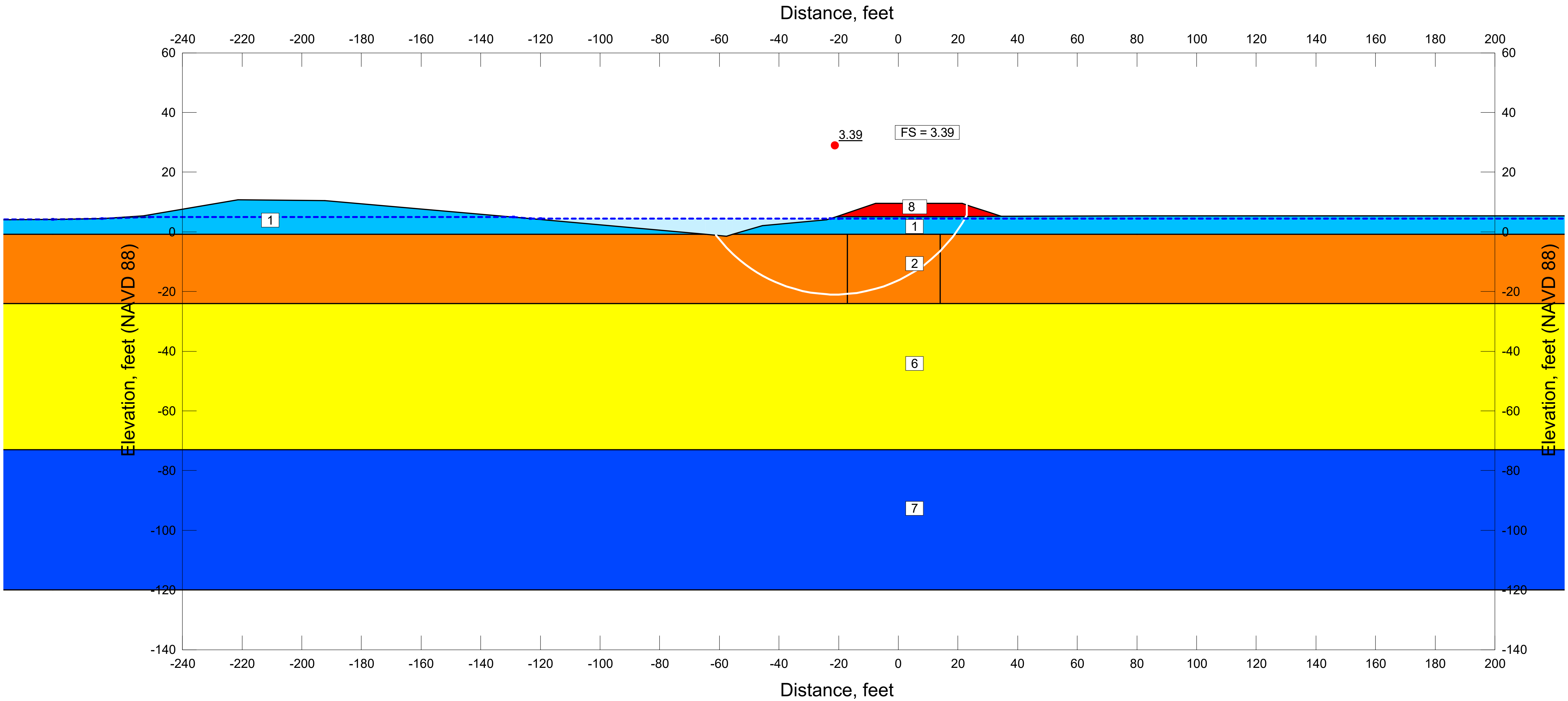
LANDSIDE WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 97+32
End-of-Construction - Landside

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FIG. D-18

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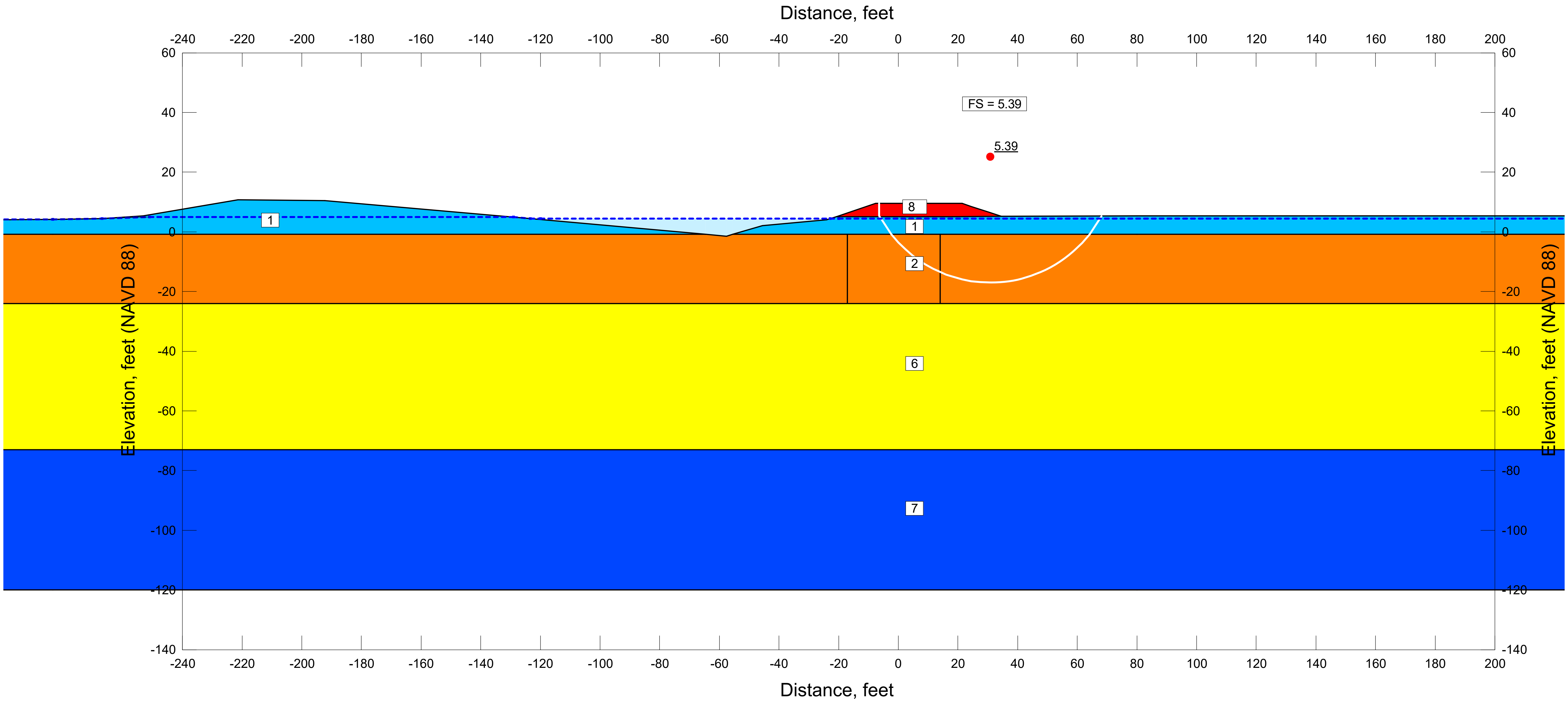
LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 97+32
End-of-Construction - Waterside

April 2024

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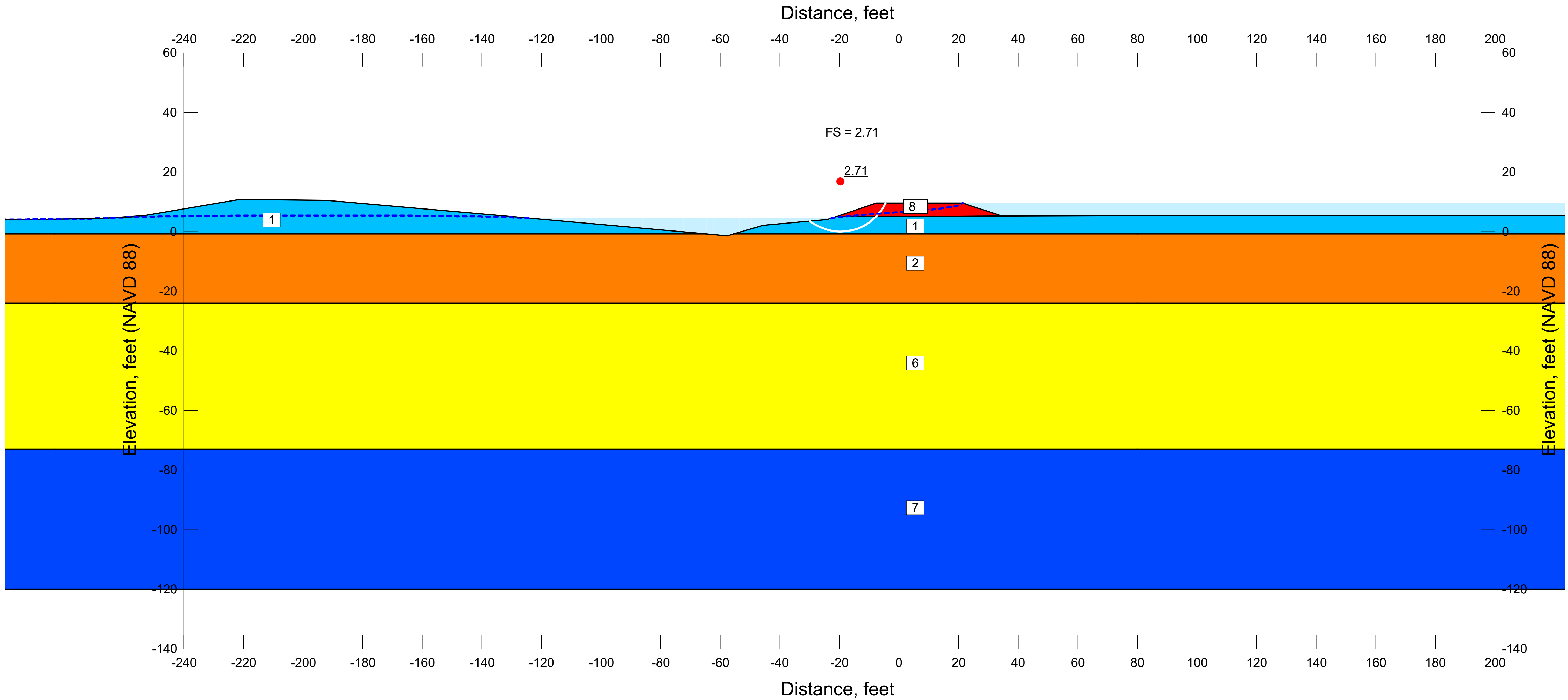
FIG. D-19

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 97+32
Long-Term Steady State

April 2024

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FIG. D-20

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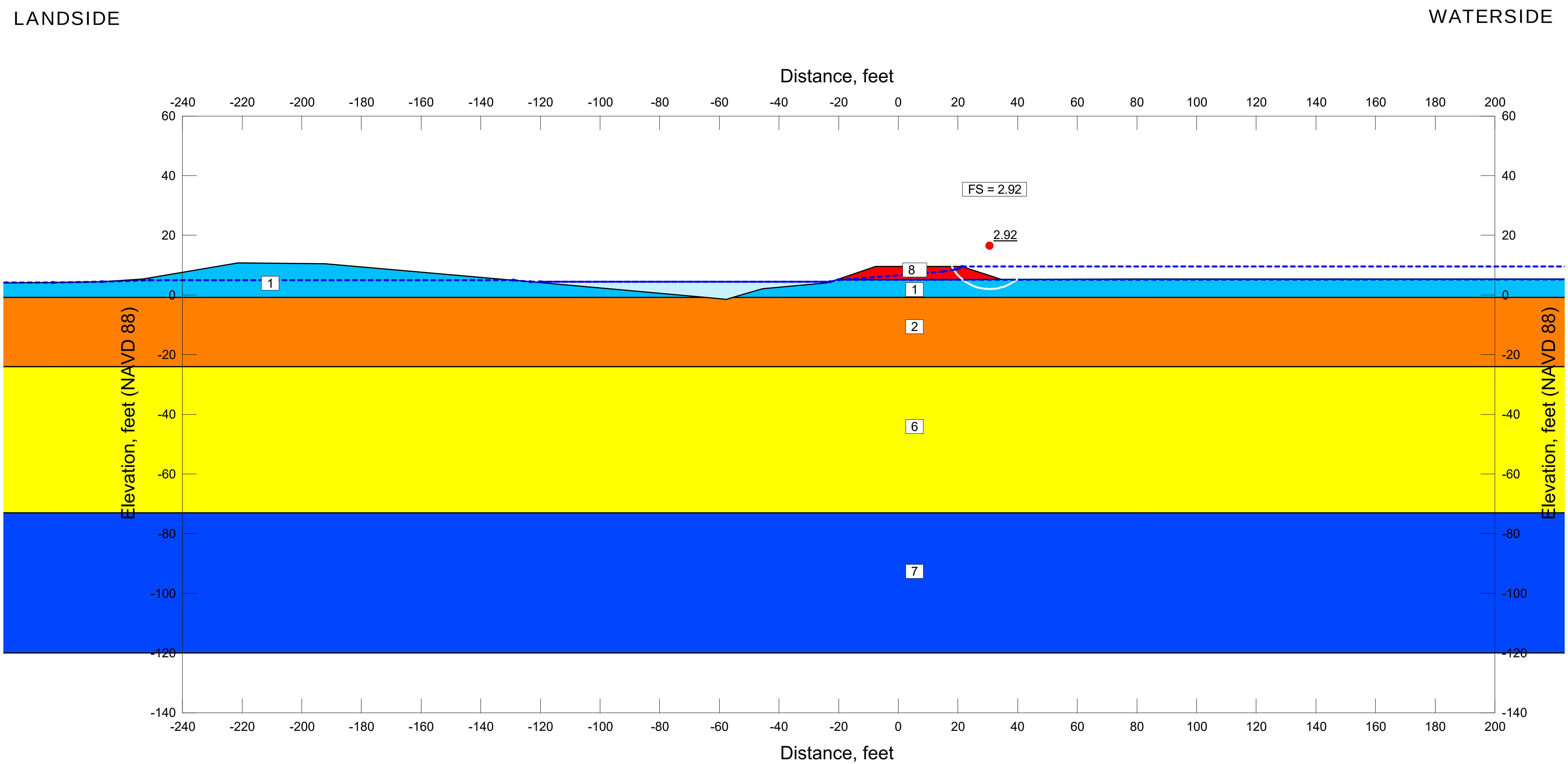

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 97+32
Rapid Drawdown

April 2024

110926

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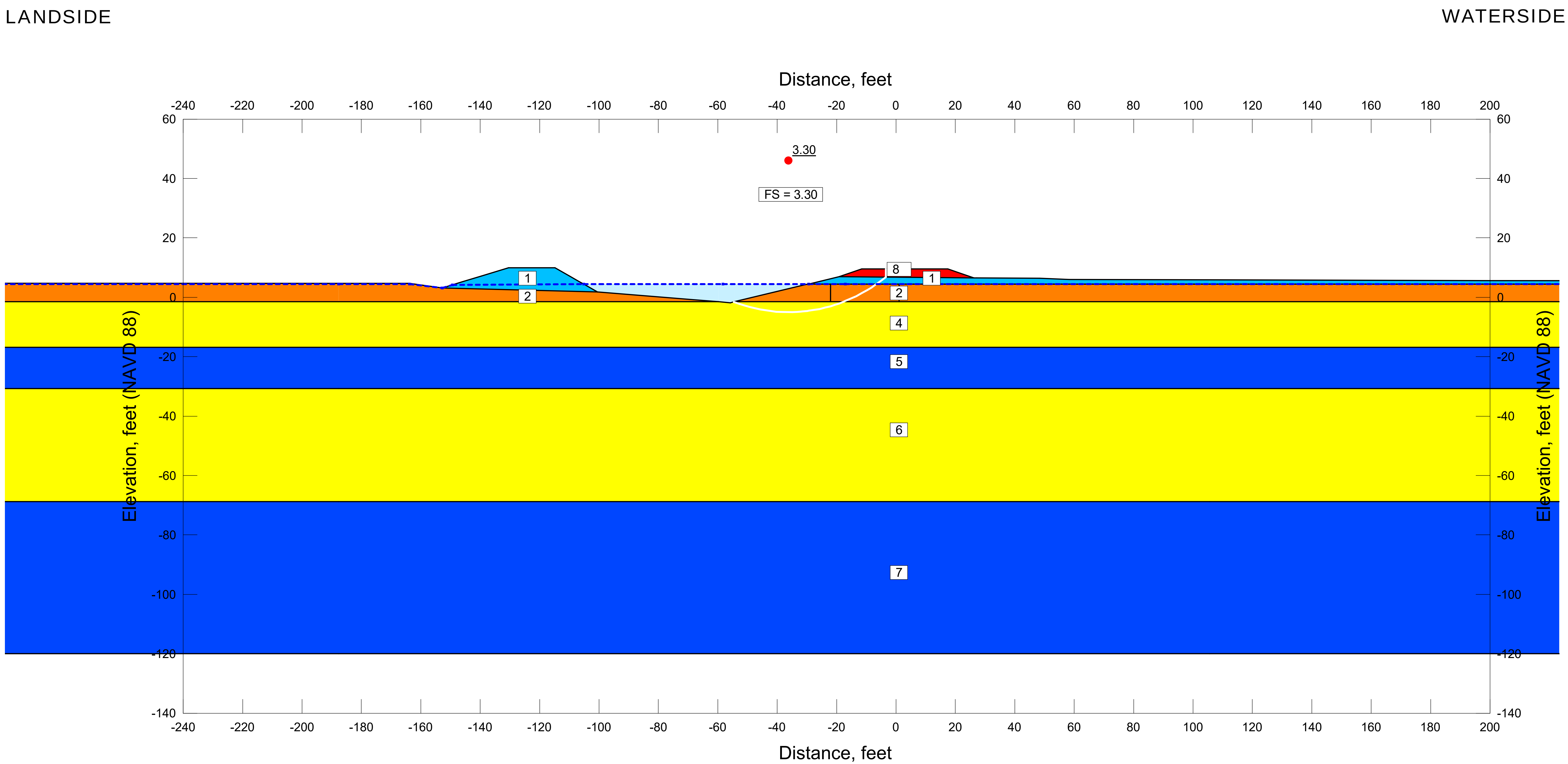
FIG. D-21

Notes:
1. See Figure D-1 for material properties.

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 103+32
End-of-Construction - Landside

April 2024

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FIG. D-22

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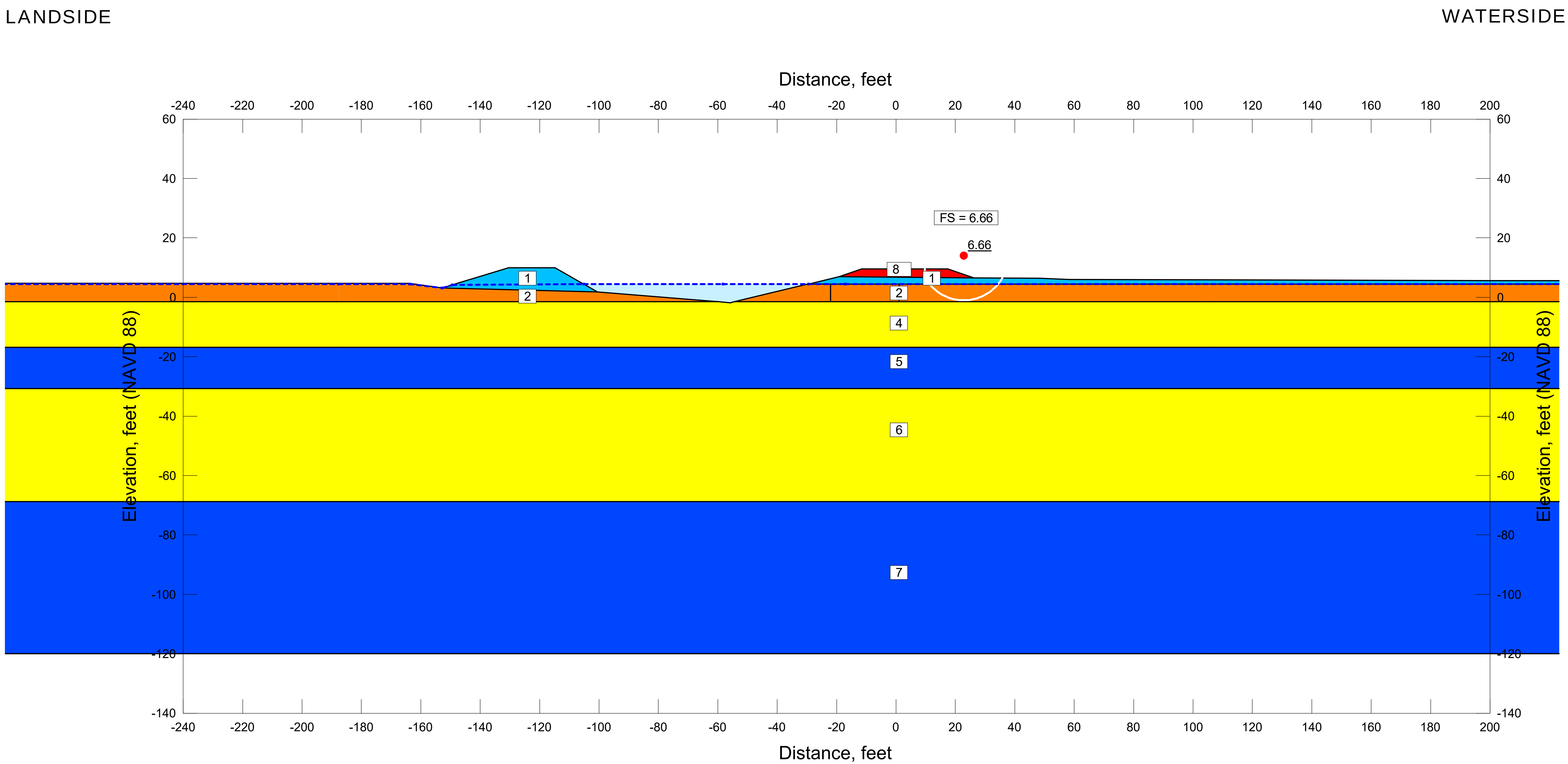

Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 103+32
End-of-Construction - Waterside

April 2024

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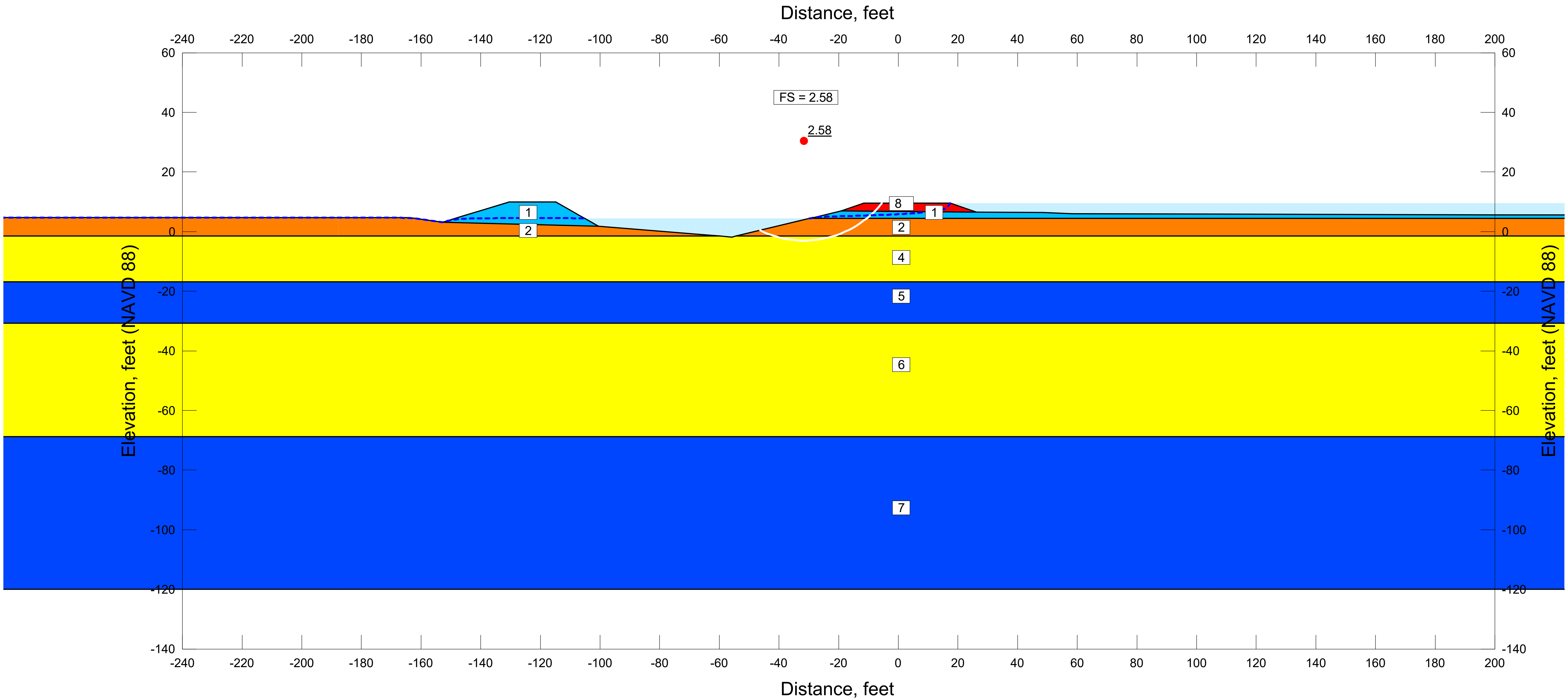
FIG. D-23

LANDSIDE WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 103+32
Long-Term Steady State

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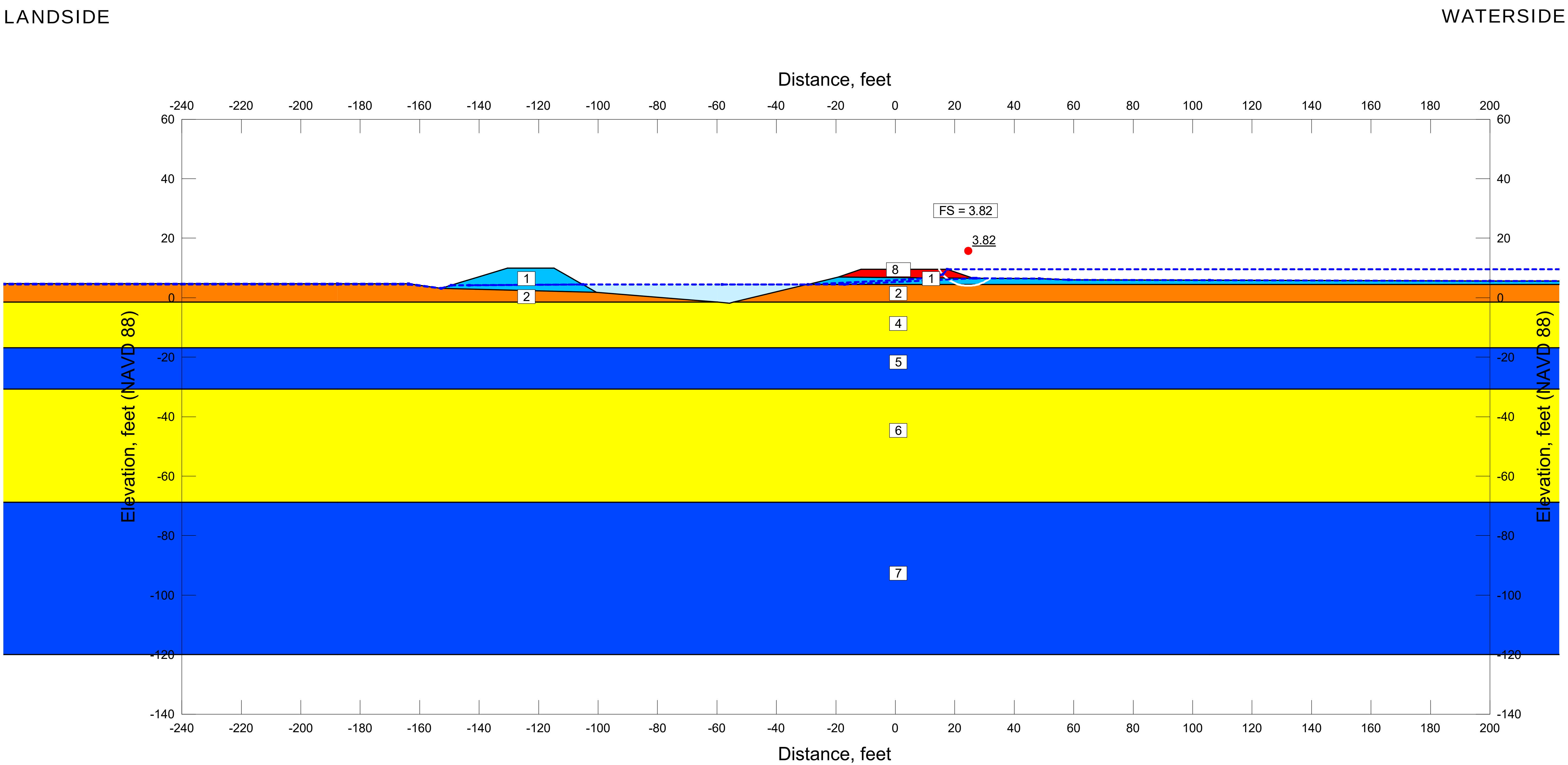
FIG. D-24

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 103+32
Rapid Drawdown

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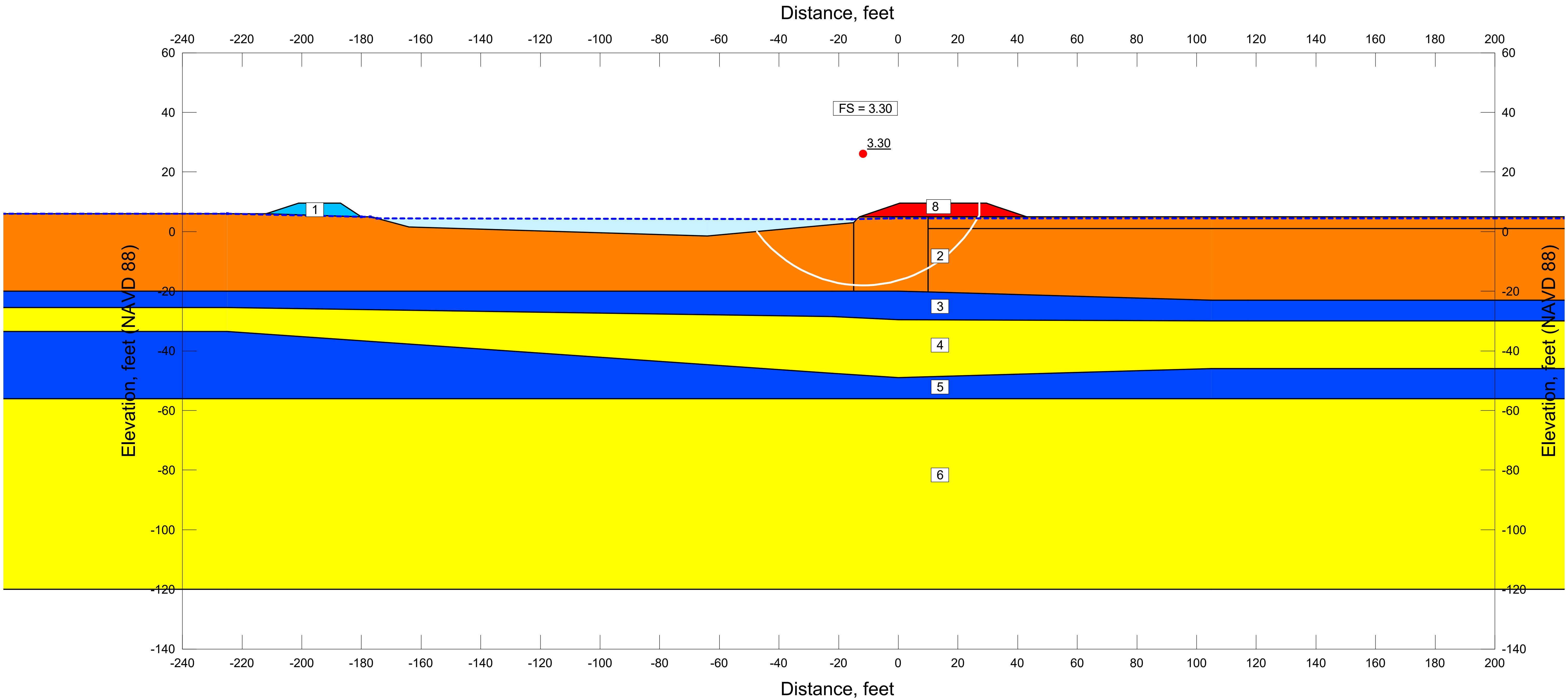
FIG. D-25

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 122+32
End-of-Construction - Landside

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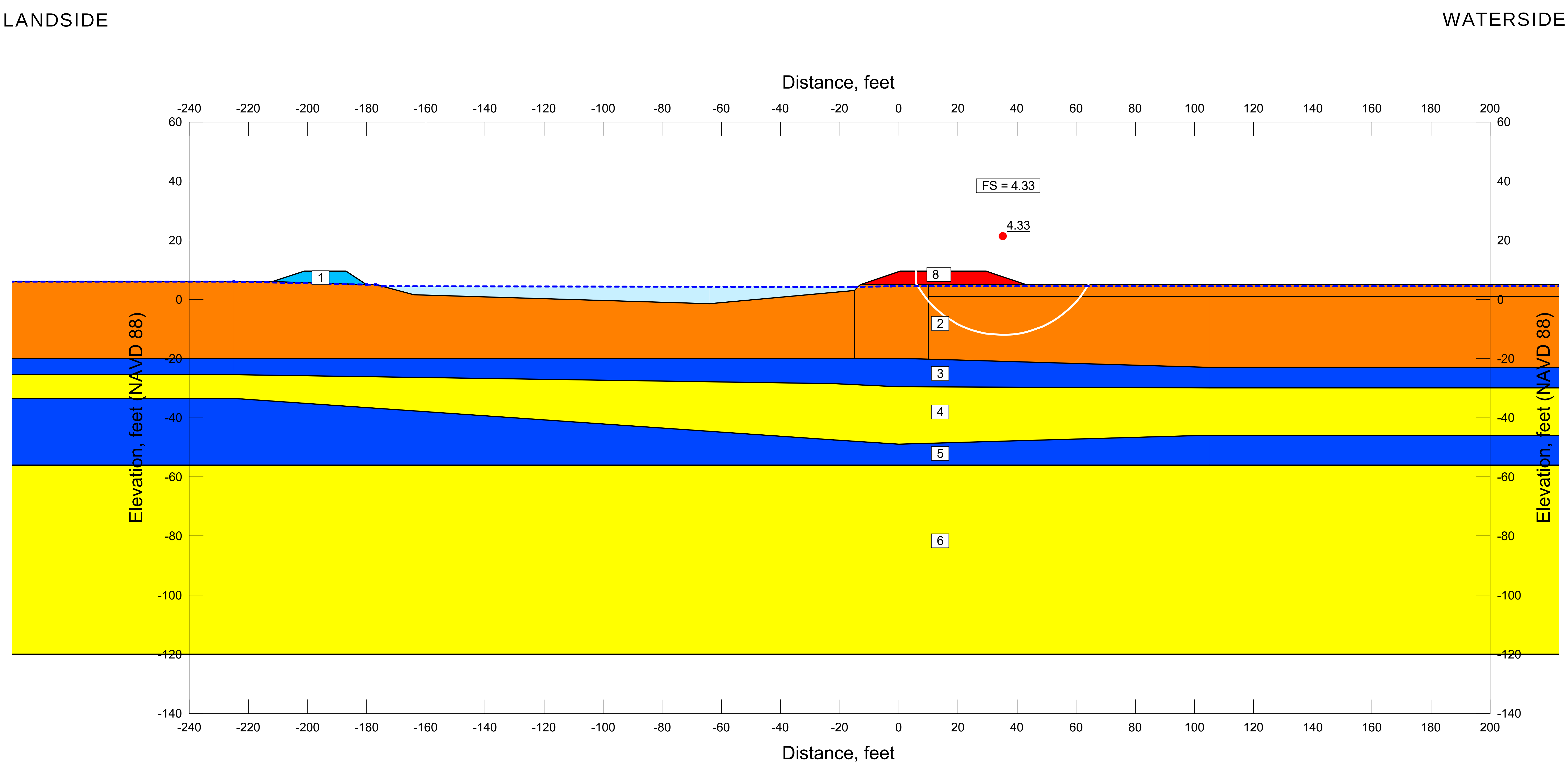
FIG. D-26



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 122+32
End-of-Construction - Waterside

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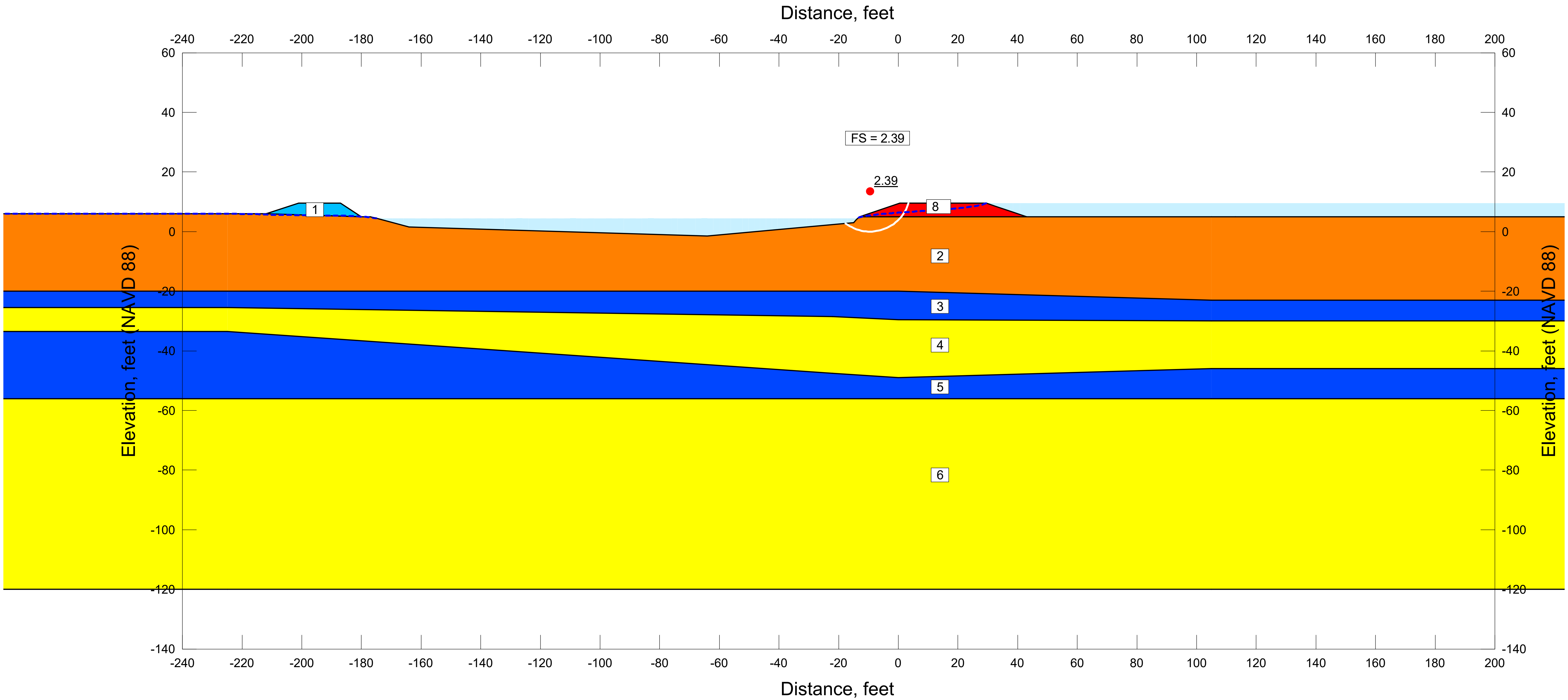
FIG. D-27

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 122+32
Long-Term Steady State

April 2024

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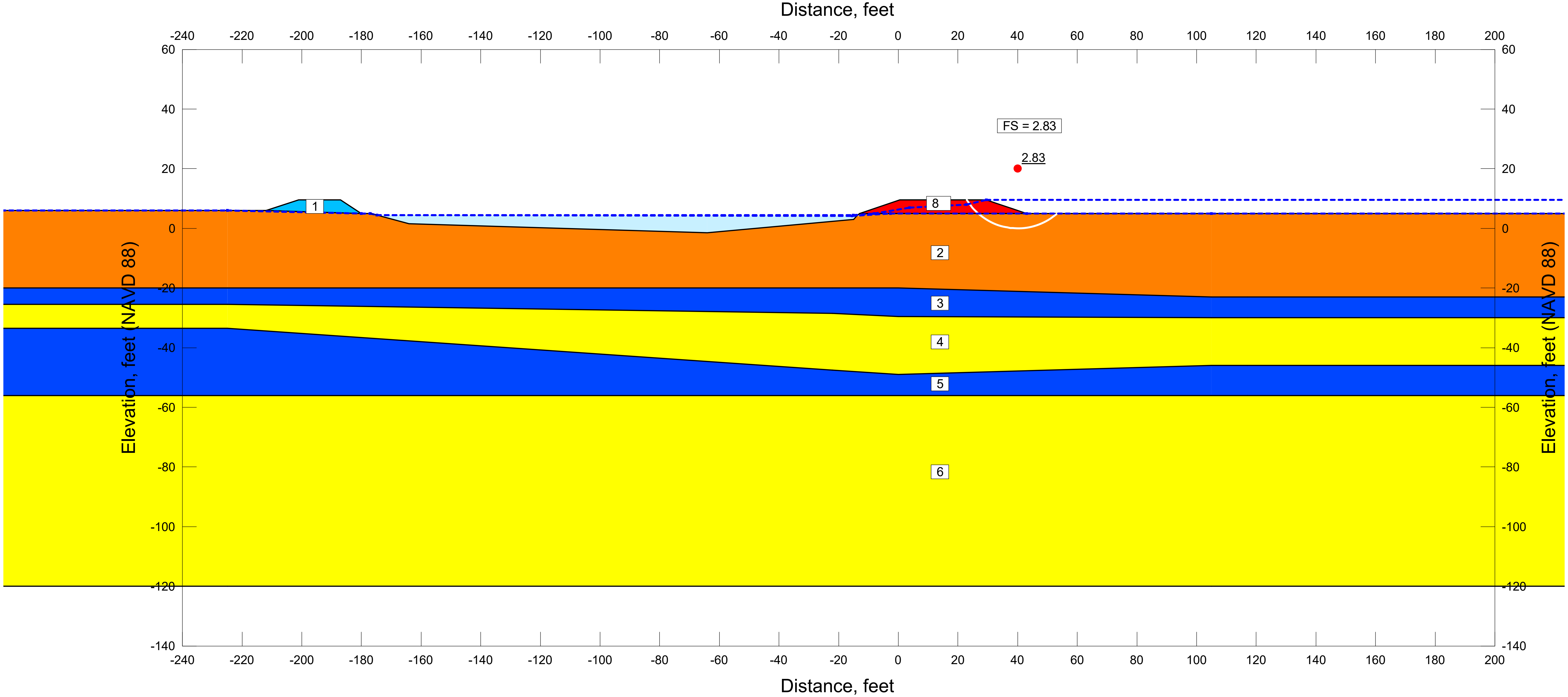
FIG. D-28

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 122+32
Rapid Drawdown

April 2024

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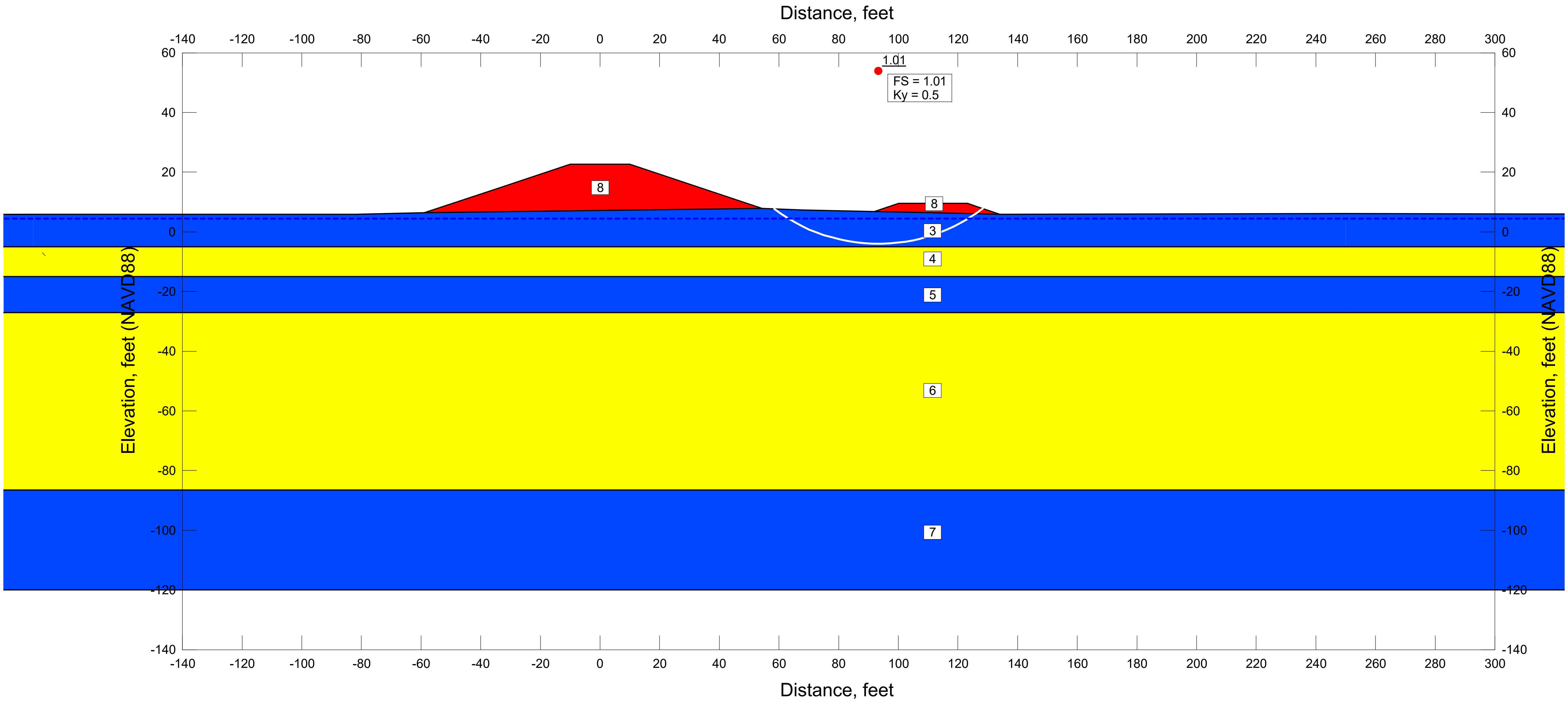
FIG. D-29

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 21+32
Pseudo-Static - Landside

April 2024

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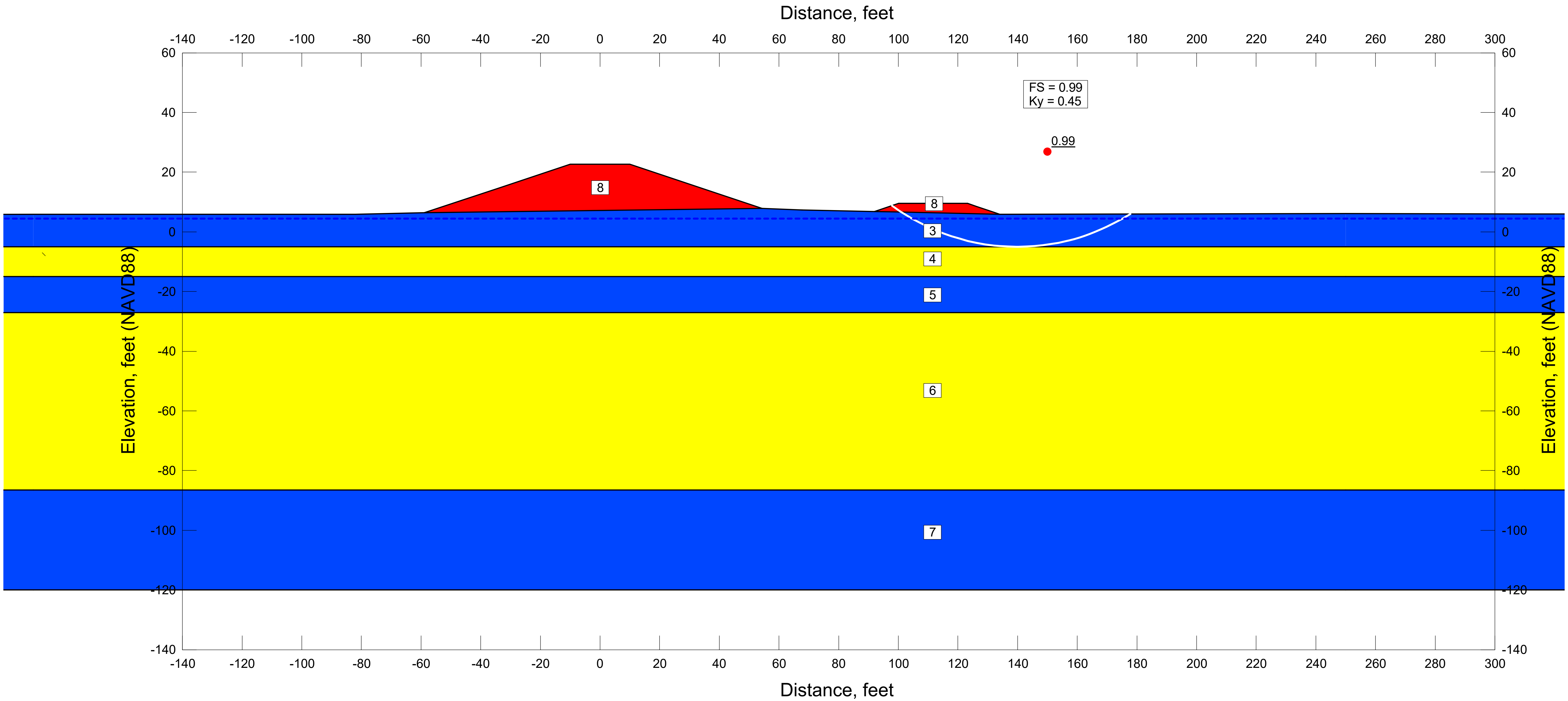
FIG. D-30

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 21+32
Pseudo-Static - Waterside

April 2024

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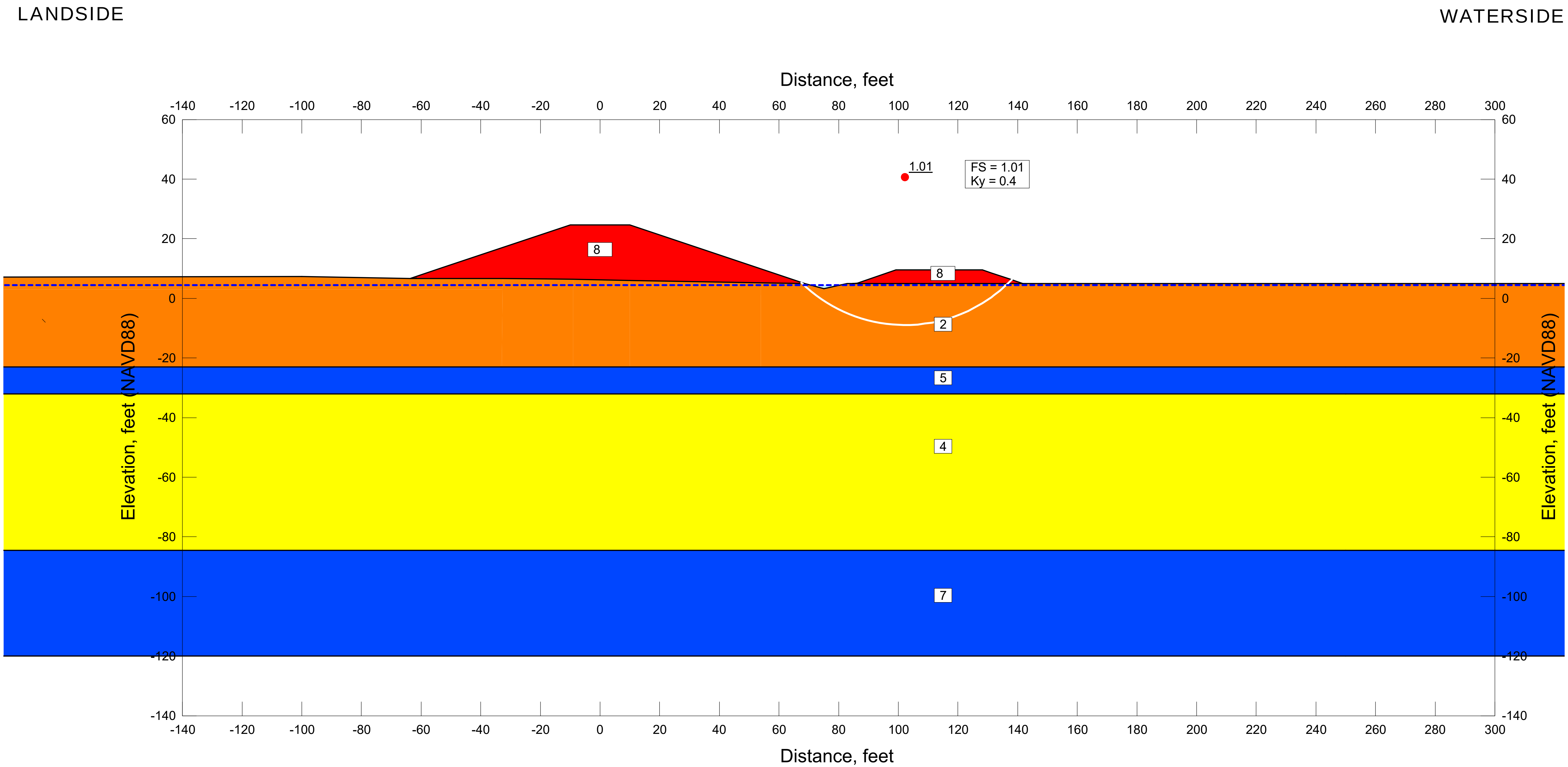
FIG. D-31

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 36+32
Pseudo-Static - Landside

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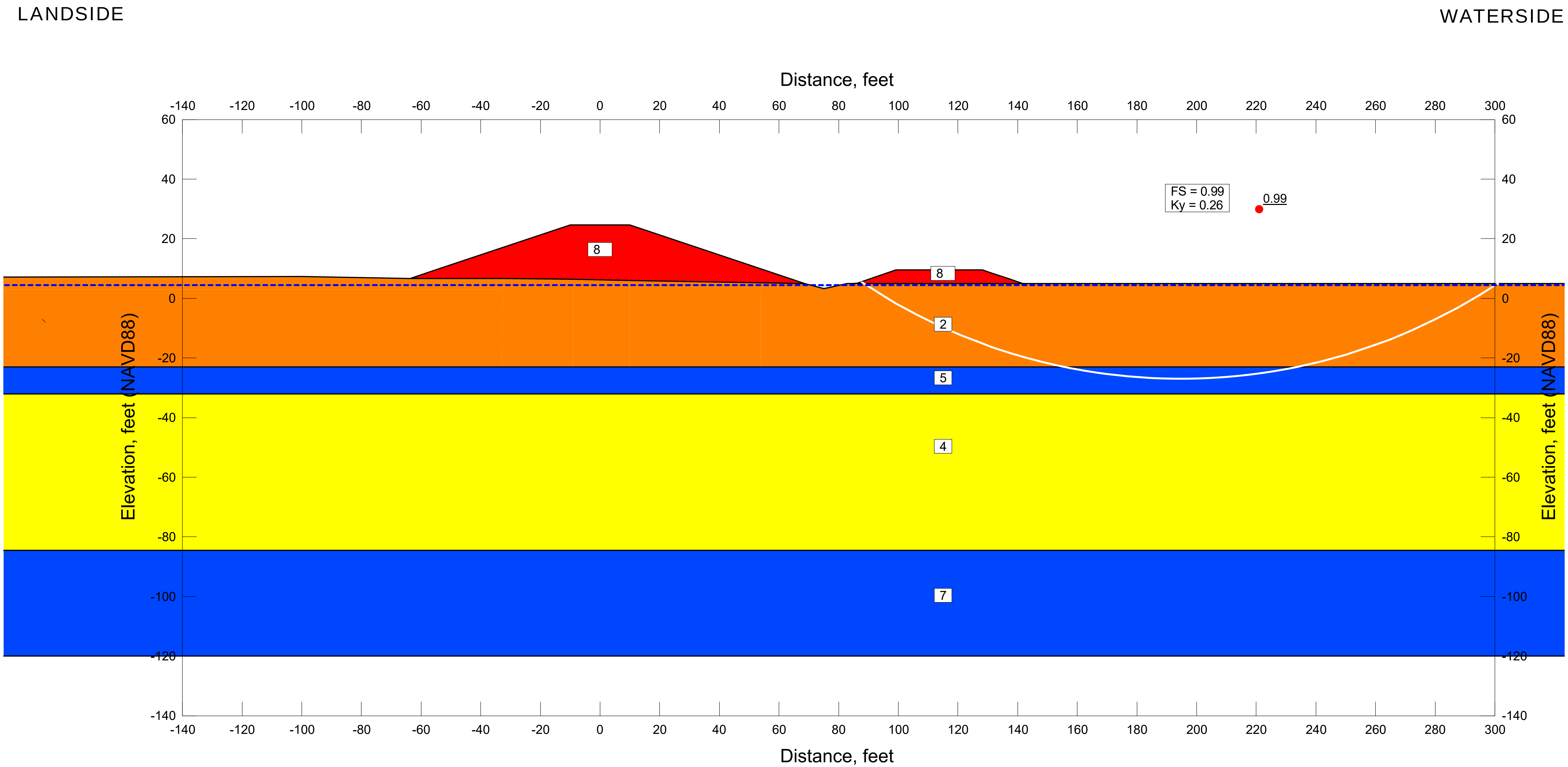
FIG. D-32

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 36+32
Pseudo-Static - Waterside

April 2024

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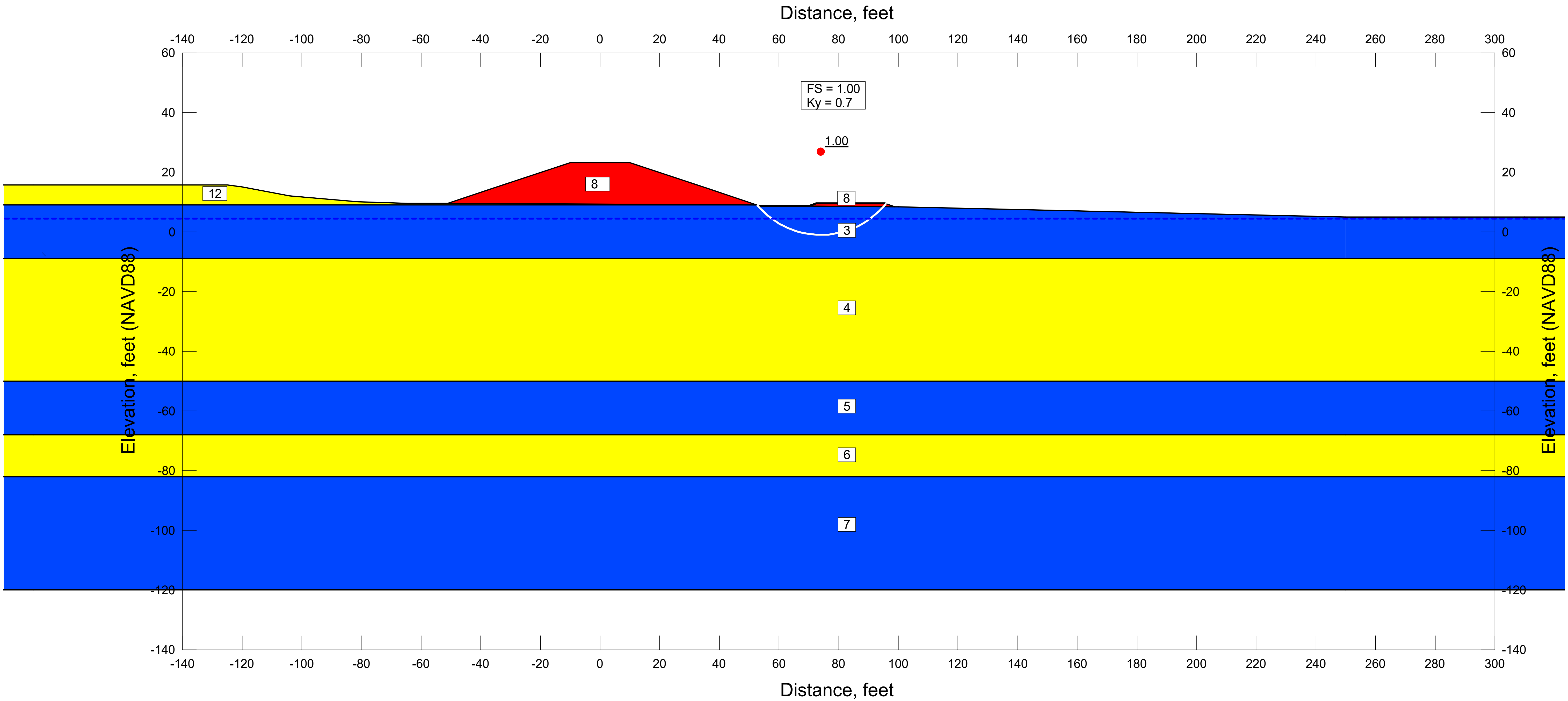
FIG. D-33

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 56+32
Pseudo-Static - Landside

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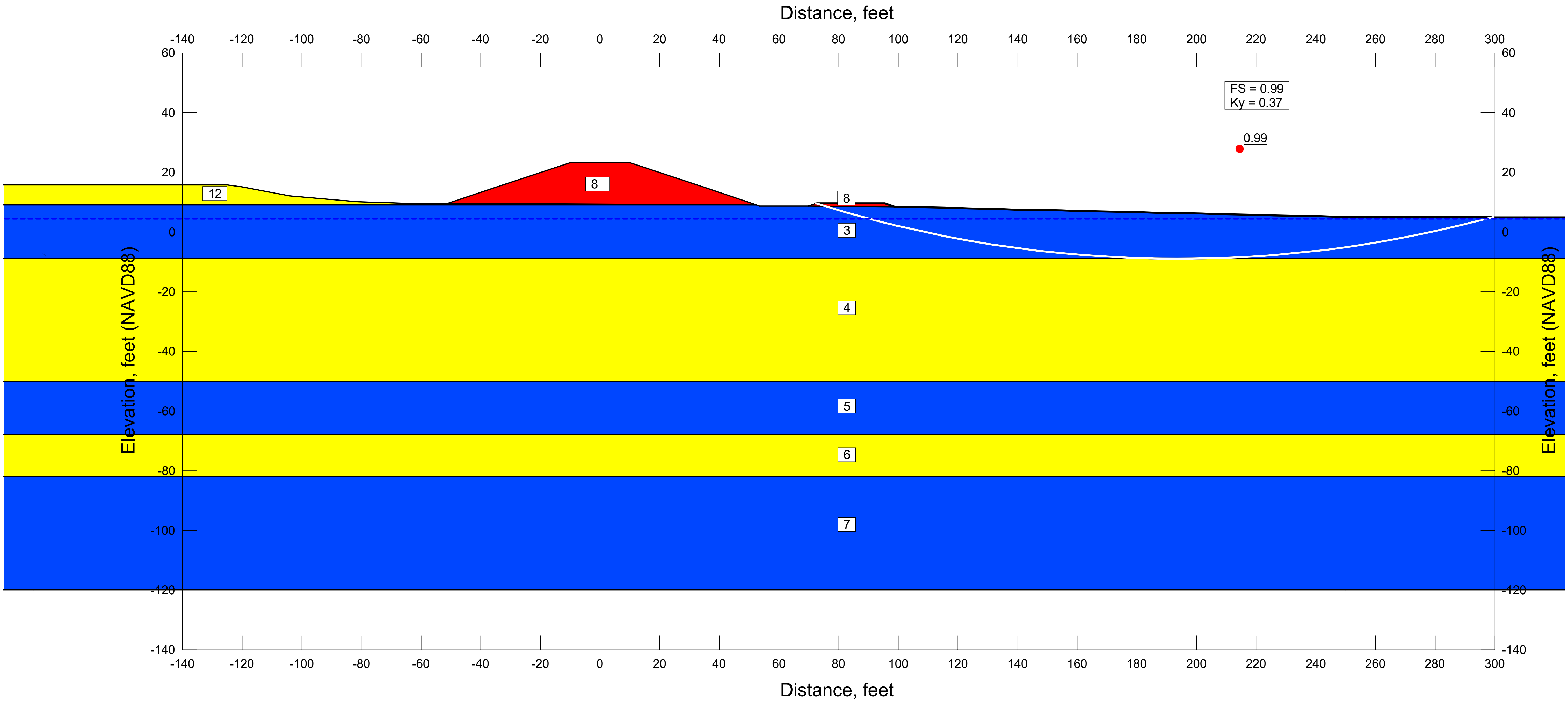
FIG. D-34

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 56+32
Pseudo-Static - Waterside

April 2024

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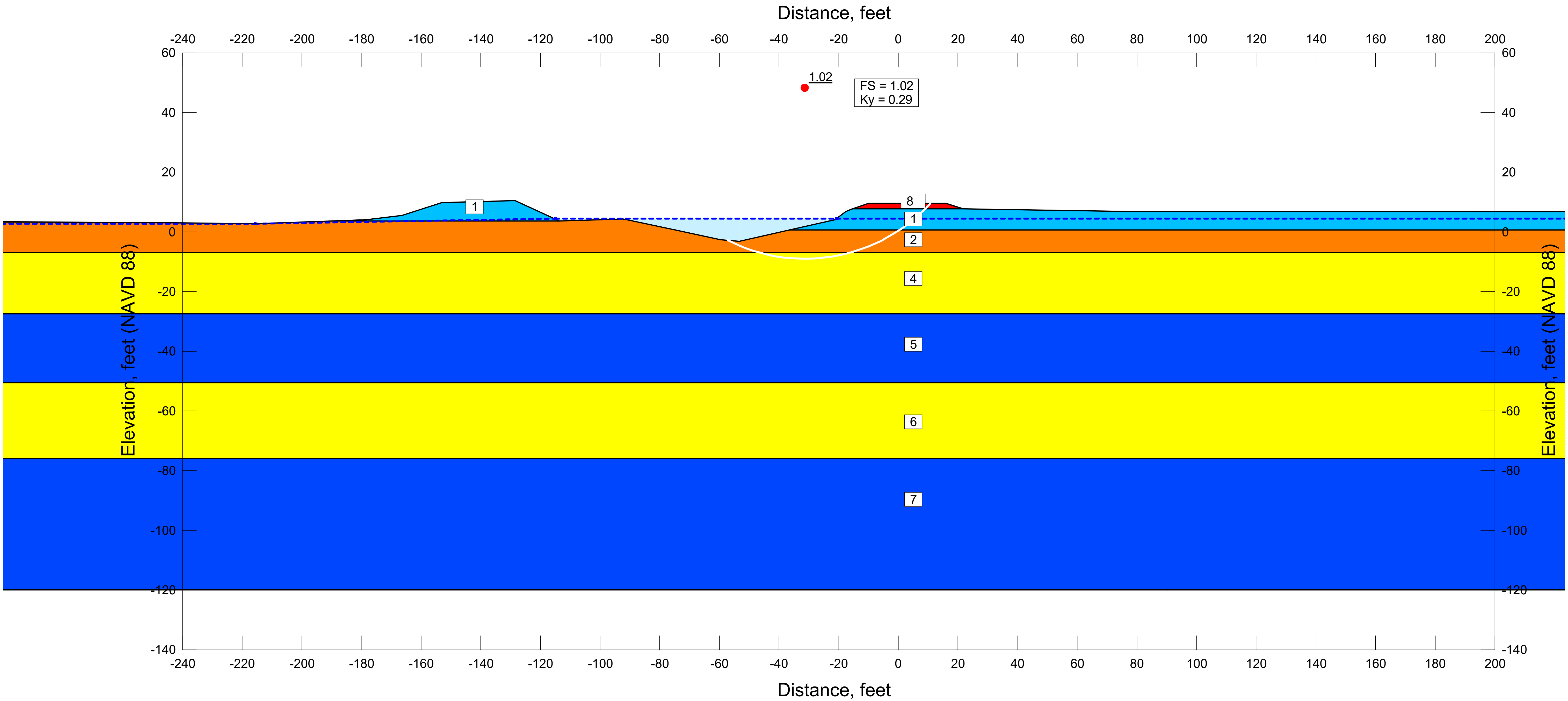
FIG. D-35

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\

LANDSIDE

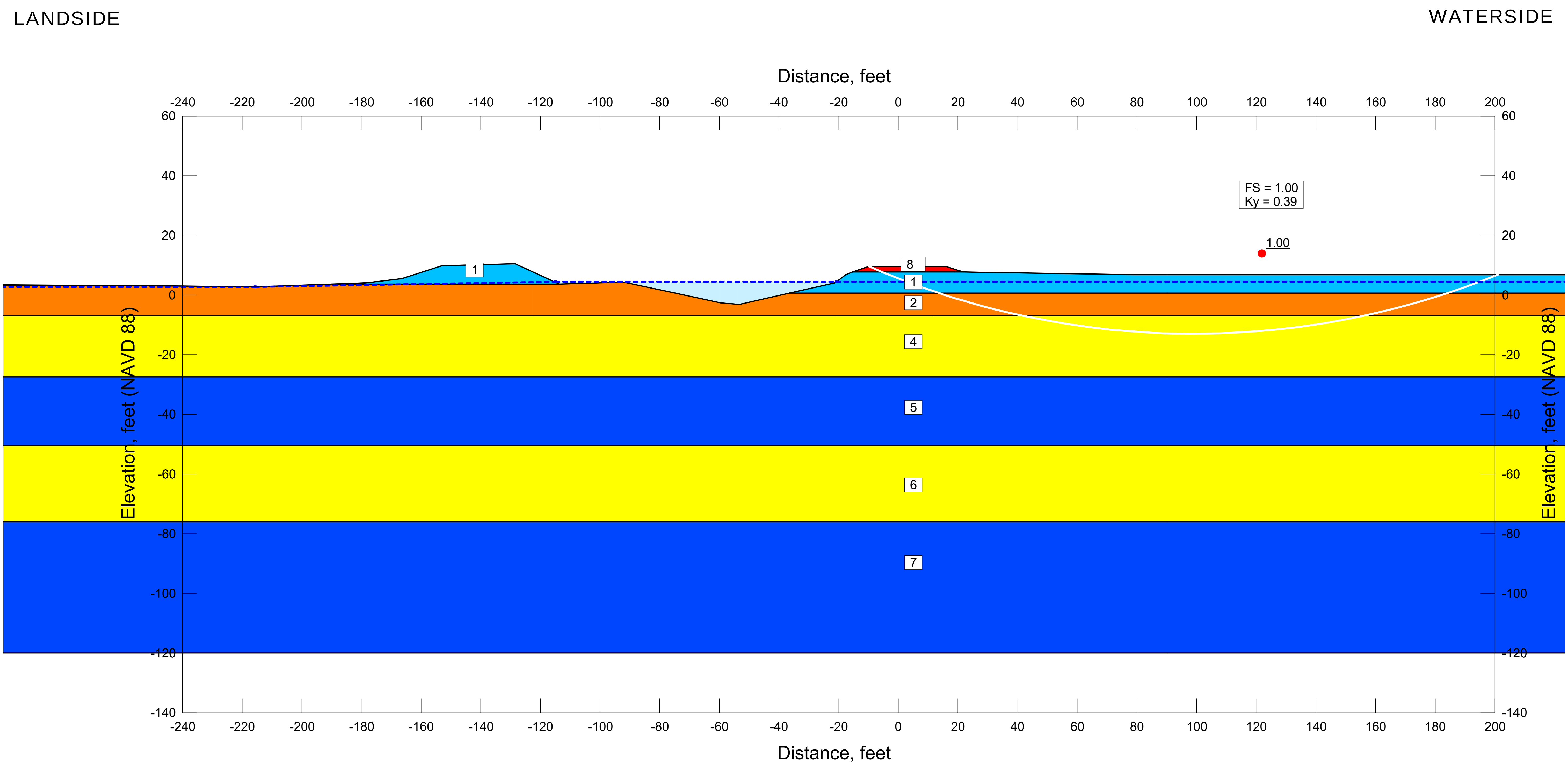
WATERSIDE



Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\

Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank Solano County, California
Slope Stability Analysis **Station 90+32** **Pseudo-Static - Landside**
April 2024 110926
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants — **FIG. D-36**



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 90+32
Pseudo-Static - Waterside

April 2024 110926

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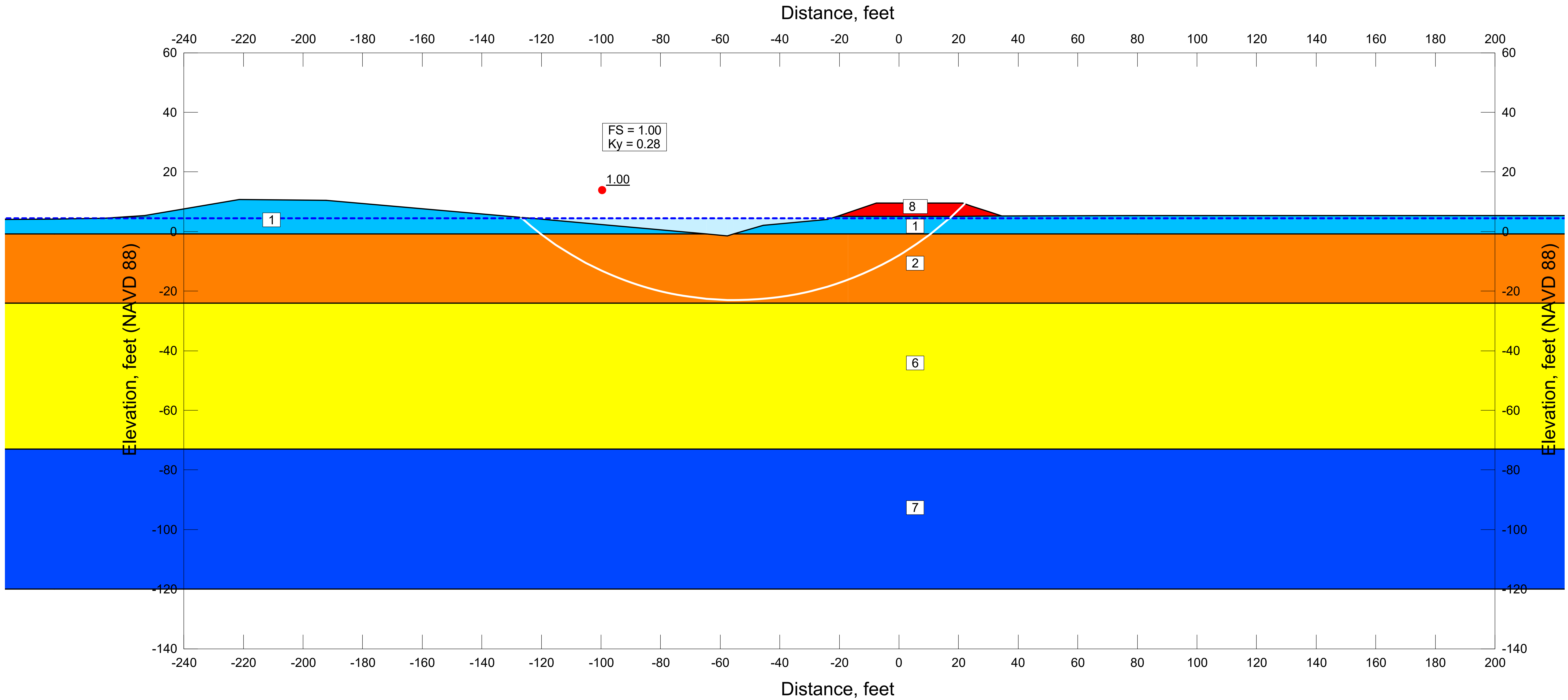
FIG. D-37

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 97+32
Pseudo-Static - Landside

April 2024

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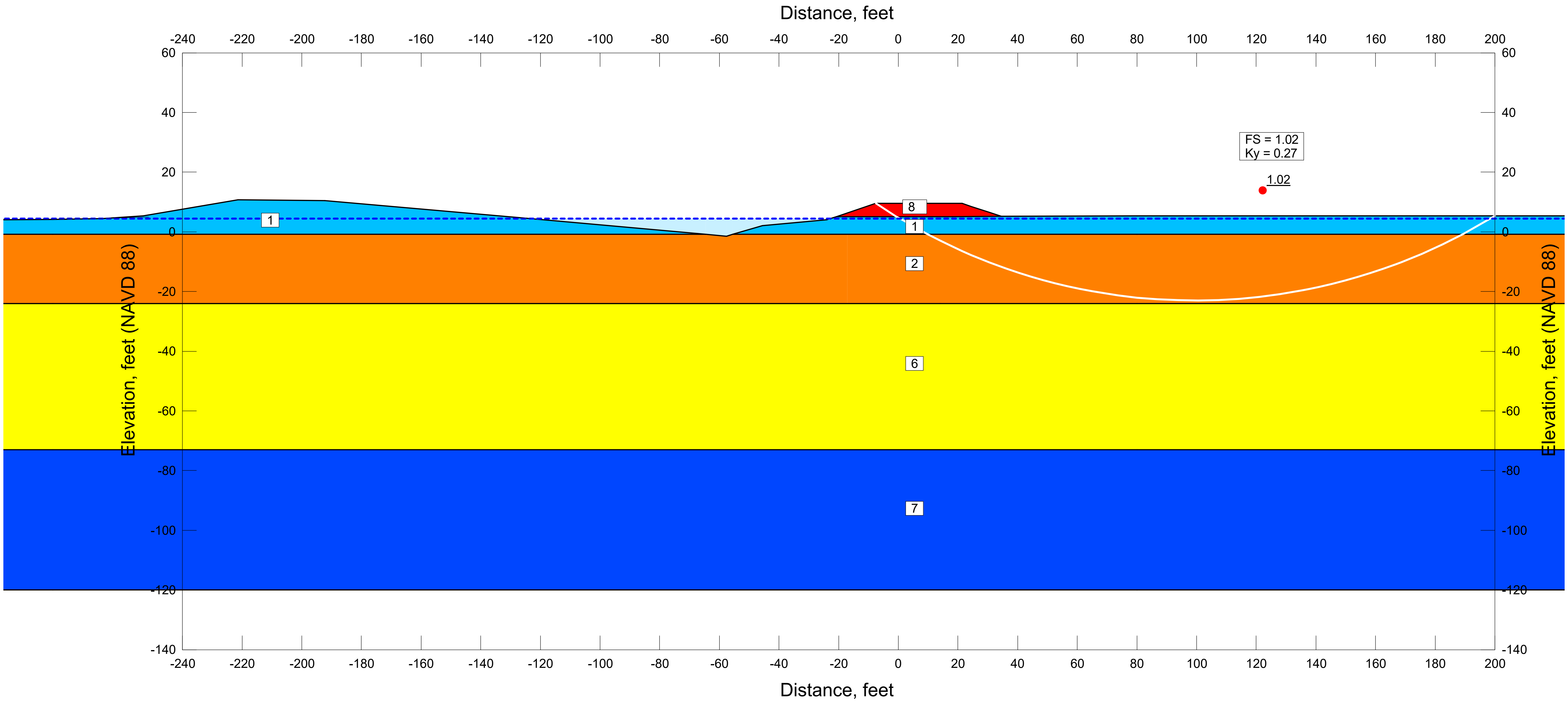
FIG. D-38

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\

LANDSIDE WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 97+32
Pseudo-Static - Waterside

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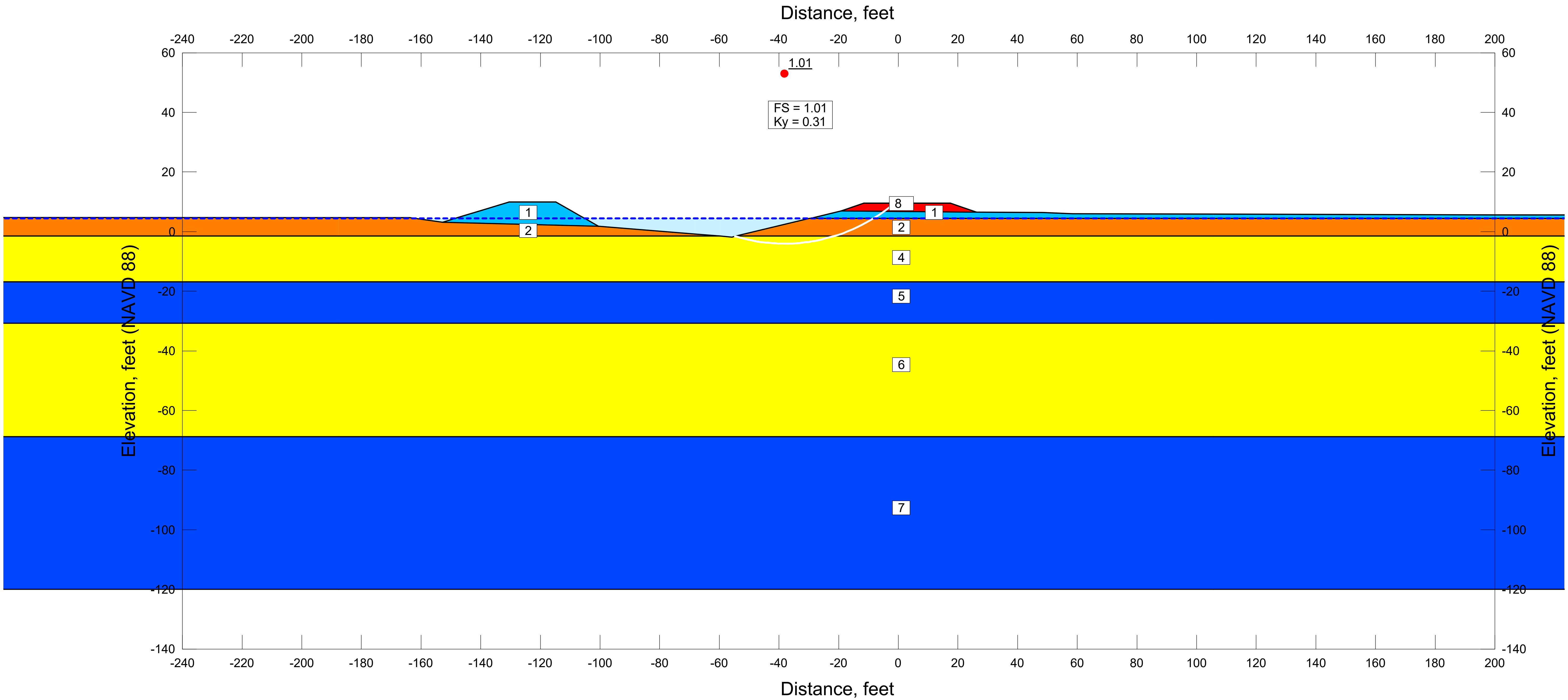
FIG. D-39

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 103+32
Pseudo-Static - Landside

April 2024

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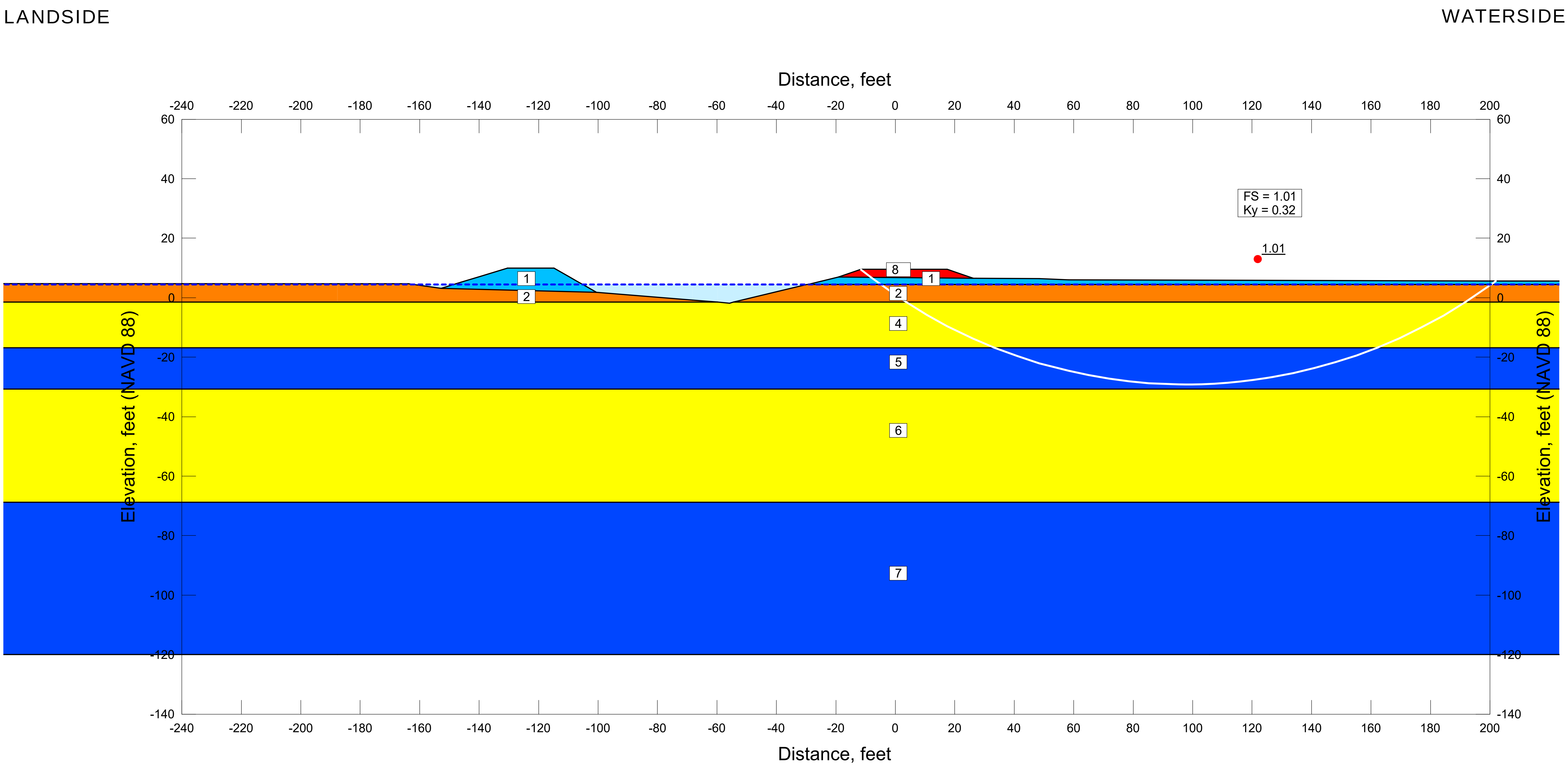
FIG. D-40



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 103+32
Pseudo-Static - Waterside

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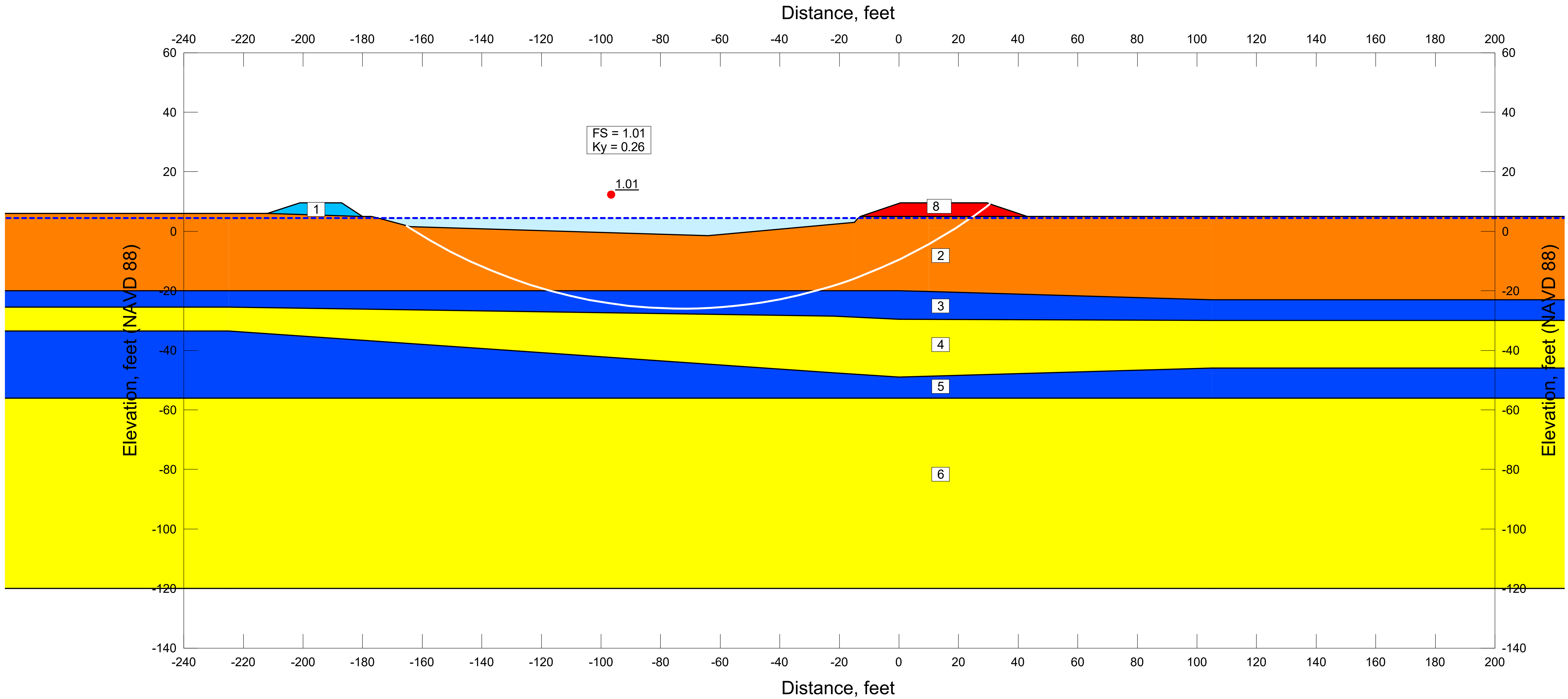
FIG. D-41



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 122+32
Pseudo-Static - Landside

April 2024

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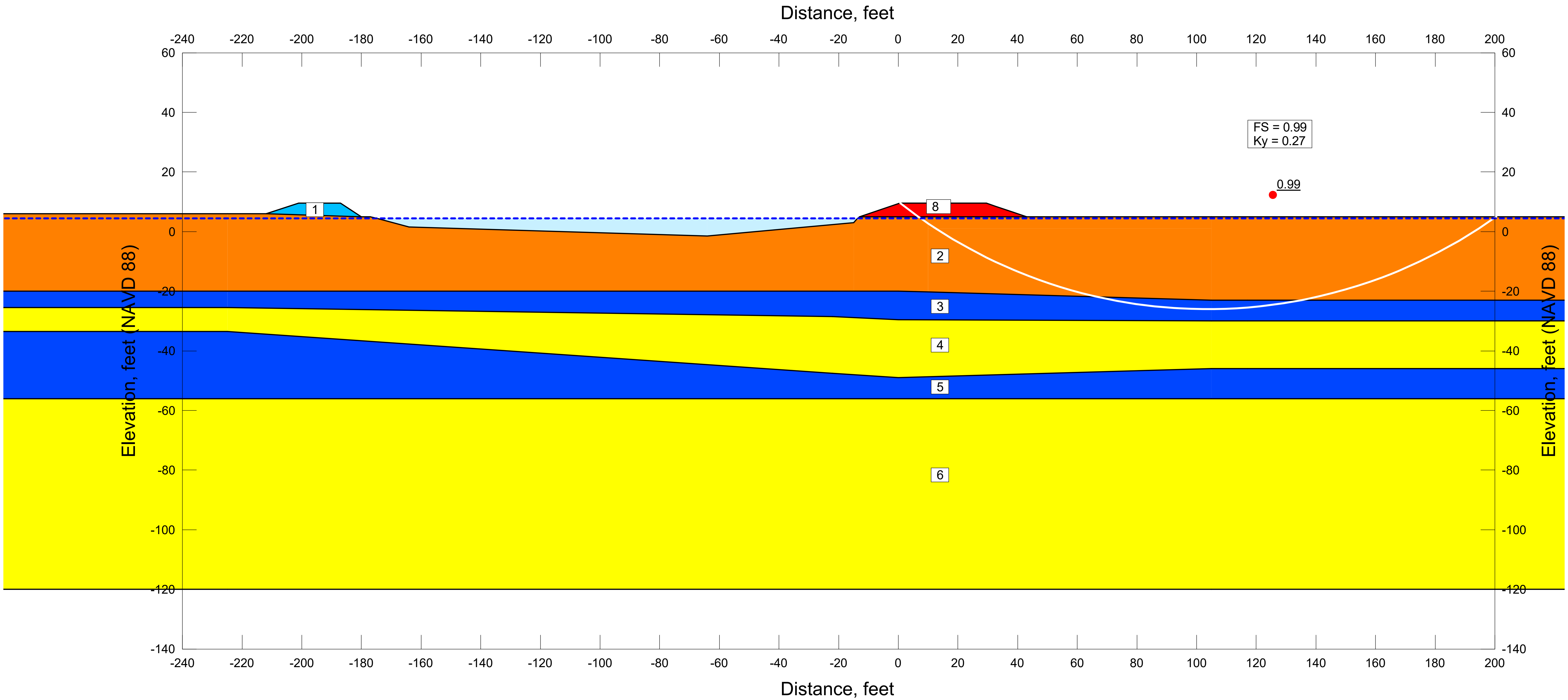
FIG. D-42

Directory: P:\CCD\110000s\110926 Cache Slough Mitigation Bank\Geotechnical Investigation\Analysis\Seepage and Slope Stability\parts for figures\Stability\

LANDSIDE

WATERSIDE



Notes:
1. See Figure D-1 for material properties.

Cache Slough Mitigation Bank
Solano County, California

Slope Stability Analysis
Station 122+32
Pseudo-Static - Waterside

April 2024

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FIG. D-43

IMPORTANT INFORMATION

Important Information

About Your Geotechnical Report

IMPORTANT INFORMATION

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 that 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

IMPORTANT INFORMATION

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.

IMPORTANT INFORMATION

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 GBA, Silver Spring, Maryland