Final Supplemental Watershed Plan No. 1 - Environmental Assessment

United States Department of Agriculture

Natural Resources Conservation Service

Central Sonoma Watershed - Matanzas Creek Dam Rehabilitation Project

Sonoma County, California

Sponsoring Local Organization: Sonoma County Water Agency

Lead Agency: Natural Resources Conservation Service (NRCS)

October 2026

Appendix D TM003



TM003 – Soil Mechanics Report

Soil Mechanics Report for Matanzas Creek Dam

USDA NRCS – California State Office
Davis, California

Project Sponsor: Sonoma County Water Agency

NID ID: CA00794

AECOM Project No.:
JDE Project No.: 2006-048
May 6, 2022

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Soil Mechanics Report for Matanzas Creek Dam

Dear Mr. Ernesto de la Riva:

Jones and DeMille Engineering (JDE) and AECOM Technical Services, Inc. (AECOM) has prepared this report on the analysis of the stability and seismic performance for Matanzas Creek Dam in Sonoma County, California. The report is based on historical explorations and explorations performed as part of this study in 2021 as summarized in the Geotechnical Investigation Report (GIR).

This report summarizes our material characterization, stability, and seismic deformation calculations for the embankment and the landslide mapped on the left abutment of the dam.

Thank you for the opportunity to assist with this project. Please call Darin Robinson at 435.896.8266 if you have any questions or comments regarding the information presented in this report.

Yours sincerely,

Darin Robinson, P.E. (UT)
Principal, Project Manager
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Quality information

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Revision History

Revision	Revision date	Details	Authorized	Name	Position
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1. Introduction

1.1 Project Overview

The NRCS is assisting Sonoma Water in preparing a supplemental watershed project plan and environmental assessment of the Matanzas Creek Dam, which is located in Northern California, immediately south of Santa Rosa, as shown in Figure 1-1. The results of the assessment will be combined into a single document, called the Supplemental Plan-EA. The Supplemental Plan-EA will be prepared to comply with the National Environmental Policy Act (NEPA). The purpose of the Supplemental Plan-EA is to develop a watershed project plan so that NRCS can decide whether to provide technical and financial assistance for implementation of the alternative selected by Sonoma Water.

1.2 Background

Matanzas Creek Dam was originally constructed as a “High” Hazard Dam in 1964 and is currently still classified as a High Hazard dam according to NRCS standards based on the results of hydraulic analyses included in a 2016 dam assessment report prepared by HDR/Kleinfelder. After construction the dam was operated with a reservoir, however in more recent years the lowest outlet has been left open resulting in a dry reservoir.

The dam was constructed as part of the Central Sonoma Watershed Protection and Flood Prevention Project, which included the construction of five other dams, diversion structures, and other facilities intended to reduce flooding risk for the city of Santa Rosa. Since the dam has exceeded its 50-year project life, the 2016 assessment was to determine if the dam meet NRCS dam safety and performance standards set forth in NRCS TR-210-60 and California Department of Water Resources - Division of Safety of Dams (“DSOD”) hydrologic design criteria (HDR/Kleinfelder, 2016). The 2016 assessment concluded that the existing dam does not meet some of the stated hydraulic criteria and is therefore out of compliance with current NRCS hydrologic and hydraulic criteria and performance standards. The 2016 HDR/Kleinfelder assessment did not include evaluation of geologic or geotechnical criteria for the dam.

1.3 Purpose and Scope

The JDE/AECOM team was contracted perform preliminary geologic investigation and geotechnical engineering services in support of the subject Supplemental Plan-EA. Specifically, the JDE/AECOM team’s scope of work includes performance of a site-specific preliminary field Geologic Investigation (GI), preparation of Geologic Investigation Report (GIR), and preparation of an Soil Mechanics Report (SMR). Specific objectives of the geologic/geotechnical scope of work are to aid in evaluating the feasibility of various alternatives to be considered in the Supplemental Plan-EA document, and to investigate the existing geologic/geotechnical conditions relating to the following identified dam performance risks:

- Evaluate static and seismic stability of the existing dam embankment;
- Investigate the mapped Matanzas Creek Fault which is inferred to cross under the dam embankment;
- Evaluate stability of the mapped landslide at the left abutment; and
- Evaluate erodibility of the existing auxiliary spillway.

Preliminary subsurface investigations were performed as part of the field GI to investigate these conditions, the findings and interpreations of which are documented in a GIR prepared under separate cover (AECOM, 2022). The GIR also provided recommended geologic input parameters for analysis of auxiliary spillway erodibility. Additional details regarding the field GI and brief summary of the GIR findings are discussed later in Section 5.1 of this report.

The purpose of SMR is to present and analyze the results of the geotechnical laboratory testing program, document material characterization and geotechnical engineering analyses results, and provide geotechnical recommendations for required dam improvements and/or further study (if necessary). Specific tasks included as part of the SMR scope of work for the Matanzas Creek Dam include the following:

1) Develop a laboratory testing program for soil and rock samples collected during the field GI, and analyze and interpret laboratory test results;

2) Examine available data from historic explorations and the recent geotechnical exploration to develop material properties for use in analysis;

3) Perform site-specific Probabilistic Seismic Hazard Analysis (PSHA) and Deterministic Seismic Hazard Analysis (DSHA) to estimate seismic loading during design earthquake events according to DSOD and NRCS criteria;

4) Perform geotechnical material characterization of the existing embankment and foundation materials and assign properties for analysis;

5) Perform seepage, filter compatibility, slope stability, and seismic analyses related to the existing dam embankment according to NRCS criteria;

6) Perform slope stability analysis (and back analysis) of the mapped landslide on the left abutment; and

7) Prepare this SMR.

The following sections cover the scope described above.

1.4 Authorization

This Geologic Investigation Report was completed by AECOM Technical Services, Inc., (AECOM) under subcontract to Jones & DeMille Engineering, Inc.(JDE) for the United States Department of Agriculture – Natural Resources Conservation Service – California State Office (NRCS-CSO) in accordance with the Statement of Work described in Order No. 12FPC320F0273 and Requisition No. 1002719, and executed under the terms and conditions of Contract No. 12SPEC18D0020, and modification No. P00001 and Requisition No. 1028141. .

2. Review of Available Records

As part of preparation of this report, JDE and AECOM received and reviewed the documents listed below to gain an understanding of the planning, design, construction, maintenance, and operation of the project.

Sonoma County Flood Control and Water Conservation District (SCFCWCD), 1960, Standard Specifications, Revised November 1960.

Sonoma County Flood Control and Water Conservation District (SCFCWCD), 1962, Central Sonoma Watershed protection and flood prevention project, Sonoma County, California, Plans for the construction of Matanzas Creek Reservoir, in cooperation with Soil Conservation Service, April 25 to 27.

Sonoma County Flood Control and Water Conservation District (SCFCWCD), 1962, Central Sonoma Watershed protection and flood prevention project, Sonoma County, California, Matanzas Creek Reservoir Special Specifications.

Sonoma County Flood Control and Water Conservation District (SCFCWCD), 1963, Central Sonoma Watershed protection and flood prevention project, Sonoma County, California, Matanzas Reservoir Materials Tests During Construction.

Sonoma County Flood Control and Water Conservation District (SCFCWCD), 1963, Central Sonoma Watershed protection and flood prevention project, Sonoma County, California, Matanzas Reservoir Soil Tests and Field Logs.

Stevenson, J.C., 1963, Trip report Central Sonoma W.P.P. – Matanzas Creek Dam: United States Government memorandum to F.K. Muceus.

Woodward-Clyde-Sherrard and Associates, 1959, Analysis of preliminary design and geological studies, Matanzas Creek Dam, Central Sonoma Watershed Project, Sonoma County, California; consultant's report prepared for U.S. Department of Agriculture, Soil Conservation Service, , 239 College Ave., Santa Rosa, California, November 6.

Woodward-Clyde-Sherrard and Associates, 1960, Supplemental field and laboratory investigation in principal and emergency spillway areas, Matanzas Creek Dam: consultant's report prepared for U.S. Dept. of Agriculture, Soil Conservation Service, 239 College Ave., Santa Rosa, California, July 12.

Woodward-Clyde-Sherrard and Associates, 1963, Drilling and Testing, Matanzas Creek Dam, Santa Rosa, California: consultant's report prepared for U.S. Dept. of Agriculture, Soil Conservation District, 2020 Milvia St., Berkeley, California, July 24.

3. Geologic and Seismotectonic Setting

This section provides an overview of the geologic and seismotectonic setting in the region and in the vicinity of Matanzas Creek Dam. This is an abbreviated summary, and more details can be found in the Geologic Investigation Report (GIR) (JDE and AECOM, 2022). A summary geologic map is shown in Figure 3-1 and fault locations are shown in Figure 3-2.

3.1 Site Setting

Matanzas Creek Dam is oriented across the north/northwest-flowing Matanzas Creek channel. The creek flows within an incised channel along the southwest edge of Bennett Valley; the east-facing slope of Taylor Mountain rises west of the creek and Bennett Valley lies east of the creek. A hill underlain by volcanic rocks creates the right abutment of the dam within Bennett Valley.

The drainage basin for Matanzas Creek reservoir has a history of generating sediment. On several occasions when the reservoir is dry in late summer and fall, this sediment has been removed to maintain flood control reservoir storage capacity. Between times of sediment removal, a delta forms and gradually extends farther into the reservoir from the upstream inlet of Matanzas Creek.

Published USGS mapping indicates the site is underlain by the sandstone, shale, and mudstone of the late Tertiary-age Petaluma Formation, and andesite, basalt, and tuff of the late Tertiary-age Sonoma Volcanics Group (McLaughlin et al., 2008). Rocks of the Petaluma Formation and Sonoma Volcanics Group unconformably overlie the much older Mesozoic basement rocks that do not crop out at or near Matanzas Creek Dam. The earthfill embankment appears to be constructed on relatively hard resistant andesite and basalt on the right abutment and on landslide deposits formed of the same volcanic material on the left abutment (McLaughlin et al., 2008). Alluvial deposits have been mapped upstream and downstream of the dam in the valley bottom.

Landslides have been mapped on most of the east-facing slope west of the reservoir and dam at the left abutment (McLaughlin et al., 2008; Huffman and Armstrong, 1980). These landslides encompass most of the western reservoir shoreline and extend upslope westward to near the ridge line of Taylor Mountain. It should be noted that this mapping was based upon large-scale regional interpretation and aerial photo review and not on detailed site-specific mapping or subsurface investigations. The dam's left abutment is believed to have been constructed on landslide debris and colluvium. One recent landslide was noted up slope of the right abutment and the auxiliary spillway during geologic reconnaissance visits but is relatively small – roughly 20-feet wide.

3.2 Seismotectonic Setting

The seismotectonic setting of the San Francisco Bay region is controlled by its location within the plate boundary zone between the Pacific and North American tectonic plates where the Pacific plate, to the west is traveling northwest relative to the North American plate at a rate of about 50 mm/yr (deMets et al., 2010). Most of this plate motion is accommodated on a series of strike-slip faults that constitute the San Andreas Fault System, that in this part of northern California include the San Andreas, the Hayward, Calaveras, Rodgers Creek, and Maacama faults. The project site is located approximately 34 kilometers northeast of the San Andreas fault (USGS, 2020). The Rodgers Creek along with the Healdsburg and Maacama faults are considered to be the northern extensions of the active right-lateral Hayward Fault System and represent the most significant earthquake hazard east of the San Andreas Fault in the northern San Francisco Bay region.

The Rodgers Creek fault (also called the Healdsburg-Rodgers Creek fault) is 82 km long, and its geomorphic expression is similar to that of the Hayward fault with small-scale (meters to hundreds of meters) right steps in drainage channels and ridgelines (McLaughlin et al., 2008). The nearest trace within the principal displacement zone is mapped by USGS about 1,000 meters southeast of Matanzas Creek Dam (Hecker and Randolph Loar, 2018). The calculated slip rate for the Rodgers Creek fault is 9 ± 2 mm/year (Schwartz et al., 1992).

The Bennett Valley fault is also a sinuous, braided strike-slip fault within the San Andreas Fault System that is mapped about 1,400 meters north-northeast of the project site (USGS, 2020). This fault zone extends for approximately 30 km southeastward from the Spring Valley Reservoir (Spring Lake) area in eastern Santa Rosa to Sonoma Valley. Bennett Valley fault and associated unnamed faults are associated with northeastward partitioning of slip between the Rodgers Creek and the Maacama Fault Zones (McLaughlin et al., 2008).

The Matanzas Creek fault has been identified at the southeast end of the reservoir where Petaluma Fm gravels on southwest are juxtaposed against Sonoma Volcanics basaltic andesite on the northeast (McLaughlin et al., 2008). As mapped by McLaughlin et al. (2008), the fault goes beneath the center of Matanzas Creek Dam. However, the fault is not considered active by California Geological Survey (CGS) or USGS and is not included in the USGS Quaternary fault database and is, therefore, not a "…fault that demonstrates geological evidence of coseismic surface deformation in large earthquakes during the Quaternary.".

4. Construction and Dam History

This section summarizes the available data from prior to the current investigation and study including investigations prior to construction of the dam, records from during construction, and significant findings from inspections since completion of the embankment.

Matanzas Creek Dam is an earthen embankment dam located in Sonoma County, California. The dam was part of the Central Sonoma Watershed project, which was developed under the NRCS PL-566 program. Matanzas Creek Dam was constructed in 1964 with the purpose of providing flood control to downstream infrastructure, which includes the City of Santa Rosa. The dam is located approximately five river miles upstream from the confluence of Santa Rosa Creek and Matanzas Creek. Matanzas Creek Dam is currently operated and maintained by Sonoma County Water Agency (Sonoma Water). Table 4-1 provides a summary of parameters for Matanzas Creek Dam.

Table 4-1: As-Built Data on Matanzas Creek Dam

Parameter	Value
Year Completed:	1964
Drainage Area:	11.6 miles^2
Storage Capacity:	1,520 acre-feet (auxiliary spillway crest) 245 acre-feet (principal spillway crest)
Principal Spillway Crest Elevation:	377.6 feet (NAVD88)
Auxiliary Spillway Crest Elevation:	412.6 feet (NAVD88)
Crest Elevation:	427.6 feet (NAVD88)
Height of Dam:	95 feet
Crest Width:	24 feet
Side Slopes:	3H:1V (Horizontal:Vertical) Upstream 2.5H:1V Downstream

The dam includes a structural principal spillway and a vegetated auxiliary spillway. The principal spillway consists of a drop shaft intake structure, 5.25-foot square (inside dimension) reinforced concrete conduit, and outlet structure with concrete chute and stilling basin. The auxiliary spillway consists of an unlined vegetated channel cut into the right abutment of the embankment that transitions to natural grade downstream of the dam centerline and discharges into the present-day creek alignment.

As-builts show that Matanzas Dam consists of a compacted embankment shell on the upstream and downstream sides of the embankment, with a compacted impervious core in the center of the embankment. A gravel drain was installed on the downstream side of the core and along the downstream foundation below the downstream shell. At least 2.5 feet of stripping was performed underneath the embankment before placement of fill with a cutoff trench installed under the core that was excavated to rock. A plan view and typical section from the as-builts is shown in Figure 4-1.

During construction a relief trench was added left of the principal spillway due to the observation of a sand seam beneath the embankment. The relief trench was excavated approximately 11 to 15 feet below the foundation of the rest of the embankment and backfilled with filter material (same as gravel drain material). In addition, several relief wells were installed to relieve potential high pore pressures within the sand seam. Figure 4-2 shows a plan view and typical section from the as-builts that show the relief trench and relief wells.

Further details on the embankment classification is covered in later sections.

4.1 Investigations Prior to Construction

Several investigations were performed prior to construction of the dam, including:

- 1959 Preliminary Design and Geological Studies (Woodward, Clyde, Sherard and Associates)
  - Included several shallow hand auger borings and dozer trenches along the dam axis, seven borings in the vicinity of the dam embankment, 24 borings in possible spillway sites, seven borings at potential borrow areas, as well as excavated trenches in potential spillways, and test pits in potential borrow areas.
  - Laboratory tests were conducted on recompacted soil samples from potential borrow areas, including strength and permeability tests, moisture, density mechanical analyses, Atterberg Limits, and triaxial shear tests. Tests for stability and settlement studies were also conducted on undisturbed soil samples from beneath the proposed embankment. Undisturbed samples were tested for water content, dry density, mechanical analyses, Atterberg Limits, triaxial shear, permeability, and unconfined compressive strength
- 1960 Final Analysis of Design Calculations and Estimates (Woodward, Clyde, Sherard and Associates)
  - A total of seven borings were completed to further investigate the foundation properties of the proposed principal and emergency spillways and the dam embankment.
  - Samples from these borings were tested for moisture content, dry density, unconfined compressive strength, and consolidation tests.
- 1963 Dam Foundation Memorandum (Woodward, Clyde, Sherard and Associates)
  - An additional 12 borings and four relief wells were drilled. Borings downstream of the dam toe were proposed to delineate the extent of sand and “organic ash beds” (later identified as low-grade coal), and to eventually help relieve water pressure that could develop in the foundation of the dam.
  - Laboratory testing of these samples included four permeability tests and four sieve analyses on sand samples, and two triaxial shear tests (one each) on the organic ash and sand samples.
- 1963 Fill Material Testing (USDA-SCS)
  - This investigation was initiated because tests of material sampled during construction (likely from the upstream blanket area) showed low friction angles of 9° to 11°. Laboratory testing on samples from this 1962 to 1963 investigation included gradation, Atterberg limits, moisture content, dry density, strength and permeability, and unconfined compressive strength.
- 1964 Drilling and Testing (Woodward, Clyde, Sherard and Associates)
  - Woodward, Clyde, Sherard and Associates conducted an investigation to further characterize the foundation beneath the dam footprint. Laboratory testing on samples from this 1964 investigation included gradation, moisture content, dry density, permeability, and consolidated and unconsolidated undrained triaxial shear tests.

Data from these reports was digitized and is shown along with data from the most recent investigation on material plots, as discussed in Section 6.

4.2 Specifications, Plans, and Construction Records

Review of the specifications found several requirements for construction of Matanzas Dam that are relevant to evaluation of the embankment:

- Embankment Fill
  - Moisture during placement was required to be plus/minus 2 percent of the optimum water content
  - Compaction was to be performed with a sheepsfoot roller to a density greater than 95 percent of ASTM D698 modified with 40 blows/layer instead of 25 or 56 blows/layer (for method A/B or C, respectively)
  - Coarser material from the borrow was to be placed on the downstream portion of the embankment with clay placed in the clay core
- Filter and Drain Material
  - Material placed in 9-inch list and compacted with at least 4 passes of a tracked 20-ton tractor
  - Filter and drain material had identical specification with the following gradation specification

Sieve Size	Allowable Percent Passing
2.5-inch	100
1.5-inch	80-100
¾-inch	60-95
No. 4	35-65
No. 8	25-50
No. 30	5-25
No. 200	0-3

During construction field density tests and laboratory testing were performed to confirm that the materials and compactive effort met the requires of the specifications. Construction records included the following data:

- Field density tests including moisture content – 187 tests
- Compaction tests (modified ASTM D698) – 149 tests
- Gradation tests, mainly on filter and drain material – 36 tests

Results of the lab testing by the contractor and any appropriate specifications are included on relevant plots of the different materials (see Section 6).

4.3 Performance Since Construction

Performance of the embankment has been documented in recent years through dam inspection reports. Dam inspections by DSOD, HDR/Kleinfelder, and Jones & DeMille Engineering/AECOM are summarized below:

- April 2016 HDR/Kleinfelder Field Inspection
  - No indications of settlement in foundation.
  - Auxiliary spillway was unobstructed and lined with mowed grass.
  - Dike is in good condition.
  - Abutments are in satisfactory condition.
  - Occasional animal burrows found.
  - No signs of deterioration or cracking at the principal spillway inlet structure.
  - Outlet structure is satisfactory with minor concrete degradation.
  - No indications of seepage.

- October 2016 CA DSOD Inspection
  - Required removal of vegetation and driftwood around principal spillway.
  - Required installation of riprap around outlet of principal spillway.

  - No indication of surficial distress or instability on dam faces, crest, and abutment contacts.
  - No indications of seepage.

- July 2018 CA DSOD Inspection
  - Required emerging trees and bushes to be removed from embankment, groins, abutments, and auxiliary spillway annually.
  - Noted that riprap was placed along principal spillway outlet wing walls.
  - 18-inch low-level outlet was partially exercised, but accumulated sediment prevented fully cycling.
  - No indications of seepage.

- August 2019 CA DSOD Inspection
  - Required removal of vegetation within 5 feet of upstream and downstream left groins.
  - Required removal of emerging bushes on downstream face of dam.
  - Noted that emerging trees in auxiliary spillway were removed.
  - No indications of seepage.

- January 2021 Jones & DeMille Engineering/AECOM Inspection
  - Dam in relatively good condition with no items of immediate concern
  - Embankment Observations
    - A few small depressions (less than 6 inches deep) were observed on the downstream face and crest
    - Minor erosion noted in the groins
    - Small animal burrows observed on the lower portions upstream and downstream face, and left groin
  - Intake Structure Observations
    - Some spalling of concrete and corrosion on exposed rebar
    - Slumping of an steep slope next to the intake structure
    - Some sediment and a tree growing in the sediment within the intake structure
  - Outlet Structure and Stilling Basin Observations
    - Joints did not have filling and appeared to have expanded by up to an inch over time
    - Erosion/sloughing observed at the end of the stilling basin
  - Auxiliary Spillway Observations
    - Erosion within a natural drainage feature in the middle of the spillway
    - Some small rodent and gopher holes observed

In addition to the above observations, discussions with Sonoma County staff have indicated that the reservoir tends to silt up over time and has been dredged and cleared in the past. No other past performance issues were noted including after earthquakes, the largest of which was a magnitude 5.7 earthquake approximately 10 kilometers from the site on October 2, 1969. Overall, performance appears to have been good and no major modifications have been made.

5. Geotechnical Investigations

5.1 Subsurface Explorations

The site investigation tasks completed in support of the seismic hazard analysis of the dam embankment consisted of drilling and sampling four borings (601 to 604) through the embankment into the underlying foundation materials and completing eight ER geophysical surveys on the downstream (ER-1 to ER-5) and upstream (ER-6) sides of the dam and transverse to the dam axis (ER-7 and ER-8) as shown on Figure 5-1. Borings 601 and 604 were drilled on the downstream bench to depths of 56.5 and 50.0 feet, respectively. Borings 602 and 603, were drilled on the dam crest and upstream bench to total depths of 130.3 and 111.8 feet, respectively. Geotechnical soil samples were collected in each boring for laboratory testing to assist with the evaluation of the embankment's physical properties and the associated potential seismic vulnerability. Seismic S-wave and P-wave velocities were measured in borings 602 and 603 to assist with the seismic stability assessment. Televiewer survey was also performed in boring 603, the boring that went deepest into the underlying foundation materials, to produce a continuous image of the borehole walls and record discontinuity and geotechnical data throughout the borehole.

The embankment portion of the 600-series borings was drilled with augers and without the utilization of drilling fluid to avoid the risk of hydrofracturing the existing embankment. Once each boring was advanced through the embankment and any unconsolidated native materials and the underlying rock had been confirmed, a steel casing was installed and driven at least 1 foot or to refusal into the foundation to act as a conductor for the drilling fluid used to advance the core drilling through the foundation. The conductor casing prevented hydraulic pressure associated with pumping the drilling fluid from impacting the embankment.

Additional details and the boring logs can be found in the Geologic Investigation Report (JDE and AECOM, 2022).

5.2 Laboratory Testing

Representative samples obtained from the exploratory borings were tested by Inspection Services Inc. (ISI) in Berkeley, California and Cooper Testing Labs in Palo Alto, California. The following geotechnical tests were performed:

- Atterberg Limits (liquid and plastic limits tests) – ASTM D4318
- Moisture Content – ASTM D2116
- Moisture-Density – ASTM D7263
- Chunk Density – ASTM D7263
- Percent Fines – ASTM D1140
- Particle Size Distribution (Gradation) – ASTM D6913
- Particle Size Distribution (Gradation) Using Hydrometer – ASTM D7928
- Crumb Test – ASTM D6572
- Double Hydrometer – ASTM D4221
- Pinhole Dispersion – ASTM D4647
- Fully Softened Torsional Ring Strength – ASTM D7608
- Residual Torsional Ring Strength – ASTM D6467
- Unconfined Compressive Strength – ASTM D7012
- Isotopically Consolidated Undrained Triaxial Strength – ASTM D4767

A summary of the results of the laboratory tests are shown in Table 5-1 and Table 5-2 and discussed in Section 6.

Table 5-1. Historic Laboratory Test Data

Sample Information							Moisture-Density			Atterberg Limit			Sieve Summary				Dispersiveness	Strength Test		
Boring	Sample ID	Top Depth (feet)	Sampler Type	Field Classification	Interpreted Unit	Field Blowcount (bpf)	Water Content (%)	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	LL	PL	PI	Percent Gravel	Percent Sand	Percent Fines	Percent Clay		Type	Confining Stress (ksf)	Max Deviator (ksf)
01-63	NA	1			Unknown		40	106	76											
01-63	NA	2			Unknown		155	173	68											
04-59	4-4	16.5	SPTO	CL	Surficial Deposit	48							55	33	12					
04-59	4-5	21.5	SPTO	CL	Surficial Deposit	67							56	24	20					
04-59	4-6	26.5	SPTO	GC	Surficial Deposit	144							61	23	16					
04-63	NA	2			Unknown		23	121	98											
04-63	NA	34			Unknown								6	88	6					
05-59	5-2	6.5	SPTO	GC	Surficial Deposit	43							18	28	54					
06-59	6-4	16.5	SPTO	Tuff	Sonoma Volcanics	27	36	113	83											
06-63	NA	28			Unknown		69	89	53											
08-63	NA	9			Unknown								2	77	21			TX-CU	2	19
10-59	10-3	11.5	SPTO	GC	Surficial Deposit	105							36	30	34					
10-59	10-4	16.5	SPTO	GC	Surficial Deposit	51	27	121	95											
10-59	10-7	31.5	SPTO	CL	Surficial Deposit	25	42	112	79											
10-59	10-8	41.5	SPTO	Shale	Sonoma Volcanics	27	22	129	106											
10-63	NA	1			Unknown		19	104	87											
10-63	NA	14			Unknown								2	90	8			TX-CU	4	28
10-63	NA	33			Unknown		102	87	43											
11-59	11-3	11.5	SPTO	CL	Surficial Deposit	15	33	116	87											
12-59	12-2	6.5	SPTO	Tuff	Sonoma Volcanics	33	28	115	90											
12-59	12-3	11.5	SPTO	Tuff	Sonoma Volcanics	26	38	115	83									TX-CU	2,8	6,18

Sample Information							Moisture-Density			Atterberg Limit			Sieve Summary				Dispersiveness	Strength Test		
Boring	Sample ID	Top Depth (feet)	Sampler Type	Field Classification	Interpreted Unit	Field Blowcount (bpf)	Water Content (%)	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	LL	PL	PI	Percent Gravel	Percent Sand	Percent Fines	Percent Clay		Type	Confining Stress (ksf)	Max Deviator (ksf)
12-59	12-4	16.5	SPTO	Tuff	Sonoma Volcanics	28	21	128	106									TX-CU	4,16	12,17
12-59	12-5	21.5	SPTO	Tuff	Sonoma Volcanics	46	12	120	107											
12-59	12-6	26.5	SPTO	Tuff	Sonoma Volcanics	66	11	123	111											
12-59	12-7	31.5	SPTO	Tuff	Sonoma Volcanics	24	8	107	99											
12-59	12-8	36.5	SPTO	Tuff	Sonoma Volcanics	38	16	122	105											
12-59	12-9	46.5	MCO	Tuff	Sonoma Volcanics	100	51	107	71											
12-63	NA	43			Unknown		85	88	48											
13-59	13-1	6	SPTO	Volcanic Sand	Sonoma Volcanics	52	66	98	59											
13-59	13-2	11	SPTO	Volcanic Sand	Sonoma Volcanics	48	81	94	52									TX-CU	2	15
13-59	13-3	16	SPTO	Volcanic Sand	Sonoma Volcanics	80	51	98	65									TX-CU	4	18
13-59	13-4	21	SPTO	Volcanic Sand	Sonoma Volcanics	200	60	98	61											
13-59	13-5	26	SPTO	Volcanic Sand	Sonoma Volcanics	360	41	103	73									TX-CU	8	41
14-59	14-2	9.5	SPTO	CL	Surficial Deposit	39	34	115	86				41	36	23			TX-CD	2	6
14-59	14-3	14	SPTO	CL	Surficial Deposit	32	41	97	69											
14-59	14-4	19.5	SPTO	CL	Surficial Deposit	31	32	114	86											
14-59	14-6	29.5	SPTO	CL	Surficial Deposit	40	25	110	88											
14-59	14-7	34.5	SPTO	CL	Surficial Deposit	40	21	122	101											
14-59	14-8	41.5	SPTO	CL	Surficial Deposit	49	23	116	94											
15-59	15-1	1.5	SPTO	CL	Surficial Deposit	32	21	113	93											
15-59	15-2	6.5	SPTO	CL	Surficial Deposit	42	21	113	93											
15-59	15-4	16.5	SPTO	CL	Surficial Deposit	50	19	105	88				70	21	9					
15-59	15-5	21.5	SPTO	CL	Surficial Deposit	42	29	119	92											
15-59	15-6	25.5	SPTO	Tuff	Sonoma Volcanics	70	28	116	91											

Sample Information							Moisture-Density			Atterberg Limit			Sieve Summary				Dispersiveness	Strength Test		
Boring	Sample ID	Top Depth (feet)	Sampler Type	Field Classification	Interpreted Unit	Field Blowcount (bpf)	Water Content (%)	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	LL	PL	PI	Percent Gravel	Percent Sand	Percent Fines	Percent Clay		Type	Confining Stress (ksf)	Max Deviator (ksf)
15-59	15-7	31.5	SPTO	Tuff	Sonoma Volcanics	100	22													
16-59	16-3	11.5	SPTO	Tuff	Sonoma Volcanics	31	33	102	77											
18-59	18-2	6.5	SPTO	GM	Surficial Deposit	35	25	123	98											
18-59	18-3	11.5	SPTO	Basalt	Sonoma Volcanics	90	27	110	87											
19-59	19-1	4.5	SPTO	CL	Surficial Deposit	42	37	118	86									TX-CD	8	10
19-59	19-2	9.5	SPTO	CL	Surficial Deposit	42	23	118	96									TX-CD	4	6
19-59	19-4	21.5	SPTO	Basalt	Sonoma Volcanics	55	41	102	72											
20-59	20-1	4.5	SPTO	Tuff	Sonoma Volcanics	42							24	49	27					
20-59	20-2	12.5	SPTO	Tuff	Sonoma Volcanics	70	27	126	99									TX-CD	2,4	9,14
20-59	20-3	21.5	SPTO	Tuff	Sonoma Volcanics	57	21	129	107									TX-CD	12	24
20-59	20-4	31.5	SPTO	Breccia	Sonoma Volcanics	90	19	133	112											
20-59	20-5	36.5	SPTO	Breccia	Sonoma Volcanics	80	19	136	114											
20-59	20-6	47.5	SPTO	Breccia	Sonoma Volcanics	60	26	125	99											
21-59	21-1	1.5	SPTO	Tuff	Sonoma Volcanics	33	34	96	72											
21-59	21-2	6.5	SPTO	Tuff	Sonoma Volcanics	50	21	113	93									TX-CD	8	13
21-59	21-3	11.5	SPTO	Tuff	Sonoma Volcanics	38	15	83	72									TX-CD	2,4	9,8
22-59	22-2	12	SPTO	Tuff	Sonoma Volcanics	9	30	117	90											
22-59	22-4	19.5	SPTO	Tuff	Sonoma Volcanics								59	29	12					
22-59	22-5	33	SPTO	Basalt	Sonoma Volcanics	25	54	103	67											
22-63	22-1	7.5	SPTO	Breccia	Sonoma Volcanics	40												TX-CU	1	9
30-60	1	37	SPTO	Rock Fragments	Other	80							45	49	6					
30-60	2	42	SPTO	Rock Fragments	Other	340							37	27	36					
30-60	3	47	SPTO	Rock Fragments	Other	370							34	58	8					

Sample Information							Moisture-Density			Atterberg Limit			Sieve Summary				Dispersiveness	Strength Test		
Boring	Sample ID	Top Depth (feet)	Sampler Type	Field Classification	Interpreted Unit	Field Blowcount (bpf)	Water Content (%)	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	LL	PL	PI	Percent Gravel	Percent Sand	Percent Fines	Percent Clay		Type	Confining Stress (ksf)	Max Deviator (ksf)
31-60	1	2	SPTO	CL	Surficial Deposit	30	22	117	96											
31-60	2	5	SPTO	ML	Surficial Deposit	24	15	71	62											
31-60	3	8	SPTO	CL	Surficial Deposit	48	24	128	103											
31-60	4	13	SPTO	CL	Surficial Deposit	60	24	128	103											
31-60	5	18	SPTO	ML	Surficial Deposit	42	31	119	91											
31-60	6	23	SPTO	ML	Surficial Deposit	40	37	115	84											
31-60	7	28	SPTO	ML	Surficial Deposit	118	46	110	75											
32-60	1	2	SPTO	CL	Surficial Deposit	54	44	94	65											
32-60	2	7	SPTO	Tuff	Sonoma Volcanics	184	28	93	73											
32-60	3	12.5	SPTO	Tuff	Sonoma Volcanics	110	36	87	64											
32-60	4	17	SPTO	Tuff	Sonoma Volcanics	510	33													
33-60	1	8	SPTO	ML	Surficial Deposit	32	53	96	63											
33-60	2	11.5	SPTO	ML	Surficial Deposit	140	75	82	47											
33-60	3	16.5	SPTO	ML	Surficial Deposit	300	56	94	60											
35-60	1	2.5	SPTO	CL	Surficial Deposit	24	37	116	85											
35-60	2	7	SPTO	SM	Surficial Deposit	84	22	109	89											
35-60	3	11.5	SPTO	GC	Surficial Deposit		16	137	118				45	43	12					
35-60	4	18	SPTO	Sandstone	Sonoma Volcanics	320	27	136	107											
35-60	5	23	SPTO	Sandstone	Sonoma Volcanics	220	17	130	111											
B-1-59	Grab1	0			Borrow					63	27	36	2	6	92	48				
B-1-59	Grab2	14			Borrow					58	28	30								
B-1-59	Grab3	24			Borrow					77	32	45								
B-2-59	Grab1	0			Borrow					51	23	28	5	18	77	39				

Sample Information							Moisture-Density			Atterberg Limit			Sieve Summary				Dispersiveness	Strength Test		
Boring	Sample ID	Top Depth (feet)	Sampler Type	Field Classification	Interpreted Unit	Field Blowcount (bpf)	Water Content (%)	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	LL	PL	PI	Percent Gravel	Percent Sand	Percent Fines	Percent Clay		Type	Confining Stress (ksf)	Max Deviator (ksf)
B-2-59	Grab2	7			Borrow					58	25	33	11	23	66	31				
B-3-59	Grab1	15.5			Borrow					58	27	31	43	29	28	10				
B-4-59	Grab2	0			Borrow					45	21	24	4	44	52	25				
B-4-59	Grab1	12			Borrow					63	26	37	0	8	92	53				
B-5-59	Grab1	4			Borrow					NP	NP	NP	4	50	46	13				
Composite A	A	0			Borrow													TX-CD TX-CU	CD: 2,8,12,16,19 CU: 2,8,12,19	CD: 8,10,11,14,12 CU: 3,6,8,11
Composite B	B	0			Borrow													TX-CD TX-CU	CD: 4,6,8 CU: 2,4,8,12,16	CD: 3,4,5 CU: 3,5,6,7,8
Composite C	C	0			Borrow								3	56	41			TX-CD TX-CU	CD: 4,8,12 CU: 4,9,12	CD: 7,12,16 CU: 6,10,11
DH-1-59	1	0			Borrow					62	26	36	2	20	78	43				
DH-1-59	2	3.5			Borrow					59	28	31	9	23	68	37				
DH-1-59	3	9			Borrow					58	25	33	13	21	66	35				
DH-4-59	1	0			Borrow					56	35	21	29	31	40					
DH-6-59	1	0			Borrow					59	26	33	0	19	81	50				
L-2-59	L2-1	12	SPTO	Tuff	Sonoma Volcanics	49	46	77	53											
L-2-59	L2-2	17	SPTO	Tuff	Sonoma Volcanics	75	25	89	71											
L-2-59	L2-3	22	SPTO	Tuff	Sonoma Volcanics	50	55	82	53											
L-2-59	L2-4	27	SPTO	Tuff	Sonoma Volcanics	33	46	99	68											
L-2-59	L2-5	36.5	SPTO	Tuff	Sonoma Volcanics	70	76	90	51											
L-2-59	L2-6	46.5	SPTO	Tuff	Sonoma Volcanics	88	50	102	68											

Table 5-2. Laboratory Test Data from This Study

Sample Information							Moisture-Density			Atterberg Limit			Sieve Summary				Dispersiveness	Strength Test		
Boring	Sample ID	Top Depth (feet)	Sampler Type	Field Classification	Interpreted Unit	Field Blowcount (bpf)	Water Content (%)	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	LL	PL	PI	Percent Gravel	Percent Sand	Percent Fines	Percent Clay		Type	Confining Stress (ksf)	Max Deviator (ksf)
601	1-1	2.5	SPTO	CL	Fill	26	20													
601	2-1	5	SPTO	CL	Fill	17	18								35					
601	3-1	7.5	SPTO	CL	Fill	15	27			60	33	27	34	35	31	14	Double Hydrometer: Intermediate Dispersiveness			
601	5-1	12.5	SPTO	CL	Fill	14	31								39					
601	6-1	15	SPTO	CL	Fill	27	27													
601	7-1	17.5	SPTO	CL	Fill	21	24			64	29	35	45	27	28	12				
601	8-1	20	SPTO	CL	Fill	26	29													
601	9-1	22.5	SPTO	CL	Fill	18	34										Double Hydrometer: Nondispersive			
601	11-1	27.5	SPTO	CL	Fill	21	25								33					
601	14-1	35	SPTO	CL	Fill	14	33													
601	15-1	37.5	SPTO	SM	Gravel Drain	46	14						29	48	23					
601	16-1	40	SPTO	SM	Gravel Drain	26	7						31	63	6					
601	17-1	42.5	SPTO	SM	Gravel Drain	38	18						40	42	18					
601	18-1	45	SPTO	CL	Alluvium	56	17								16					
601	R1	46.7	CORE	GC	Alluvium								92	5	3					
601	R4	54	CORE	Tuff	Sonoma Volcanics		6	60	56											
602	1-1	5	SPTO	CL	Fill	14	32													
602	2-1	10	SPTO	CL	Fill	23	21						38	33	29					
602	3-1	15	SPTO	CL	Fill	50	29			55	32	23								
602	4-1	20	SPTO	CL	Fill	63	10													

Sample Information							Moisture-Density			Atterberg Limit			Sieve Summary				Dispersiveness	Strength Test		
Boring	Sample ID	Top Depth (feet)	Sampler Type	Field Classification	Interpreted Unit	Field Blowcount (bpf)	Water Content (%)	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	LL	PL	PI	Percent Gravel	Percent Sand	Percent Fines	Percent Clay		Type	Confining Stress (ksf)	Max Deviator (ksf)
602	5-1	25	SPTO	CL	Fill	22	33			58	26	32					Crumb Test: Nondispersive			
602	6-1	30	SPTO	CL	Fill	13	30													
602	7-1	35	SPTO	CL	Fill	21	27													
602	8-1	40	CAO	GC	Fill	30	24										Double Hydrometer: Nondispersive			
602	9-1	45	CAO	GC/CL	Fill	34	28			51	22	29					Crumb Test: Dispersive			
602	10-1	50	CAO	CL	Fill	35	16													
602	11-1	55	CAO	CL	Fill	33	25			55	24	31	8	25	67	34	Crumb Test: Highly Dispersive			
602	12-1	60	CAO	CL	Fill	43	26													
602	13-1	65	CAO	CL	Fill	52	17			61	28	33	44	28	28	0	Crumb Test: Highly Dispersive			
602	14-1	70	CAO	CL	Fill	60	21			65	31	34	56	19	25		Crumb Test: Highly Dispersive			
602	15-1	80	CAO	CL	Fill	30	27			62	23	39	41	22	37	15	Crumb Test: Highly Dispersive			
602	16-1	85	CAO	CL	Fill	44	31													
602	17-1	90	CAO	CL	Fill	43	28													
602	R1	91.5	CORE	CL	Fill					56	22	34								
602	R7	110.3	CORE	Pumiceous Tuff	Sonoma Volcanics								3	72	25					
603	4	10	SPTO	CL	Fill	27	20						22	34	44	18				
603	5	15	SPTO	CL	Fill	17	28													
603	6	20	SPTO	CL	Fill	31	24			52	29	23					Crumb Test: Dispersive			
603	7	25	SPTO	CL	Fill	19	25			54	30	24			44		Crumb Test: Highly Dispersive			
603	8	30	SPTO	CL	Fill	14	28								36					
603	9	32.5	SPTO	SM	Alluvium	26	23						32	32	36					
603	10	35	SPTO	SC	Alluvium	18	31								61					

Sample Information							Moisture-Density			Atterberg Limit			Sieve Summary				Dispersiveness	Strength Test		
Boring	Sample ID	Top Depth (feet)	Sampler Type	Field Classification	Interpreted Unit	Field Blowcount (bpf)	Water Content (%)	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	LL	PL	PI	Percent Gravel	Percent Sand	Percent Fines	Percent Clay		Type	Confining Stress (ksf)	Max Deviator (ksf)
603	11	40	SPTO	Tuff	Sonoma Volcanics	43	56													
603	12	42.5	SPTO	Pumiceous Tuff	Sonoma Volcanics	35	43													
603	13	45	SPTO	Pumiceous Tuff	Sonoma Volcanics	54	48													
603	R4	51.8	CORE	Pumiceous Tuff	Sonoma Volcanics													UC	0	18
603	R7	66.8	CORE	Pumiceous Tuff	Sonoma Volcanics		1	79	78				6	78	16					
603	R9	76.8	CORE	Pumiceous Tuff & Mudstone	Sonoma Volcanics								2	57	41					
603	R13	96.8	CORE	Pumiceous Tuff	Sonoma Volcanics													UC	0	4
603	R14	101.8	CORE	Pumiceous Tuff	Sonoma Volcanics													UC	0	8
604	1	4	CAO	CL	Fill	39	17	105	90											
604	3	15	CAO	CL	Fill	32	26	95	75											
604	5	25	CAO	CL	Fill	29	37	53	39											
604	6	30	CAO	CL	Fill	18	16													
604	7	32.5	SPTO	CL	Fill	7	30													
604	8	37.5	SPTO	CL	Fill	12	44													
604	9	40	SPTO	SW	Gravel Drain	36	34													
604	10	42.5	SPTO	SW	Gravel Drain	46	3													
604	11	45	CAO	SW	Gravel Drain	64	3	132	128				39	56	5					
604	12	48.5	CAO	Tuff	Sonoma Volcanics	90												UC	0	15
701	R2	12	CORE	GC	Colluvium/Landslide								38	27	35	11				
701	R5	19.5	CORE	CL	Colluvium/Landslide					39	20	19	3	29	68	23				
701	R10	34.5	CORE	CL	Colluvium/Landslide					64	22	42	0	8	92	49				
701	R12	39.5	CORE	CL	Colluvium/Landslide					31	15	16	7	45	48	18				

Sample Information							Moisture-Density			Atterberg Limit			Sieve Summary				Dispersiveness	Strength Test		
Boring	Sample ID	Top Depth (feet)	Sampler Type	Field Classification	Interpreted Unit	Field Blowcount (bpf)	Water Content (%)	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	LL	PL	PI	Percent Gravel	Percent Sand	Percent Fines	Percent Clay		Type	Confining Stress (ksf)	Max Deviator (ksf)
701	R17	54.5	CORE	Tuff	Sonoma Volcanics													UC	0	24
701	R21	72	CORE	Tuff	Sonoma Volcanics													UC	0	20
701	R28	107	CORE	Sandy Tuff	Sonoma Volcanics		9	66	61				5	58	37					
702	R4	11.5	CORE	GC	Colluvium/Landslide		9	124	114	72	29	43	50	19	31	13				
702	R7	21.5	CORE	SM	Alluvium					31	13	18	4	57	39	13				
702	R9	31.5	CORE	Pumice	Sonoma Volcanics		6	54	51											
702	R18	64.3	CORE	Pumice	Sonoma Volcanics													UC	0	4
702	R19	66.5	CORE	Pumice	Sonoma Volcanics													UC	0	4
702	R22	81.5	CORE	Pumice	Sonoma Volcanics		17	71	61				2	59	39					
703	R3	10	CORE	GW-GM/GM	Colluvium/Landslide		5	123	117	60	32	28	35	27	38	13				
703	R7	19	CORE	CL	Colluvium/Landslide					64	26	38			76					
703	R11	29	CORE	CL	Colluvium/Landslide					76	25	51	3	23	74	36				
703	R14	36.5	CORE	CL	Colluvium/Landslide					83	32	51			48					
703	R21	54	CORE	CL	Colluvium/Landslide		6	114	108	68	28	40	16	26	58	25				
703	R28	71.5	CORE	Basalt	Sonoma Volcanics													UC	0	304
704	R2	7.5	CORE	CL	Colluvium/Landslide					47	31	16	16	44	40	9				
704	R6	16.5	CORE	CL	Colluvium/Landslide					74	31	43	13	19	68					
704	R12	31.5	CORE	CL	Colluvium/Landslide		12	112	100	87	25	62	1	7	92	60				
704	R18	46.5	CORE	CL	Colluvium/Landslide					94	35	59	6	39	55	27				
704	R23	59	CORE	CL	Colluvium/Landslide		11	129	116	73	26	47	0	20	80	45				
704	R28	71.5	CORE	CL	Colluvium/Landslide		3	129	125	44	19	25	27	38	35	13				
704	R34	86.5	CORE	GC	Colluvium/Landslide					36	17	19	19	36	45	15				
704	R40	109	CORE	Pumiceous Tuff	Sonoma Volcanics													UC	0	7

Sample Information							Moisture-Density			Atterberg Limit			Sieve Summary				Dispersiveness	Strength Test		
Boring	Sample ID	Top Depth (feet)	Sampler Type	Field Classification	Interpreted Unit	Field Blowcount (bpf)	Water Content (%)	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	LL	PL	PI	Percent Gravel	Percent Sand	Percent Fines	Percent Clay		Type	Confining Stress (ksf)	Max Deviator (ksf)
Composite Samples	1	1	GRAB		Fill					63	24	39	38	31	31	15	Pinhole: Slightly Dispersive	TX-CU	2,4,8	2,4,7
Composite Samples	2	2	GRAB		Fill					64	30	34	47	21	32	15	Pinhole: Slightly Dispersive			
Composite Samples	3	3	GRAB		Colluvium/Landslide					58	32	26	61	13	26	8				
Composite Samples	4	4	GRAB		Colluvium/Landslide					52	20	32	2	34	64	28		TX-CU, Ring Torsion	3,6	5,7
Composite Samples	5	5	GRAB		Colluvium/Landslide					79	30	49	32	29	39	20		Ring Torsion		
Composite Samples	7	7	GRAB		Fill					62	26	36	39	15	46	21				
Composite Samples	8	8	GRAB		Colluvium/Landslide					34	16	18	33	32	35	12				

6. Material Characterization and Properties for Analysis

6.1 Cross Section Development

Zoning and selection of material properties for the different soil and rock encountered were completed based on the historical explorations and explorations completed as part of this study along with any associated lab testing. In addition, observations and records from during construction of the embankment were used to inform our decisions. Based on the data, the subsurface materials in the vicinity of the embankment consist of embankment fill, gravel drain/filter material, alluvium, landslide debris/colluvium, and Sonoma Volcanics rock. In addition, a layer of organic mudstone/low grade coal was observed beneath the embankment. Lake sediment is also assumed to exist on the upstream side of the embankment based on the difference in ground surface between what's shown in the as-builts and the 2021 survey. No information was gathered on the lake sediment so typical properties from similar properties were assumed for analysis.

An embankment section was developed for analysis as part of this report, with the location shown on Figure 6-1. The embankment section location was chosen based on the location of explorations performed as part of this study, where the maximum height of the embankment occurs. In addition to the analysis of the maximum section, a modified maximum section was produced to examine the stability of the embankment in the vicinity of the relief trench. Figure 6-2 shows the developed cross sections for the maximum section of the embankment with green lines showing the approximate location of the relief trench. With limited recent investigations for the current embankment, there are some uncertainties. The two main uncertainties in the embankment cross sections are:

1) The extent of the alluvium underneath the embankment. Borings performed as part of this project indicated pockets of alluvium underneath the embankment that were not noted in as-builts or in pre-construction explorations. Further explorations would help refine the extent of the alluvium.

2) The lake sediment shown at the upstream toe of the embankment may not be present. The ground surface shown is based on recent LiDAR while the embankment geometry is based on as-built records. When comparing the elevation of the upstream toe in the as-built records and the upstream toe in the LiDAR, there was discrepancy, which was filled in with lake sediment for the purpose of this analysis. It is also possible that either a) there is some material other than lake sediment located at the upstream toe, such as waste material, that was not shown in the as-builts, or b) the as-built section is inaccurate or not applicable for the section selected for analysis. Explorations at the toe of the embankment would provide information to better model this area of the embankment.

One landslide cross-section was developed for analysis, the location of which is shown on Figure 6-1. Figure 6-3 shows the developed cross section for the landslide section. The location of the section was selected based on the location of the axis of the dam and the mapped location of the landslide. The purpose of this section is to check if the mapped landslide may push on the left abutment of the embankment and cause cracking or deformation that may impact the stability of the dam. There is a decrease in the elevation of the landslide at boring 703 that was flattened for analysis (see Section 9). It is possible that the landslide shown in the section is actually two or more overlapping landslides overlapping around boring 703, but there is no surface evidence to support this, so a larger single slide plane was analyzed.

The cross sections were developed using recent survey data, as-built plans and elevations, and both historic and recent exploration data. For analysis, the landslide section was modified slightly to create a more typical slip surface shape and a lower factor of safety.

6.2 Embankment Materials

Based on the as-builts and specifications, the embankment for Matanzas Dam consists of a compacted clay core with compacted shells on the upstream and downstream side of the core. Between the downstream shell and the clay core is a gravel grain, which also runs underneath the downstream shell. In addition, the specifications indicate that the material placed on the upstream side of the embankment should be finer than the material placed on the downstream side of the embankment.

Based on the results of index testing shown in Figure 6-4 to 6-10 (and discussed further below), the embankment appears relatively homogenous with most of the data falling within a similar range. Based on the field borings logs for boring 603 and the gradation and percent fines tests performed on that boring, it appears the upstream shell may be slightly finer compared to the downstream shell and core. However, the data is limited and still falls within the scatter from the other explorations, so the embankment fill was treated as a single homogenous material.

6.2.1 Embankment Fill

6.2.1.1 Index Testing and Density Summary

Index test results are shown in Figures 6-4 to 6-10. As shown in those plots, the embankment fill generally consists of a clayey gravel or sand with approximately equal parts of gravel, sand, and clay. Fines generally classify as a fat clay (CH) with a few tests classified as high plasticity silts (MH). Water contents of the embankment fill are approximately 25 percent with no significant difference between the upstream portions of the dam and the downstream portion of the dam. Moisture contents from construction averaged about 30 percent, slightly higher than the current water content. Water contents are generally near or below the plastic limits, with none of the water contents being greater than halfway between the plastic and liquid limits. Three unit weight tests were performed as part of this study, all from boring 604. In addition, unit weight tests from during construction are available and shown on Figure 6-9 and 6-10. The tests from during construction indicate that the moist unit weight of the embankment fill generally varies between 95 and 130 pcf, with an average of about 115 pcf. Similarly, dry unit weights generally vary between 80 and 100 pcf with an average of about 90 pcf. Tests performed as part of this study were on the low end of construction data and one wet unit weight of about 53 pcf was found. It is likely that the 53 pcf wet unit weight is an outlier or an error.

Based on the results of index testing and construction data, a moist unit weight of 115 pcf was selected for analysis. While the data from this study indicated a lower unit weight, there is significantly more unit weight data from construction and the method to obtain that data was likely more accurate. The permeability of the unit was estimated using an average gradation, the dry unit weight of the soil, and relations by Kozeny-Carmen as presented in Charbeneau (2000), and a relation by Hazen (1982). In addition, based on the gradation and plasticity of the fill, the material was assumed to be non-free draining during rapid drawdown and seismic shaking. Material properties are summarized in Table 6-1.

Table 6-1: Summary of Selected Material Parameters

Material	Moist Unit Weight (pcf)	Representative Blowcount, $(N_1)_{60}$	Strength Parameters						Permeability Parameters			Seismic Parameters	
			Effective Strengths		Total Strengths				Porosity, n	Horizontal Permeability, k_h (cm/s)	Anisotropy Coefficient, k_h/k_v	Shear Wave Velocity or K2Max	Residual Strength (psf)
			ϕ' (deg)	C' (psf)	ϕ (deg)	C (psf)	$\psi_{Kc=1}$ (deg)	$d_{kc=1}$ (psf)					
Embankment Fill	115	18	29	0	15	200	18	150	0.466	$3x10^{-5}$	4	1200 ft/s	S_r/σ'_{vc} = 0.15 Min. = 100 psf
Filter/Drain	130	26	35 Sens. = 29	0	Drained Only				0.406	$5x10^{-3}$	1	1200 ft/s	S_r/σ'_{vc} = 0.25 (sens. = 0.15) Min. = 100 psf
Alluvium	120	25	35	0	15	200	18	150	0.406	$2x10^{-4}$	10	1200 ft/s	S_r/σ'_{vc} = 0.25 Min. = 100 psf
Sonoma Volcanics	80	40	Selected: c = 0 psf and ϕ = 39 deg (Blowcount Based) Sensitivity: Hoek-Brown (UCS=12ksf, GSI=35, mi=13) Equivalent Mohr-Coulomb From H-B: ϕ = 20 deg; C = 1000 psf						0.525	$1x10^{-3}$	2	1300 ft/s	Not Susceptible to Liquefaction
Organic Mudstone/Low Grade Coal	Assumed Same Properties as Sonoma Volcanics								0.474	$1x10^{-4}$	1	Assumed Same as Sonoma Volcanics	
Colluvium/Landslide Debris	130	Not Required for Analysis	29 ($\phi_{Fully\ Softened}$ = 21 $\phi_{Residual}$ = 9)	0	19	500	Not Required for Analysis		Not Required For Analysis			1300 ft/s	Water Level Below Bottom of Landslide
Lake Sediment	90	Not Required for Analysis	25	0	K2max = 35		Not Required for Analysis		0.600	$3x10^{-6}$	1	Not Required for Analysis	

6.2.1.2 Blowcount Data and Strength Testing

Blowcount data was collected using Standard Penetration Test (SPT) and Modified California (Mod Cal) samplers. Plots of field blowcounts and SPT equivalent, energy corrected, normalized blowcounts (referred to as $(N_1)_{60}$ herein) as a function of elevation are shown in Figure 6-11 and 6-12, respectively. As shown in the gradation plot (Figure 6-4), significant portions of gravel up to the max size of the sampler were observed in the samples, indicating that gravel interference may be an issue with these blowcounts. Therefore, when selecting a representative blowcount, greater weight was given to the samples with lower blowcounts. Based on the information presented, a representative $(N_1)_{60}$ of 18 was selected for analysis, which corresponds to approximately a 20th percentile value, with only 7 of the 47 recorded blowcounts in the fill being less than that value.

As part of this study three isotropically-consolidated undrained triaxial strength tests were performed on reconstituted samples of fill material, with material coming from multiple samples in all three borings. Reconstituted tests were performed instead of tests on relatively undisturbed samples due to the high gravel content in the embankment and related difficulties in pushing Shelby tubes. The composite sample consisted of material from the samples shown in Table 6-2. A comparison of the gradation and plasticity of the composite sample to the rest of the embankment is shown in Figure 6-4 and Figure 6-8, respectively, and in Table 6-2.

Table 6-2: Embankment Composite Samples

Composite ID	Boring ID	Sample ID	Sample Depths (feet)	Gradation Results			Atterberg Limit Results			USCS
				Gravel	Sand	Fines	PL	LL	PI	
1	602	12, 16, and 17	60, 85, and 90	38%	31%	31%	24	63	39	GC
2	604	2, 3, 4	10, 15, 20	47%	21%	32%	30	64	34	GC
7	602	6 and 7	30 and 35	39%	15%	46%	26	62	36	GC
Embankment Median				**39%**	**26%**	**35%**	**29**	**57**	**28**	**GC**

Notes:
PL = Plastic Limit
LL = Liquid Limit
PI = Plasticity Index
USCS = Unified Soil Classification System

As shown, composite samples 1 and 2 were similar while composite 7 had more fines, although all composite samples were still classified as a clayey gravel. To have a sufficient quantity of material for strength testing, composite samples 1 and 2 were combined prior to testing. The strength tests were reconstituted to a dry unit weight of approximately 90 pcf with a water content of 25 percent. The tests were run at confining pressures of 2000 psf, 4000 psf, and 8000 psf, which for a dry embankment would be approximately 17 ft, 35 ft, and 70 feet below the ground surface.

Effective (drained) and total (undrained) results are shown in Figure 6-13 and Figure 6-14, respectively. Figure 6-15 shows the effective and total strength envelopes using methodology by Duncan et al. (2005), which was used to develop a bilinear envelope for analysis (see Section 8.1). Sample behavior plots showing deviator stress and excess porewater pressure versus axial strain are shown on Figure 6-16 and show contractive materials with positive excess porewater pressures at all strains and confining pressures.

Strength testing results from historic tests (from borrow material prior to construction) and reconstituted strength tests performed as part of this study were used to selected embankment fill strengths. Historic testing involved both consolidated-drained tests and consolidated-undrained tests without pore pressure measurements. For examination of the strengths, the consolidated-drained tests were assumed to be representative of effective strengths while consolidated-undrained tests were representative of total strengths. Based on the strength results, an effective strength with a friction angle of 29 degrees with no cohesion was selected for analysis. The selected effective strength has a higher friction angle but lower cohesion than is apparent from the historic consolidated-drainedtriaxial

tests from the borrow material, although those tests may not have been loaded slowly enough to determine an actual drained strength. A total strength with a friction angle of 15 degrees and a cohesion of 200 psf was selected for analysis, which generally agrees with the historic consolidated undrained triaxial test results shown. Using methodology by Duncan et al. (2005), a total stress strength envelope with a friction angle of 18 degrees and cohesion of 150 psf was selected for analysis. Selected strength properties are summarized in Table 6-1.

6.2.1.3 Seismic Parameters

Seismic shear wave velocities are shown on Figure 6-17. As shown, the embankment fill shear wave velocity varied from about 800 feet/second to about 1400 feet/second, with an average of about 1200 feet/second and no significant trend with depth. Therefore, a seismic shear wave velocity of 1200 feet/second was selected for analysis. A residual shear strength ratio of 0.15 was selected based on recommendations by Idriss and Boulanger (2008), with a minimum strength of 100 psf.

6.2.2 Gravel Drain/Filter

6.2.2.1 Index Testing and Density Summary

Four gradations tests were performed on samples of the gravel drain and are shown on Figure 6-18 along with gradations from construction and the specification for the gravel drain/filter. As shown, the gravel drain generally consists sand with gravel with 5 to 25 percent fines, between 30 and 40 percent gravel, and maximum particle sizes of approximately 0.75-inch to 1.5-inch. However, it should be noted that it is possible that the maximum particle size and the gradation could be affected by the limited size of the sampler used. Obtaining a more representative sample may result in a significant increase in the coarser portion of the gradation and corresponding reduction in the finer portion of the gradation. No Atterberg limit tests were performed on gravel drain material but the fines were classified in the field as low to non-plastic. Six water content tests were performed, with water contents ranging from 3 to 35 percent, with an average of about 13 percent. One unit weight test was performed as part of this study and had a moist unit weight of 132 pcf and a dry unit weight of 128 pcf.

Based on the results of index testing, blowcount data (discussed below), and our understanding of how the gravel drain/filter was placed, a moist unit weight of 130 pcf was selected for analysis. The permeability of the unit was estimated using an average gradation, the dry unit weight of the soil, and relations by Kozeny-Carmen as presented in Charbeneau (2000), and a relation by Hazen (1982). In addition, based on the gradation and plasticity of the fill, the material was assumed to be free draining during rapid drawdown and seismic shaking. Material properties are summarized in Table 6-1.

6.2.2.2 Blowcount Data and Strength Testing

Blowcount data was collected using SPT and Mod Cal samplers. Six blowcounts were recorded within the gravel drain with $(N_1)_{60}$ values of 26, 36, 38, 46, 46, and 64. As shown in the gradation plot for the gravel drain, significant portions of gravel up to the max size of the sampler were observed in the samples, indicating that gravel interference may be an issue with these blowcounts. Therefore, when selecting a representative blowcount, the lowest recorded blowcount of 26 was selected for analysis.

No strength tests were performed on the gravel drain. An effective (drained) strength friction angle was selected based on relations with $(N_1)_{60}$ by Hatanaka and Uchida (1996) and Peck, Hanson, and Thornburn (1974). Based on those relationships and a blowcount of 26, a friction angle of 35 degrees was selected for analysis. Given the limited data available for the filter/drain, a sensitivity strength equal to the effective strength for the embankment fill (29 degrees) was also selected for analysis. Selected strength properties are summarized in Table 6-1.

6.2.2.3 Seismic Parameters

No seismic shear wave velocities are available for the gravel drain. Based on experience with similar materials, and the gravel drain's similarity to the embankment, a shear wave velocity of 1200 feet/second was selected for analysis. A residual shear strength ratio of 0.25 was selected based on recommendations by Idriss and Boulanger (2008), with a minimum strength of 100 psf. Given the limited data available for the filter/drain, a sensitivity strength equal to the residual strength ratio for the embankment fill (0.15) was also selected for analysis.

6.3 Foundation Materials

Underlying the embankment are foundational materials consisting of alluvium and Sonoma Volcanics, primarily consisting of Tuffs with varying weathering and strength. The alluvium was found in borings 601 and 603, while none of the historical borings in the vicinity of 601 and 603 showed alluvium, so the alluvium was interpreted as being in

two pockets on the upstream and downstream side of the dam (as shown in the cross sections). However, it is possible that additional alluvium exists underneath the embankment and that some of the rock logged in the historical borings is actually alluvium. Additional investigations would be required to determine the full extent of the alluvium underneath the embankment.

Sonoma Volcanics encountered in borings performed as part of this study and previous studies generally consist of tuffs and breccia, which can be very loosely welded, and strong basalts. The landslide on the left abutment is also underlain by Sonoma Volcanics with similar appearance and quality to that observed beneath the embankment. Therefore, the same properties were assumed for the Sonoma Volcanics underlaying the embankment and landslide.

6.3.1 Alluvium

6.3.1.1 Index Testing and Density Summary

Index test results are shown in Figures 6-19 to 6-23. As shown in those plots, the alluvium has a wide range of gradations with fines between 3 percent and 61 percent, averaging about 20 to 30 percent. Gravels range from 4 percent to 92 percent, averaging about 30 percent. A single Atterberg limit test was performed, with the fines being classified as CL. Three water content tests were performed on the alluvium with values of about 17, 23, and 31 percent. Water contents generally fall withing the range of the plastic and liquid limit from the Atterberg limit test. No unit weight tests were performed as part of this study.

Based on the results of index testing, blowcount data (discussed below), and experience with similar materials, a moist unit weight of 120 pcf was selected for analysis. The permeability of the unit was estimated using an average gradation of the alluvium, an assumed dry unit weight of the soil, and relations by Kozeny-Carmen as presented in Charbeneau (2000), and a relation by Hazen (1982). In addition, based on the gradation and plasticity of the alluvium, the material was assumed to be non-free draining during rapid drawdown and seismic shaking. Material properties are summarized in Table 6-1.

6.3.1.2 Blowcount Data and Strength Testing

Blowcount data was collected using SPT and Mod Cal samplers as shown in Figure 6-24 and Figure 6-25. As shown in the gradation plot for the alluvium, significant portions of gravel up to the max size of the sampler were observed in the samples, indicating that gravel interference may be an issue with these blowcounts. Therefore, when selecting a representative blowcount, greater weight was given to the samples with lower blowcounts. Data from the 300 series borings are also shown on the figures, with the blowcounts from under the embankment indicated with an “X” mark. Based on the information presented, a representative $(N_1)_{60}$ of 25 was selected for analysis, which corresponds to approximately a 20^{th} percentile value, with only 8 of the 68 recorded blowcounts in the alluvium being less than that value.

No strength tests were performed on the alluvium. An effective (drained) strength friction angle was selected based on relations with $(N_1)_{60}$ by Hatanaka and Uchida (1996) and Peck, Hanson, and Thornburn (1974). Based on those relationships and a blowcount of 25, a friction angle of 35 degrees was selected for analysis. The total (undrained) strength was assumed to be equal to the embankment fill since the average gradation for the alluvium is relatively similar and since a significant portion of the embankment was sourced from a borrow within the channel upstream of the dam, which was likely alluvial in origin. Selected strength properties are summarized in Table 6-1.

6.3.1.3 Seismic Parameters

Seismic shear wave velocities are shown on Figure 6-17. Only three readings were taken in the alluvium, ranging from about 1150 feet/second to 1300 feet/second. Based on the seismic data obtain and the alluvium’s similarity (and likely similar source as discussed above) to the embankment fill, a seismic shear wave velocity of 1200 feet/second was selected for analysis. A residual shear strength ratio of 0.25 was selected based on recommendations by Idriss and Boulanger (2008), with a minimum strength of 100 psf.

6.3.2 Sonoma Volcanics

6.3.2.1 Index Testing and Density Summary

Due to the weak nature of much of the breccia and tuff, soil index tests were performed on some of the samples of Sonoma Volcanics. Destructive testing, such as gradations, were only performed on samples that were so weakly welded that no strength testing could be performed. Index test results are shown in Figures 6-26 to 6-29. As shown in those plots, the weak portion of the Sonoma Volcanics (tuffs and breccia) is generally a sand with silt or clay, with between 15 and 40 percent fines and less than 5 percent gravel. Water contents of the Sonoma Volcanics are up to

to about 55 percent with one historical test at about 36 percent. The core samples tested likely dried some prior to testing so the higher water contents were given higher weight. No Atterberg limit tests were performed. Moist unit weights range from about 50 pcf to 140 pcf. All of the tests performed as part of this study were less than 90 pcf but that is likely mainly due to the samples drying out some prior to testing. Dry unit weights range from 50 pcf to 120 pcf with significantly more overlap in the historic data and the data collected as part of this study but the data from this study still was significantly lower than the historical data (all less that about 80 pcf).

Based on the results of index testing, blowcount data (discussed below), and experience with similar materials, a moist unit weight of 90 pcf was selected for analysis, which corresponds to a dry unit weight of about 70 pcf and a water content of 30 percent. The permeability of the unit was estimated using an average gradation of the Sonoma Volcanics, the dry unit weight of the rock, experience with similar materials, and relations by Kozeny-Carmen as presented in Charbeneau (2000), and a relation by Hazen (1982). Material properties are summarized in Table 6-1.

6.3.2.2 Blowcount Data and Strength Testing

Blowcount data was collected using SPT and Mod Cal samplers. Only 4 blowcounts were recorded as part of this study, but 83 were gathered as part of previous studies. Blowcounts within rock are difficult to relate to any relations dues to the welded nature of rock; however, the Sonoma Volcanics at this site is often only loosely welded and can be sandy so blowcounts were examined when developing strengths. Field blowcounts and values of $(N_1)_{60}$ are shown in Figure 6-30 and Figure 6-31, respectively. Based on the information presented, a representative $(N_1)_{60}$ of 40 was selected for analysis, which corresponds to approximately a 20^{th} percentile value.

Unconfined compressive strength (UCS) tests were performed on samples of the Sonoma Volcanics whenever strong enough to be transported to the lab. Results, presented on Figure 6-32, showed that the tuff and similar materials such as pumiceous tuff had UCS values of 4 to 24 ksf, with a 30^{th} percentile value around 6 ksf. One test on a sample of basalt had a UCS value of about 304 ksf.

Given the almost soil-like conditions mixed with weak rock and some blocks of strong rock (basalt blocks), two strengths were developed for analysis. The primary selected strength was selected based on the representative blowcount mentioned above and a drained an effective (drained) strength friction angle was selected based on relations with $(N_1)_{60}$ by Hatanaka and Uchida (1996) and Peck, Hanson, and Thornburn (1974). Based on those relationships and a blowcount of 40, a friction angle of 39 degrees was selected for analysis. A sensitivity strength was then selected based on procedures recommended by Hoek-Brown (2002), with a UCS value of 12 ksf (80 psi) and geologic strength index (GSI) value of 35. A UCS value of 12 ksf was selected based on an average of the available data. An average was used since the slip surface will be pushing through a wide range of materials including stronger blocks. Selected strength properties are summarized in Table 6-1 and a comparison of the strengths is shown in Figure 6-33.

6.3.2.3 Seismic Parameters

Seismic shear wave velocities are shown on Figure 6-17. Shear wave velocities ranged from 900 feet/second to 2300 feet/second, with a minor increase observed at a depth of approximately 80 feet, although limited data is available below 80 feet. Based on the available data a shear wave velocity of 1300 feet/second was selected for analysis.

6.4 Landslide Debris/Colluvium

Landslide debris/colluvium was logged in the borings on the left abutment where a landslide was previously mapped, as shown in the cross section on Figure 6-3. Near the base of the landslide boulders and/or clay layers were observed but no obvious slide plane of residual strength soil was observed. The landslide is underlain by Sonoma Volcanics, the strength and properties of which was previously discussed.

6.4.1 Index Testing and Density Summary

Index test results are shown in Figures 6-34 to 6-39. As shown in those plots, the landslide debris/colluvium generally consists of a sandy clay with gravel or a clayey sand with gravel, although the material can be variable. Fines generally classify as a fat clay (CH) or a lean clay (CL) with a couple tests classified as high plasticity silts (MH) or low plasticity silts (ML). Water contents of samples of the landslide debris/colluvium are all less than 15 percent however that is mainly due to the samples drying out prior to testing. Unit weights were obtained as a mix of chunk density tests and standard density tests, with dry unit weights between 90 and 125 pcf and an average of about 110 pcf. Moist unit weights are not show due to the samples drying out prior to testing resulting in inaccurate water

contents. For analysis, a water content of 20 percent was assumed, resulting in a moist unit weight of 130 pcf. Material properties are summarized in Table 6-1.

6.4.2 Strength Testing

As part of this study two isotropically-consolidated undrained triaxial strength tests and two sets of torsional ring shear tests were performed on reconstituted samples of landslide debris/colluvium material, with material coming from multiple samples in all four borings. Reconstituted tests were performed instead of tests on relatively undisturbed samples due to the high gravel contents and the types of tests to perform. The composite samples consisted of material from the samples shown in Table 6-3. A comparison of the gradation and plasticity of the composite sample to the rest of the embankment is shown in Figure 6-34 and Figure 6-38, respectively.

Table 6-3: Landslide Debris/Colluvium Composite Samples

Composite ID	Boring ID	Sample ID	Sample Depths (feet)	Gradation Results			Atterberg Limit Results			USCS
				Gravel	Sand	Fines	PL	LL	PI	
3	701 and 703	R2 and R3	12 and 10	61%	10%	26%	32	58	26	GM
4	701, 703, and 704	701: R3 703: R7 & R11 704: R6 & R12	701: 34.5 703: 19 & 29 704: 16.5 & 31.5	2%	34%	64%	20	52	32	CH
5	703 & 704	703: R14 & R21 704: R18 & R23	703: 36.5 & 54 704: 46.5 & 59	32%	29%	39%	30	79	49	GC
8	701 & 704	701: R12 704: R28 & R34	701: 39.5 704: 71.5 & 86.5	33%	32%	35%	16	34	18	GC

Notes:
PL = Plastic Limit
LL = Liquid Limit
PI = Plasticity Index
USCS = Unified Soil Classification System

As shown, the composite samples had varying gradations and plasticity but are generally clayey with a significant portion of gravel. The composites were generally grouped to by depth with samples ranging from relatively shallow to relatively deep near the base of the landslide.

Two isotropically consolidated undrained triaxial strength tests were performed on reconstituted tests from composite 4. The two tests were reconstituted to a dry unit weight of approximately 100 pcf with a water content of 25 percent. The dry unit weight selected was slightly lower than the average discussed above due to some uncertainty in the dry unit weight and to produce a more conservative result. The tests were run at confining pressures of 3000 psf and 6000 psf, which would be approximately 23 feet and 46 feet below the ground surface. Effective (drained) and total (undrained) results are shown in Figure 6-40 and Figure 6-41, respectively. Sample behavior plots showing deviator stress and excess porewater pressure versus axial strain are shown on Figure 6-42 and show contractive materials with positive excess porewater pressures at all strains and confining pressures. Based on the strength results, an effective strength with a friction angle of 29 degrees with no cohesion was selected for analysis. A total strength with a friction angle of 19 degrees and a cohesion of 500 psf was selected for analysis. Selected strength properties are summarized in Table 6-1.

In addition to the triaxial strength tests, two sets of fully softened and residual strengths were obtained using a torsion ring shear test. One of the sets of strengths was tested on landslide samples that were shallower (less than 35 feet below grade, composite 4) and the other set was tested on samples that were deeper (between 35 and 60 feet below grade, composite 5). Results of the torsion ring shear tests are shown on Figure 6-43 and Figure 6-44 for fully softened and residual strengths, respectively. As shown, fully softened strengths were slightly less for the deeper landslide samples, with a friction angle of approximately 21 degrees. For the residual strengths the deeper samples

had a significantly lower strength compared to the shallower samples, with the deeper samples having a friction angle of about 9 degrees. Based on these results, a fully softened friction angle of 21 degrees and a residual friction angle of 9 degrees were selected for analysis.

6.4.3 Seismic Parameters

No shear wave velocity data was obtained in the landslide debris/colluvium. Based on experience in similar materials and based on the shear wave velocities of other materials onsite, a shear wave velocity of 1300 feet/second was assumed for analysis.

7. Embankment Seepage and Erosion Analyses

7.1 Steady-State Seepage

7.1.1 Methodology

Seepage analyses were performed using the two-dimensional finite element computer program SEEP/W (Geo-Slope, 2021). Steady-state seepage calculations were performed to estimate the location of the phreatic surface through the embankment with a reservoir level at the principal spillway level (Elevation 377.6 feet[1]) and auxiliary spillway level (Elevation 412.6 feet). While steady-state seepage condition with water at the spillway level are unlikely to occur due to lack of time to saturate the embankment, results were still used in analysis to be conservative. A steady-state seepage analysis was also performed for evaluating the stability of the embankment at the flood surge level (Elevation 423 feet) but those results are not shown in this section because steady-state conditions are unlikely to occur during the flood surge event. The locations of the estimated phreatic surfaces were used in corresponding stability analyses, which are presented in Section 8.

To model steady-state seepage, appropriate boundary conditions were applied with SEEP/W. The applied boundary conditions generally included the following:

- A constant-head boundary condition at the ground surface was used along the downstream vertical side boundary to model the expected shallow groundwater conditions in this area during a flood event.
- A constant-head boundary condition corresponding to the reservoir level was used along the upstream vertical side boundary and along the reservoir floor and upstream embankment slope.
- A no-flow boundary was applied along the base of the model.
- A potential-seepage boundary condition was assumed on the downstream slope. The potential-seepage boundary allows seepage to exit the model as necessary to represent a condition of unbounded flow.

The applied boundary conditions are shown in the analysis sections presented herein.

Material parameters for the seepage analyses were presented in Section 6. As described, hydraulic conductivities and anisotropy ratios of the embankment and foundation materials were estimated based on the results of laboratory tests, in-situ permeability tests, and experience with similar materials. For materials that may be not be fully saturated (i.e. the phreatic surface may cross through them), unsaturated/saturated material properties were developed, using the porosity and typical curves for different soils as provided by Geo-Slope (2021).

NRCS and DSOD criteria for seepage are summarized in Table 7-1. In addition to the requirements in Table 7-1, seepage results are evaluated for other potential concerns such as through-seepage. Conditions for uplift are not present at Matanzas as there is not aquifer/blanket so uplift was not evaluated.

[1] All referenced elevations in this section and below are NAVD88

Table 7-1: Seepage Analysis Design Criteria

Category	Requirement	DSOD/USACE
Filter Compatibility	Meet requirements of NRCS Part 633 Soils Engineering National Handbook, Chapter 26, Gradation Design of Sand and Gravel Filters	Meet requirements of USBR Design Standard No. 13 (Embankment Dams), Chapter 5: Protective Filters
Heave (cohesionless soils)	Minimum Factor of Safety of 4.0	Minimum Factor of Safety of 1.5
Uplift (blanket-aquifer condition)	Minimum Factor of Safety of 3.0 (effective stress method) Minimum Factor of Safety of 1.5 (total stress method)	Minimum Factor of Safety of 1.5

7.1.2 Results

An example model setup for the seepage analysis with a reservoir level at the principal spillway is shown on Figure 7-1. Results of the seepage analyses for the maximum section and relief trench section at both the principal and auxiliary spillway level are shown in Figure 7-2 to Figure 7-5. Total head contours and the location of the phreatic surface at steady state are shown on the result figures. It is important to note that these analyses assume steady-state seepage conditions under the different flood event, which is a conservative assumption given the short duration of the flood events. Figure 7-6 shows results from transient analyses at the auxiliary spillway level, which is discussed in Section 7.3.

Analysis results for the auxiliary spillway reservoir level for the maximum section show the phreatic surface at the ground surface approximately 2 to 3 feet above the top of the filter/drain while in all the other models the phreatic surface at the ground surface within the filter/drain. If water were to seep out of the embankment fill, fines could be eroded, and a piping failure could occur. However, given that steady-state seepage at the auxiliary spillway reservoir level is unlikely to occur due to the short timeframe that the level would be at the auxiliary spillway and the fact that the phreatic surface is only 2 to 3 feet above the filter/drain, piping erosion is unlikely to occur.

Calculated gradients at the toe of the embankment with steady-state seepage with the reservoir at the principal spillway level were up to approximately 0.14 and up to 0.19 with the reservoir at the auxiliary spillway level. The dry unit weight of the Sonoma Volcanics was approximately 70 pcf resulting in a saturated unit weight of 107 pcf, assuming a specific gravity of 2.7. Using this data a factor of safety against heave of 5.1 and 3.7 were calculated for the reservoir at the principal spillway and auxiliary spillway, respectively. While the value at the auxiliary spillway level is less than the NRCS requirement of 4.0, it is close to the requirement and as stated before, steady-state seepage is unlikely to occur at the auxiliary spillway level. For this reason, we consider the embankment to meet criteria for heave.

It should be noted that the estimated phreatic surface is based on the estimated representative hydraulic conductivities of materials. Increased seepage through the foundation could occur if localized, continuous zones of higher hydraulic conductivity bedrock are present beneath the embankment. In addition, the estimated phreatic surface and total head contours are based on a two-dimensional analysis. The actual conditions will be affected by three-dimensional flow conditions (such as downslope flow from each of the abutments).

7.2 Drain/Filter Compatibility

This section describes the filter/drain compatibility analyses for the filter/drain zone against the embankment fill and foundation materials. Filters are designed to prevent piping (internal erosion) of materials in the dam or from the dam foundation due to the seepage forces through and under the dam and to collect, control and filter seepage flows. Potential seepage pathways that could lead to internal erosion include the following:

- From the embankment fill through the chimney drain/filter or downward migration of the downstream shell into the blanket drain/filter.
- From the foundation upward through the drain/filter

Filter compatibility was checked with both methodology by the Natural Resource Conservation Service (NRCS, 2017) and the United States Bureau of Reclamation (USBR, 2011).

Crumb tests, double hydrometers, and pinhole tests were performed to assess the dispersiveness of the embankment fill. Results are summarized in Table 5-1. Out of the 14 tests performed, 4 were classified as nondispersive, 5 were classified as slightly or intermediately dispersive, and 5 were classified as highly dispersive. The two pinhole tests, which were performed on composites of embankment materials, were both classified as slightly dispersive. Based on the results of these tests, the embankment fill was classified as dispersive.

Representative gradations for the base soils (embankment fill and foundation alluvium and rock) and the filter (gravel filter/drain) were selected. The representative gradations each had a coarser and finer gradation curve which was based on 10^{th} and 90^{th} percentile gradations or minimum and maximum gradations depending on the number of individual gradations for the specific material and spread in the data. The selected representative gradation bands for the base and filter soils are shown in Figures 7-7 through 7-9. The gradation for the filter/drain material includes gradations from the time of construction of the dam, and another gradation based on the results of laboratory testing performed as part of the current study. Upon comparison of the two gradations, it is apparent that the percentage of finer material and sand in the filter/drain material has increased since construction of the embankment, although the gradation may also be affected by the size of the sampler as well.

The results of the filter compatibility evaluation for the embankment fill as the base and the filter/drain material as the filter are presented in Figures 7-10 and 7-11 for existing conditions and time of construction, respectively. The figures show the control points for particle retention and permeability requirements for both NRCS and USBR criteria. To meet criteria, the percent retained must be between the pairs of control points. As shown in the figure, the filter/drain material does not meet compatibility requirements against the embankment fill. The fact that the filter/drain did not meet particle retention requirement at the start of construction and the increase of finer particles in the filter/drain over time suggests that erosion of the embankment material into the filter/drain could be occurring.

The results of the filter compatibility evaluation for the alluvium and foundation rock as the base and the filter/drain material as the filter are presented in Figure 7-12. As shown in the figure, the filter/drain material is close to meeting particle retention requirements against the alluvium and foundation rock; however, the amount of fines in the filter/drain exceeds the permeability requirements for the filter.

Based on the results of the filter compatibility analyses, an erosion analysis was performed using recommendations from Foster and Fell (2001). The method uses correlations with the filter and base soil gradations to classify the filter and base soil combination into one of the following categories:

- No Erosion: Significant erosion is unlikely to develop
- Some Erosion: Erosion may occur but is not likely to be significant
- Excessive Erosion: Erosion is more likely to occur and may be significant but is likely to plug up the pore spaces eventually
- Continuing Erosion: Continuous erosion may occur and may result in piping failure of the embankment

Results of the analyses are presented in Figure 7-13 to Figure 7-15. Figure 7-13 shows results for the embankment against the existing filter/drain, Figure 7-14 shows results for the embankment against the filter/drain gradation from construction, and Figure 7-15 shows results for the foundation against the filter/drain. As shown, the filter/drain from construction was found to be in the erosion zone but not quite in the excessive erosion zone while the other two cases were in the no erosion zone. Based on the results, and as seen in the filter compatibility results, it seems the filter/drain placed during construction may have had material from the embankment erode into the filter/drain. However, the increase in fines in the filter/drain material has led to an increase in the filter compatibility of the filter/drain and it is now unlikely to continue to erode.

7.3 Transient Seepage During Drawdown

As discussed in Section 8, rapid drawdown from the reservoir at the auxiliary spillway level to empty was not meeting criteria with simplified methods. However, failures have not been observed during drawdowns when the outlet gate was fully open (i.e. the maximum drawdown rate). Therefore, a transient drawdown analysis was performed so that stability analyses could be performed with the actual maximum drawdown rate. Transient analyses use the same boundary conditions as those for steady-state seepage but with a reservoir side boundary condition that varies with

time. Based on hydraulic analyses, the drawdown rate from the auxiliary spillway to empty is approximately 45 hours so a linear function was defined with the maximum reservoir level being at Elevation 412.6 (auxiliary spillway level) at 0 hours and Elevation 350 feet (approximately ground surface elevation at upstream toe of the embankment) at 45 hours. Results of this analysis are summarized in Figure 7-7 and are utilized in the drawdown stability analyses, as discussed in Section 8.

7.4 Surface Erosion

No riprap was observed on the upstream face of the embankment. However, no signs of significant erosion from waves or runoff were observed on the upstream (or downstream) face of the embankment. Limited riprap/rockfill was noted in the groins on the upstream and downstream face and erosion was observed (likely due to rainfall runoff). The erosion in the groins does not represent an immediate issue but should be monitored and riprap/erosion protection added if needed.

While the upstream face lacks riprap as is standard for modern dam designs, the lack of apparent erosion since construction of the embankment indicates that riprap is not likely needed. The upstream face should be monitored and if signs of erosion appear then riprap or other erosion control measures may be required.

8. Embankment Slope Stability and Seismic Deformation

This section presents the methodology and results of slope stability and seismic deformation analyses. Material properties were defined earlier in Section 6.

8.1 Methodology and Design Criteria

Limit equilibrium, seismic deformation, and liquefaction analyses were performed as part of this study. Details on the methodology used for those analyses are covered in this section along with the criteria for those analyses.

8.1.1 Limit Equilibrium Analysis

Limit equilibrium slope stability analyses were performed using the computer program Slope/W (Geo-Slope, 2021). The limit equilibrium slope stability calculations presented herein were performed using methodology by Morgenstern-Price with a half-sin interslice side function, which satisfies both moment and force equilibrium simultaneously.

Limit equilibrium slope stability analyses were performed for the following loading conditions:

- Long-term conditions with steady-state seepage pore water pressures calculated using SEEP/W for the reservoir at the principal spillway (Elevation 377.6 feet), the auxiliary spillway (Elevation 412.6 feet), and the flood surcharge (Elevation 423 feet) levels
- Rapid drawdown of the reservoir from the principal spillway and auxiliary spillway reservoir level to empty
- Pseudo-static analysis to determine yield coefficients with the reservoir at the principal spillway level
- Post-earthquake conditions with residual reduced strengths for the embankment fill, alluvium, and filter/drain as applicable

Limit equilibrium stability analyses were performed for the maximum section and a section at the relief trench.

Material strength parameters for stability analyses were described in Section 6. Rapid drawdown and pseudo-static analyses were performed using the staged analysis procedure described by Duncan et al. (1990), as implemented in Slope/W for non–free-draining materials. In addition, rapid drawdown analyses were performed using a bilinear strength envelope, as recommended by NRCS (2019), which is a lower bound of the effective and total strength envelopes. Methodology by Duncan et al. (2005) was used to develop the total strength parameters for the bilinear envelope. Any material shown in Table 6-1 with a total strength shown was assumed to be non-free draining during rapid drawdown.

For the flood surcharge analysis, drained strengths and the bilinear failure envelope used for rapid drawdown analysis were used.

Based on the results of liquefaction analysis (discussed below) residual strengths were assigned to saturated materials as applicable and post-seismic stability analyses were performed.

Criteria for the different limit equilibrium analyses are summarized in Table 8-1.

Table 8-1: Limit Equilibrium Criteria

Case	NRCS TR-60 (2019)	DSOD/USACE
Long-Term with Steady-State Seepage	1.5 (1.3 for near surface failures)	1.5
Auxiliary Spillway and Flood Surcharge	1.4*	1.4
End-of-Construction and Short-Term Loading (construction)	1.4 (slip surfaces extending into foundation) 1.3 (slip surfaces confined to embankment)	1.3
Rapid Drawdown	1.2 (1.1 for near surface failures)*	1.3
Post-Seismic Stability with Reduced Strengths Applied as Applicable	1.2	1.25

*NRCS TR-60 recommends use of bilinear failure envelope combining total and effective strength envelopes

NRCS = Natural Resources Conservation Service

DSOD = California Department of Water Resources Division of Safety of Dams

USACE = United States Army Corp of Engineers

8.1.2 Seismic Evaluation

Seismic deformation and stability analyses were performed to assess the stability of the embankment during and after a seismic event. For a dam passing the seismic evaluation, deformation should not result in embankment overtopping, internal erosion from cracking, or damage to appurtenances to the extent that could result in dam failure. Dam failure is defined as major damage that may lead to uncontrolled loss of the reservoir. After the earthquake and prior to repair, the dam and appurtenances must be able to pass the principal spillway flood without failure.

Per NRCS TR-60 (2019), Matanzas Dam is classified as a “High Consequence” of seismic failure, since the normal pool breach maps prepared as part of this contract indicate inundation of downstream residential areas by up to 2 feet (JDE, 2021). Therefore, seismic evaluations require a design ground motion return period of 10,000-years and an operating basis earthquake with a return period of 500-years. In addition to the NRCS criteria, analysis was performed using an 84th percentile deterministic motion as required by DSOD. Details of the methodology for the seismic evaluation are provided in the following subsections.

8.1.2.1 Seismic Hazard

A site-specific probabilistic seismic hazard analysis (PSHA) based on the methodology developed by Cornell (1968) was performed to estimate the hazard at the project site. Additionally, a deterministic seismic hazard analysis (DSHA) was performed to developed scenario spectrum at the 84th percentile level. OYO-suspension logging in borehole 603 (Section 5.1) was used to determine the time-averaged shear-wave velocity over a depth of 30 m (VS30) at the base of the dam of 380 meters/second (1200 ft/s). The values of Z1.0 and Z2.5, the depth to a velocity of 1 km/sec and 2.5 km/sec, respectively, were adopted from the United States Geological Survey (USGS) Bay Area seismic velocity model version 8.3.0 (Brocher, 2008).

The fault model used in the PSHA was adopted from a model developed as part of the California Department of Water Resources’ Delta Risk Management Strategy Project (URS Corporation/Jack R. Benjamin & Associates, 2007), with fault geometries and slip-rates updated as necessary from the statewide rupture forecast project; the Uniform California Earthquake Rupture Forecast, version 3 (UCERF3) (Field et al. 2013). The geometry of the Rodgers Creek fault was updated with data from Hecker and Randolph Loar (2018). The Next Generation of Attenuation West2

(NGA-W2) ground motion models of Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014) and Chiou and Youngs (2014) were used is the hazard analysis, equally weighted.

The seismic hazard curves deaggregated by contribution to the total mean hazard are shown on Figure 8-1. Given the proximity of the Hayward-Rodgers Creek-Healdsburg fault system (Figure 3-2), its high slip rate (9 ± 2 mm/year), and potential for generating large earthquakes (moment magnitude (M) greater than 6.7) it is the principal contributor to the seismic hazard over all return periods of interest. The peak ground acceleration (PGA) at a return period of 10,000 years is 1.76 g and the 1.0 sec spectral acceleration is 2.51 g. The modal magnitude and distance at PGA and a return period of 10,000-years is a M 7.5 at 1.5 km.

The DSHA was calculated for a M 7.2 on the Rodgers Creek-Healdsburg fault, a strike-slip fault, at a rupture distance of 0.24 km. The fault dips slightly to the east at an angle of 77 degrees, so that the site is in the hanging wall. The Bennett Valley fault was also considered in the DSHA, with a M 6.7 at a rupture distance of 2.3 km. The same site conditions and ground motion models were used in the DSHA as in the PSHA. The largest ground motion were for the Rodgers Creek-Healdsburg fault, with a PGA at the 84th percentile level of 1.06 g.

The largest seismic event to occur at the project site since construction of the embankment was on October 2, 1969, when a magnitude 5.7 earthquake occurred 10 kilometers from the site. The earthquake caused one fatality and severe property damage in Santa Rosa, but no reports of damage to the embankment or appurtenant structures were noted. A back analysis to check the validity of the selected properties with this event is discussed in Section 8.3.

The DSHA spectra, uniform hazard spectra (PSHA) for a range of return period, and historic spectra are shown on Figure 8-2 and summarized in Table 8-3 (Section 8.3).

No site response analysis was performed for the liquefaction or slope displacement analyses. Based on relationships between peak base acceleration and peak crest acceleration (Smith et al., 2022), deamplification of the motion is anticipated from the base of the embankment to the crest due to softening of the materials during shaking. In addition, liquefaction (see discussion of liquefaction results) would also effect the motions through the embankment.

8.1.2.2 Seismic Deformation

Seismically induced permanent displacements were estimated using methodology by Bray and Macedo (2019) with a 500-year (operating basis earthquake), 10,000-year (NRCS design motion), and 84th percentile deterministic (DSOD design motion) motion. Based on the proximity of the site to the Rodger's Creek Fault, which drives much of the risk for the site, the pulse motion dataset was used for analysis to account for potential directivity effects at the site with the PulseD50 motion set used for analysis of the embankment (fault-parallel motion) and the PulseD100 motion used for analysis of the landslide (fault-normal motion).

The key input parameters for the Bray and Macedo (2019) procedure are the seismic yield coefficient of the slide mass, the fundamental period of the slide mass, the acceleration response spectrum for the earthquake, and the earthquake magnitude. The fundamental period of the representative slide masses was estimated using the estimated shear wave velocities and the slide mass geometry as recommended by Bray and Macedo (2019). The acceleration response spectrum of the outcrop motion was assumed to be appropriate for the slide surfaces although some amplification of the ground motion may occur for slip surfaces near the top of the embankment.

Pseudo-static analyses were performed to estimate the seismic yield coefficients of critical slide surfaces on the upstream and downstream sides of the dam. Yield coefficients were calculated for a range of representative slip surfaces on both the upstream and downstream slopes of the embankment. Analyses were performed using the steady-state pore water pressures calculated with the reservoir at the principal spillway elevation. The strengths of the saturated materials with total stress strengths (such as the embankment fill and alluvium) were estimated using the staged analysis procedure described by Duncan et al. (1990). Effective strength parameters were used for other materials.

8.1.2.3 Liquefaction

We performed liquefaction triggering analyses based on the information obtained from the borings performed as part of this study using methodology by Boulanger & Idriss (2014), Cetin el al. (2018, and Youd and Idriss (2001) (NCEER). The peak ground acceleration and earthquake magnitude used in analysis were selected based on the results of the seismic hazard analysis described in Section 8.1.2.1 with a 500-year (operating basis earthquake), 10,000-year (NRCS design motion), and 84th percentile deterministic (DSOD design motion) motion. Water levels were estimated using the results of the seepage analysis discussed in Section 7.

8.2 Results of Limit Equilibrium Analysis

Results of the limit equilibrium analysis are summarized in Table 8-2 with slip surfaces shown in Figure 8-3 to Figure 8-27.

Table 8-2: Summary of Limit Equilibrium Results - Embankment

Section	Reservoir Level	Case	Calculated Minimum Factor of Safety	Criteria	Result	Results Figure
Max	Principal Spillway	Long-Term	1.5 (sens.[c] = 1.5)	1.5 (1.3[a])	Meets Criteria	8-3
		Rapid Drawdown	Duncan et al. = 1.5 Bilinear Str. = 1.4	1.2 (1.1[a])	Meets Criteria	8-4 and 8-5
		Post-Seismic	Circular = 1.1 (sens.[c] = 0.9) Block = 1.0 (sens.[c] = 0.8)	1.2	Does Not Meet Criteria	8-6 to 8-11
	Auxiliary Spillway	Long-Term	Drained = 1.5 Bilinear Str. = 1.4 Undrained = 1.5	1.4 (1.2*)	Meets Criteria	8-12 to 8-14
		Rapid Drawdown	Bilinear Str. = 0.9 Duncan et al. = 1.0 Transient = 1.4	1.2 (1.1[a])	Meets Criteria	8-15 to 8-17
	Flood Surcharge	Long-Term	Drained = 1.5 Bilinear Str. = 1.4 Undrained = 1.4	1.4 (1.2*)	Meets Criteria	8-18 to 8-20
Relief Trench[b]	Principal Spillway	Long-Term	1.5 (sens.[c] = 1.5)	1.5 (1.3[a])	Meets Criteria	8-21
		Post-Seismic	Circular = 1.1 (sens.[c] = 0.8) Block = 1.1 (sens.[c] = 0.8)	1.2	Does Not Meet Criteria	8-22 and 8-24
	Auxiliary Spillway	Long-Term	Bilinear Str. = 1.5 Undrained = 1.5 Drained = 1.5	1.4 (1.2*)	Meets Criteria	8-25 to 8-27
	Flood Surcharge	Long-Term	Bilinear Str. = 1.5 Undrained = 1.5 Drained = 1.5	1.4 (1.2*)	Meets Criteria	8-28 to 8-30

Notes:

*For shallow slip surfaces

[a]For shallow slip surfaces

[b]Analyses only performed for the downstream slope since the upstream slope and phreatic line were unchanged.

[c]Sensitivity with filter/drain strength equal to the embankment fill

As shown, the embankment meets criteria except for post-seismic stability, which uses residual material strengths for materials that are predicted to liquefy in the design seismic events (see Section 8.3.2). Several other analyses had specific methods or sensitivities that did not meet criteria:

- Hoek-Brown sensitivity strengths (see Table 6-1) had no effect on the stability of the embankment. However, if a lower UCS value is used to develop the Hoek-Brown strength, long-term stability (at both reservoir levels) do not meet criteria. The strengths selected are considered appropriate based on the information obtained but further investigation of the rock strength should be performed to provide more certainty in the strength of the rock.
- Rapid drawdown analyses from the auxiliary spillway level to empty results in factors of safety below the required criteria when using methodology by Duncan et al. (1990) and the bilinear strength envelope. To further refine the analyses, transient drawdown analyses were performed, and limit equilibrium analyses performed using the results of that seepage analysis (see Section 7.3). Effective strengths and a bilinear envelope were used for the strength of the embankment fill and the resulting minimum factor of safety being calculated as 1.4. Given the significantly more rigorous methodology behind the transient drawdown analysis and the lack of observed failures during past drawdown events, the results of the transient drawdown analysis were considered to be more appropriate. However, it is important to note that the results are sensitive to changes in the permeability of the embankment, which was estimated based on the limited gradation data and no lab or in-situ permeability tests. Permeability of the embankment fill should be investigated further.

- Factors of safety of 1.0 were calculated for shallow slip surfaces during drawdown analyses at any level. However, given the lack of observed failures during drawdown since construction of the dam, these calculated lower factors of safety shallow slip surfaces were considered to be unrealistic.

8.3 Results of Seismic Analysis

This section presents the results of seismic analyses, including seismic displacement using simplified methodology by Bray and Macedo (2019) and liquefaction analyses using methodology by Boulanger & Idriss (2014), Cetin el al. (2018, and Youd and Idriss (2001) (NCEER). It should be noted that the methodology by Bray and Macedo (2019) does not consider effects of liquefaction.

8.3.1 Simplified Seismic Displacement

Results of the limit equilibrium analysis are summarized in Table 8-4 with slip surfaces shown in Figure 8-31 to Figure 8-40. As shown, median predicted displacements for the 84th percentile motion ranged from 2 feet to 4 feet and ranged from 4 to 8 feet for the 10,000-year motion. Given the lower water level behind the embankment for the design event (principal spillway level), cracking may not be deep enough that it would reach the low water level. However, it is also possible that sufficient displacement could occur (with or without liquefaction) to damage the principal spillway structure. Further (and more rigorous) analysis would be required to examine the deformation pattern and incorporate and potential displacement due to liquefaction (see Section 8.3.2).

In addition to the analysis of existing conditions, a rough check of the estimated maximum displacement for different return periods and the historic earthquake motion were made, as summarized in Table 8-3. Based on this analysis, the estimated displacement for the largest historic event is 0.1 feet or less, which would not have produced observable damage to the embankment and is consistent with reported performance.

Table 8-3: Summary of Response Spectra and Maximum Displacement Estimates

Motion	Peak Ground Acceleration (g)	Peak Spectral Acceleration (g)	Approximate Max Slope Displacement[1] (feet)
475-year PSHA Motion	0.706	1.63 (T=0.3 sec)	1
975-year PSHA Motion	0.923	2.19 (T=0.3 sec)	2
2,475-year PSHA Motion	1.24	3.03 (T=0.3 sec)	6
5,000-year PSHA Motion	1.50	3.70 (T=0.3 sec)	7
10,000-year PSHA Motion	1.76	4.44 (T=0.3 sec)	8
84th Percentile DSHA Motion	1.06	2.57 (T=0.3 sec)	4
1969 Santa Rosa Earthquake, 84th Percentile Motion (M5.7 at 10km from site)	0.324	0.777 (T=0.15 sec)	0.1

[1]Calculated using simplified procedures and without effects from liquefaction. Values presented are median estimates.

PSHA = Probabilistic Seismic Hazard Analysis

DSHA = Deterministic Seismic Hazard Analysis

Table 8-4: Summary of Simplified Seismic Displacement Calculations - Embankment

Section	Slip Surface	Static Factor of Safety	Seismic Analysis													
							84th Percentile Deterministic Motion (DSOD Criteria)					10,000-year Motion (NRCS Criteria)				
			Yield Coefficient, Ky	Slip Surface Height, H (feet)	Fundamental Period of Slip Surface, Ts (seconds)	1.3 x Ts (seconds)	Spectral Acceleration at 1.3Ts, Sa(1.3Ts) (g)	Probability of Greater Than 5 feet of Displacement	Displacement by Probability of Exceedance (feet)			Spectral Acceleration at 1.3Ts, Sa(1.3Ts) (g)	Probability of Greater Than 5 feet of Displacement	Displacement by Probability of Exceedance (feet)		
									84%	50%	16%			84%	50%	16%
Max	DS1	1.5	0.20	10	0.033	0.043	1.24	6%	1.3	2.2	3.7	2.19	52%	3.0	5.1	8.8
	DS2	1.9	0.24	65	0.217	0.282	2.54	31%	2.2	3.8	6.6	4.38	79%	4.5	7.7	13.2
	DS3	1.7	0.24	25	0.083	0.108	1.81	6%	1.3	2.2	3.7	3.38	52%	3.0	5.1	8.8
	DS4	1.9	0.23	35	0.117	0.152	2.08	18%	1.8	3.1	5.2	3.78	70%	3.9	6.7	11.4
	US1	2.3	0.26	30	0.100	0.130	1.96	8%	1.4	2.3	4.0	3.60	56%	3.2	5.4	9.2
	US2	3.0	0.41	70	0.233	0.303	2.57	2%	1.0	1.6	2.8	4.43	32%	2.3	3.9	6.6
	US3	1.8	0.26	15	0.050	0.065	1.44	2%	0.9	1.6	2.7	2.67	35%	2.4	4.0	6.9
	US4	3.1	0.28	40	0.133	0.173	2.19	8%	1.4	2.4	4.0	3.94	56%	3.2	5.5	9.3
Relief Trench	DS1	2.0	0.27	70	0.233	0.303	2.57	22%	1.9	3.3	5.6	4.43	72%	4.0	6.9	11.7
	DS2	1.7	0.29	40	0.113	0.173	2.19	7%	1.3	2.3	3.9	3.94	53%	3.0	5.2	8.9

8.3.2 Liquefaction

Liquefaction requires for the soil to be susceptible to liquefaction and if susceptible have a low enough density that liquefaction can trigger with the design motion. Typically, materials with fines contents less than about 30 to 40 percent fines are considered susceptible to liquefaction. Materials with greater percent fines can also still liquefy but require low plasticity index values.

A liquefaction assessment was performed for each boring with the susceptibility being assessed on a sample-by-sample basis. Results of these analysis are presented in Figure 8-41 to Figure 8-44 for the 84th percentile Rodger's Creek motion, Figure 8-45 to 8-48 for the 10,000-year motion, and Figure 8-49 and 8-52 for the 500-year motion. A summary of the factors of safety against liquefaction for susceptible materials are shown in Figure 8-53 to 8-55 for the 84th percentile,10,000-year, and 500-year motions, respectively. The results of these analyses show that portions of the embankment fill, filter/drain, and alluvium were susceptible to liquefaction, with the embankment fill and filter/drain triggering in all three motions although the portion triggering in the 500-year motion is slightly less than the 10,000-year motion (about one-half versus two-thirds). While the single susceptible blowcount within the alluvium did not trigger in the analyses, the sample in question (boring 601, sample 18-1 and 19-1 plus the following core run) had significant quantities of gravel that may have resulted in an increased blowcount.

Based on the results from the individual boring liquefaction analyses and a review of gradations shown in Section 6, three materials encountered as part of the investigation of Matanzas Dam are potentially susceptible to liquefaction and were shown to trigger in the design earthquakes: the embankment fill, the filter/drain material, and the alluvium. Susceptibility of these materials as a whole is discussed below.

- Embankment Fill: As shown in the gradation plots presented in Section 6 (Figure 6-4), the embankment fill generally consists of almost equal parts fines, sand, and gravel. Out of the 13 gradations or percent fines tests performed, 11 had less than 40 percent fines and 4 had less than 30 percent fines, making the material borderline susceptible. Given the low number of samples gathered relative to the size of the embankment and the large number of samples with less than 40 percent fines, the embankment fill as a whole was considered susceptible to liquefaction and likely to triggering in both of the design motions.
- Filter/Drain: As shown in the gradation plot presented in Section 6 (Figure 6-18), the filter/drain material generally consists of a gravelly sand or sandy gravel with less than 25 percent fines. It is possible for a material to be coarse enough that water drains away freely during an earthquake. However, while approximately 45 percent of the filter/drain material is gravel, only about 5 percent of that is a coarse gravel, which may not be coarse enough to fully relieve pore pressures during seismic shaking. Therefore, the filter/drain material as a whole was considered susceptible to liquefaction and likely to triggering in both of the design motions.
- Alluvium: As shown in the gradation plot presented in Section 6 (Figure 6-19), the alluvium material varies significantly, with between 3 and 61 percent fines. Out of the 5 gradations or percent fines tests performed, 4 had less than 40 percent fines and 2 had less than 30 percent fines, making the material borderline susceptible. As with the embankment fill, given the low number of samples gathered relative to the size of the embankment and the large number of samples with less than 40 percent fines, the alluvium as whole was considered susceptible to liquefaction and likely to triggering in both of the design motions.

Figure 8-56 and Figure 8-57 show the calculated shear strength ratios (S_r/σ'_v) for the samples that triggered in the individual boring liquefaction calculations for the 84th percentile and 10,000-year motions, respectively. Based on the results shown in the figures and an evaluation of the blowcounts overall (see representative blowcounts in Table 6-1), a residual shear strength ratio of 0.15 was selected for the embankment fill and a value of 0.25 selected for the filter/drain material and alluvium as shown in Table 6-1. A sensitivity of the filter/drain with a residual shear strength ratio of 0.15 was also selected. Results of the post-seismic stability analyses are presented in Section 8.2.

8.3.3 Fault Offset

As mapped by McLaughlin et al. (2008), the Matanzas Creek Fault goes beneath the center of Matanzas Creek Dam. The California Geological Survey (CGS) or United States Geologic Survey (USGS) do not consider the fault to be active and is not included in the USGS Quaternary fault database and is, therefore, not a fault that demonstrates geological evidence of coseismic surface deformation in large earthquakes during the Quaternary although limited information is available on the fault.

To further assess the fault, six soil borings and four geophysical surveys were performed downstream of the embankment. As detailed in the GIR (JDE and AECOM, 2022), the subsurface explorations showed some evidence for the fault in the approximately location mapped by McLaughlin et al. (2018) but indicated that the fault could be millions of years old and therefore may not pose a risk to the embankment.

Since the fault is not considered active by CGS or the USGS, and the investigation results indicated similar information, the fault was considered to be inactive and no fault offset was estimated.

9. Landslide Slope Stability and Seismic Deformation

As detailed in the GIR (JDE and AECOM, 2022), the landslide on the left abutment of the dam consists of a mass of colluvium and large block of volcanic rock. The boring logs and televiewer logs did not show a continuous slide plane between borings, so it is possible that the landslide is geologically old. In addition, no recent movement of the landslide has been observed indicating that the topography may have changed since the initial landslide resulting in a more stable configuration. However, it is possible that the dam is buttressing the landslide. To assess the stability the landslide stability analyses were performed, as summarized in this section.

9.1 Methodology

Limit equilibrium slope stability analyses were performed as described in Section 8.1 except with the modifications noted below. Limit equilibrium slope stability analyses were performed for the following loading conditions:

- Long-term conditions with an assumed phreatic surface based on groundwater measurements and the reservoir at the principal spillway level
- Fully softened and residual shear strengths with a defined slip surface along the base of the landside with the reservoir at the principal spillway level
- Pseudo-static analysis to determine yield coefficients with the reservoir at the principal spillway level
- Pre-construction conditions (cut slope without dam placed) and back-calculation of a strength along the bottom of the landslide resulting in a factor of safety of 1.0. As mentioned above, the landslide is likely relatively old and does not show signs of recent movement so the back-calculated strength will only provide a lower bound strength check instead of providing a potentially realistic strength along the failure plane.

A factor of safety criteria of 1.5 was selected for surfaces that impact the embankment and a factor of safety criteria of 1.3 was selected for surfaces that don't directly impact the embankment. The factor of safety of 1.5 was selected since it would directly impact the embankment and poses a risk to the embankment, so the same criteria as static stability of the dam was used.

No liquefaction or post-seismic stability calculations were performed based on the deep groundwater observed in the borings. While the water level may fluctuate in heavy rains, the chance of the design earthquake occurring during such a heavy rain event is remote and therefore not considered.

9.2 Results of Limit Equilibrium Analysis

Results of the limit equilibrium analysis are summarized in **Error! Reference source not found.** with slip surfaces shown in Figure 9-1 to Figure 9-6. The landslide plane was altered for analyses of factor of safety along the base of the landslide using fully softened and residual strengths.

Table 9-1: Summary of Limit Equilibrium Results - Landslide

Condition	Case	Slide Plane	Calculated Minimum Factor of Safety	Criteria	Result	Results Figure
Existing Condition (with Embankment)	Long-Term (Effective Strength)	Within Landslide (Global Surface)	3.6	1.5	Meets Criteria	9-1
		Within Landslide (Local Surface)	3.1	1.3	Meets Criteria	9-2
	Long-Term (Fully Softened Strength)	Landslide Slip Plane (Fully Softened Strength)	2.9 (high GW* = 1.6)	1.5	Meets Criteria	9-3
	Long-Term (Residual Strength)	Landslide Slip Plane (Residual Strength)	1.2 (high GW* = 0.7)	1.5	Does Not Meet Criteria	9-4
Pre-Construction (without Embankment)	Long-Term (Fully Softened Strength)	Landslide Slip Plane (Fully Softened Strength)	2.7 (high GW* = 1.4)	Not Applicable		9-5
	Long-Term (Residual Strength)	Landslide Slip Plane (Residual Strength)	1.1 (high GW* = 0.6)	Not Applicable		9-6
	Back-Calculated Strength for Static Instability	Landslide Slip Plane	Friction Angle = 8 degrees (high GW = 15 degrees)	Factor of Safety = 1.0		None

*Sensitivity with groundwater about 15 feet below ground surface

As shown, the landslide meets stability criteria except the long-term case using residual strengths along the landslide plane. However, based on the available information from the recent borings and borehole televiewer results documented in the GIR (JDE & AECOM, 2022), a continuous well-defined residual strength shear plane was not observed at the base of the landslide. Therefore, we consider the fully softened strengths to be more appropriate.

Without the embankment in place, the factor of safety for the landslide dropped for both the residual strength and the fully softened strength by about 0.1 or 0.2, indicating the dam provides some buttressing effect. While the presence of the embankment produces only a relatively minor increase in factor of safety, the results suggest that the embankment appears to be providing some resistance to landslide movement.

As shown in the results above, increases in groundwater elevation can significantly reduce the factor of safety, indicating the possibility for landslide movement following heavy rains. Vibrating wire piezometers and an inclinometer were installed in boring 702. The piezometers and inclinometer should be monitored to evaluate the influence of groundwater on potential slope movement, and analyses updated based on observations.

Based on the results of the geologic investigation, the instrumentation monitoring data collected to date, and the results of the stability analyses presented herein, we consider the landslide to be statically stable. The embankment may be providing some buttressing effect to the landslide, although the effect is relatively low.

9.3 Results of Seismic Analysis

Results of the limit equilibrium analysis are summarized in Table 9-2,**Error! Reference source not found.** with critical slip surfaces shown in Figure 9-7 to Figure 9-11. The geometry in the fully softened and residual strength analysis cases was conservatively modified to create a more circular slide plane failure. Seismic yield coefficients for the fully softened shear and residual strength cases with the unmodified geometry were only slightly higher (less than 0.05g) than those with the modified geometry.

As shown, median predicted displacements of slip surfaces within the landslide (not fully softened or residual strengths) for the 84th percentile motion ranged from 2 feet to 4 feet and ranged from 3 to 4 feet for the 10,000-year motion. Median predicted displacements of the landslide plane using fully softened shear strength for the 84th percentile motion and the 10,000-year motion were approximately 8 feet and 7 feet, respectively. Predicted median displacements of the landslide plane were considerably higher using residual shear strength along the slide plane.

Displacements were in excess of tens of feet would be expected if a continuous slide plane with residual strength material exists at the bottom of the landslide. However, based on the available information from the recent borings and borehole televiewer, a residual strength shear plane was not observed at the base of the landslide. For this reason, we consider the fully softened strengths and related results to be more appropriate.

For the largest historical earthquake since construction of the dam (Mw 5.7 at 10km from the site in 1969), less than 0.1 feet of movement was predicted using fully softened strengths and up to 2 feet of movement was predicted using residual shear strengths. Since no movement of the landslide was noted after the 1969 Santa Rosa earthquake, the fully softened strengths appear to be appropriate.

Table 9-2: Summary of Simplified Seismic Displacement Calculations - Landslide

Slip Surface	Static Factor of Safety	Seismic Analysis													
		Yield Coefficient, Ky	Slip Surface Height, H (feet)	Fundamental Period of Slip Surface, Ts (seconds)	1.3 x Ts (seconds)	84th Percentile Deterministic Motion (DSOD Criteria)					10,000-year Motion (NRCS Criteria)				
						Spectral Acceleration at 1.3Ts, Sa(1.3Ts) (g)	Probability of Greater Than 5 feet of Displacement	Displacement by Probability of Exceedance (feet)			Spectral Acceleration at 1.3Ts, Sa(1.3Ts) (g)	Probability of Greater Than 5 feet of Displacement	Displacement by Probability of Exceedance (feet)		
								84%	50%	16%			84%	50%	16%
Global	3.6	0.38	20	0.062	0.080	1.58	9%	1.3	2.3	4.1	2.97	17%	1.7	2.9	5.1
Local	3.1	0.36	25	0.077	0.100	1.75	15%	1.6	2.8	4.9	3.28	24%	1.9	3.3	5.8
Deeper	3.6	0.39	45	0.138	0.180	2.22	28%	2.1	3.6	6.3	3.98	34%	2.3	4.0	7.0
Landslide Plane (Fully Softened Strength)	2.9	0.28	110	0.338	0.440	2.40	79%	4.5	7.8	14	4.11	76%	4.2	7.4	13
Landslide Plane (Residual Strength)	1.2	0.05	110	0.338	0.440	2.40	~ 100%	27	47	82	4.11	~ 100%	15	26	45

10. Conclusions and Recommendations

Based on the analyses performed as part of this report, the following broad conclusions can be made:

- The existing filter/drain for the embankment is not compatible with the embankment fill and does not meet permeability requirements. However, based on analysis recommendations by Foster and Fell (2001), the filter/drain is unlikely to continue to allow eroding of the embankment or foundation, although there is some uncertainty due to the size of the gravel compared to the sampler affecting the gradation. No signs of internal erosion were observed during our site visit, but it is possible any erosion hasn't reached the ground surface or signs of internal erosion are hidden by vegetation or surface runoff. It should be noted that with the current operation plan, which leaves the outlets open allowing the reservoir to empty in about 2 days, steady-state seepage is unlikely to occur except when numerous storms occur back-to-back. Given the aforementioned conclusions, the filter/drain is considered adequate for the current operation plan (i.e. reservoir being kept empty). If the operation plan changes such that water is held behind the embankment for extended period of time, then a remediation to the filter/drain is recommended.
- Seepage analyses of the embankment showed that in steady-state flow from the auxiliary spillway level, water could exit onto the embankment face above the top of the filter/drain and the factor of safety against heave was slightly less than the NRCS requirement. As discussed above, it is unlikely steady-state conditions will form at the auxiliary reservoir level, but we recommend that if the auxiliary spillway reservoir level is reached then an inspection be performed to look for seepage near the toe of the downstream face of the dam.
- Stability analysis results for the dam embankment met NRCS and DSOD criteria except for seismic stability (e.g., post-seismic case). Key findings of the stability analyses are noted as follows:
  - Sensitivity on the long-term stability using Hoek-Brown strengths for the rock met criteria with a UCS value of 12 ksf. Lower UCS values did not meet criteria. Based on this information, the strength of the rock is important to the stability of the embankment and should be investigated further.
  - Liquefaction was predicted within the embankment fill, filter/drain, and alluvium. More rigorous analyses, such as performing a dynamic site response and/or nonlinear finite element analysis, are recommended to better understand the deformation pattern of the dam and incorporate any deformation due to liquefaction.
  - Some uncertainty exists in regard to the unit weight, gradation, strength, and density of the embankment, filter/drain, and alluvium. Further investigation including gathering of relatively undisturbed samples of the embankment fill and alluvium may be helpful.
  - Seismic deformation analysis results (without consideration of liquefaction) indicate median displacements of about 2 to 3 feet (84^{th} percentile motion) and 4 to 7 feet (10,000-year motion). Seismic deformation for the operating basis earthquake (500-year event) produced median displacements of about 1 foot. Further evaluations using more sophisticated methods of analysis (i.e. nonlinear finite element analyses) are necessary to confirm whether estimated deformations are tolerable and to assess the effect of liquefaction on displacement estimates.
- The landslide was found to be statically stable but may be subject to significant movements during the design earthquake. Given the relatively flat slopes, findings from the borings and borehole televiewer, and results of the analyses summarized in this section, the landslide is judged to be statically stable. It is postulated that the landslide is old and had moved downslope to its current stable position as a result of a large prehistoric earthquake on the Rodgers Creek fault, possibly during a wet winter or spring when groundwater levels would have been higher.
- The Matanzas Creek Fault is considered inactive (per CGS and USGS definitions), and a recent investigation appeared to confirm those findings.
- The results indicate that remediation efforts may be required to stabilize the embankment during a seismic event. Details on potential remediation alternatives will be presented in an upcoming report.

Given these findings, it is recommended that the dam continue to be operated with the gates fully open and the reservoir dry until any remediations are made.

11. Limitations

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of the engineering profession practicing in the same locality, under similar conditions and at the date the services are provided. The conclusions, opinions, and recommendations in this report are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. AECOM makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

Some background information and other data used by AECOM in preparing this Report have been furnished by third parties. AECOM has relied on this information as furnished and is neither responsible for nor has confirmed the accuracy of this information.

The assessment of the conditions reported was based on available data, limited data collected, and visual observations. The reported geologic conditions were based on observations of field conditions at the time of the field reconnaissance and limited subsurface investigation, along with data available to the team.

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AECOM Oakland CA 1/10/2022 USER ONealC1 PATH: \\na.aecomnet.com\fs\AMER\Oakland\USOAK01\DCS\Projects\GIS\Projects\60647670_MatanzasCreek\02_Maps\02_Report_Maps\FieldInvestigationPlan\Figure_1_SiteLocation.mxd

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Matanzas Creek Dam - Soil Mechanics Report

Sonoma County, California

PROJECT NO. 60647670

FIGURE 1-1

Site Location Map

Matanzas Creek Dam

af

anticline fold axis

anticline fold axis, approx.

anticline fold axis, conc. queried

anticline fold axis, concealed

axis line

axis line, approx. located

axis line, concealed

contact, approx. loc., queried

contact, approx. located

contact, certain

contact, concealed

contact, concealed, queried

fault, approx. loc., queried

fault, approx. located

fault, certain

fault, concealed

fault, concealed, queried

fault, queried

lineaments

marker bed

normal fault certain, ball & bar

sag pond

scarp

strike-slip fault, RL

syncline fold axis

syncline fold axis, approx.

syncline fold axis, concealed

thrust fault, approx.

thrust fault, approx., queried

thrust fault, certain

water boundary

Geologic Units

Surficial Deposits

af: Artificial Fill

Qha: Alluvium, undivided (Holocene)

Qhb: Basin Deposits (Holocene)

Qhc: Channels (Holocene)

Qhf: Alluvial fan and fluvial terrace deposits, undivided (Holocene)

Qhf1: Young Holocene alluvial fan and fluvial terrace deposits

Qhf2: Old Holocene alluvial fan and fluvial terrace deposits (Holocene?)

Qal: Alluvial deposits, undivided (Holocene and/or Pleistocene)

Qc: Colluvium (Holocene and/or Pleistocene)

Qhp: Alluvial deposits, undivided (Holocene and Pleistocene)

Qhpf: Alluvial fan and terrace deposits (Holocene? and/or Pleistocene)

Qhpf1: Younger alluvial fan and terrace deposits (Holocene? and/or Pleistocene)

Qhpf2: Older alluvial fan and terrace deposits (Holocene? and/or Pleistocene)

Qls: Landslide deposits (Holocene and Pleistocene)

Qoa: Older alluvium, undivided (Pleistocene)

Qpf: Alluvial fans and fluvial terrace deposits, undivided (Pleistocene)

Qt: Alluvial deposits, undivided (Holocene and Pleistocene)

Early Quaternary and Tertiary Units

QTg: Fluvial and lacustrine deposts (early Pleistocene and Pliocene)

Tertiary Rocks

Tgc: Gravel of the Cloverleaf Ranch (Pliocene?)

Tgp: Fluvial and lacustrine deposits of Humbug Creek (Pliocene)

Tp: Petaluma Formation (Pliocene and Miocene)

Tpd: Diatomite and diatomaceous mudstone

Tplg: Lignite and lignitic mudstone

Tpwd: Silicified (petrified) wood

Tprb: Breccia

Tprg: Gravel lenses

Late Tertiary Volcanic Rocks

Tsd: Dacitic flows

Tsr: Rhyolitic and rhyodacitic flows and intrusive rocks

Tst: Rhyolitic to dacitic and minor andesitic pumiceous tuff

Tstw: Crystal-rich rhyolitic to rhyodacitic welded tuff

Tstb: Andesitic to rhyodacitic breccia (Pliocene)

Tsb: Andesite, basaltic andesite and basalt

Tsbt: Andesitic to dacitic tuff, breccia and minor flows

Pre-Tertiary Rocks

Jos: Serpentinized ultramafic rocks

fcm: Undifferentiated melange

Other

H20: Water

sag: Sag

Matanzas Creek fault

Matanzas Creek Dam

DRAFT

N

0 1

Miles

ESRI Imagery, 2021
USGS geologic data: McLaughlin et al., 2008
AECOM, 2021

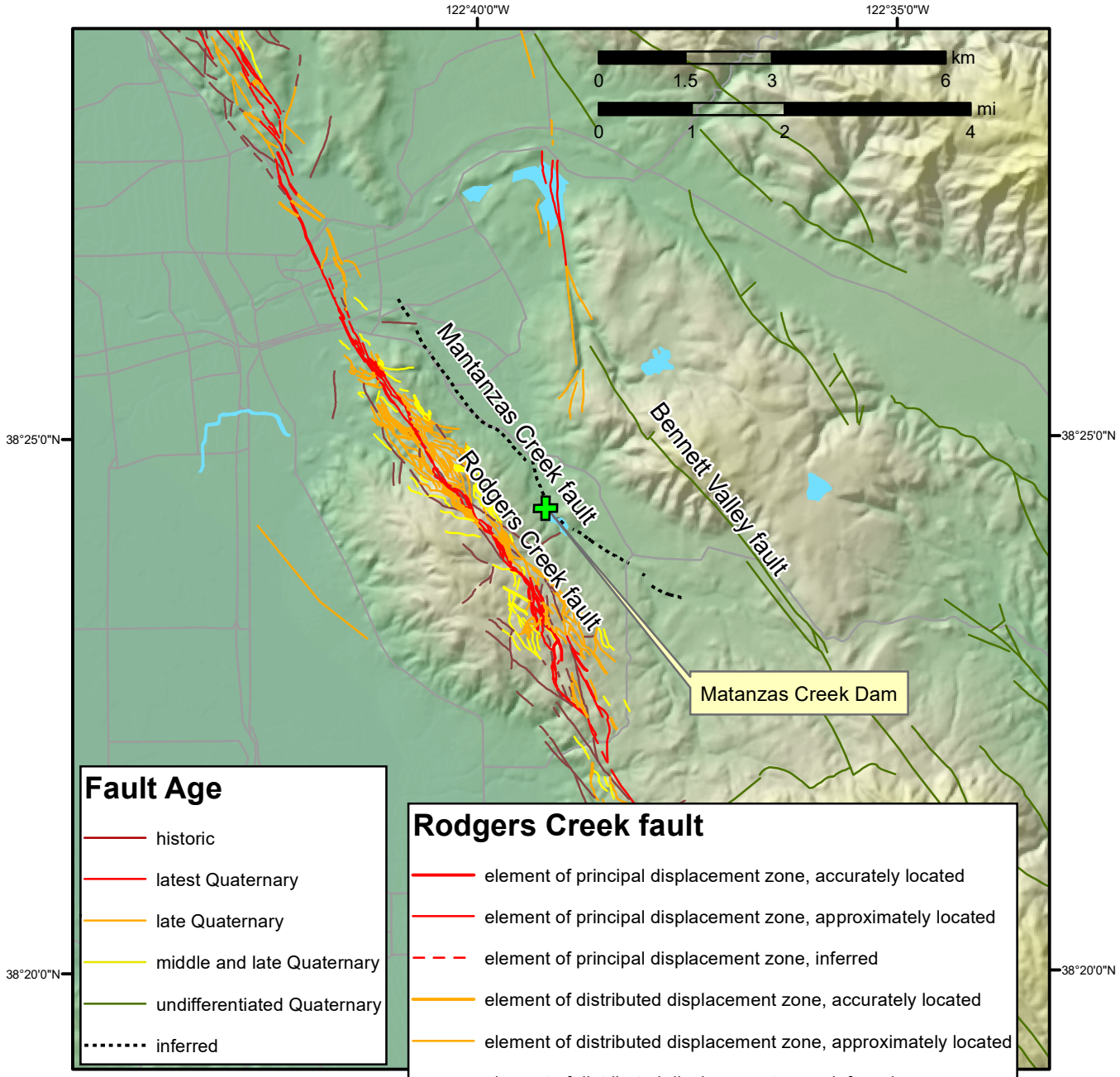
AECOM Oakland CA 1/17/2022 USER ONealC1 PATH: \\pa.aecomnet.com\ifs\AMER\Oakland\US0AK01\DCS\Projects\GIS\Projects\60647670_MatanzasCreek\02_Maps\02_Report_Maps\ExistingConditions_Geologic\Figure_4_Regional Geologic Map Portrait.mxd

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Natural Resources Conservation Service
Matanzas Creek Dam Watershed Rehabilitation Project
SOIL MECHANICS REPORT

FIGURE 3-1

Regional Geologic Map

DRAFT



notes:
Rodgers Creek/Healdsburg fault from Hecker and Randolph Loar, 2018
Matanazas Creek inferred fault from McLaughlin et al., 2008
all other faults from USGS Quaternary database, 2019

N

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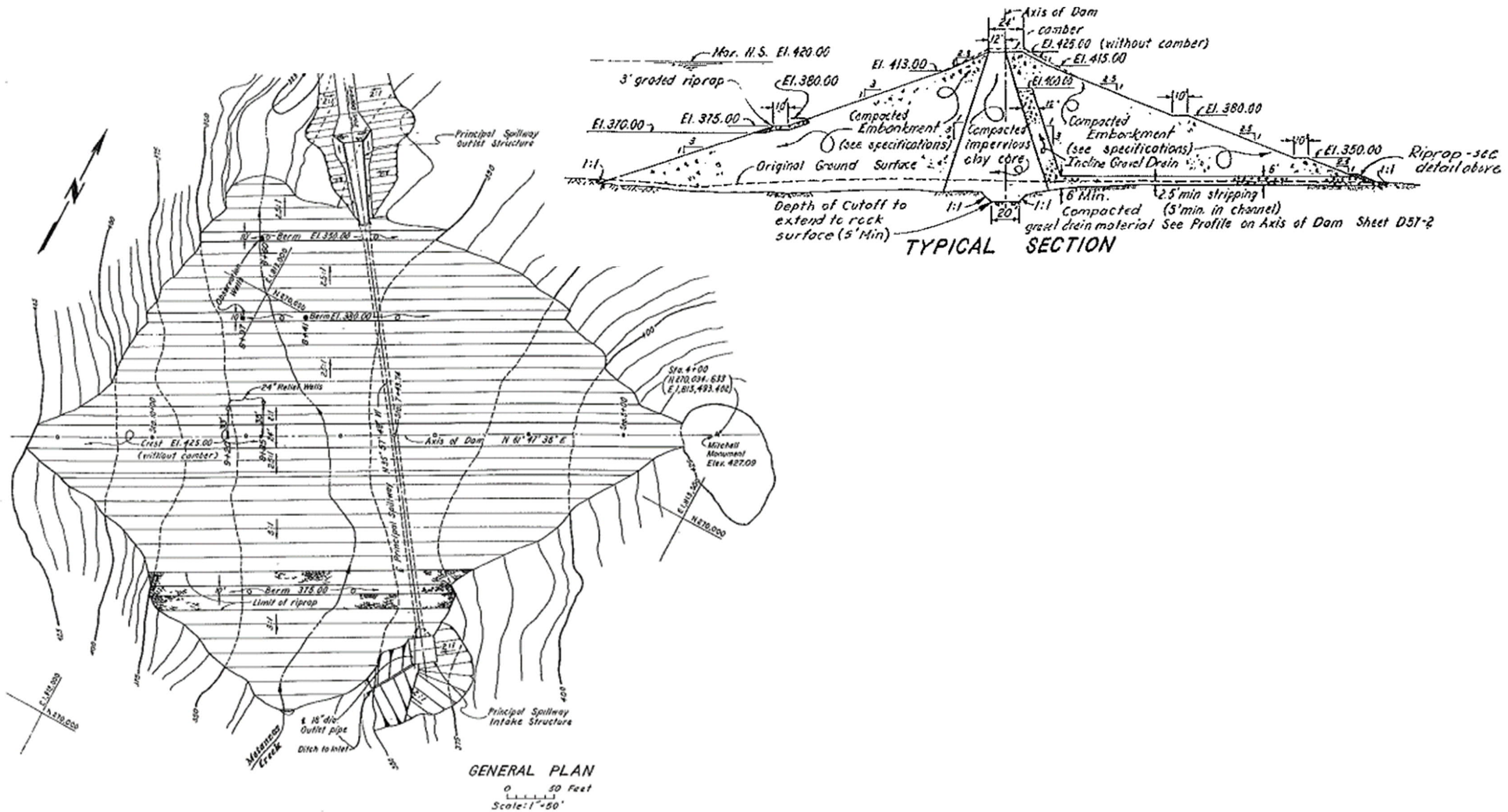
Matanzas Creek Dam - Soil Mechanics Report

Sonoma County, California

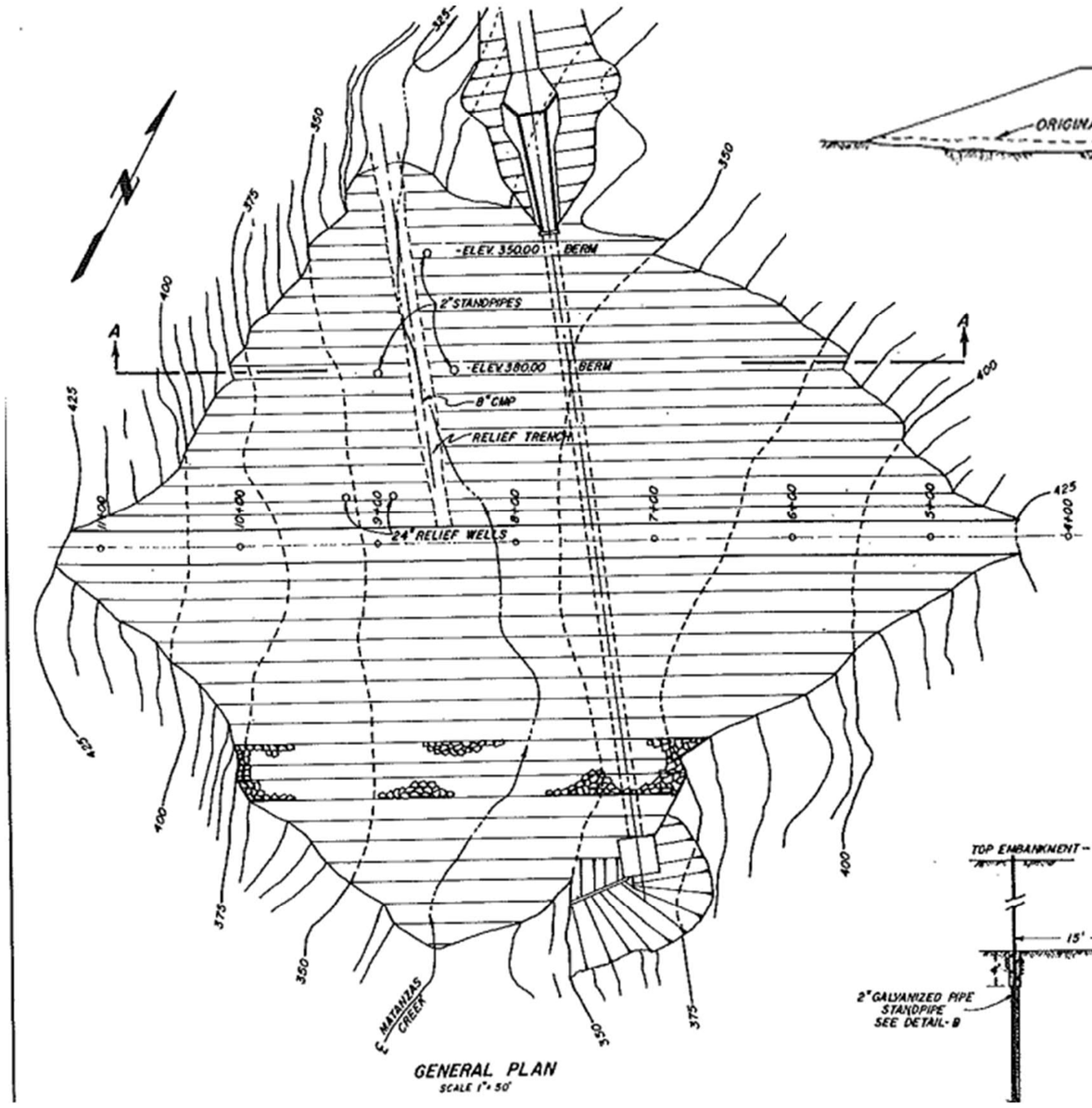
PROJECT NO. 60647670

FIGURE 3-2

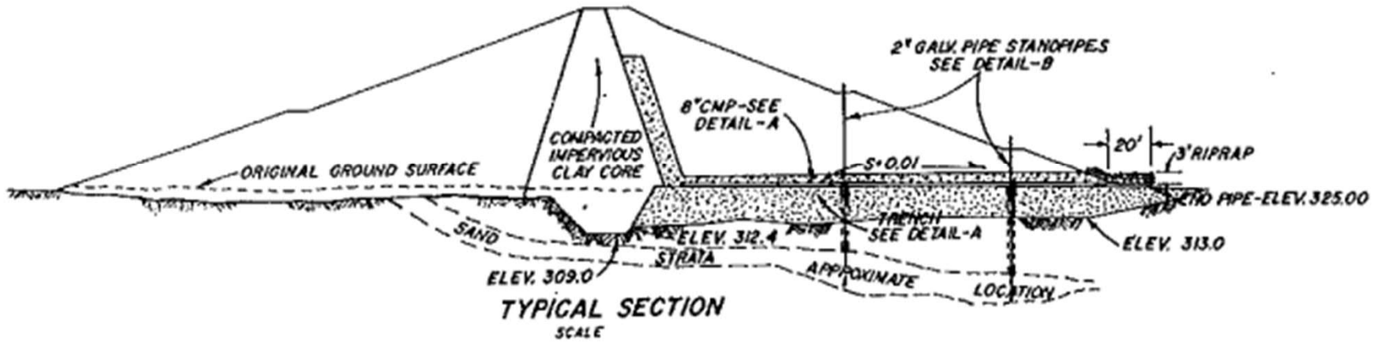
Rodgers Creek Fault and Other Local Faults



Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	As-Built Embankment Plan and Section Maximum Section	Figure 4-1
	AECOM		



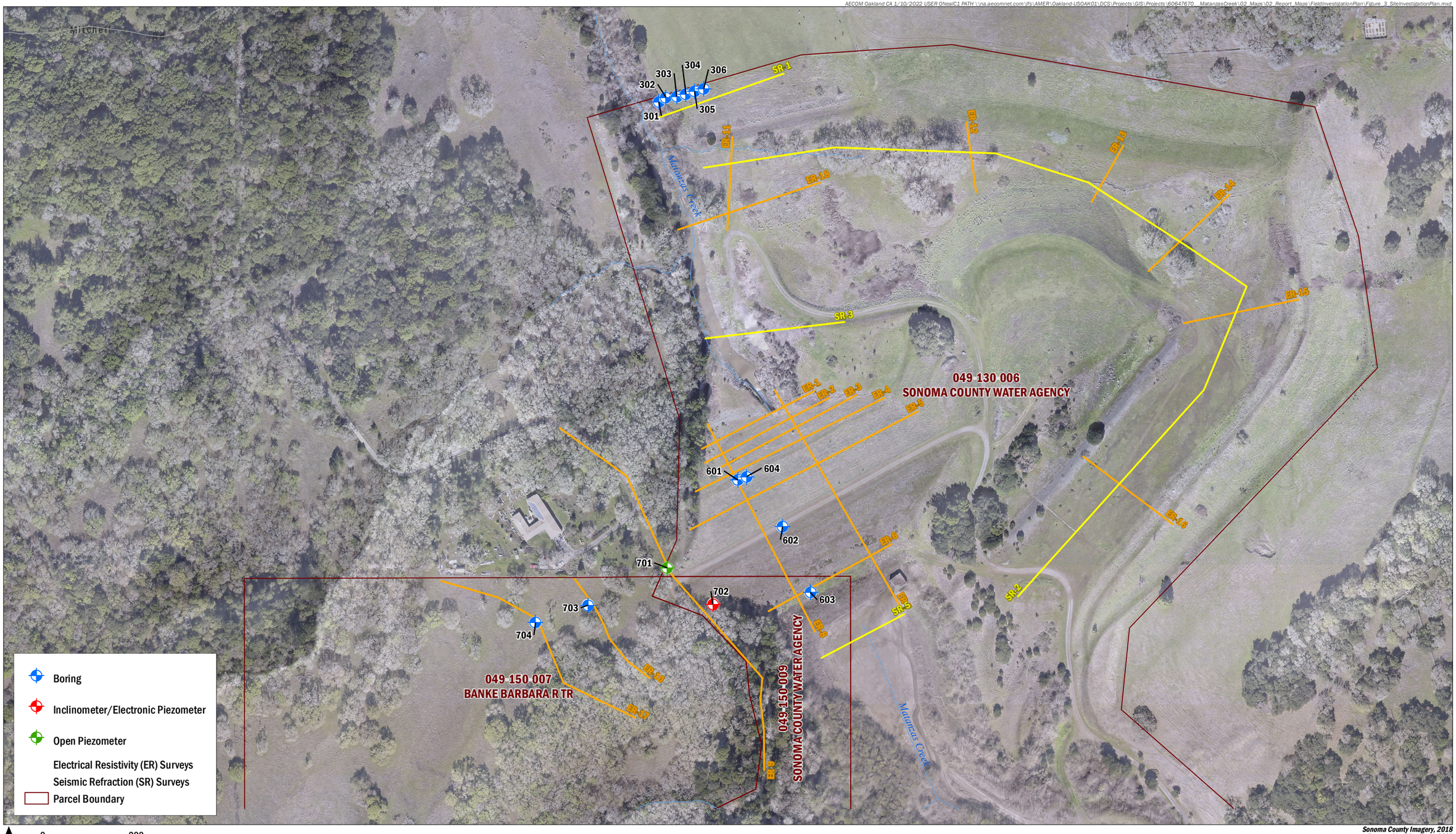
GENERAL PLAN
SCALE 1" = 50'



TYPICAL SECTION
SCALE

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	As-Built Embankment Plan and Section Relief Trench Section	Figure 4-2
AECOM			

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Sonoma County Imagery, 2018
Sonoma Veg Map Topo, 2020
Sonoma County Parcels, 2020
AECOM, 2021

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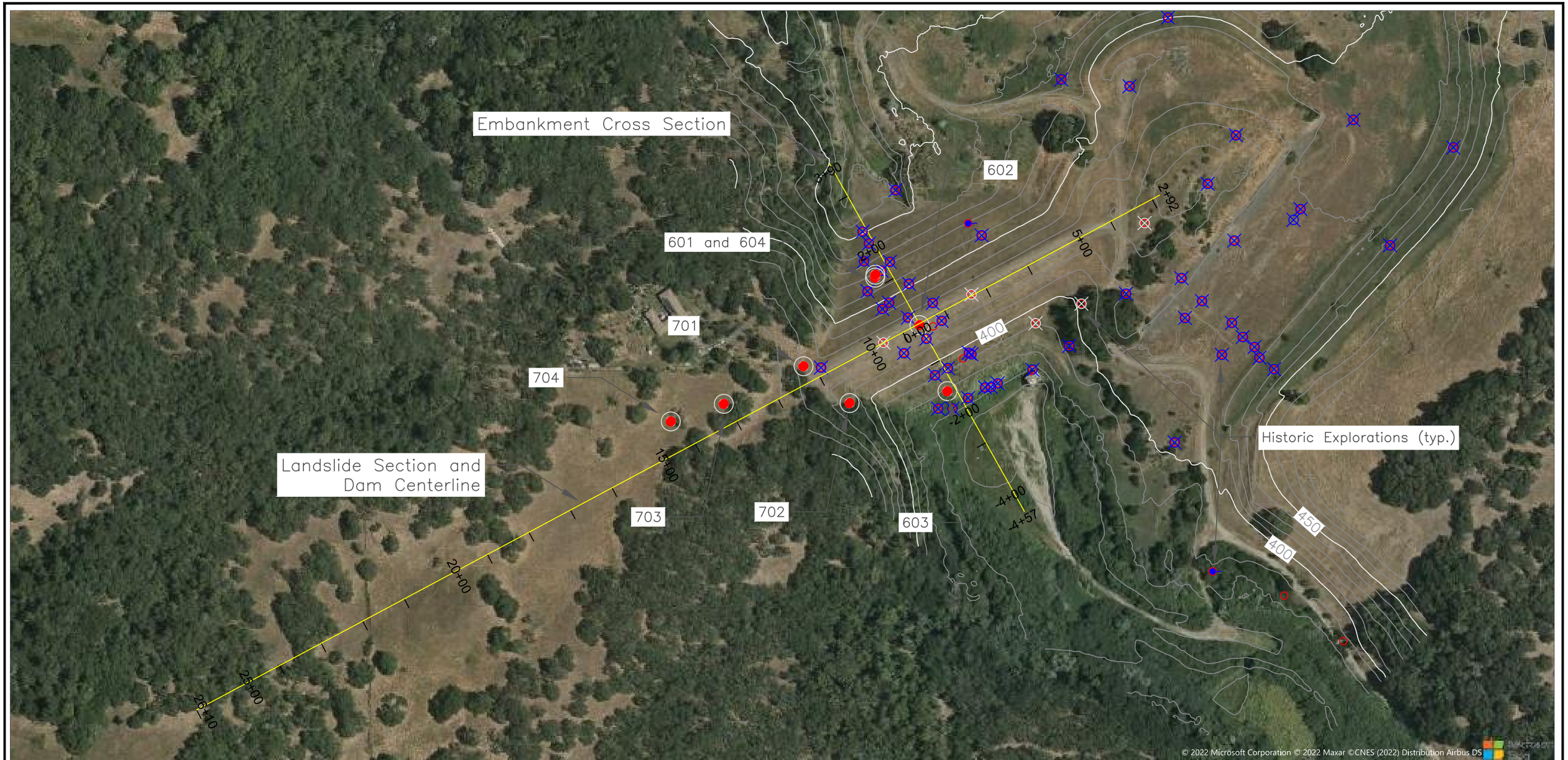
Matanzas Creek Dam - Soil Mechanics Report

Sonoma County, California

PROJECT NO. 60647670

FIGURE 5-1

Site Investigation Map



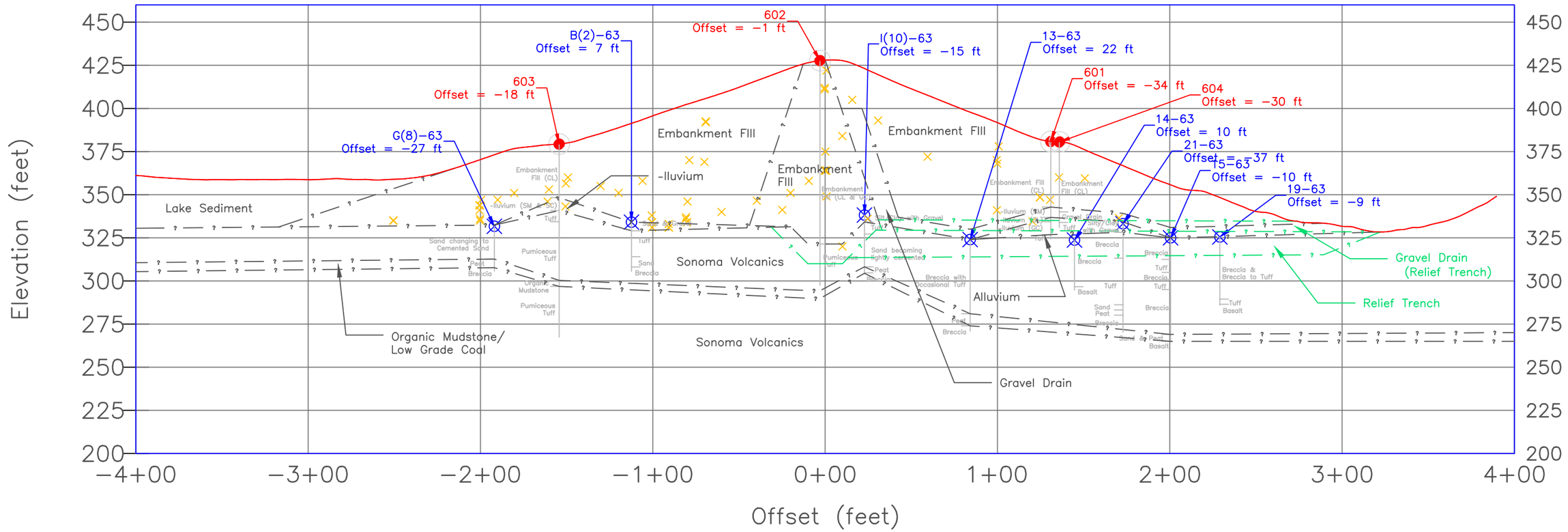
N

0 200 400 Feet

Scale: 1–inch = 200 feet

AECOM	Project No. 60647670 Sonoma County Matanzas Creek Dam Soil Mechanics Report	EMBANKMENT SECTIONS PLAN VIEW	Figure 6–1

EMBANKMENT BORINGS SECTION 2 PROFILE



2021 Exploration (This Study)

Previous Exploration (Pre−Construction)

Location of Construction Density Test

Inferred Stratigraphy

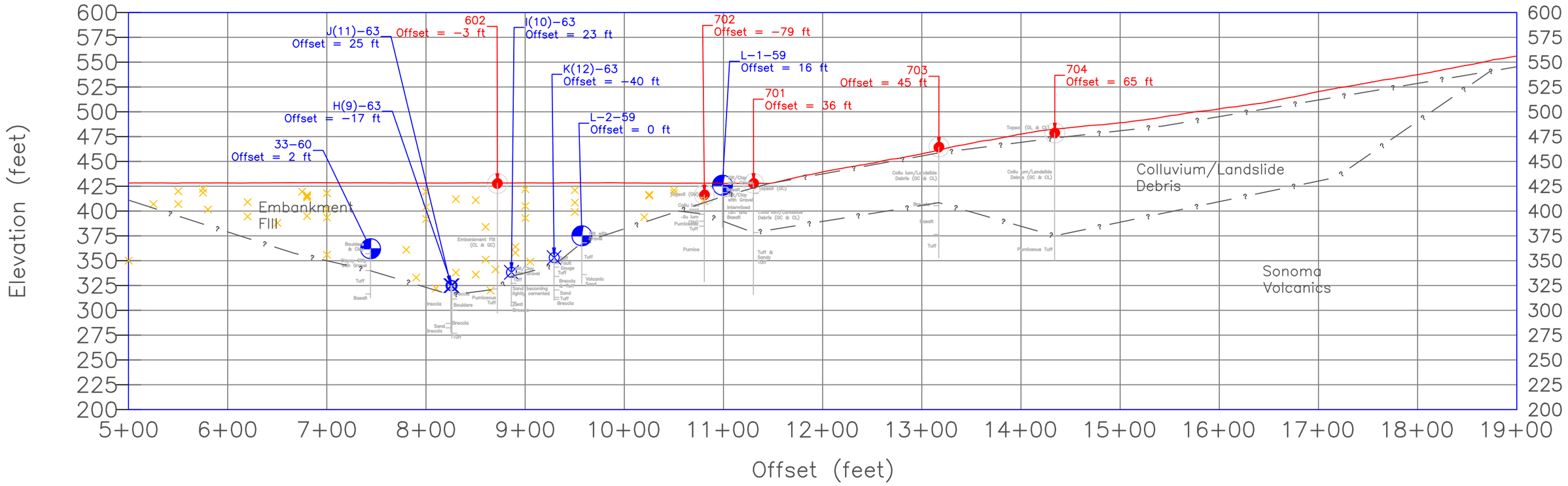
Inferred Stratigraphy (Relief Trench)

0 60 120 Feet

Scale = 1:60

AECOM	Project No. 60647670 Sonoma County Matanzas Creek Dam Soil Mechanics Report	EMBANKMENT SECTIONS EMBANKMENT CROSS SECTION	Figure 6−2

EMBANKMENT CENTERLINE & LANDSLIDE PROFILE



2021 Exploration (This Study)

Previous Exploration (Pre–Construction)

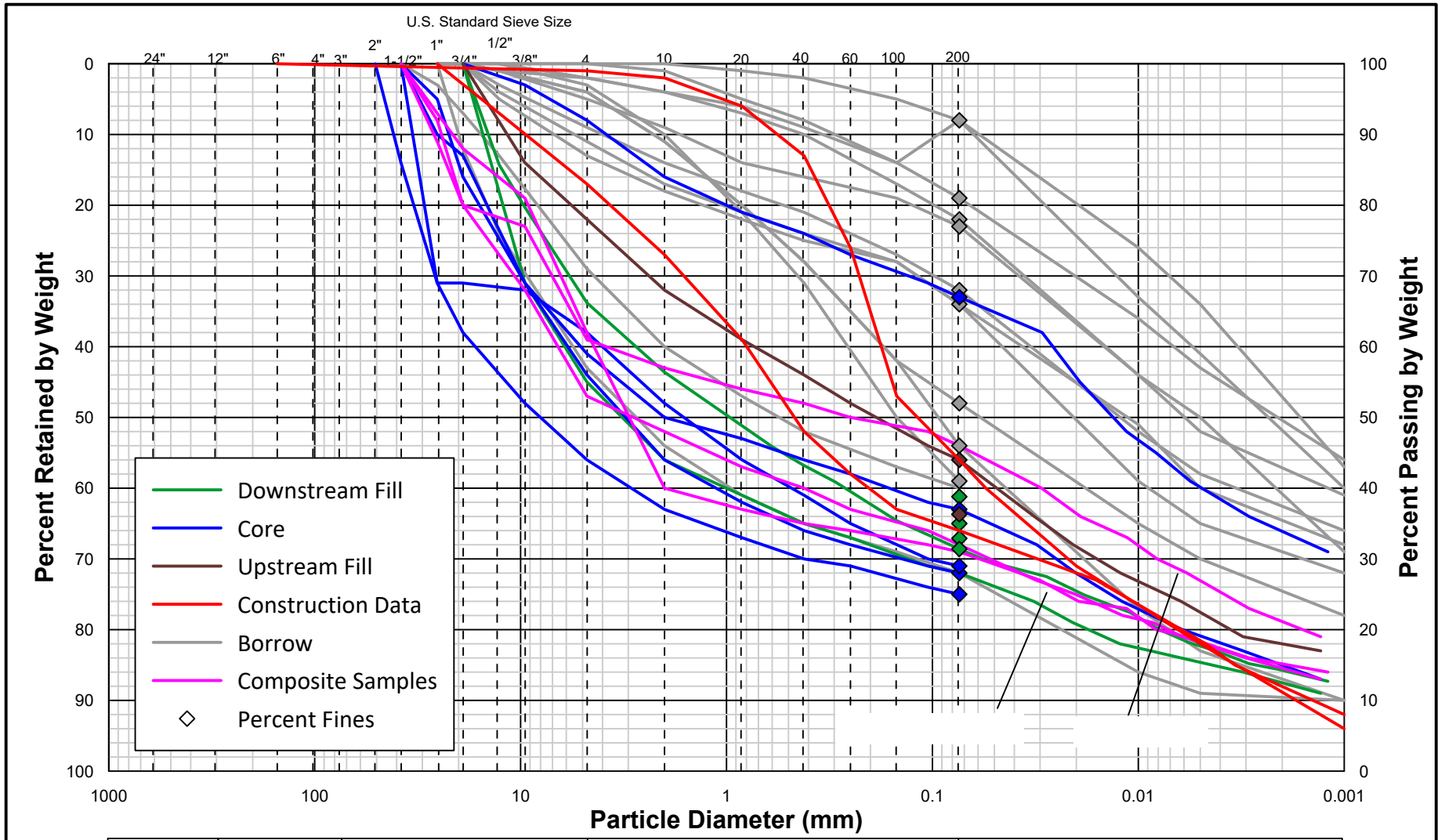
Location of Construction Density Test

Inferred Stratigraphy

0 100 200 Feet

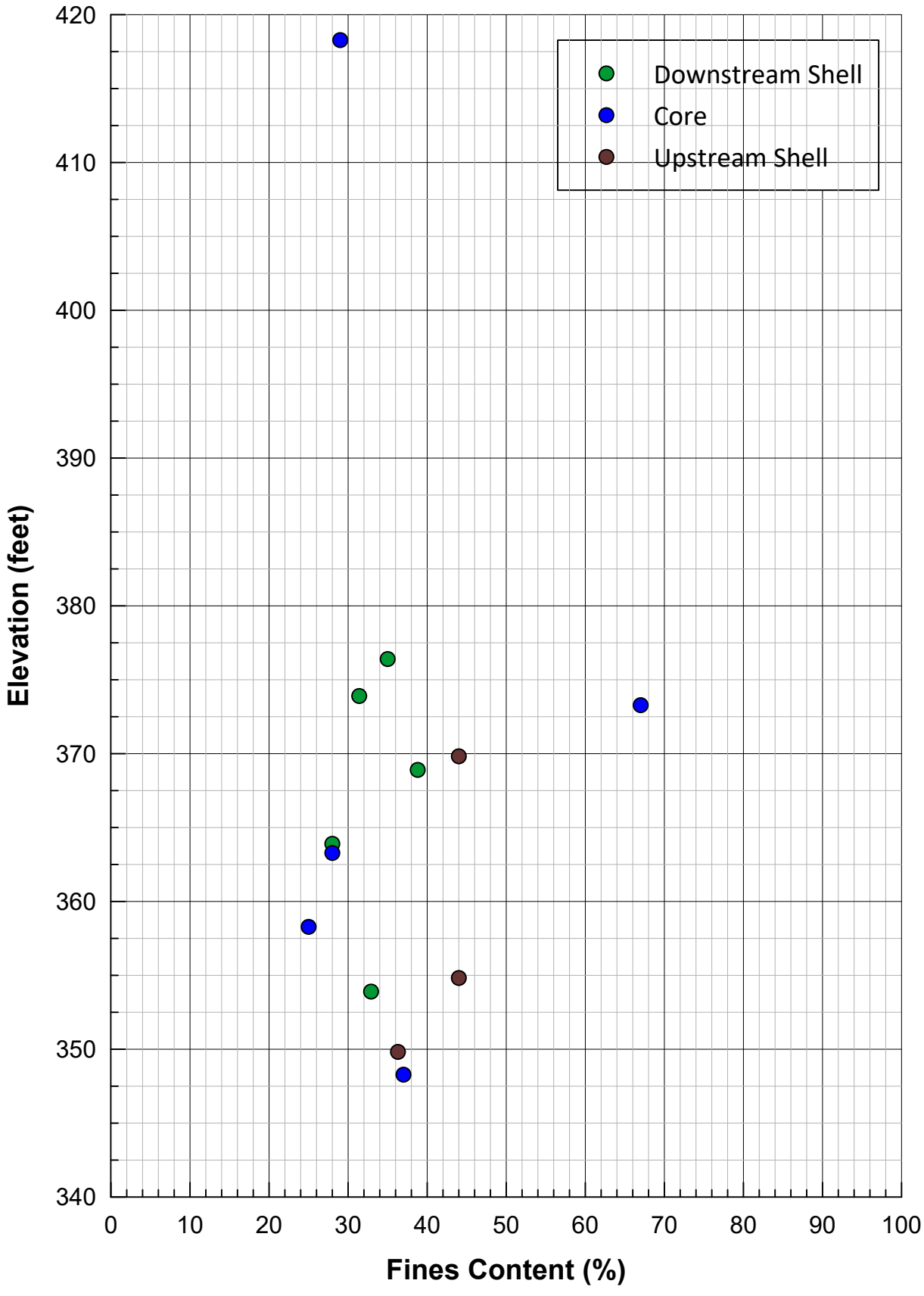
Scale = 1:100

AECOM	Project No. 60647670 Sonoma County Matanzas Creek Dam Soil Mechanics Report	EMBANKMENT SECTIONS LANDSLIDE CROSS SECTION	Figure 6–3



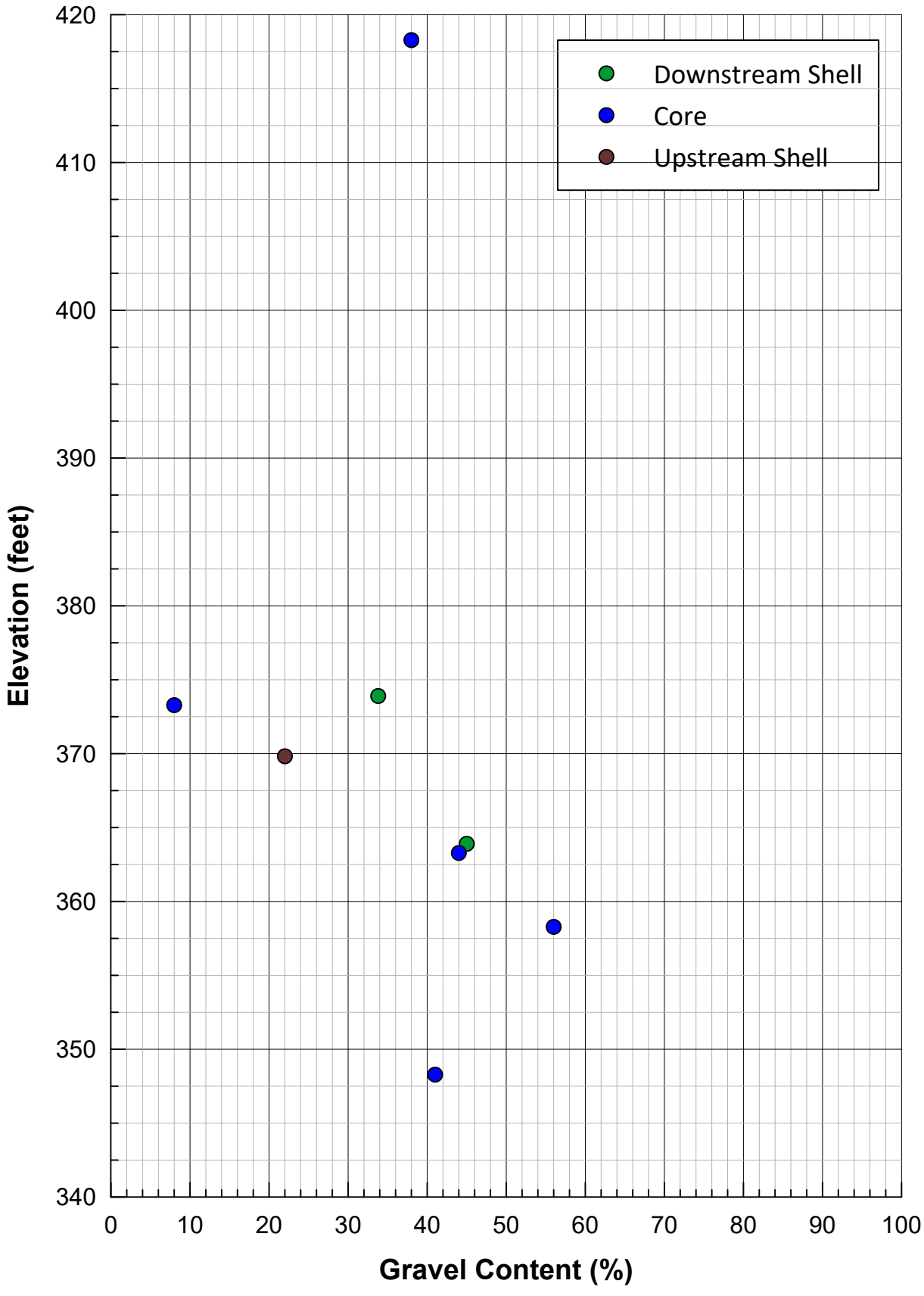
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Figure 6-4
AECOM	Embankment Fill Gradation Test Results	

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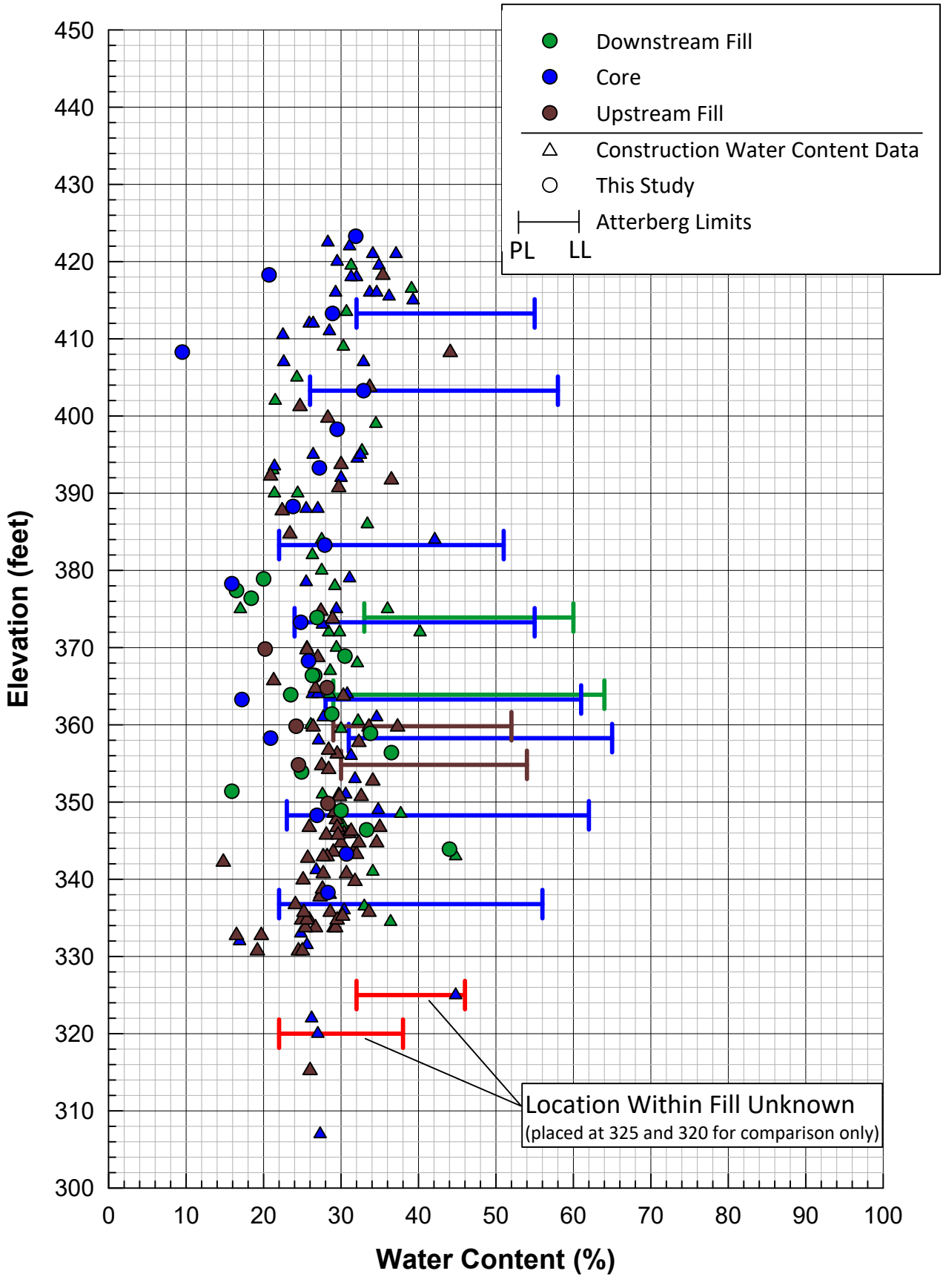
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Fines Content	Figure 6-5

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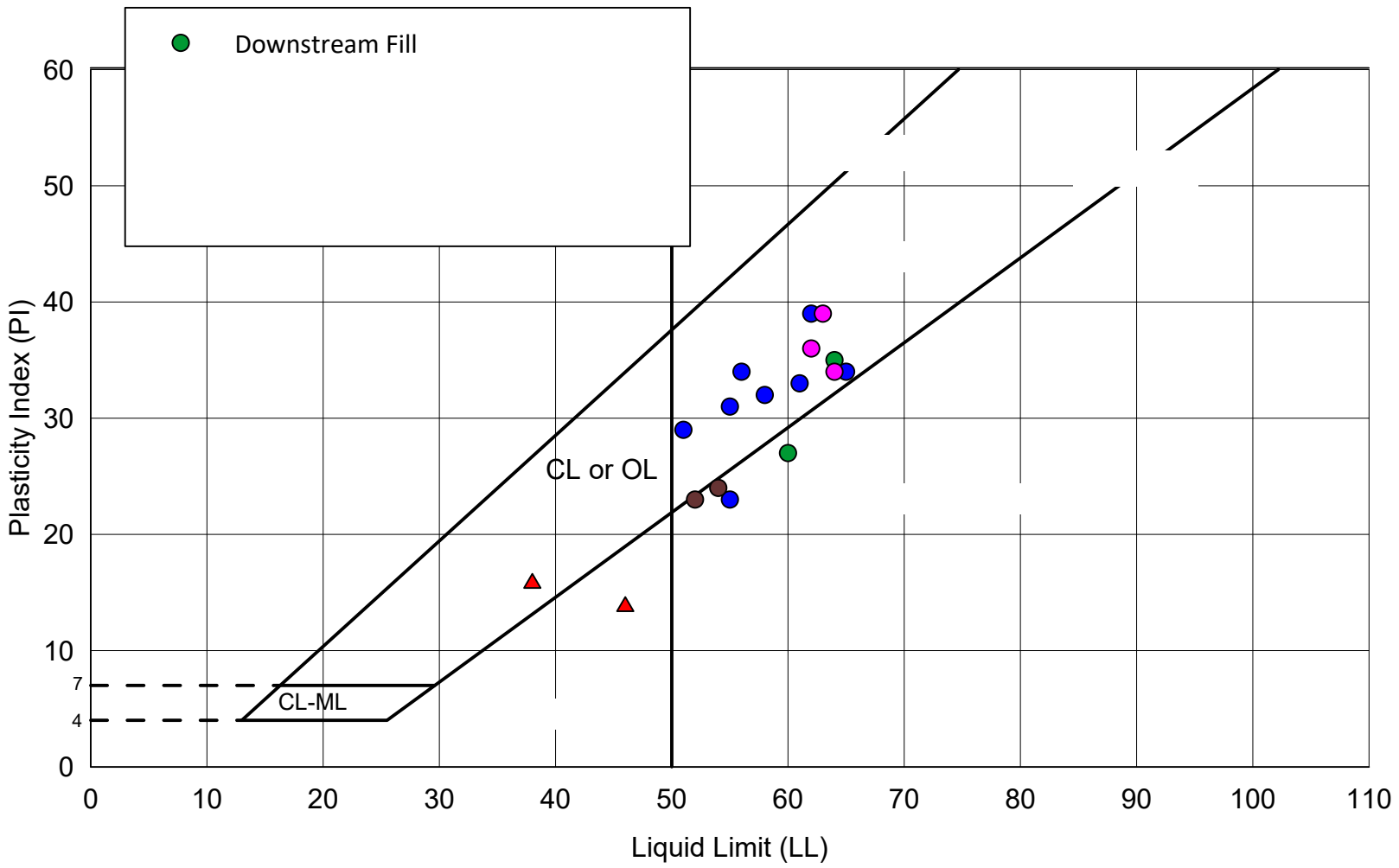
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Gravel Content	Figure 6-6

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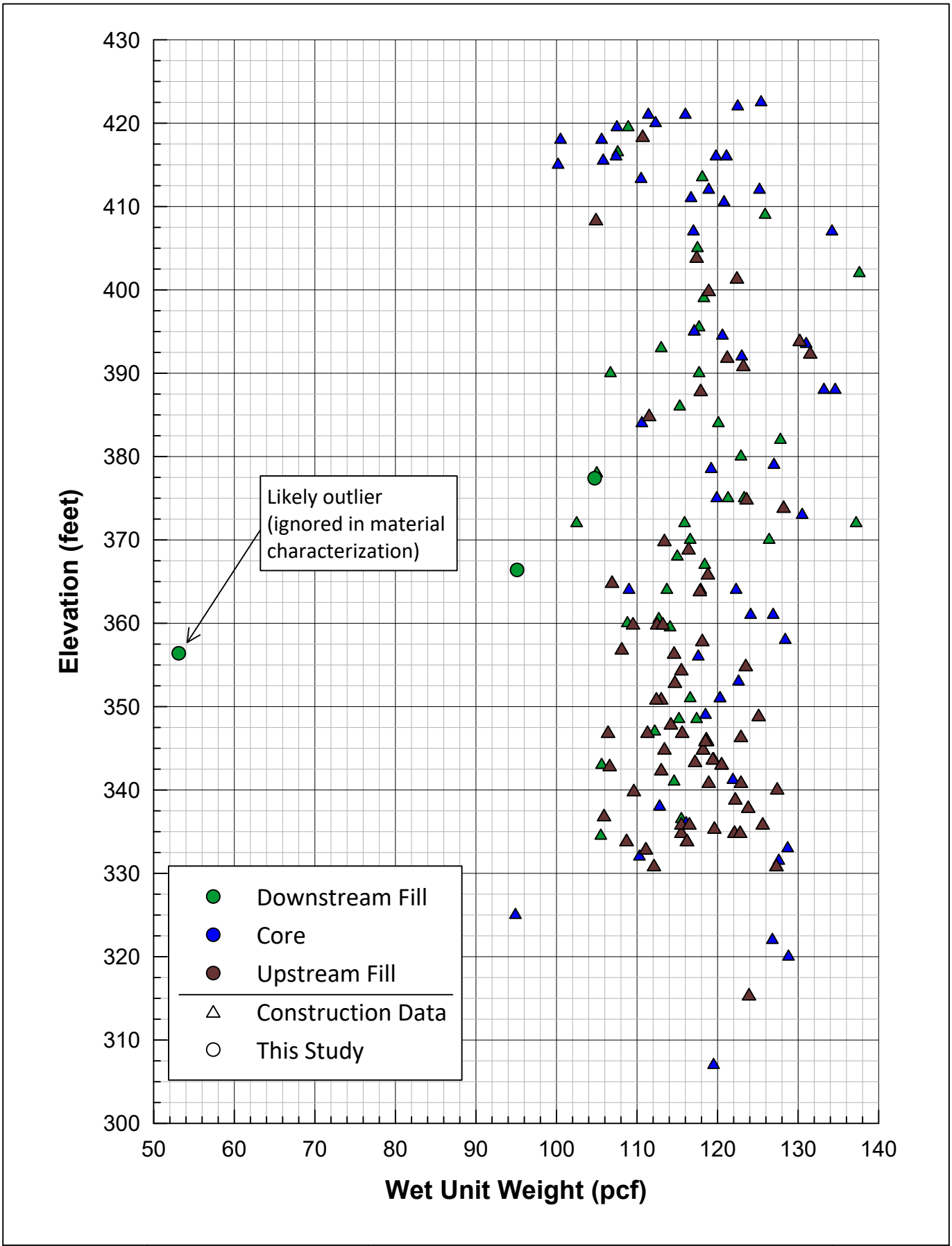
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Water Content & Plasticity	Figure 6-7

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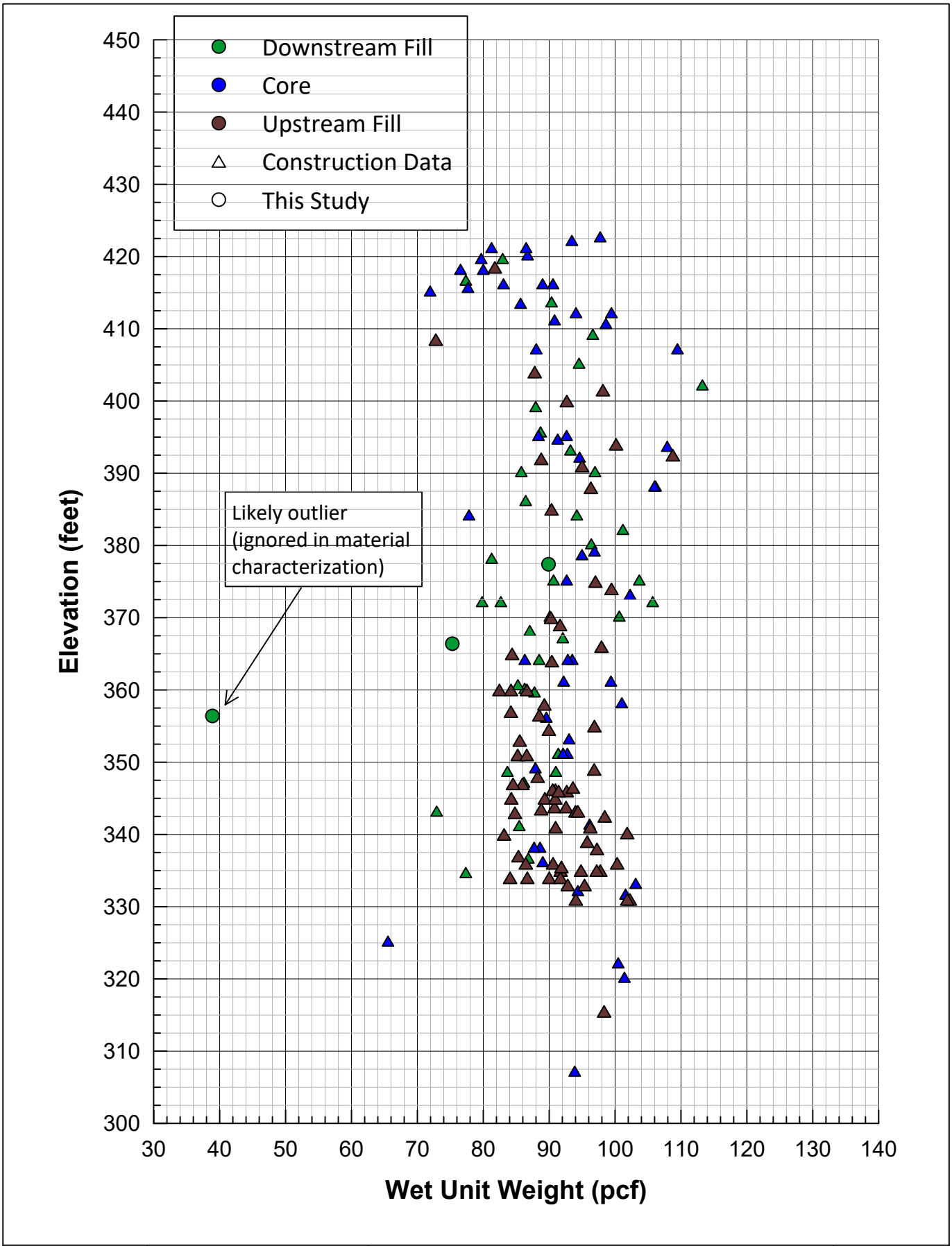
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Plasticity Chart	Figure 6-8
AECOM			

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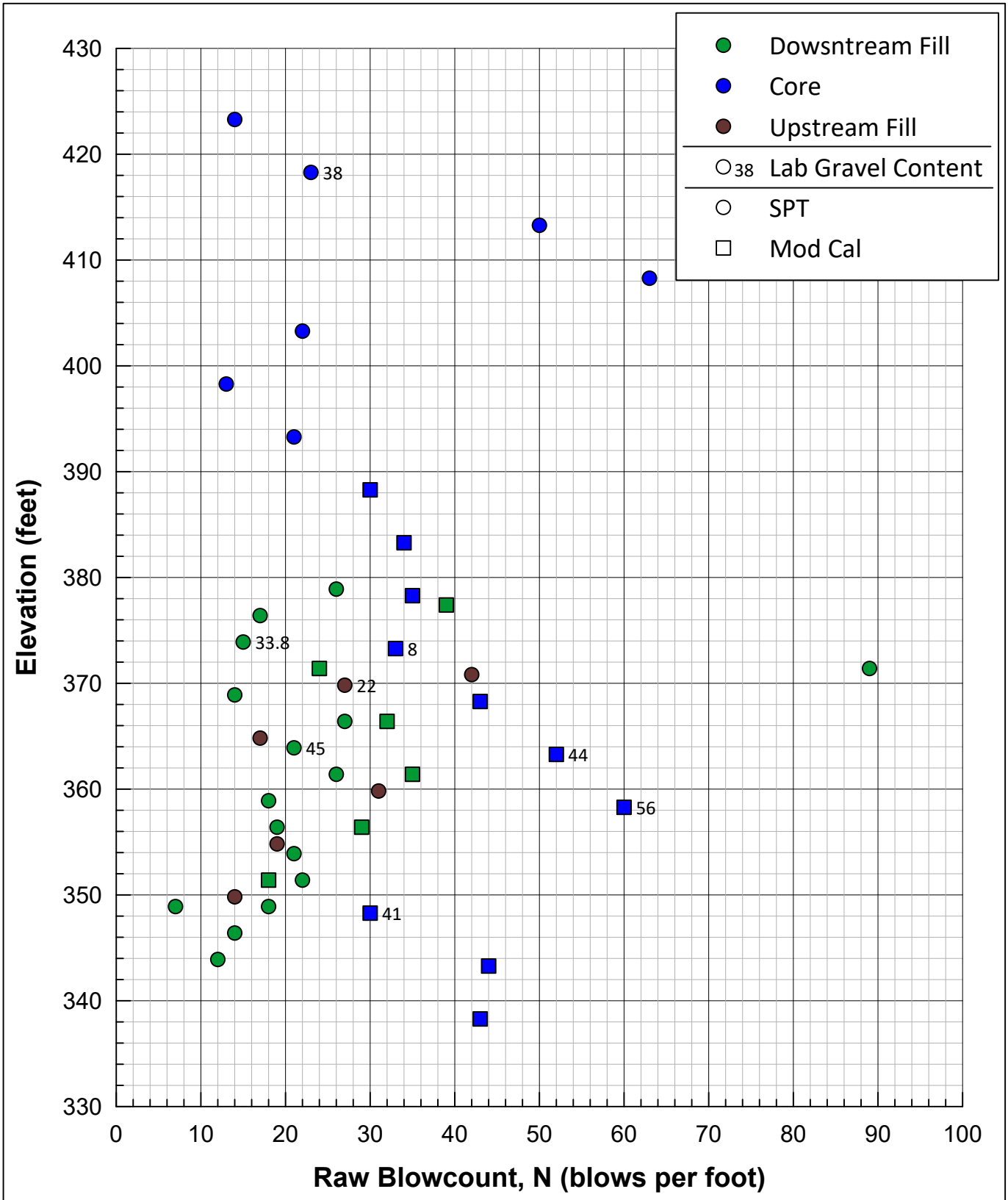
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Wet Unit Weight	Figure 6-9

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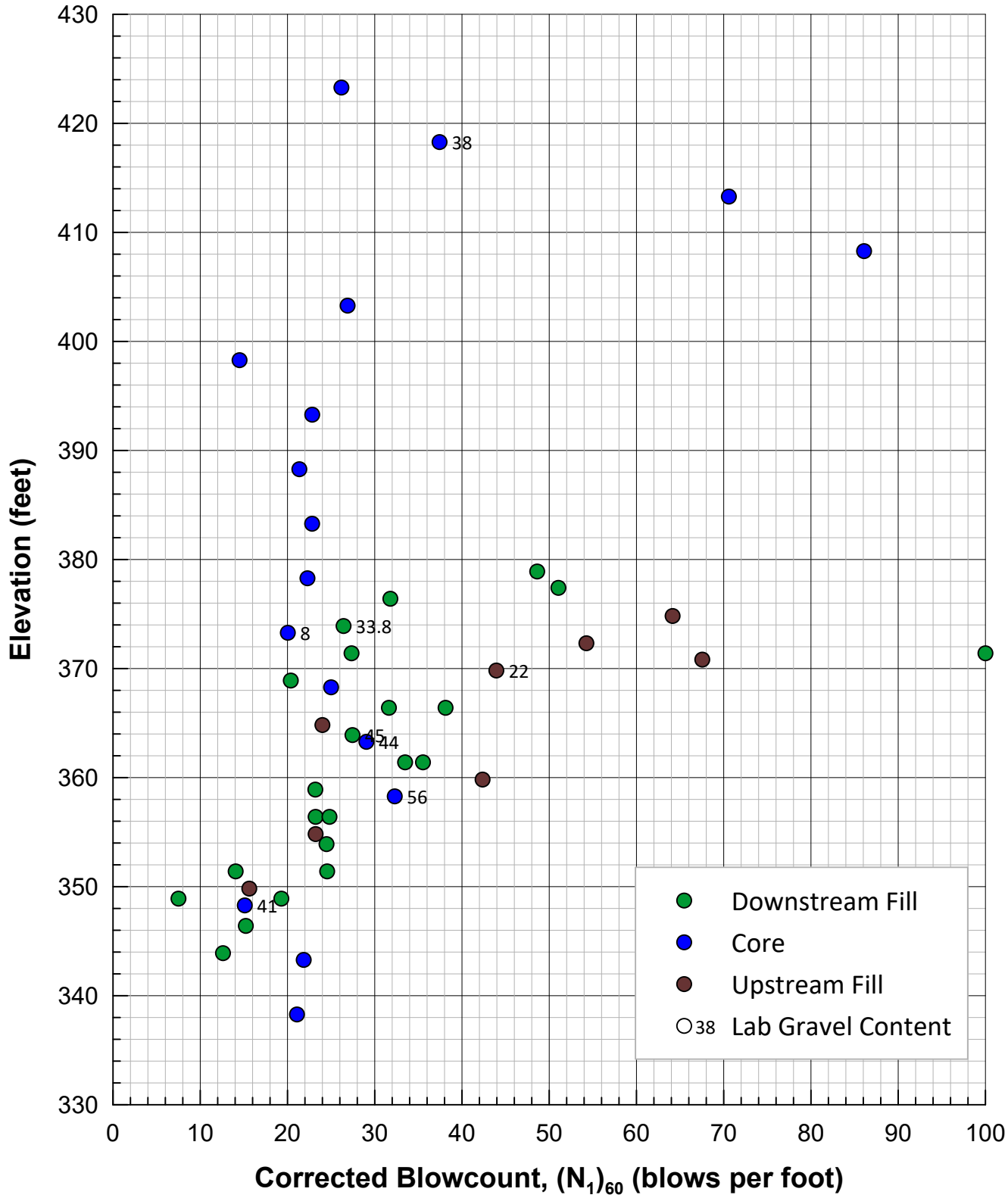
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Dry Unit Weight	Figure 6-10

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AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Uncorrected Blowcount, N	Figure 6-11

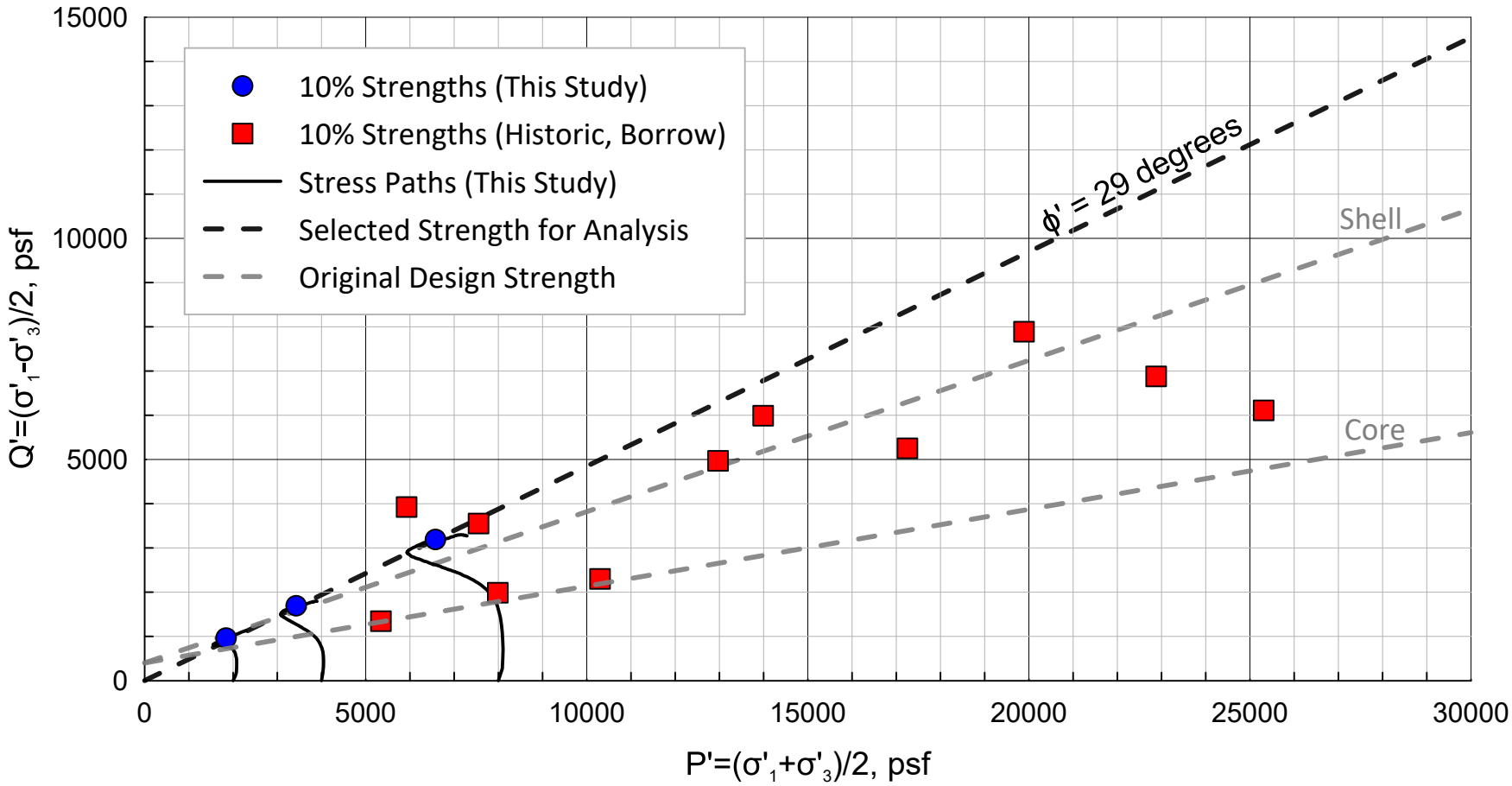
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Notes:
- Refusal and N values greater than 100 are plotted at blowcount of 100.

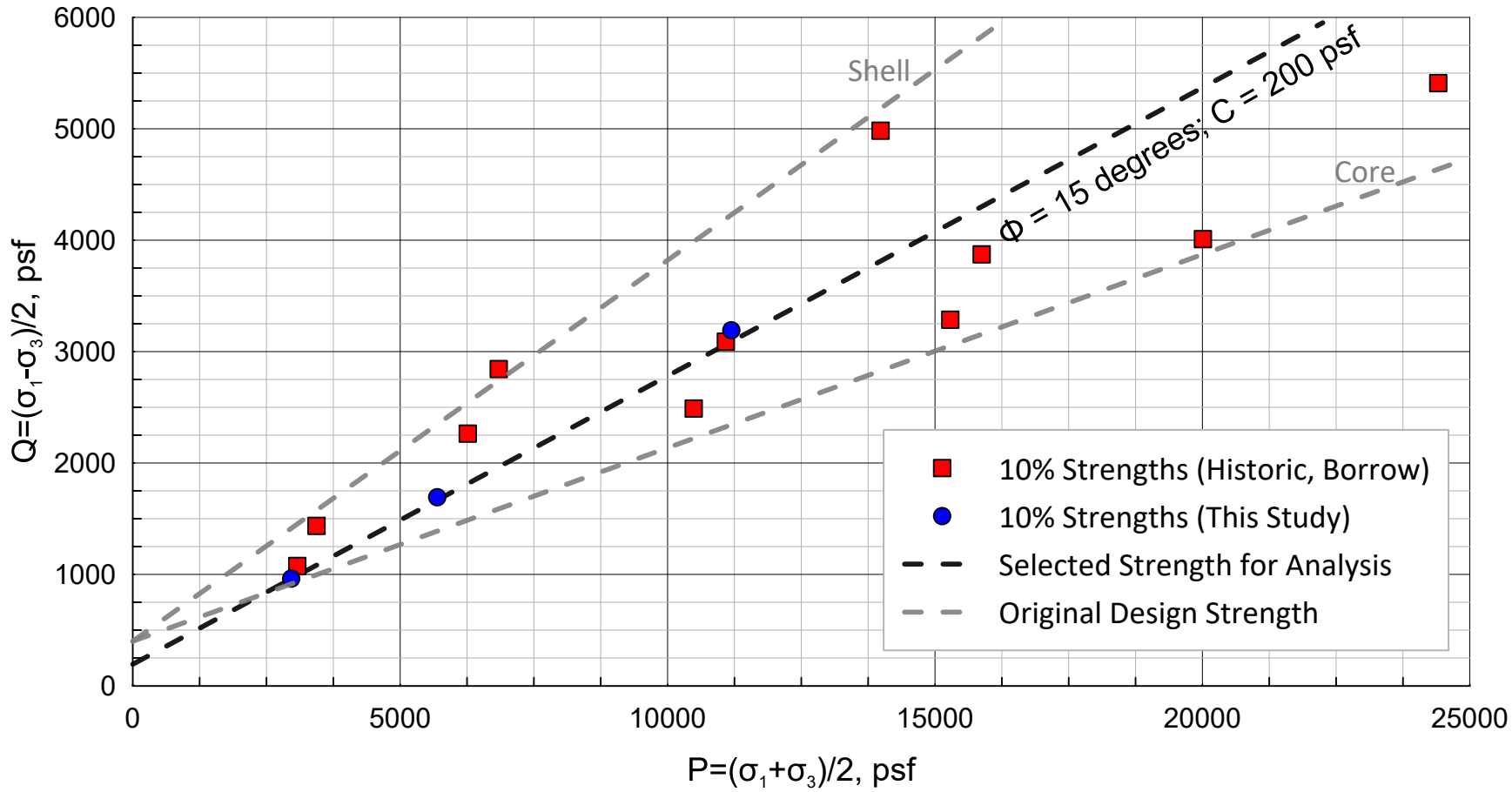
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Sampler, Energy, and Overburden Corrected Blowcount, $(N_1)_{60}$	Figure 6-12

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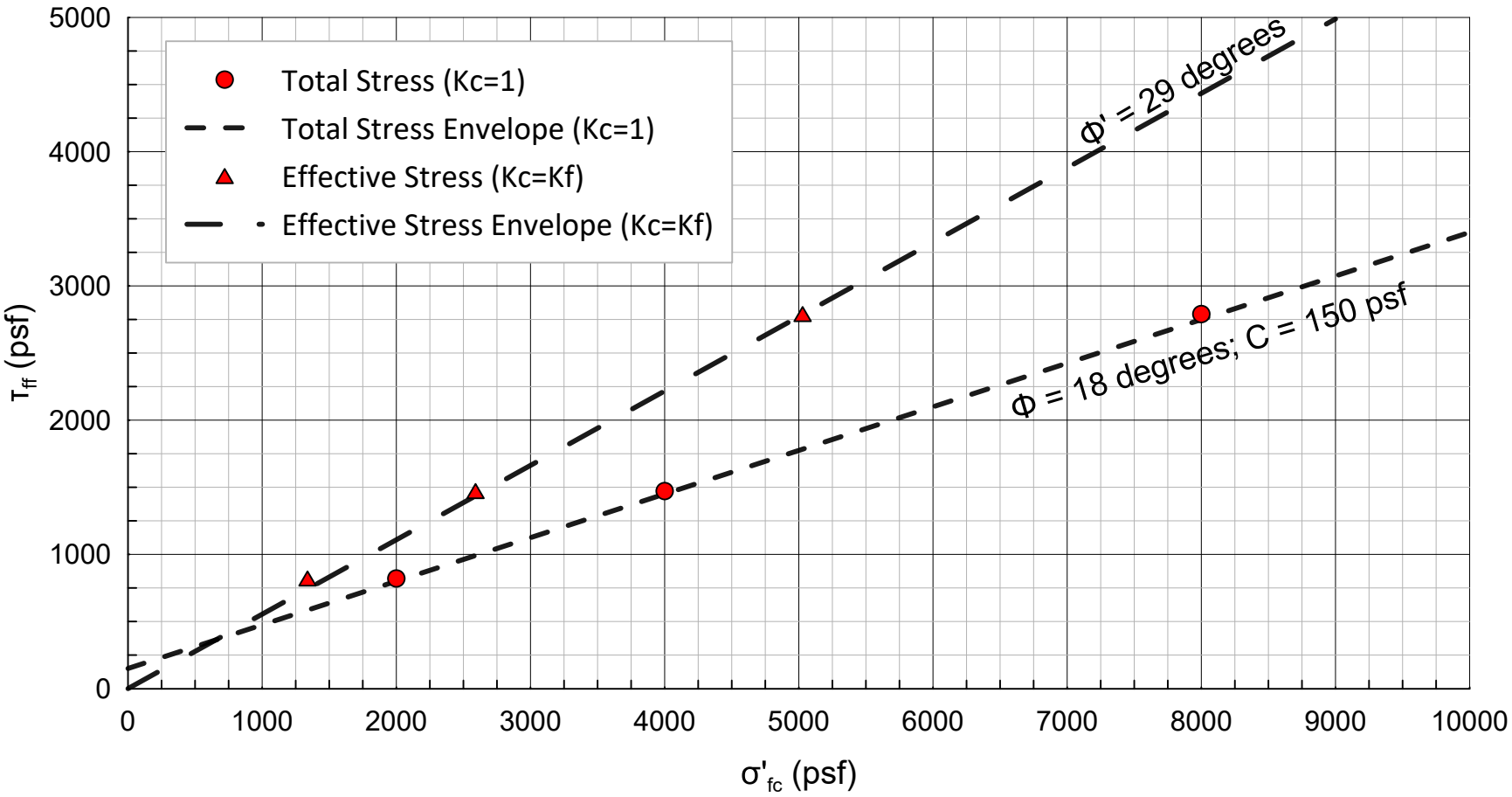
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Effective Stress Strengths	Figure 6-13
AECOM			

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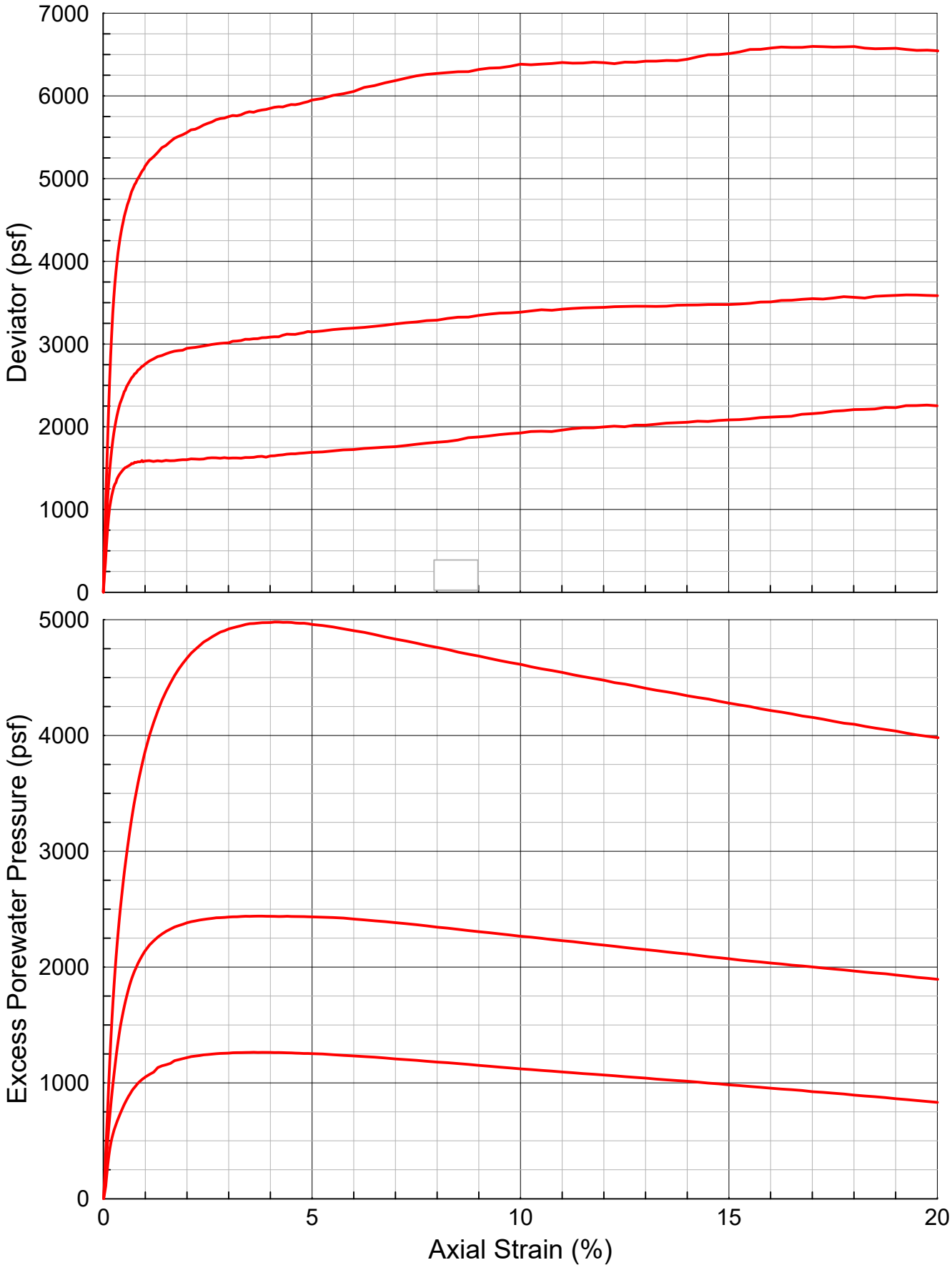
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Total Stress Strengths	Figure 6-14
AECOM			

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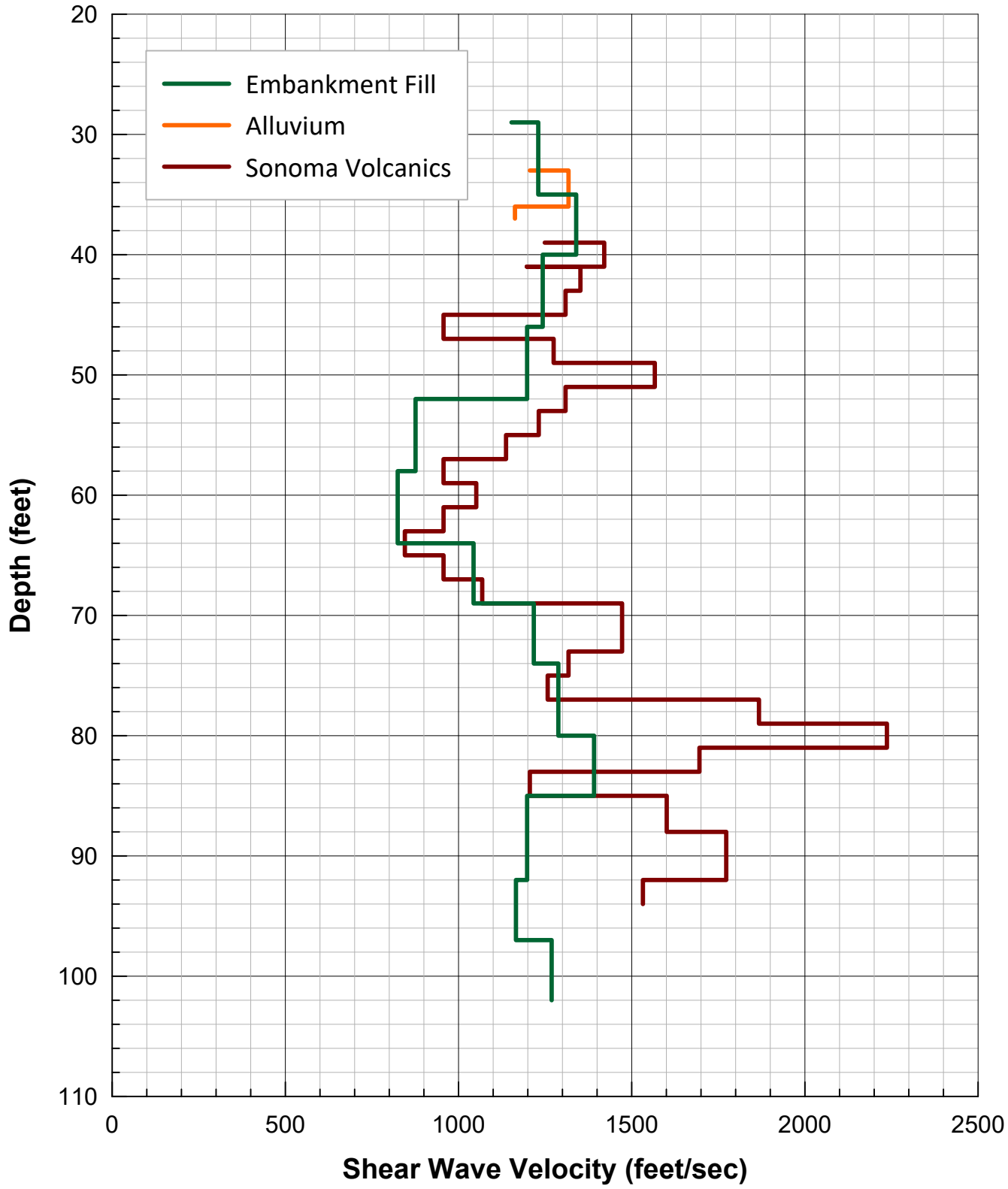
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Duncan & Wright (2005) Failure Envelopes (τ_{ff} versus σ'_{fc})	Figure 6-15
AECOM			

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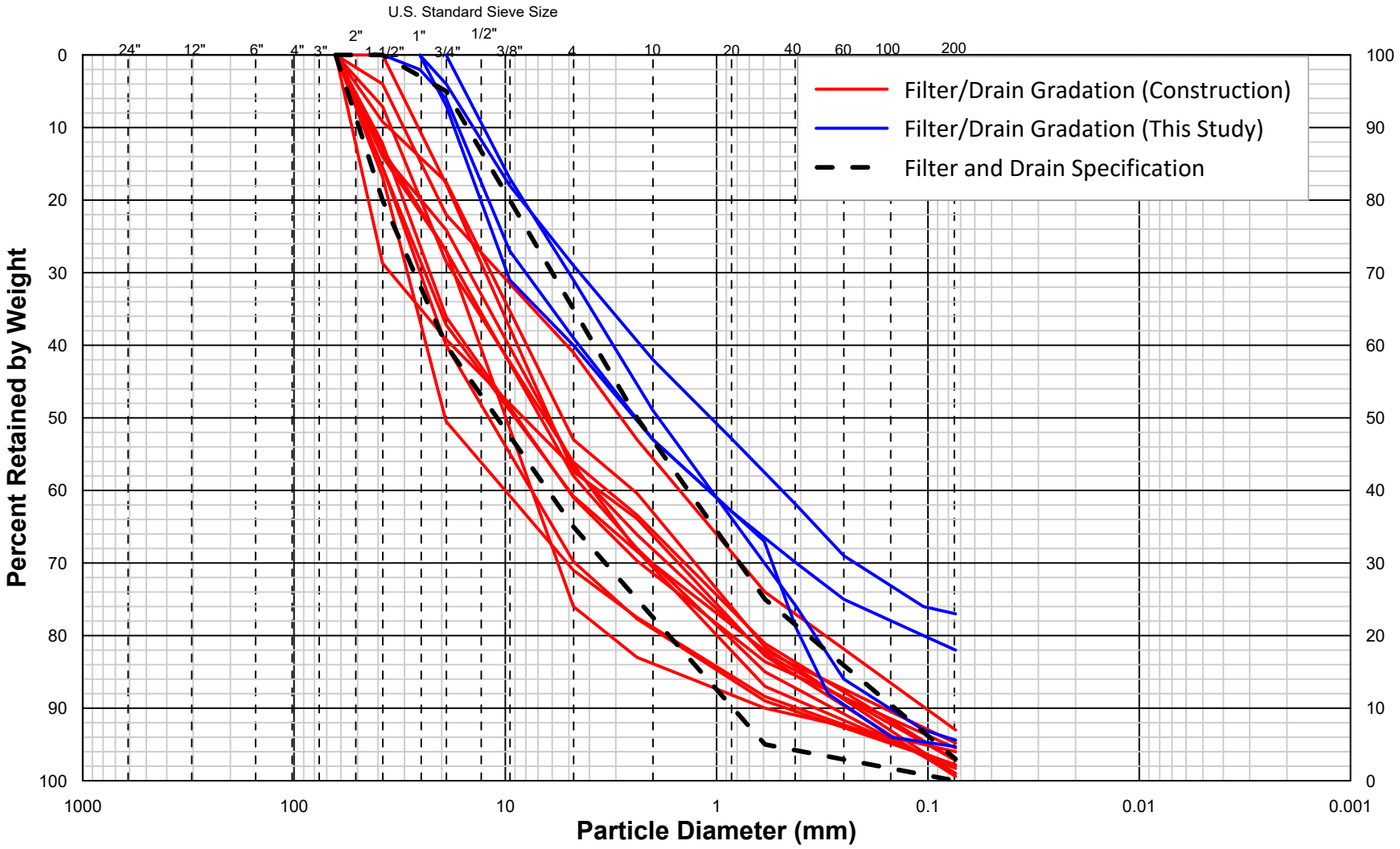
URS	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Embankment Fill Strength Test Behavior	Figure 6-16

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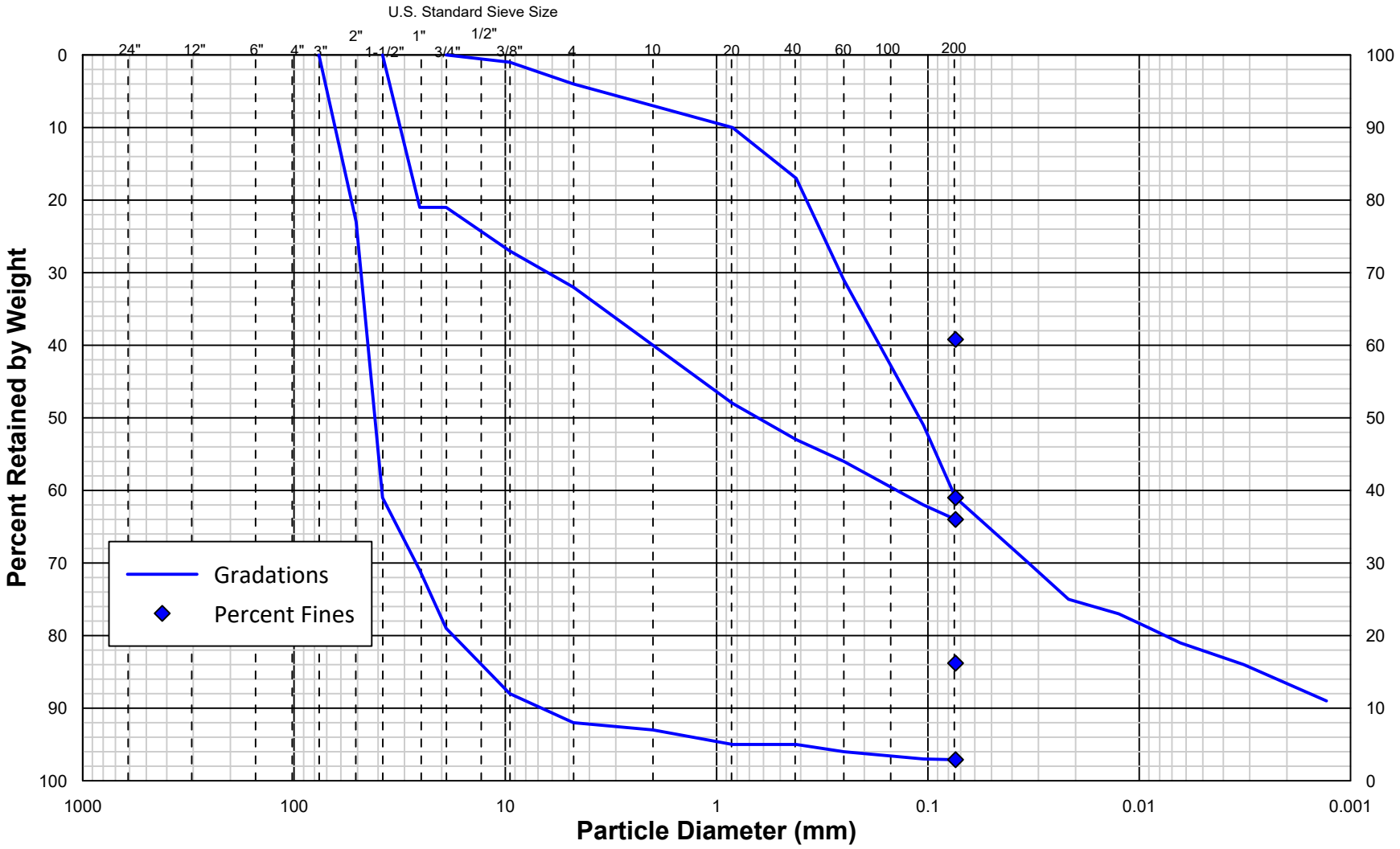
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Seismic Shear Wave Velocity Measurements	Figure 6-17

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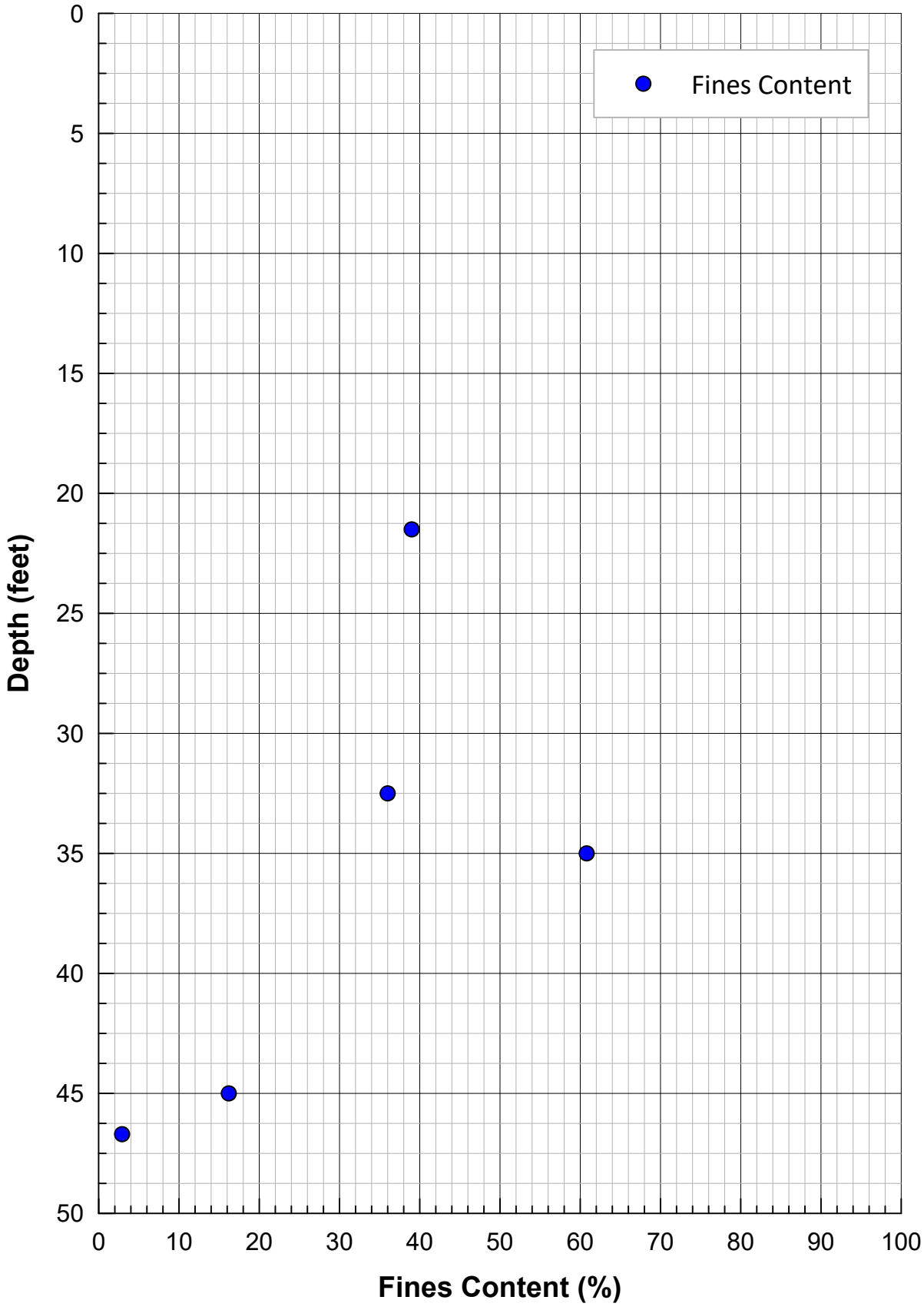
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Figure 6-18
AECOM	Filter and Drain Gradation Test Results	

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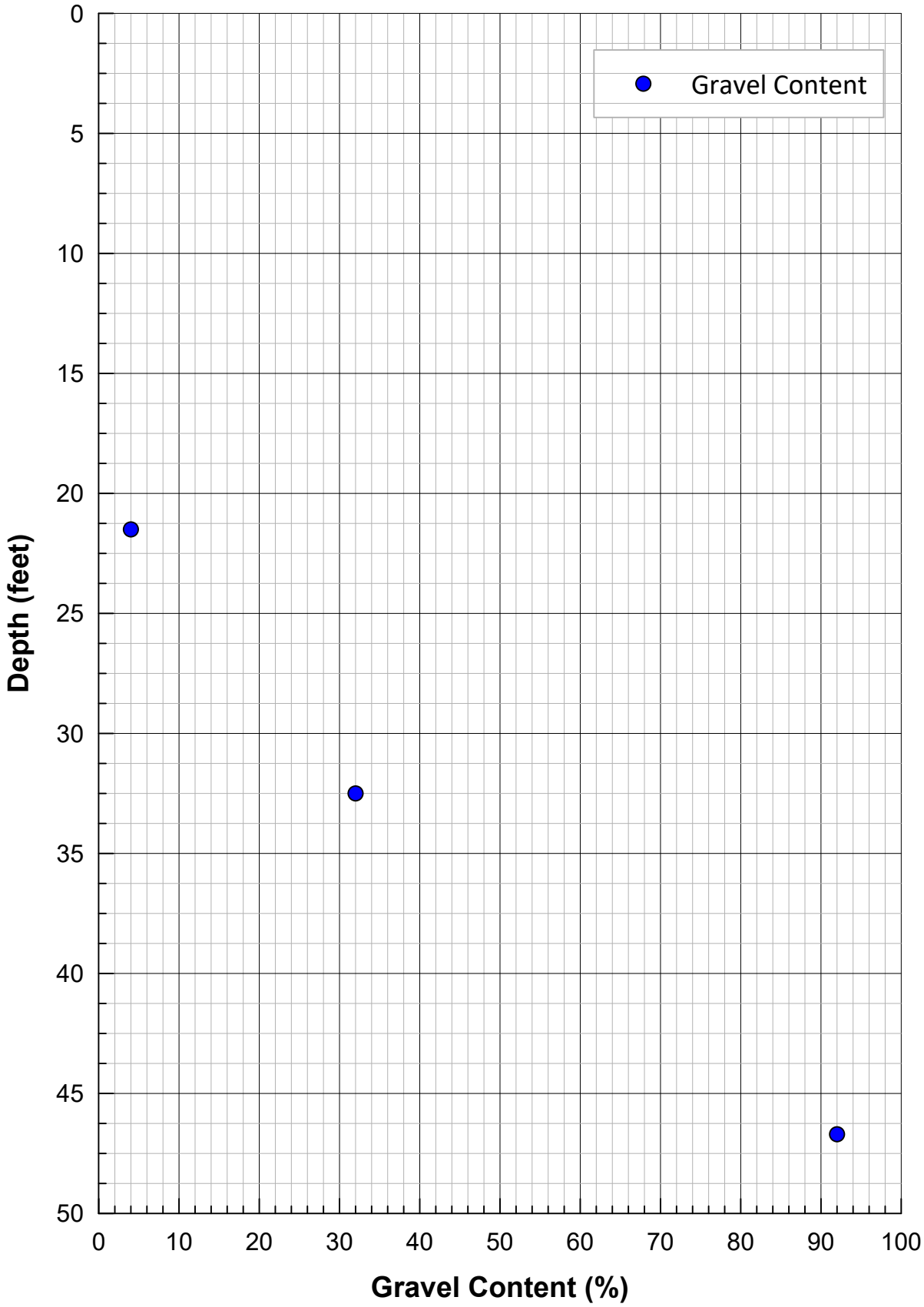
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Figure 6-19
AECOM	Alluvium Gradation Test Results	

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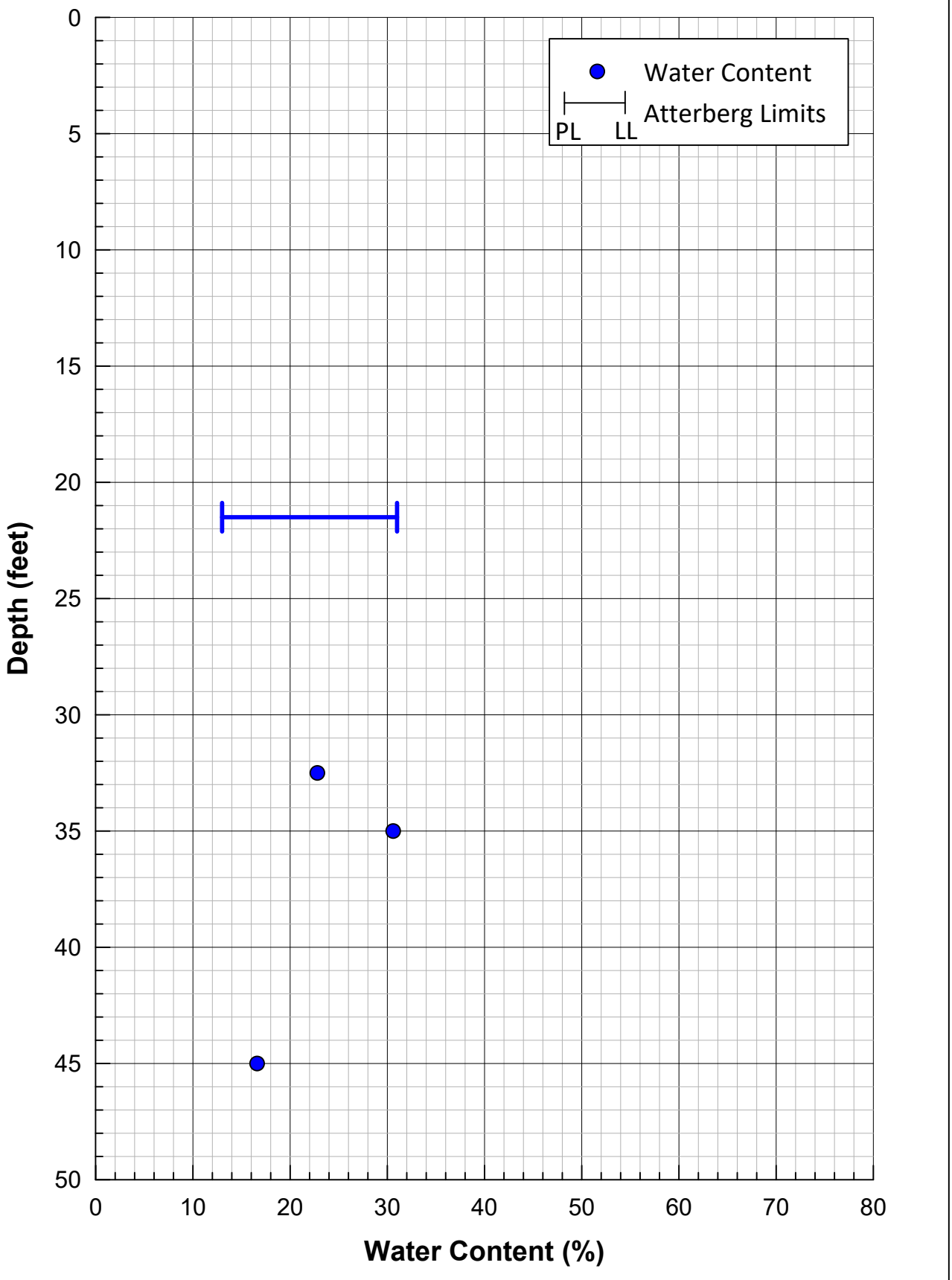
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Alluvium Fines Content	Figure 6-20

C:\Users\Benjamin.Aldridge\AECOM Directory\NRCS-CA Matanzas Creek - General\Embankment Evaluation\Deliverables\SMR\Figures\Figure 6-20_Qal_Fine Content.gpj



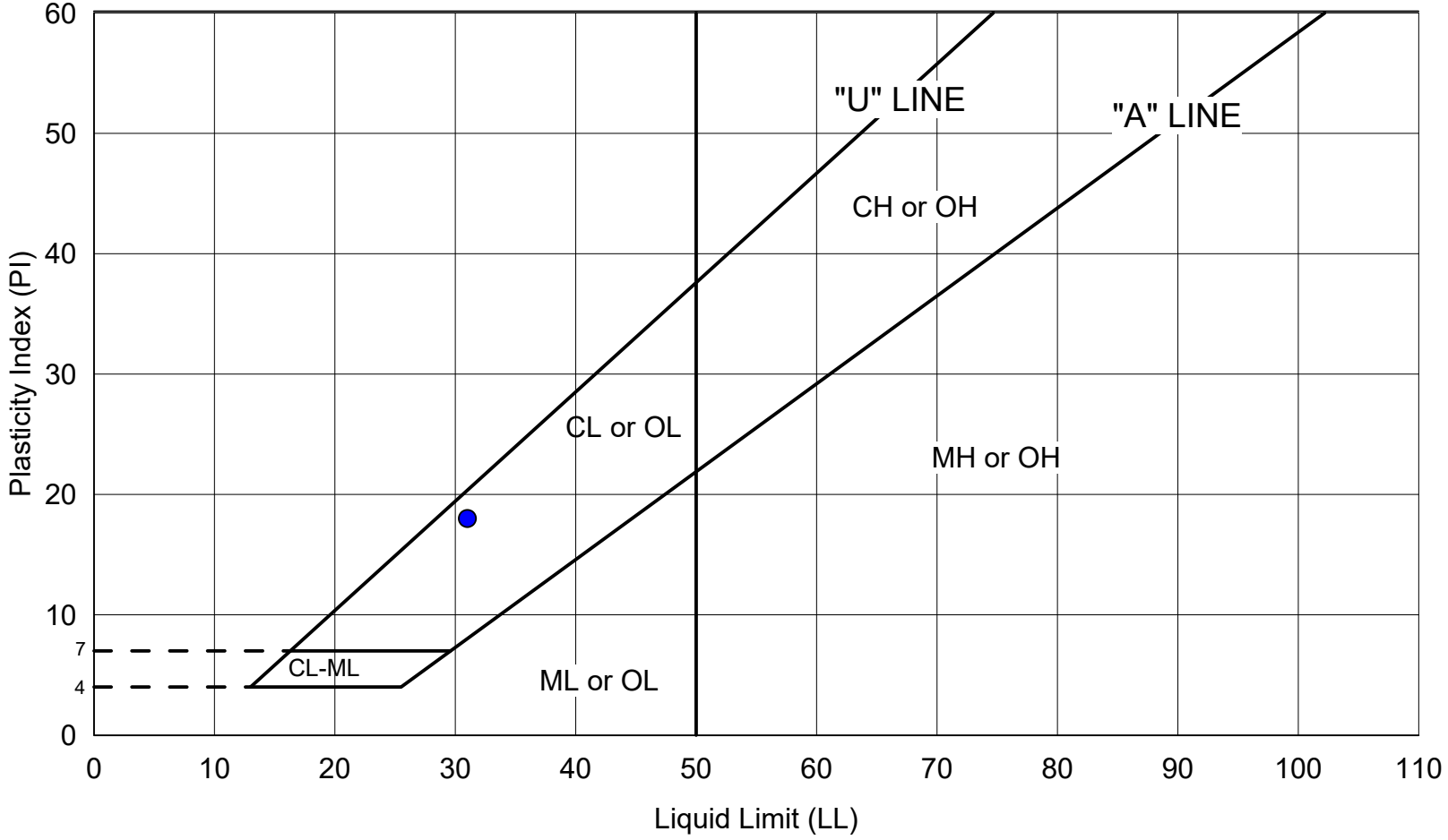
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Alluvium Gravel Content	Figure 6-21

C:\Users\Benjamin.Aldridge\AECOM Directory\NRCS-CA Matanzas Creek - General\Embankment Evaluation\Deliverables\SMR\Figures\Figure 6-21_Qal_Gravel Content.gpj



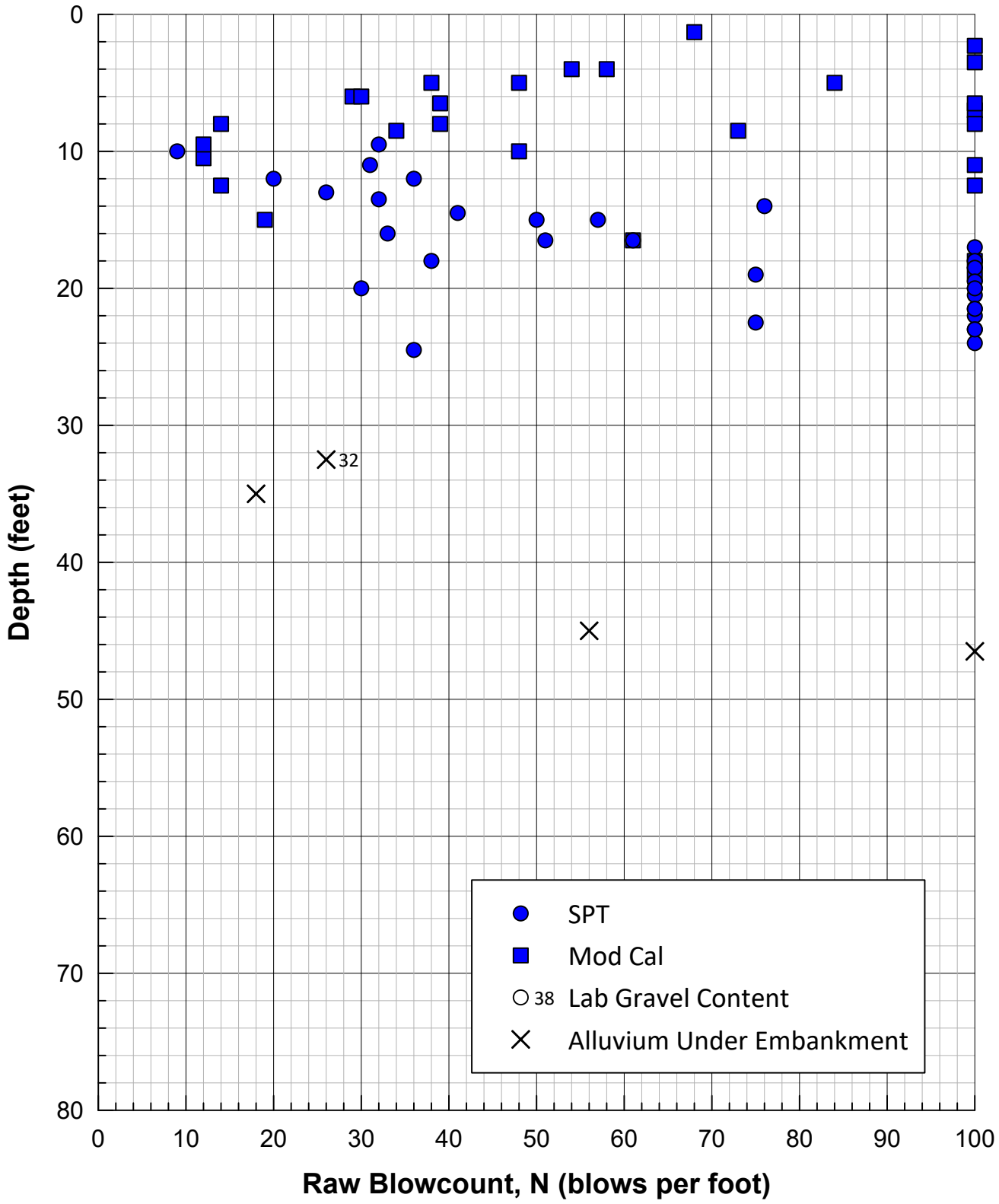
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Alluvium Water Content & Plasticity	Figure 6-22

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Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Alluvium Plasticity Chart	Figure 6-23
AECOM			

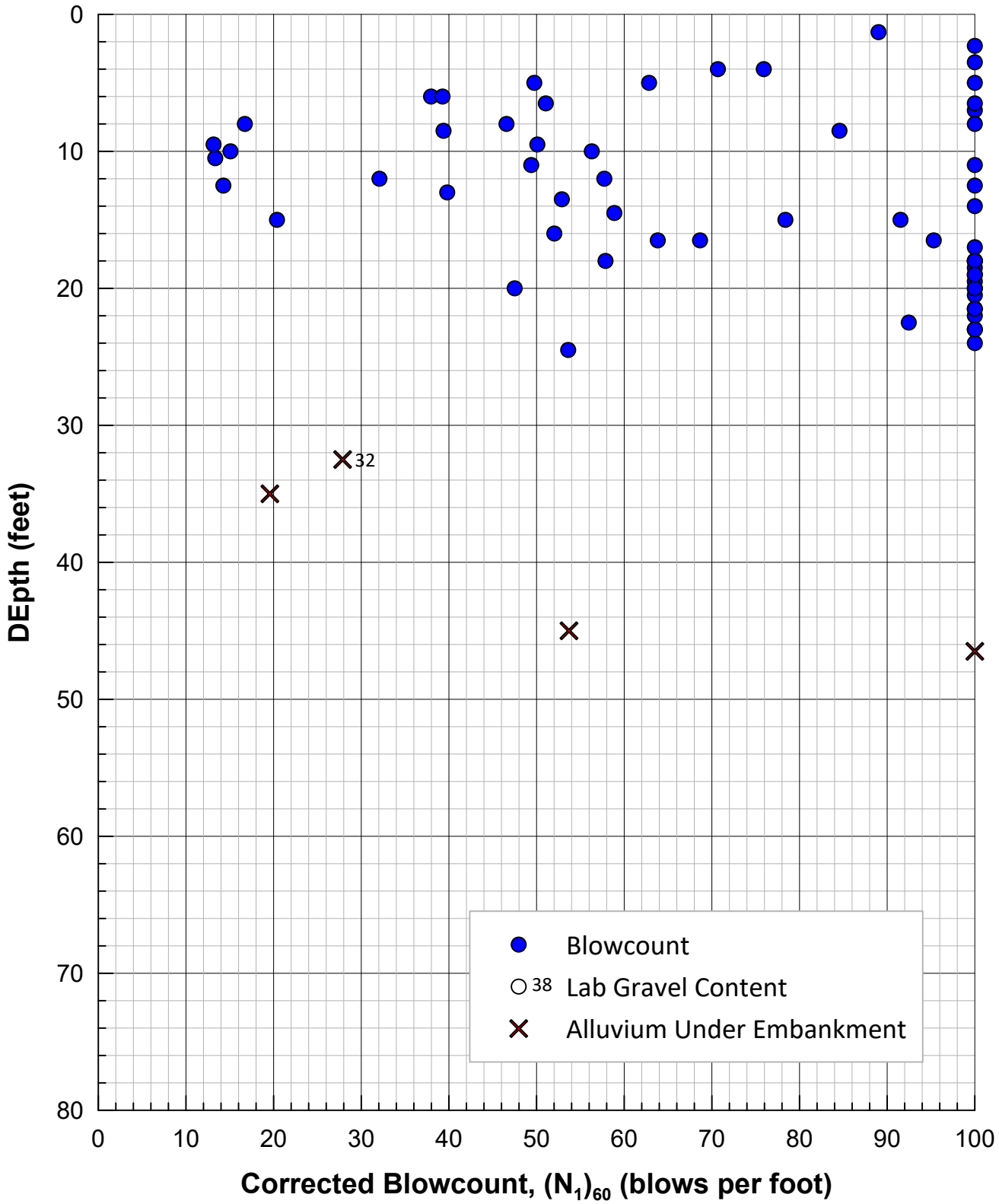
C:\Users\Benjamin.Aldridge\AECOM Directory\NRCS-CA Matanzas Creek - General\Embankment Evaluation\Deliverables\SMR\Figures\Figure 6-23_Qal_Plasticity Chart.gpj



Notes:
- Refusal and N values greater than 100 are plotted at blowcount of 100.

AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Alluvium Uncorrected Blowcount, N	Figure 6-24

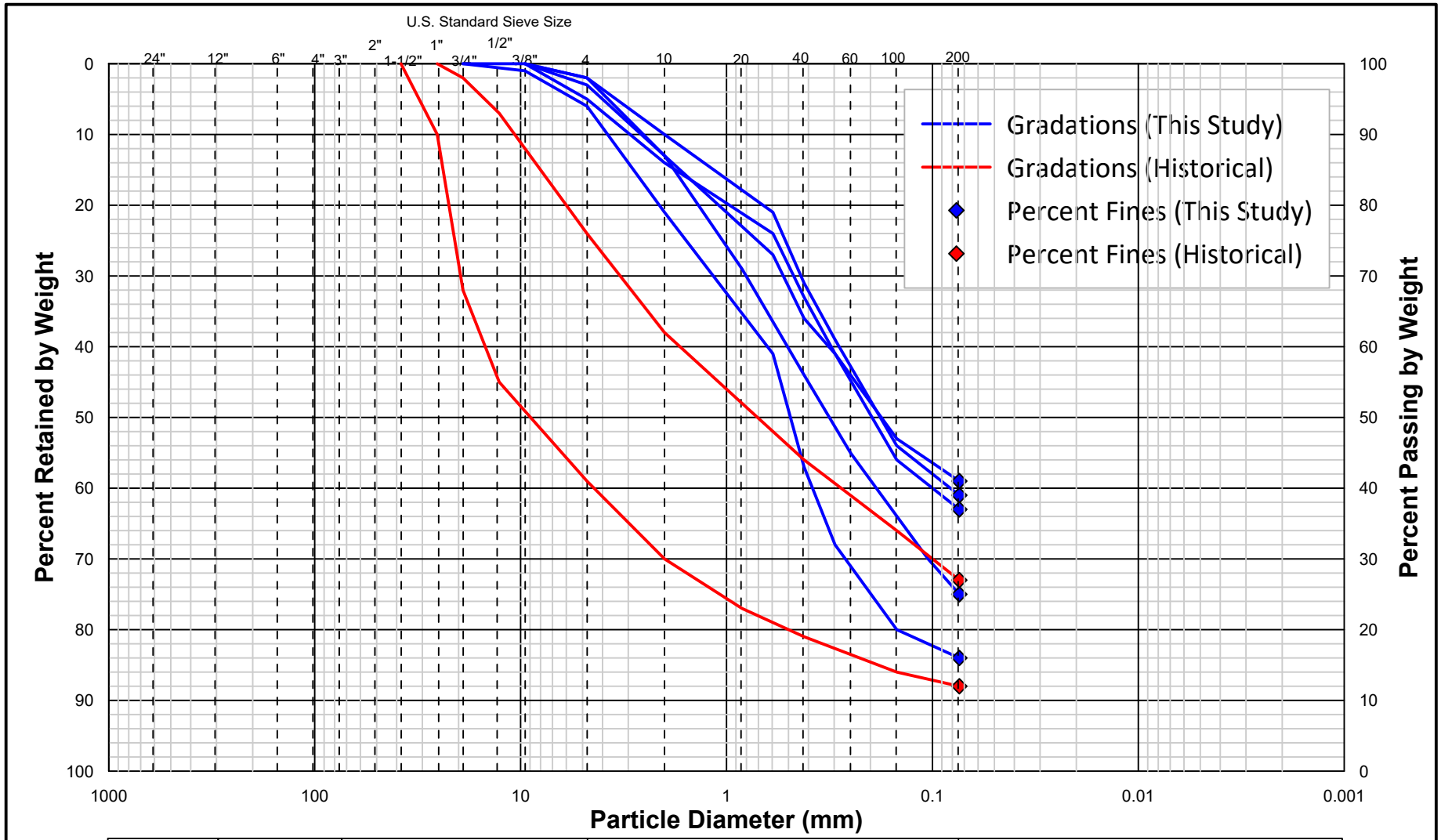
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Notes:
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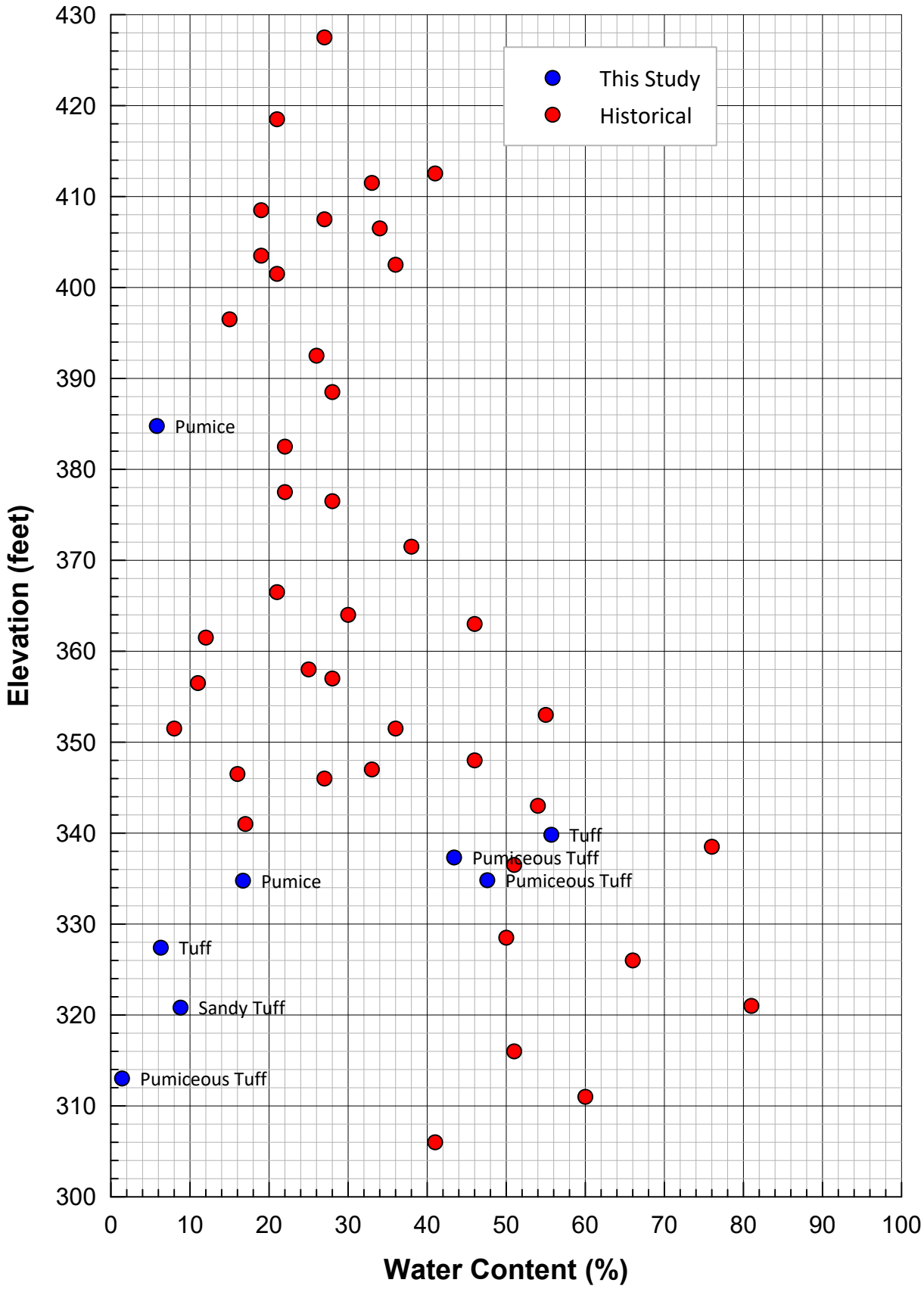
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Alluvium Sampler, Energy, and Overburden Corrected Blowcount, $(N_1)_{60}$	Figure 6-25

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Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Figure 6-26
AECOM	Sonoma Volcanics Gradation Test Results	

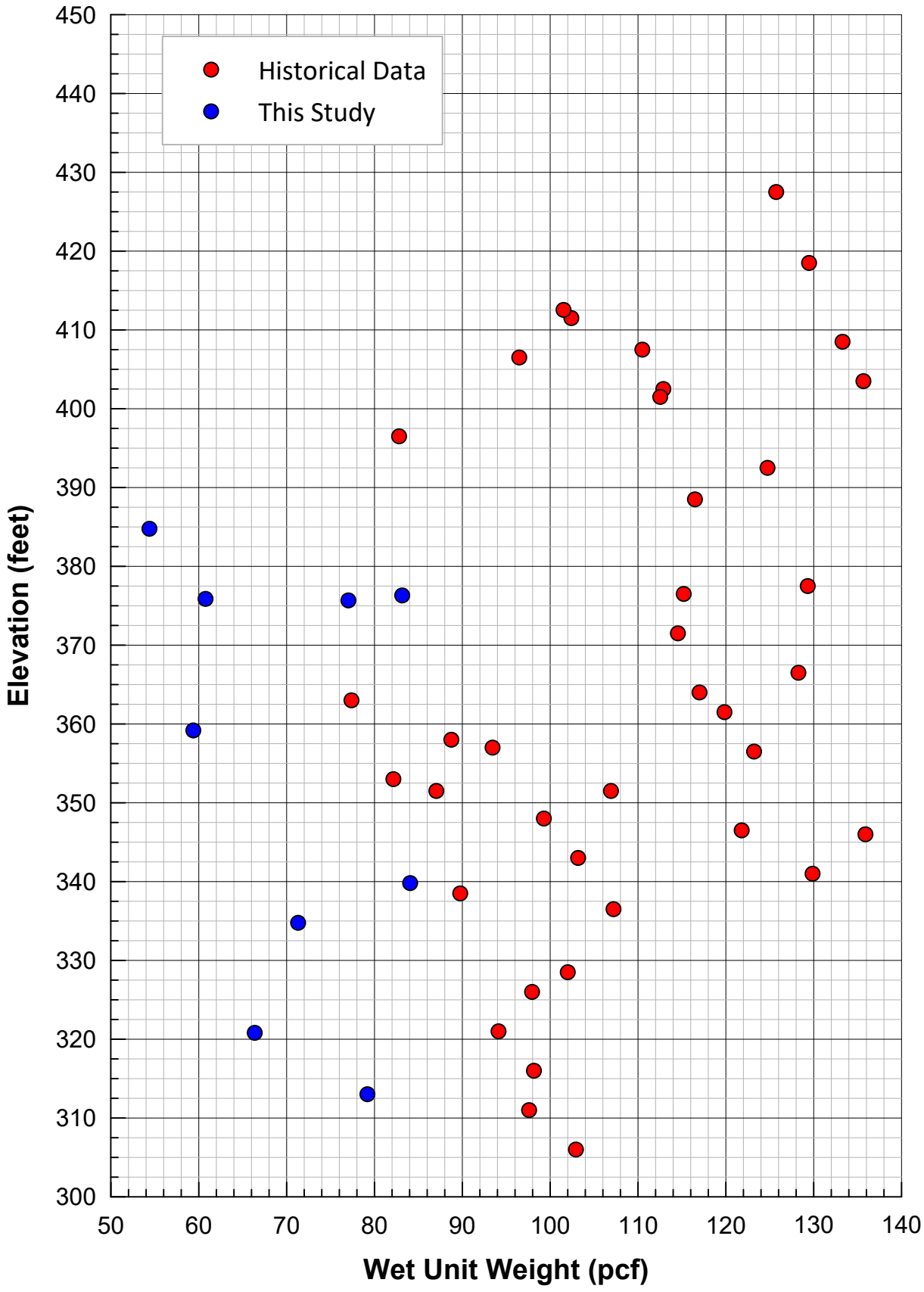
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Note: No Atterberg Limit tests performed

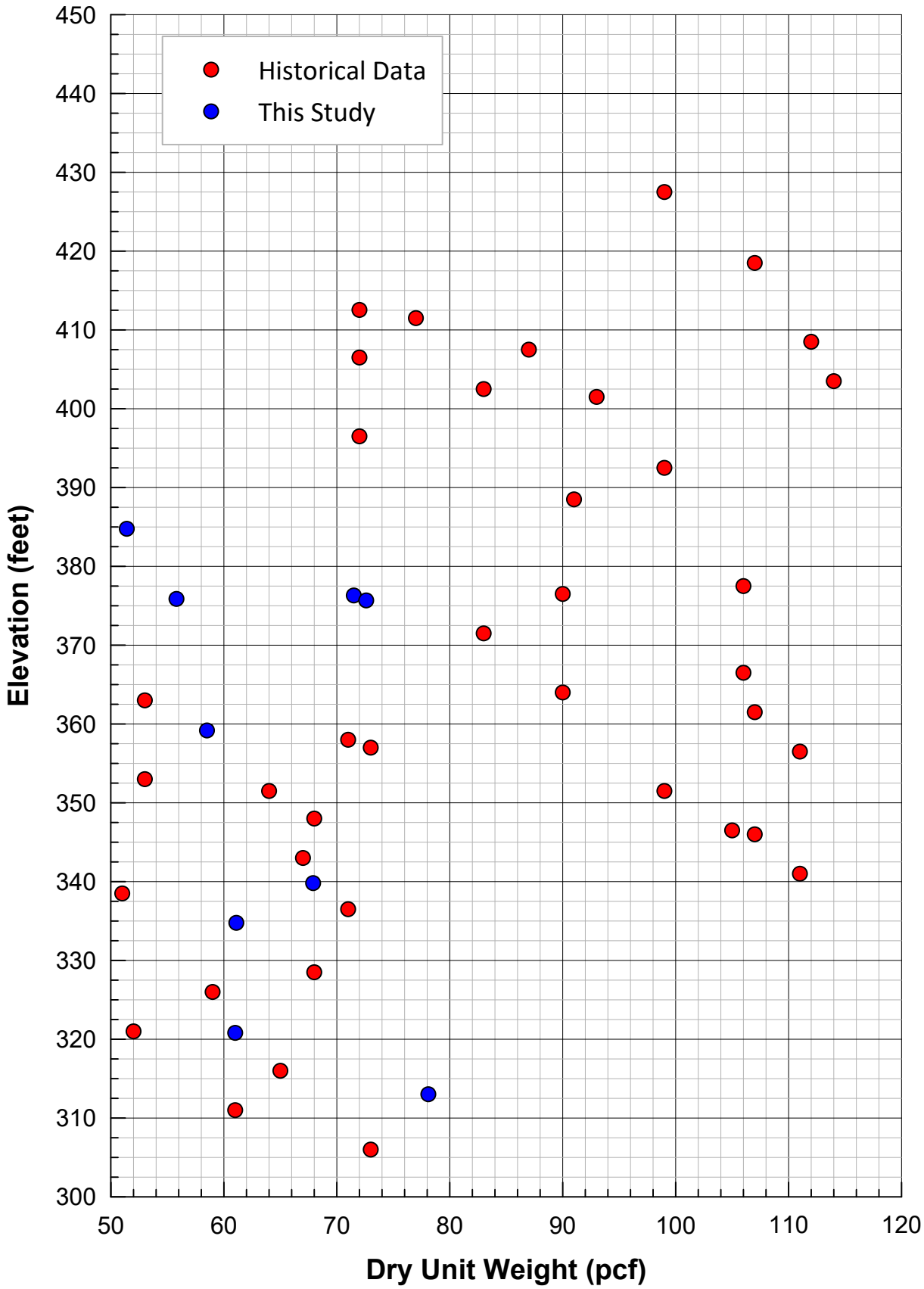
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Sonoma Volcanics Water Content	Figure 6-27

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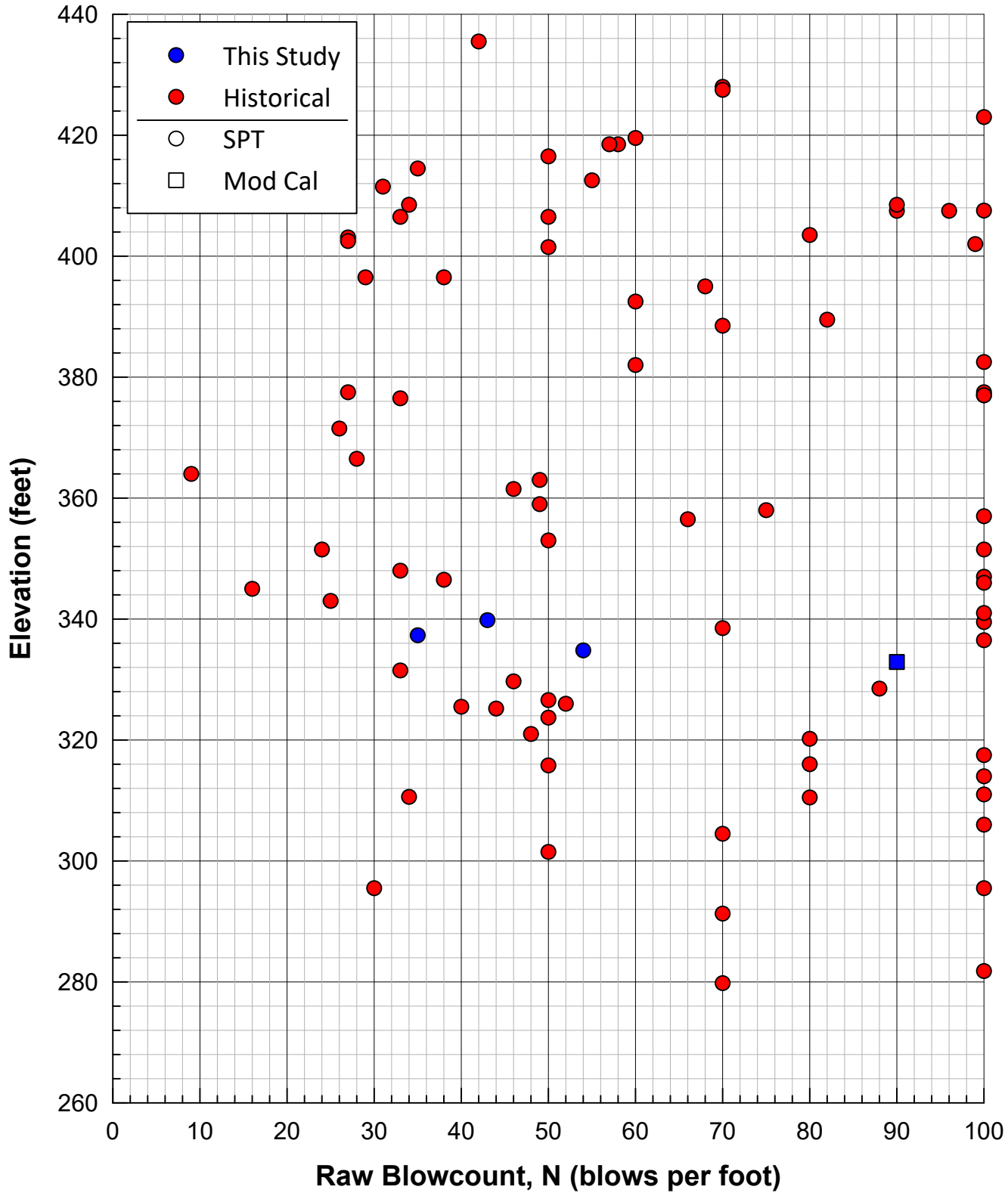
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Sonoma Volcanics Wet Unit Weight	Figure 6-28

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AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Sonoma Volcanics Dry Unit Weight	Figure 6-29

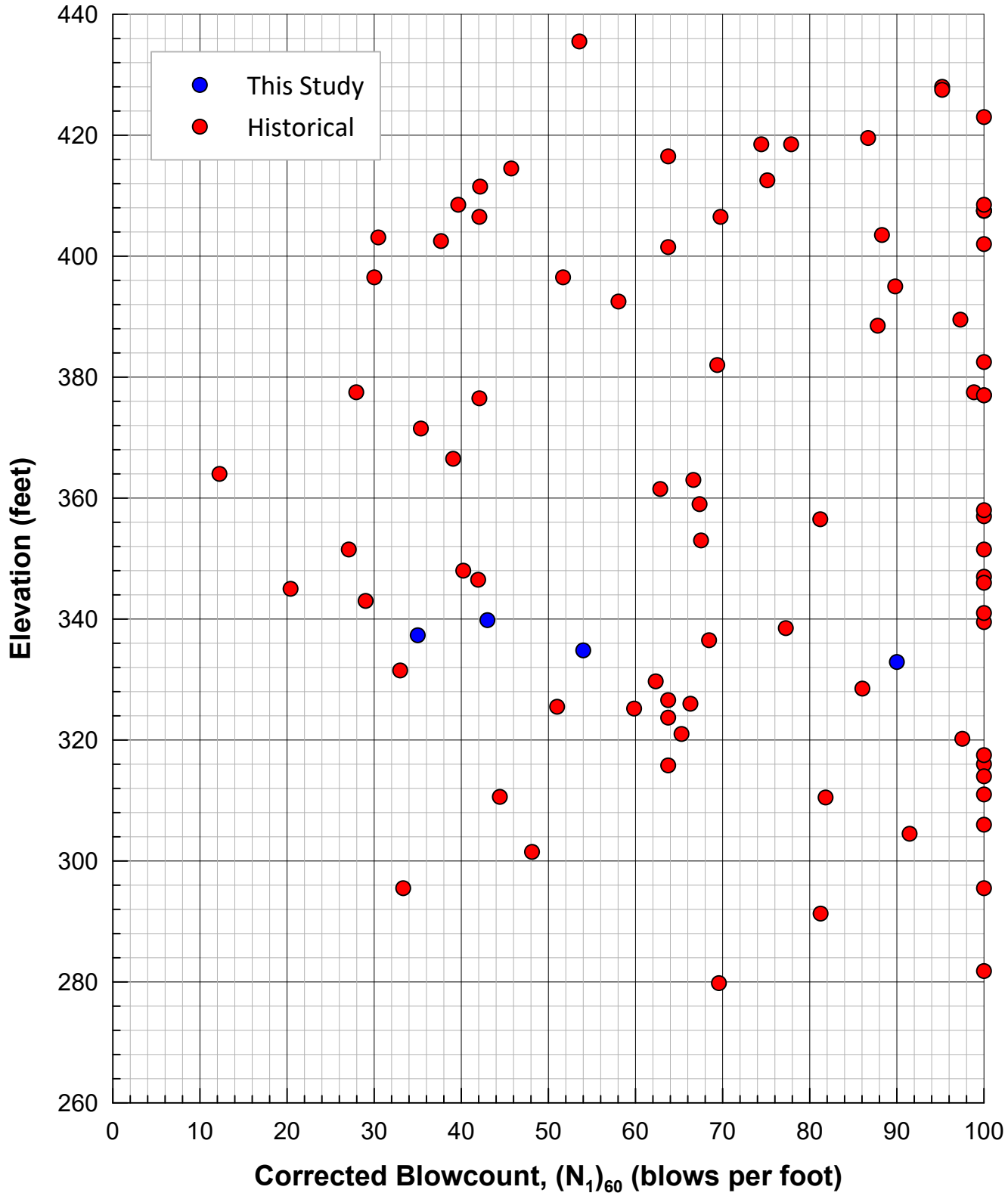
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Notes:
- Refusal and N values greater than 100 are plotted at blowcount of 100.

AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Sonoma Volcanics Uncorrected Blowcount, N	Figure 6-30

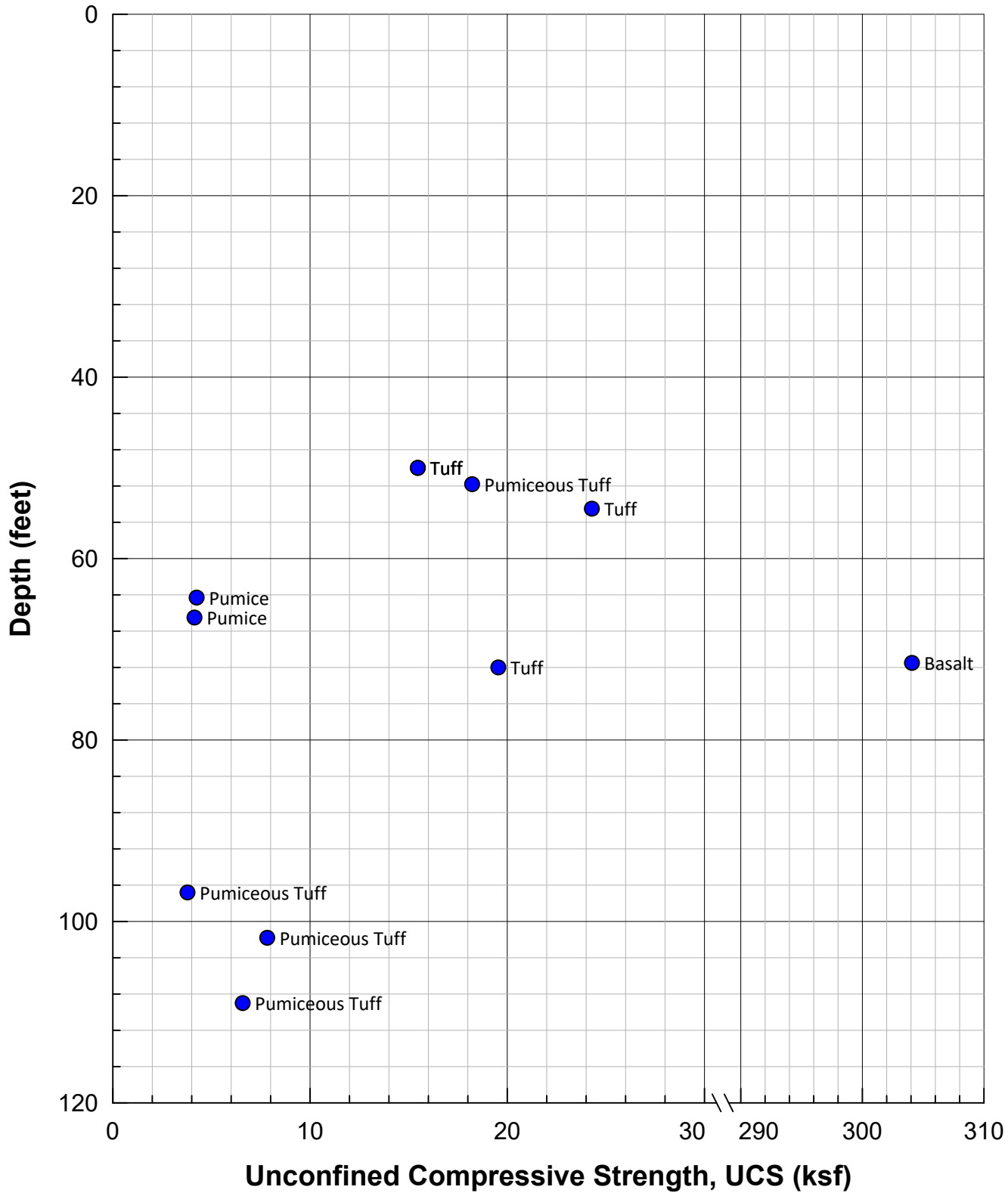
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Notes:
- Refusal and N values greater than 100 are plotted at blowcount of 100.

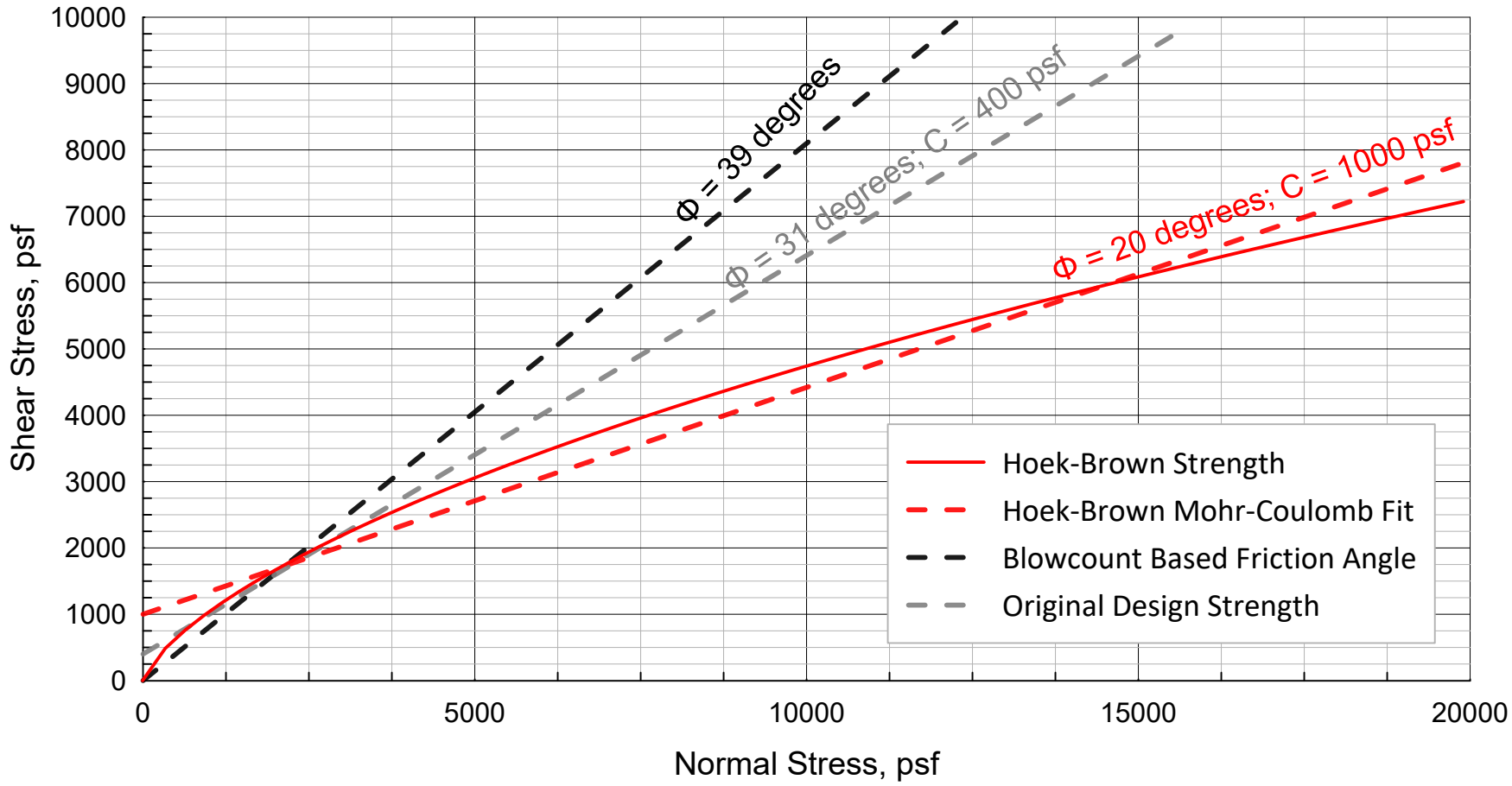
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Sonoma Volcanics Sampler, Energy, and Overburden Corrected Blowcount, $(N_1)_{60}$	Figure 6-31

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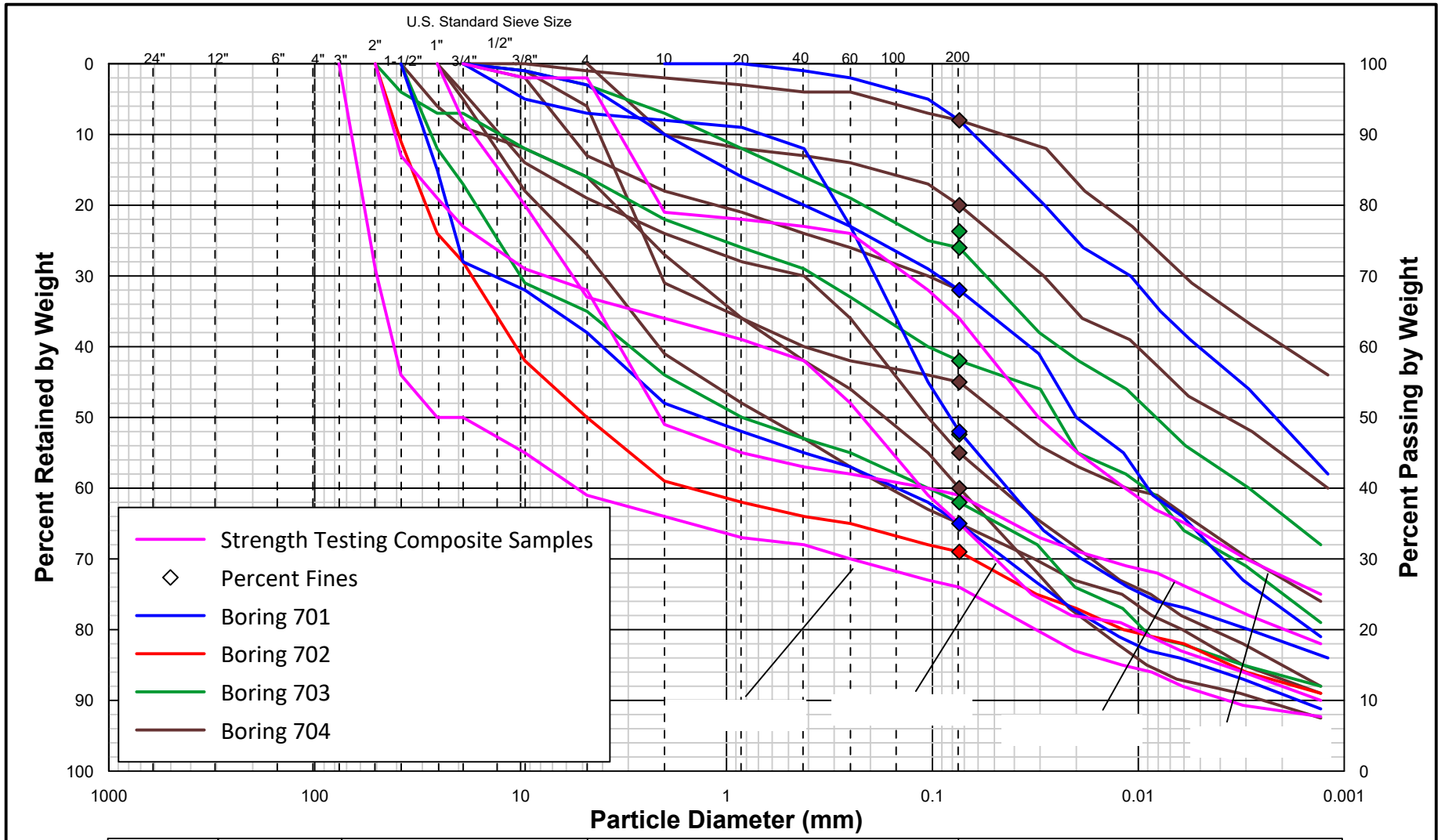
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Sonoma Volcanics Unconfined Compressive Strength	Figure 6-32

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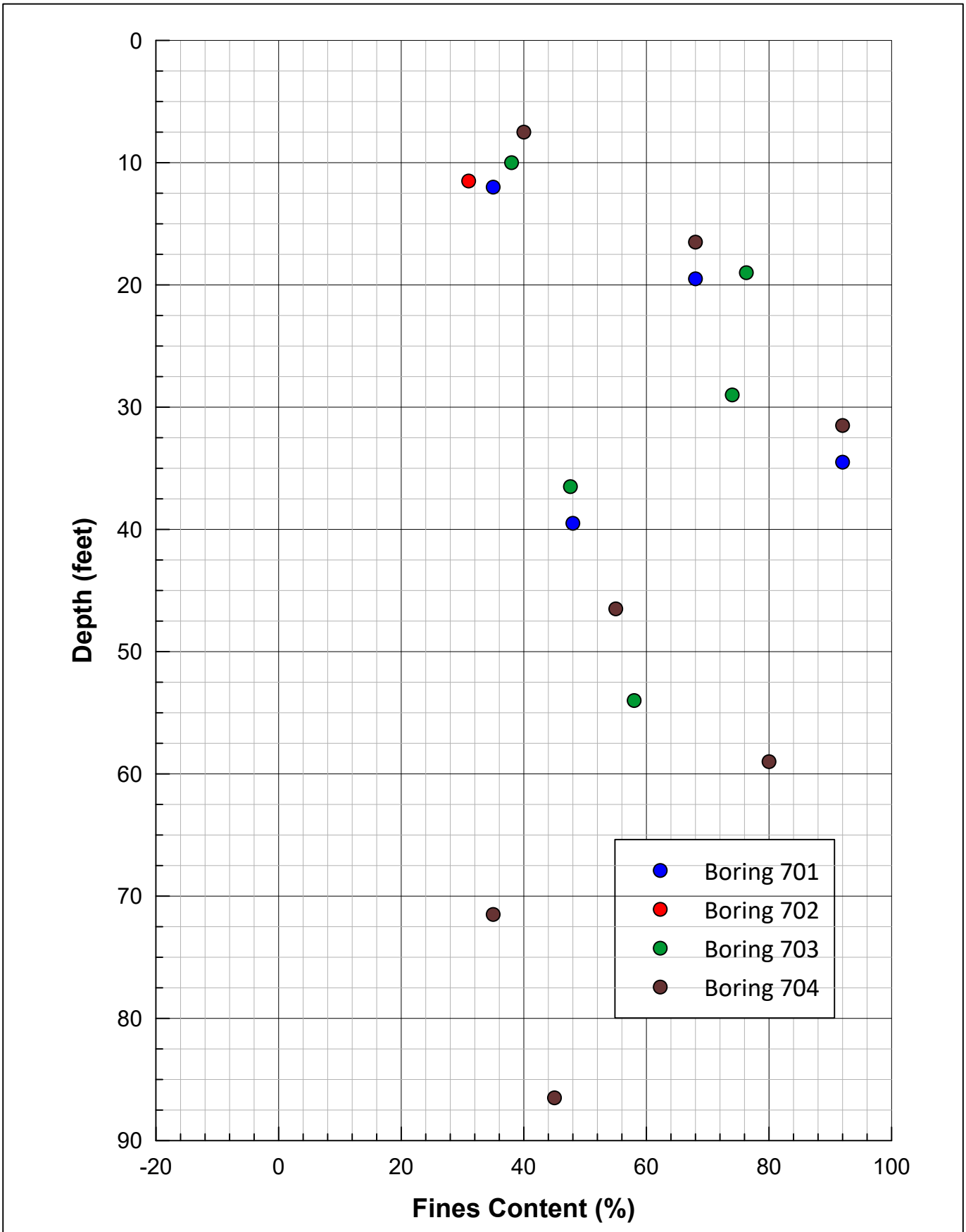
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Sonoma Volcanics Comparison of Strengths	Figure 6-33
AECOM			

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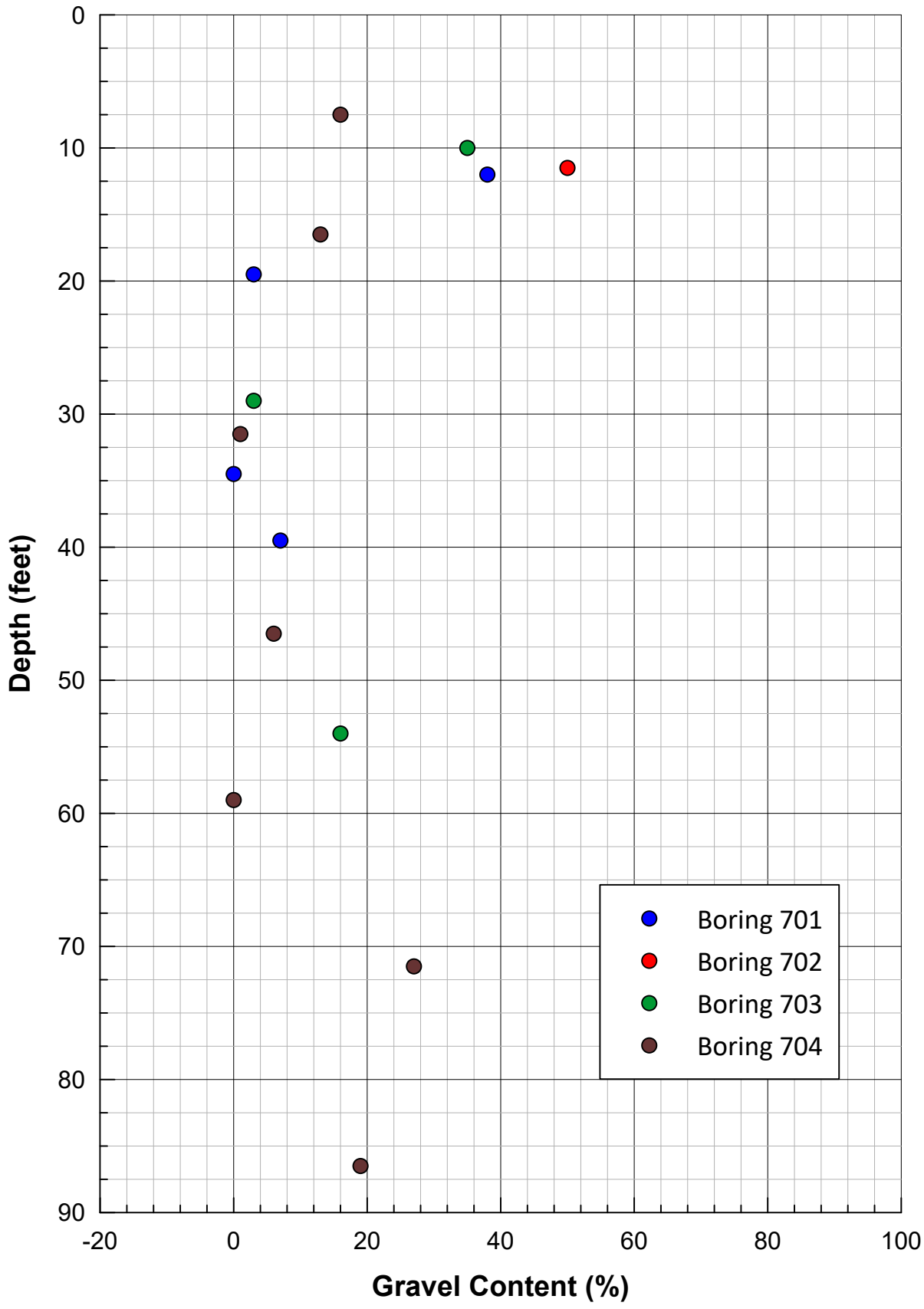
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Figure 6-34
AECOM	Colluvium/Landslide Debris Gradation Test Results	

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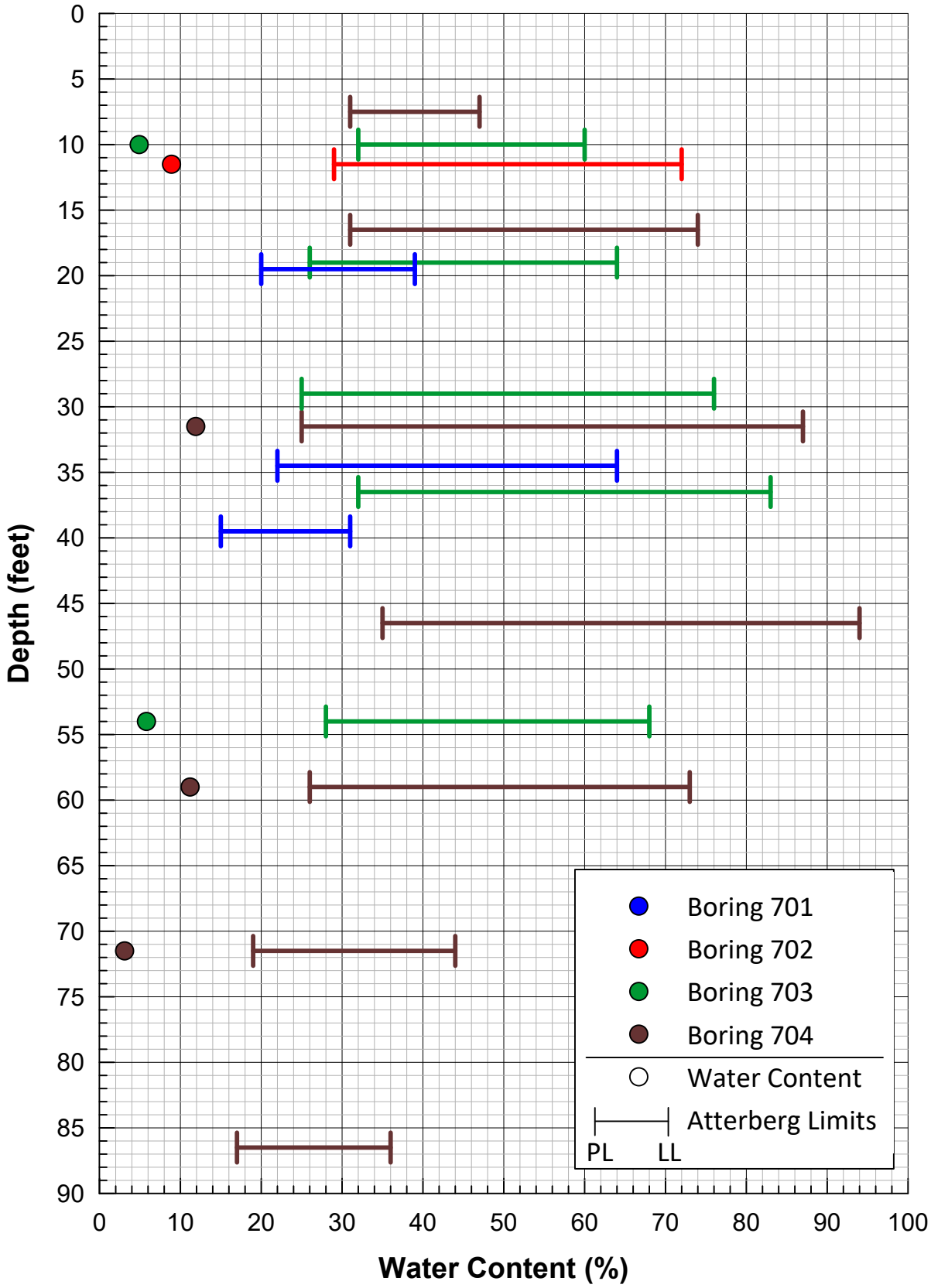
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Colluvium/Landslide Debris Fines Content	Figure 6-35

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AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Colluvium/Landslide Debris Gravel Content	Figure 6-36

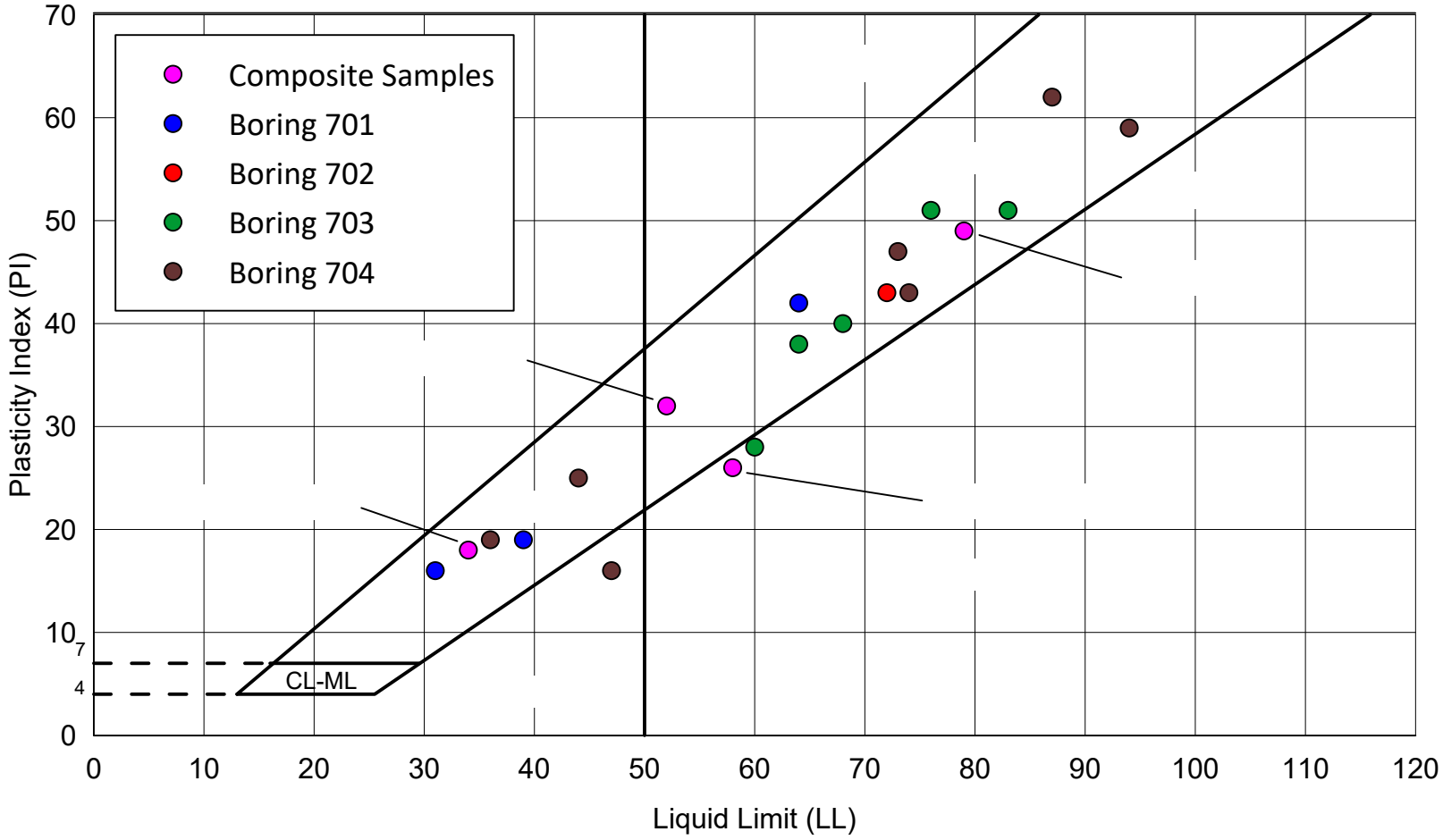
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Note: Water content samples were not wrapped
so water contents are likely lower than in-situ conditions.

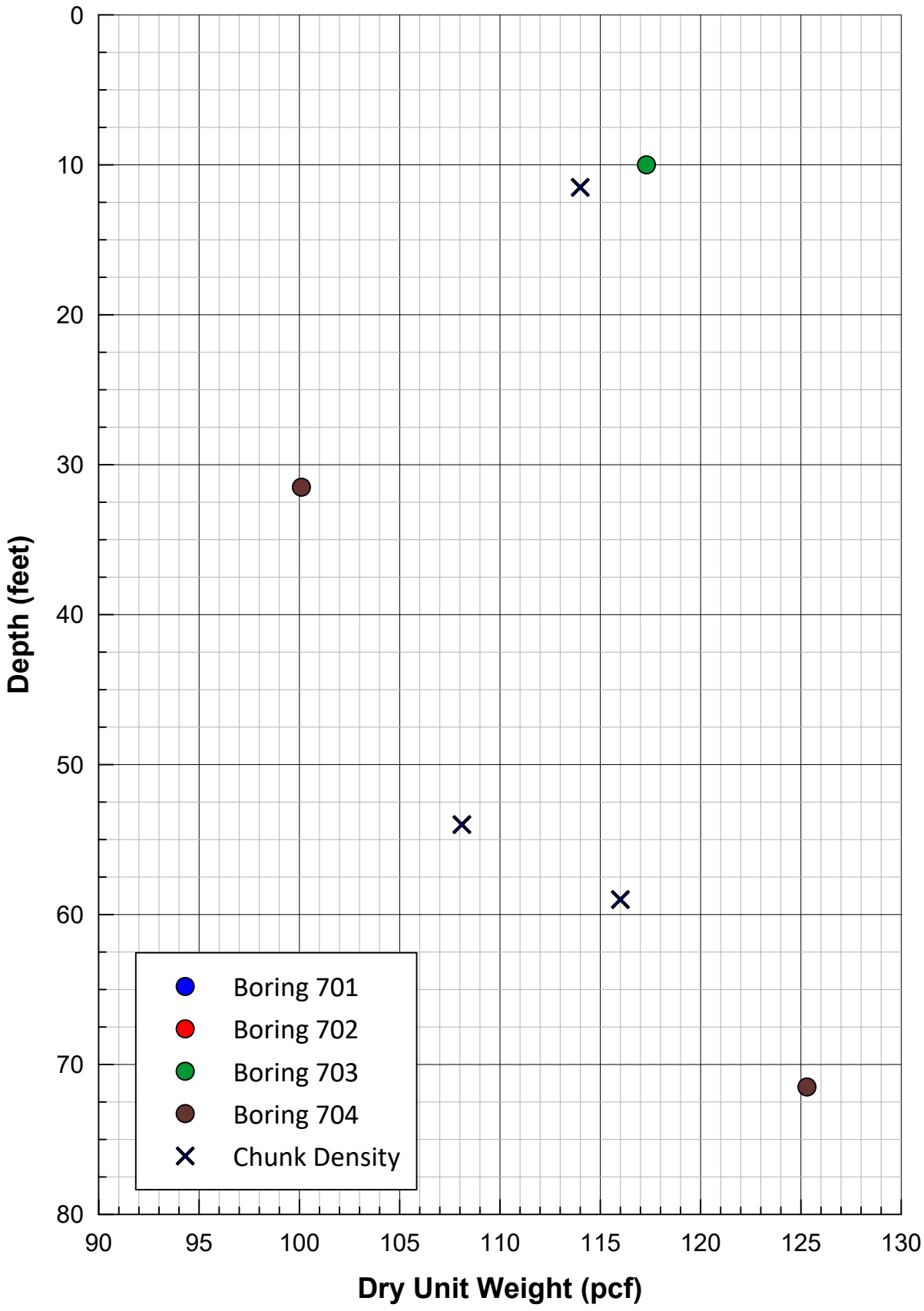
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Colluvium/Landslide Debris Water Content & Plasticity	Figure 6-37

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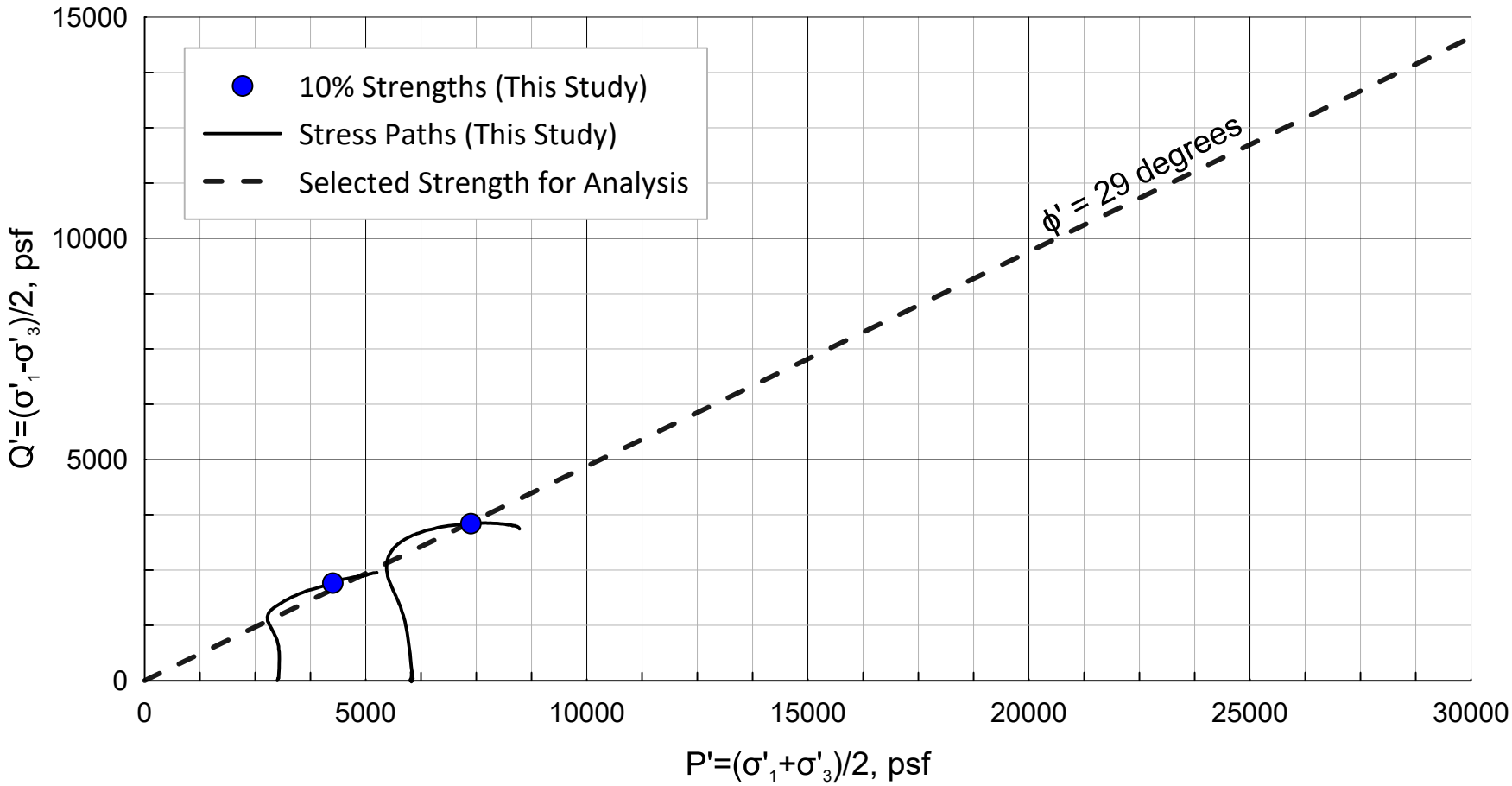
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Colluvium/Landslide Debris Plasticity Chart	Figure 6-38
AECOM			

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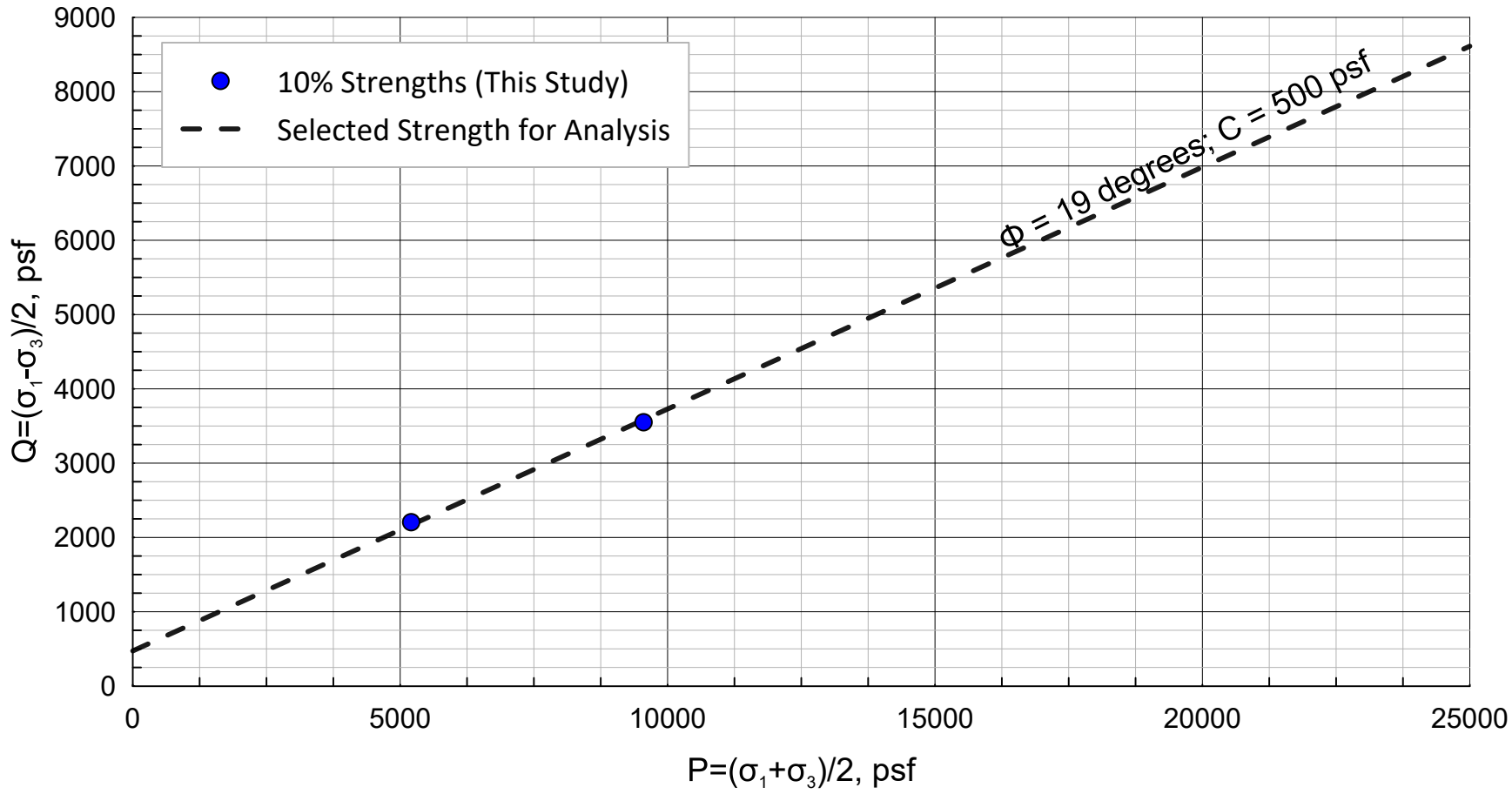
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Colluvium/Landslide Debris Dry Unit Weight	Figure 6-39

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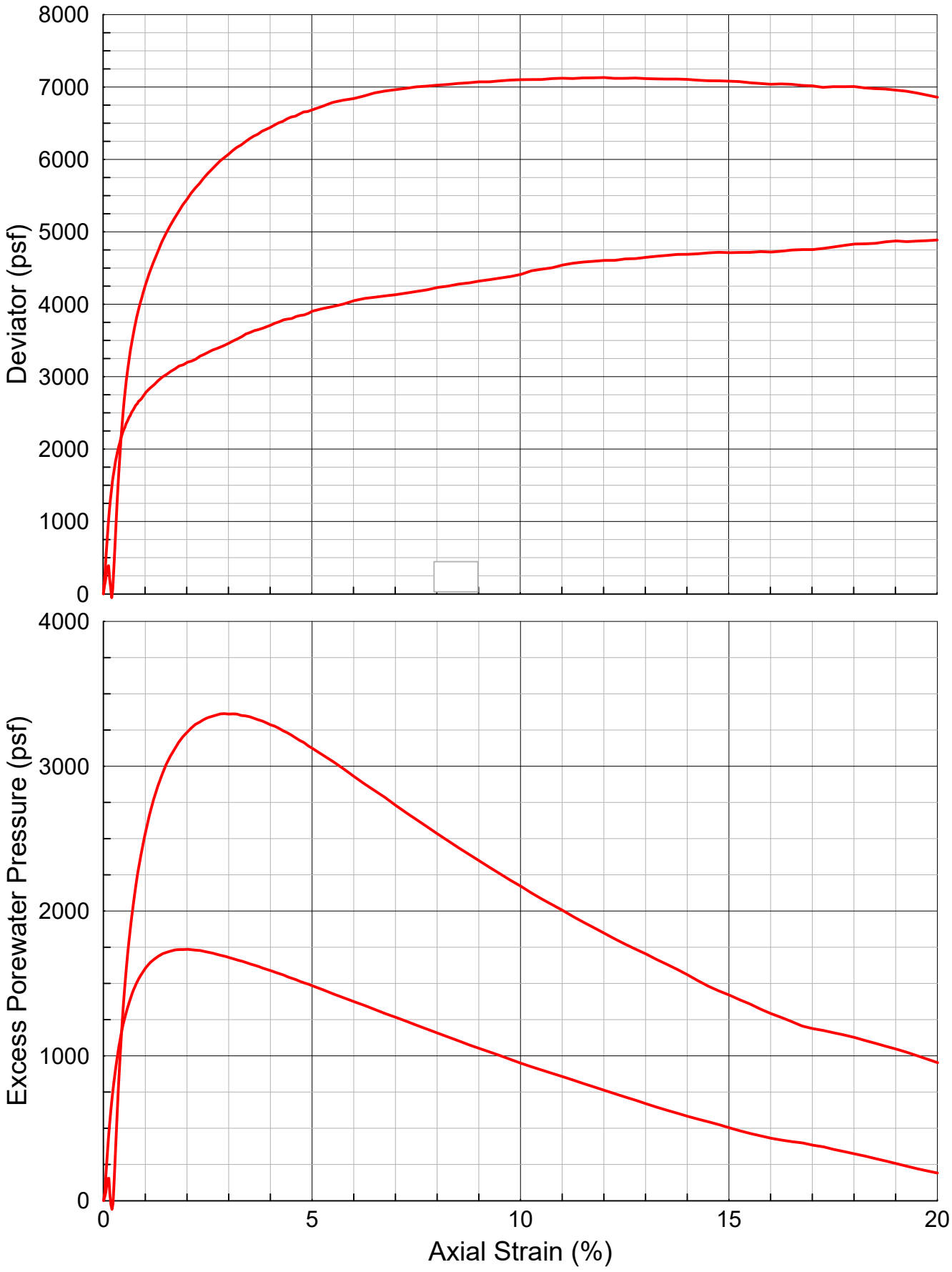
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Debris/Colluvium Effective Stress Strengths	Figure 6-40
AECOM			

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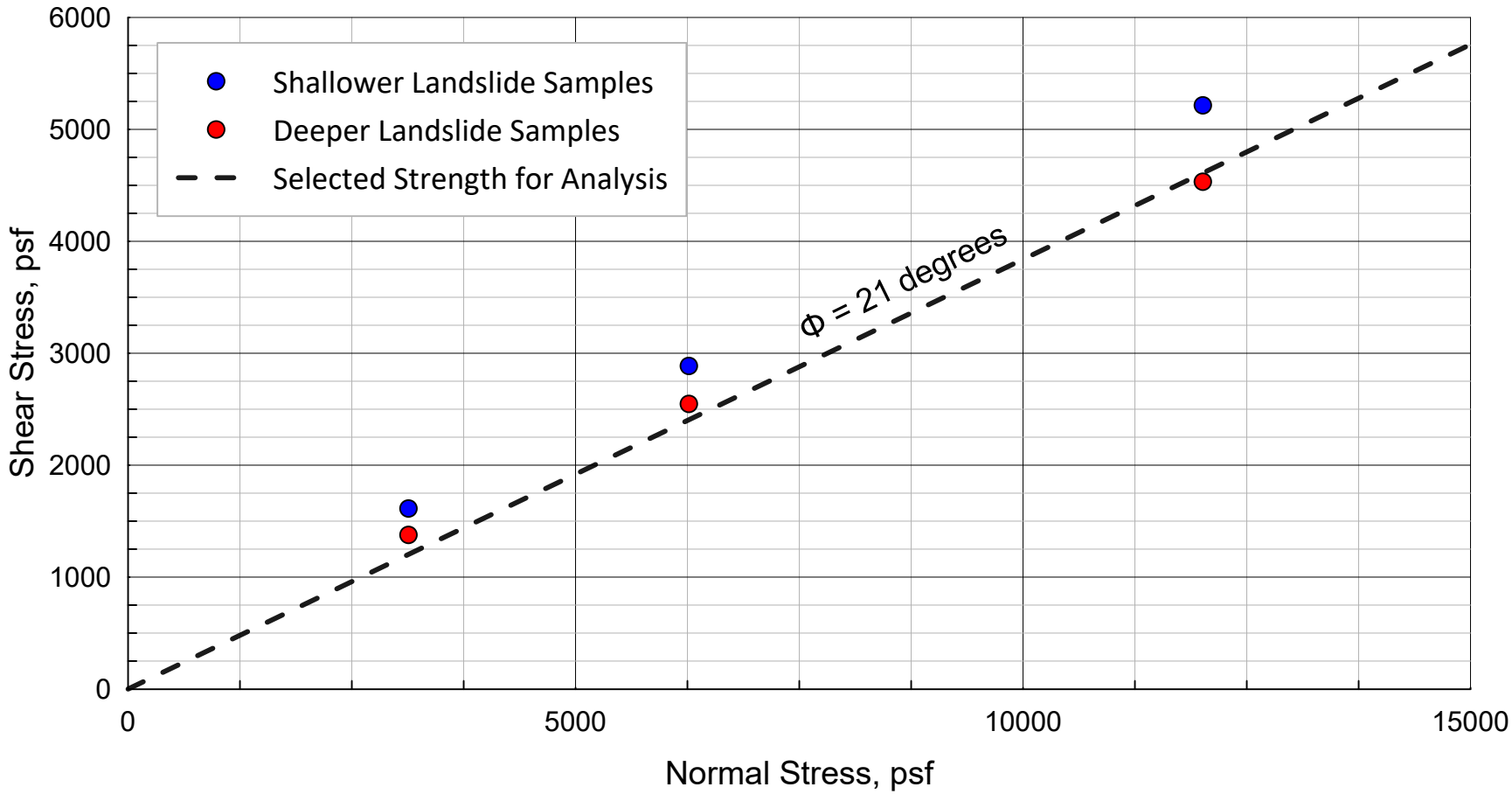
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Debris/Colluvium Total Stress Strengths	Figure 6-41
AECOM			

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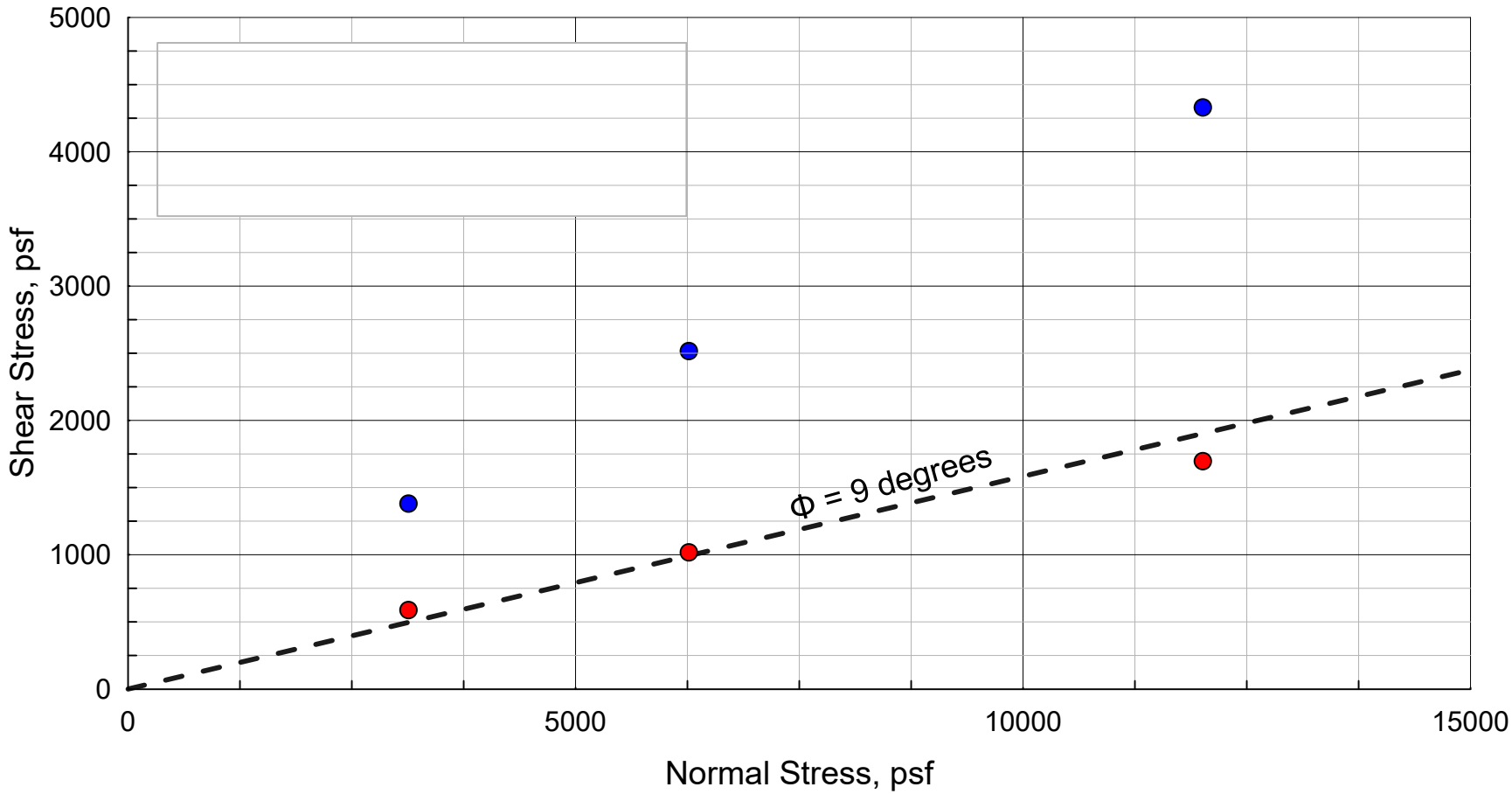
URS	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Landslide Debris/Colluvium Strength Test Behavior	Figure 6-42

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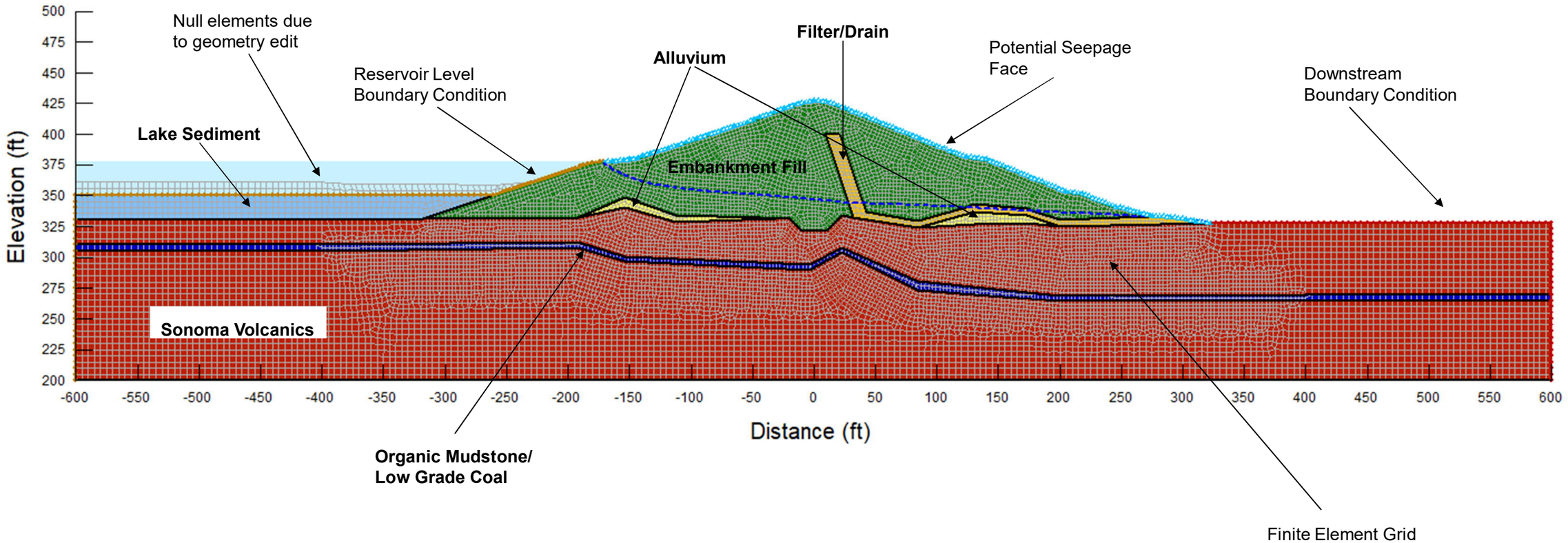
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Debris/Colluvium Fully Softened Shear Strength	Figure 6-43
AECOM			

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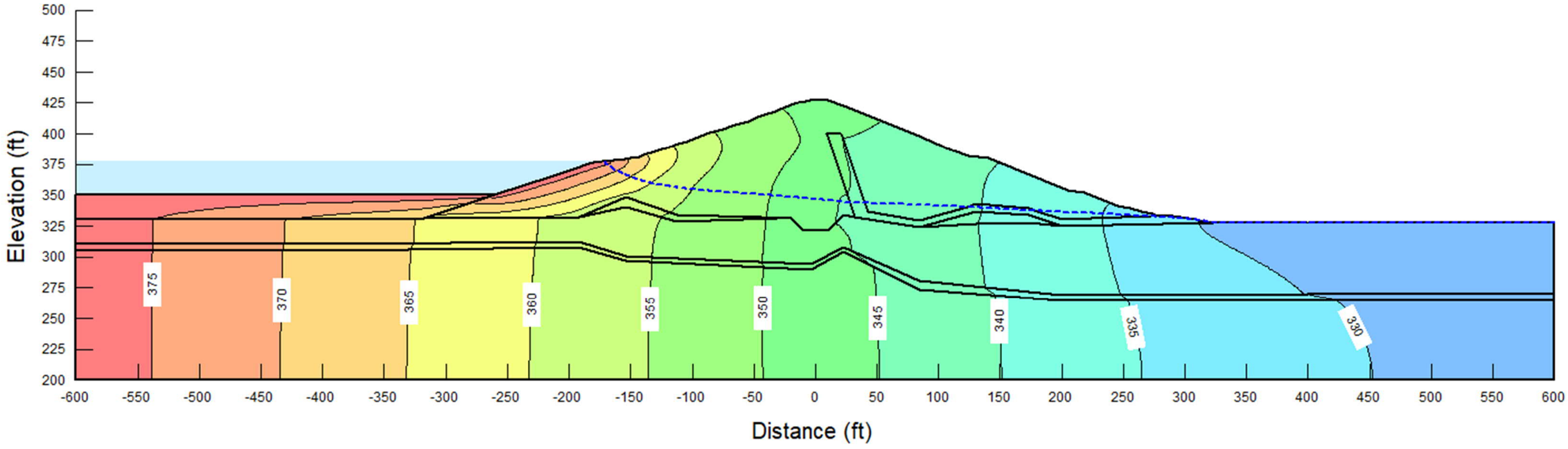


Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Debris/Colluvium Residual Shear Strength	Figure 6-44
AECOM			

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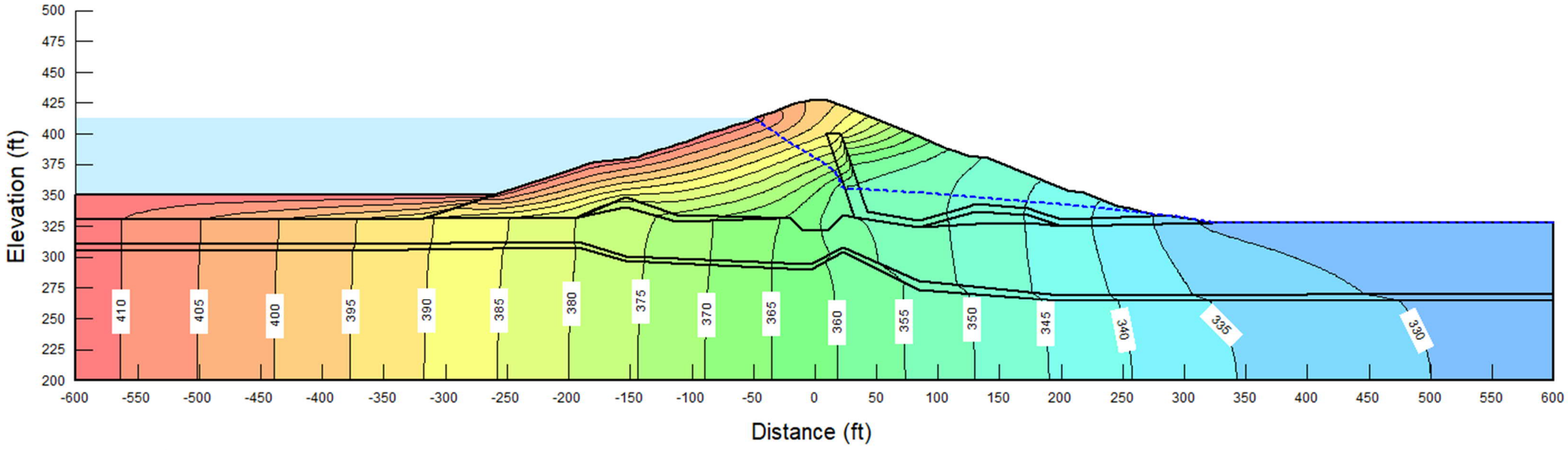


Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Seepage Analysis Example Model Definition	Figure 7-1
	AECOM		



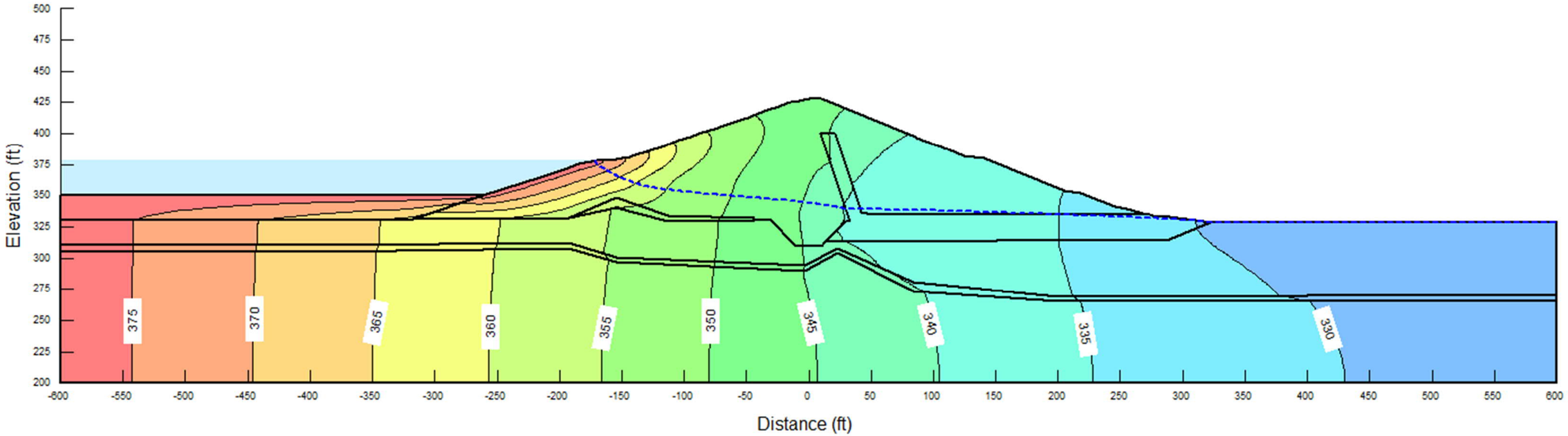
Total head contours

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Seepage Analysis Maximum Section Reservoir at Principal Spillway	Figure 7-2
AECOM			



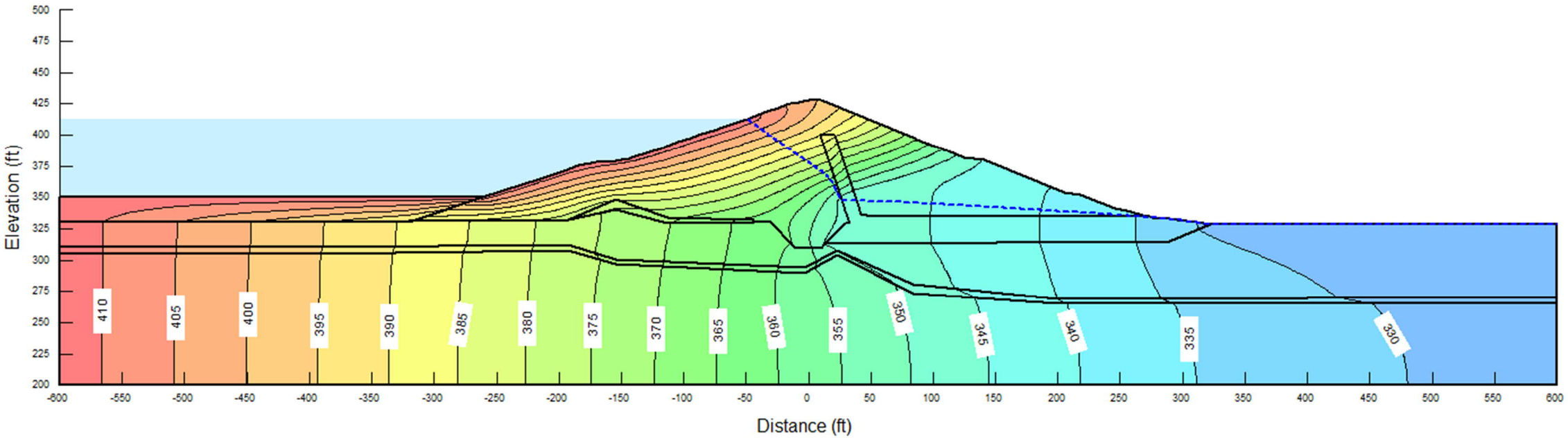
Total head contours

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Seepage Analysis Maximum Section Reservoir at Auxiliary Spillway	Figure 7-3
AECOM			



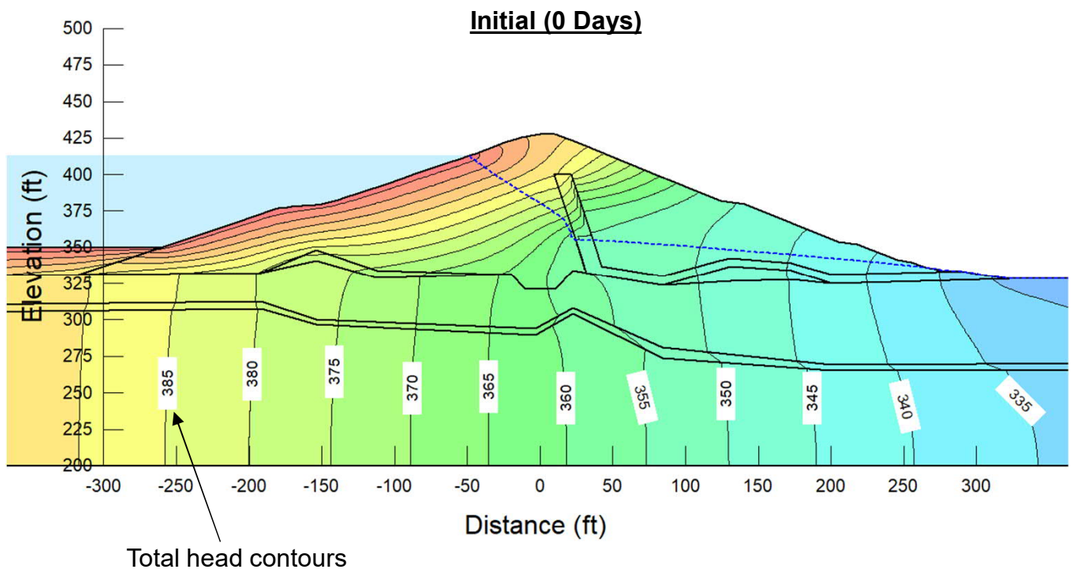
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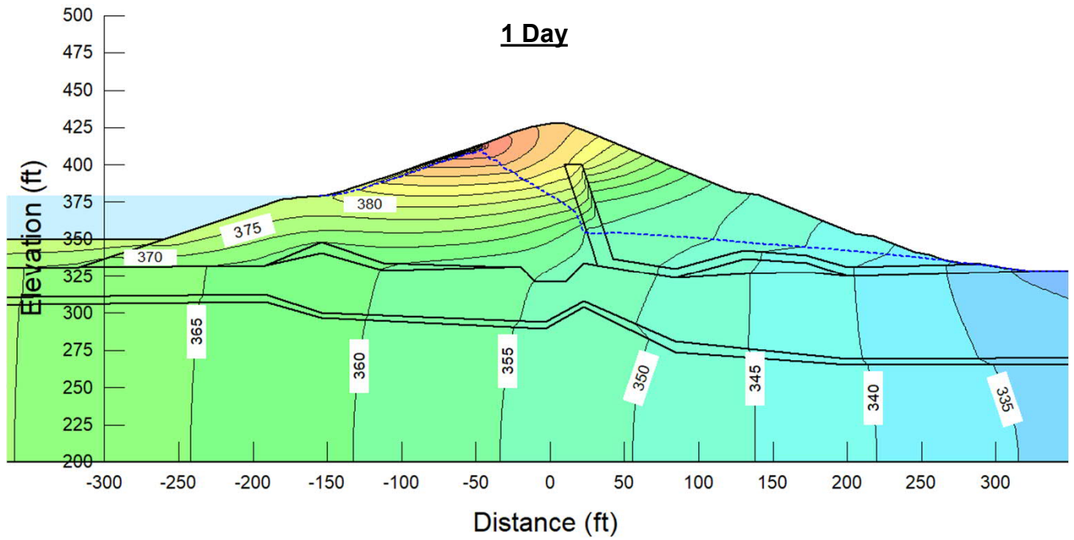
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Seepage Analysis Relief Trench Section Reservoir at Principal Spillway	Figure 7-4
	AECOM		

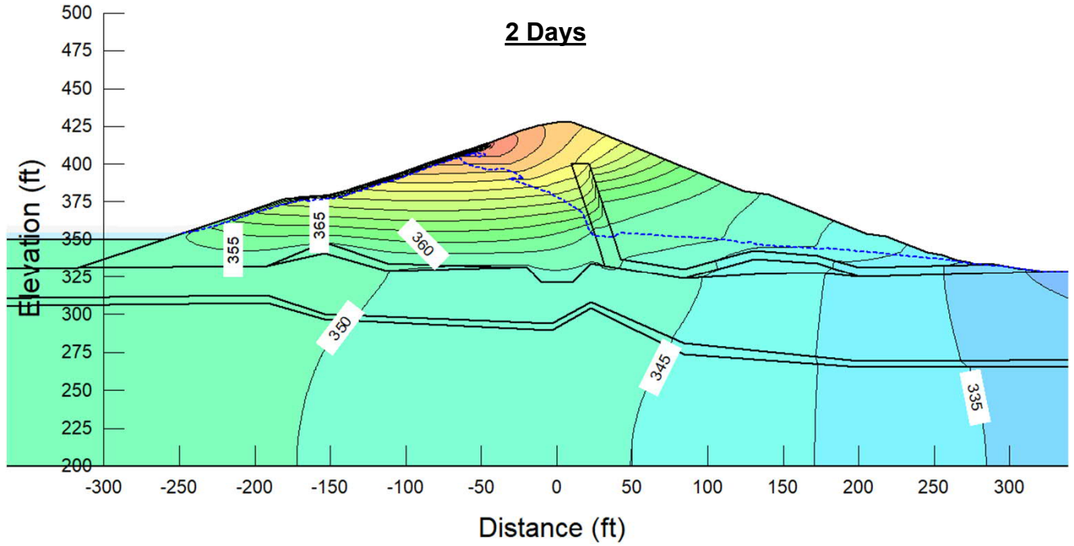


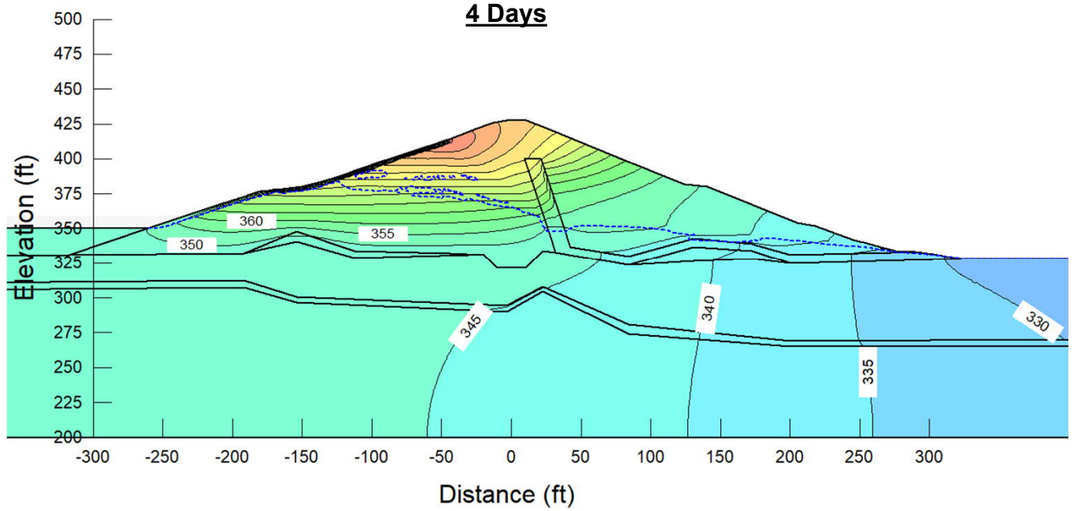
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Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Seepage Analysis Relief Trench Section Reservoir at Auxiliary Spillway	Figure 7-5
AECOM			

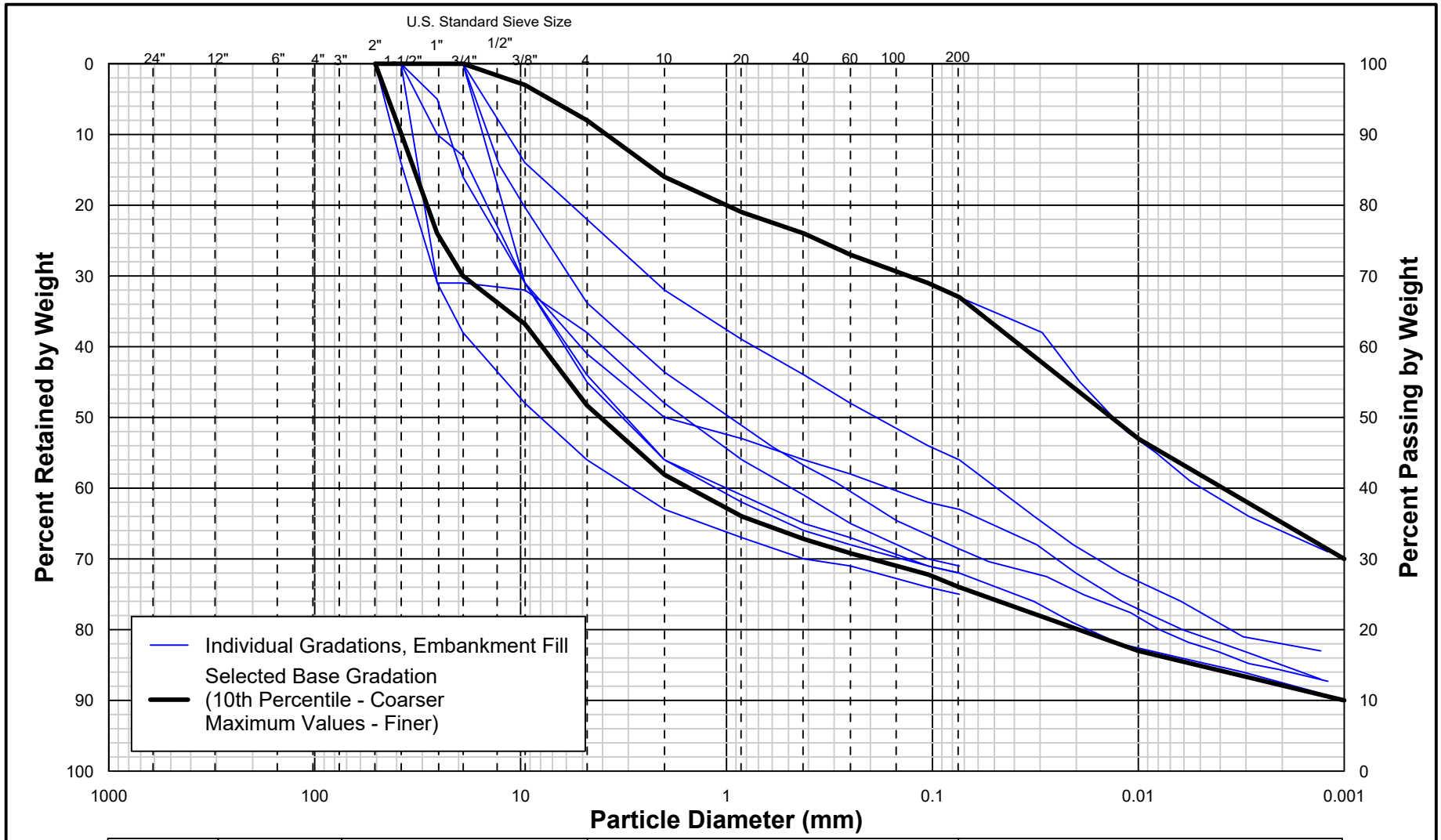






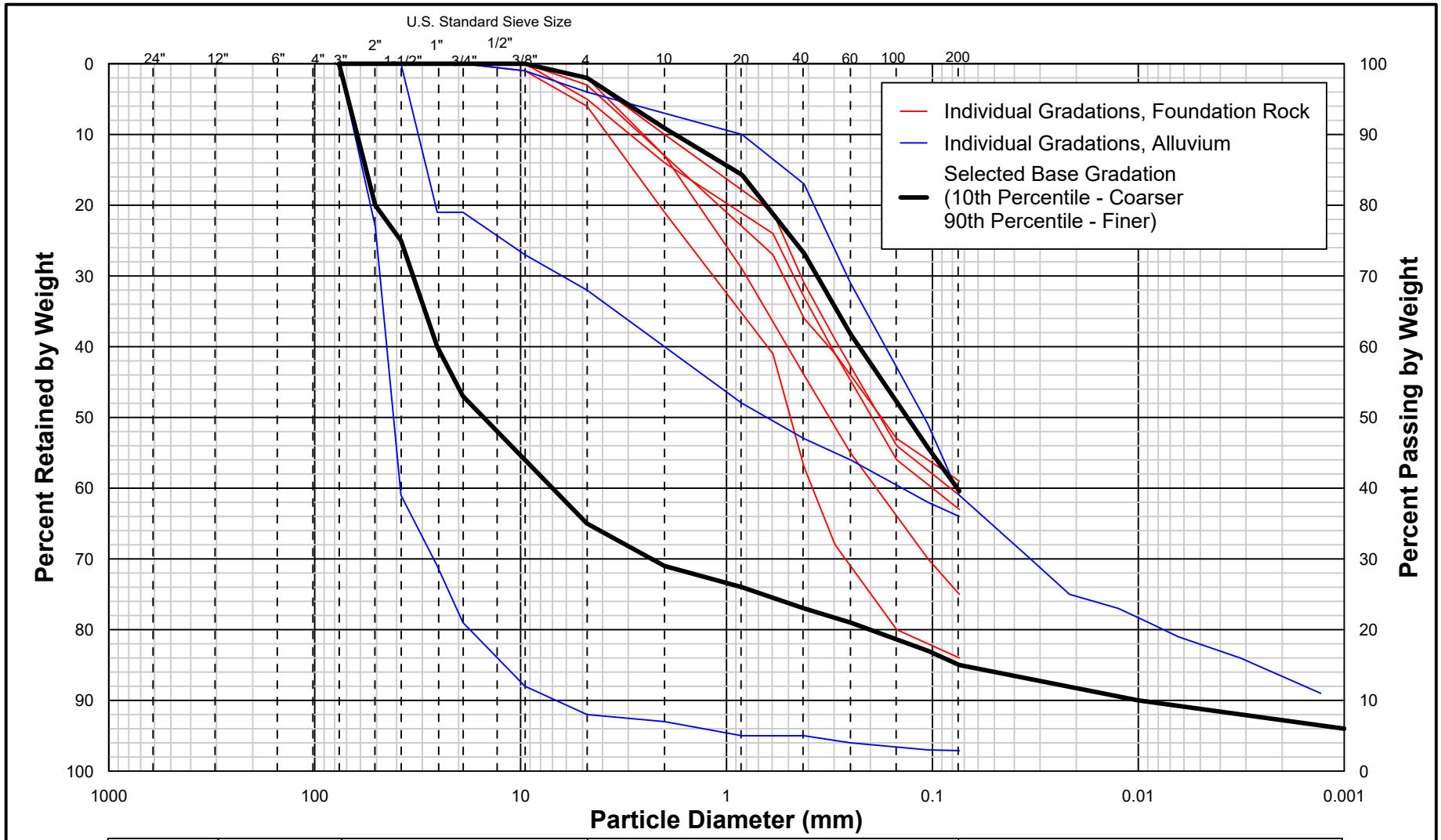


Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Seepage Analysis Maximum Section Reservoir at Auxiliary Spillway Transient Drawdown Analysis	Figure 7-6
	AECOM		



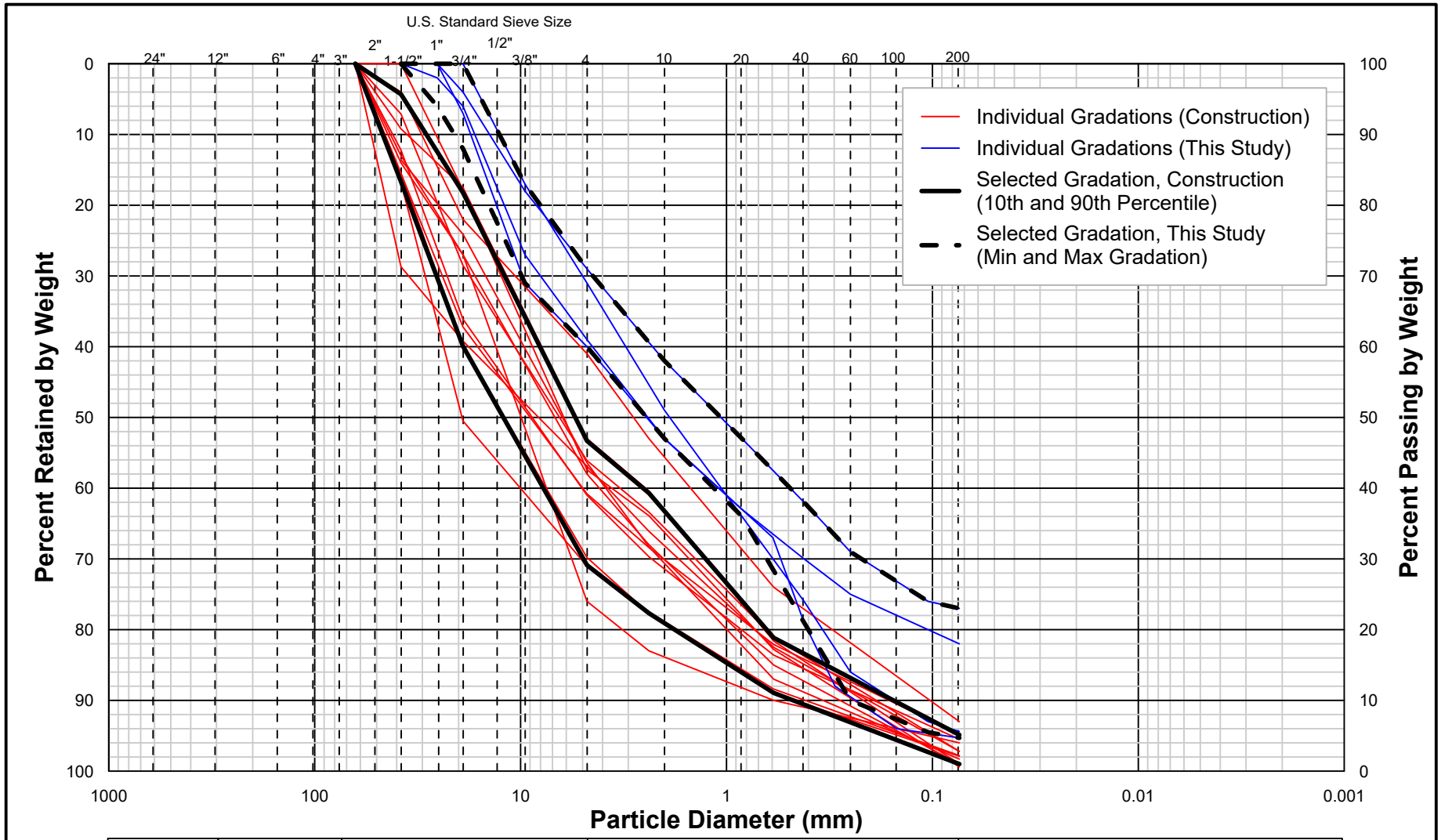
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Figure 7-7
AECOM	Filter Compatibility Embankment Fill Selected Base Soil Gradation	

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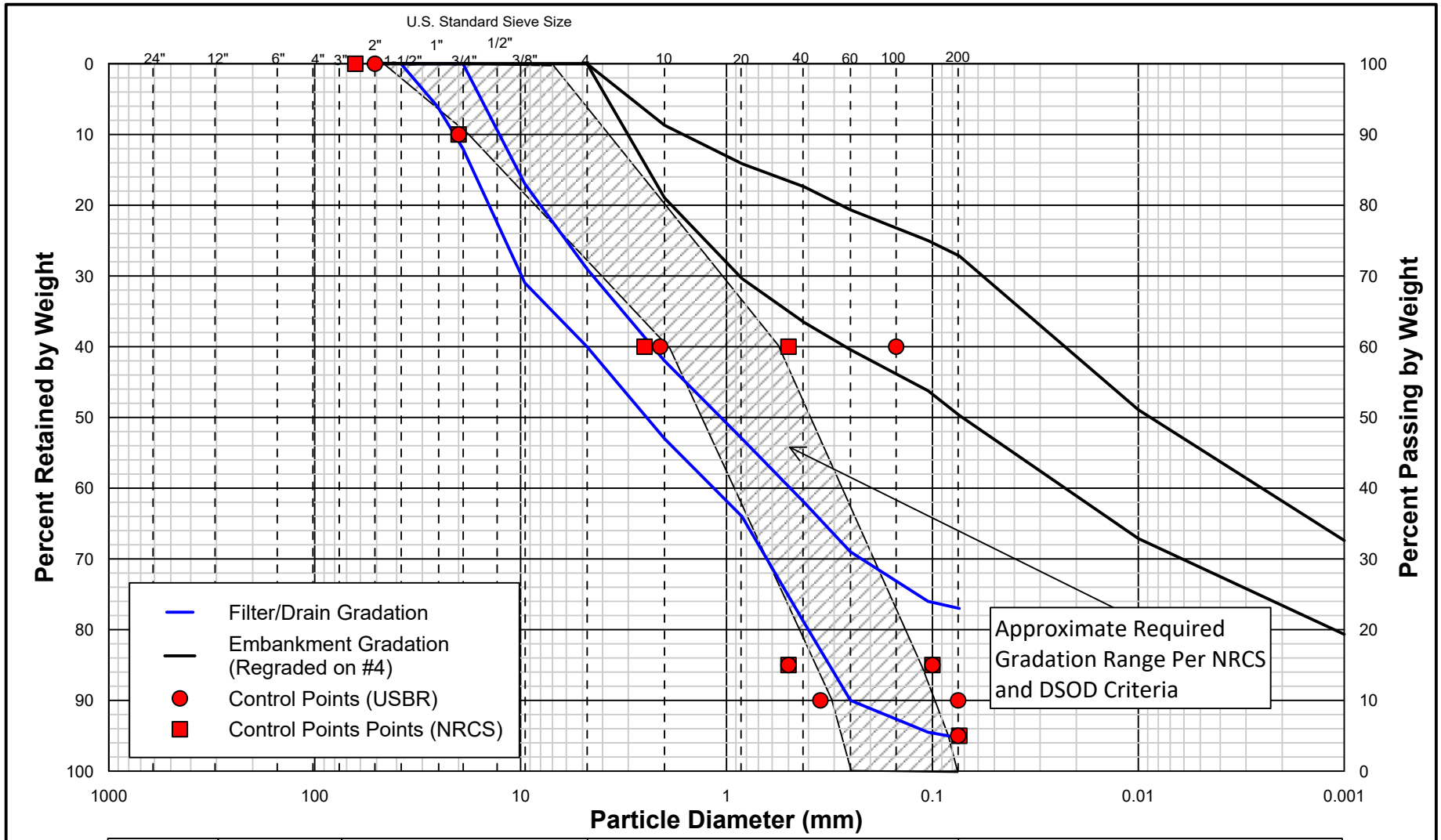
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Figure 7-8
AECOM	Filter Compatibility Alluvium and Foundation Rock Selected Base Soil Gradation	

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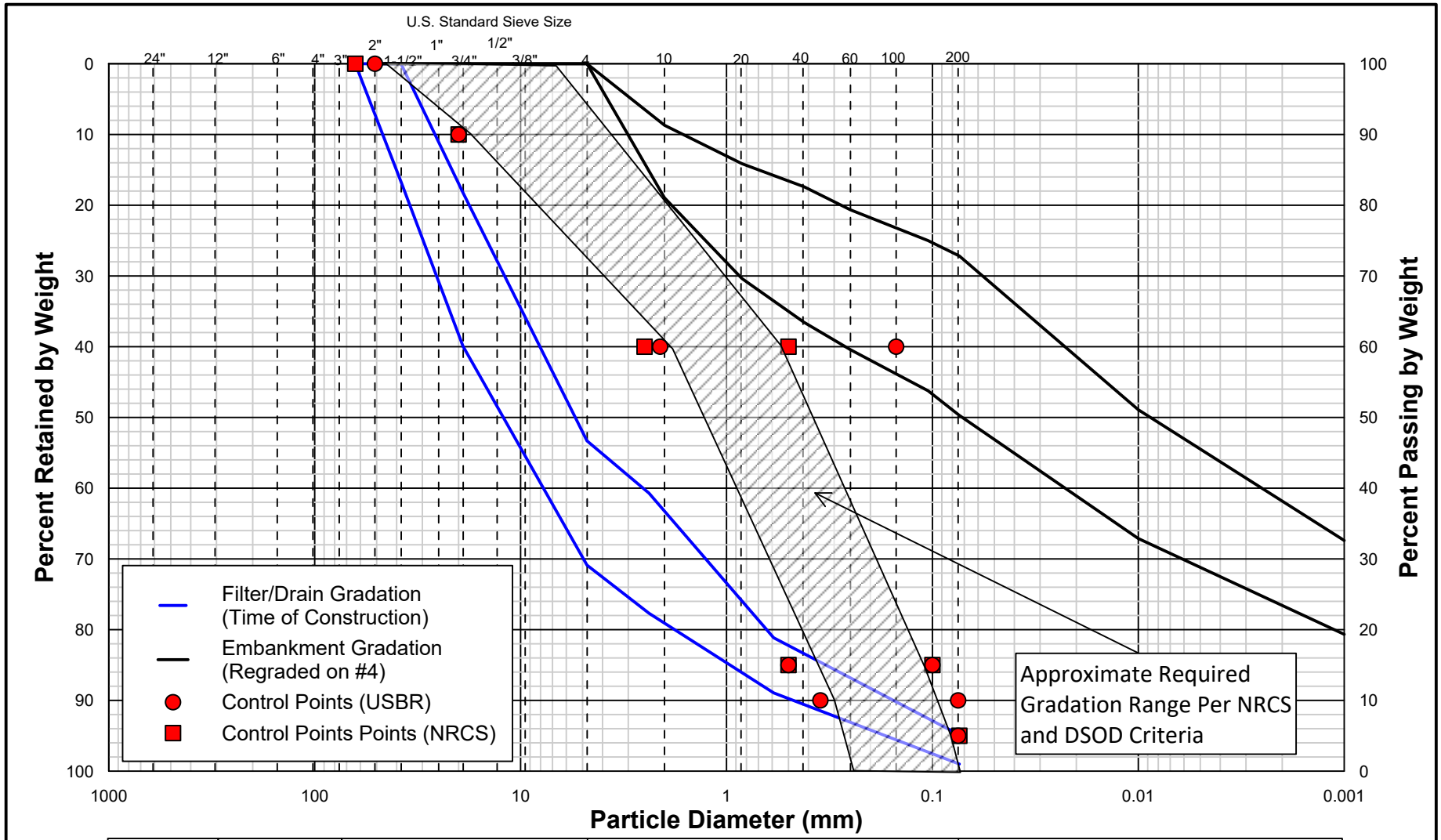
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Filter Compatibility Filter/Drain Selected Gradation
AECOM		Figure 7-9

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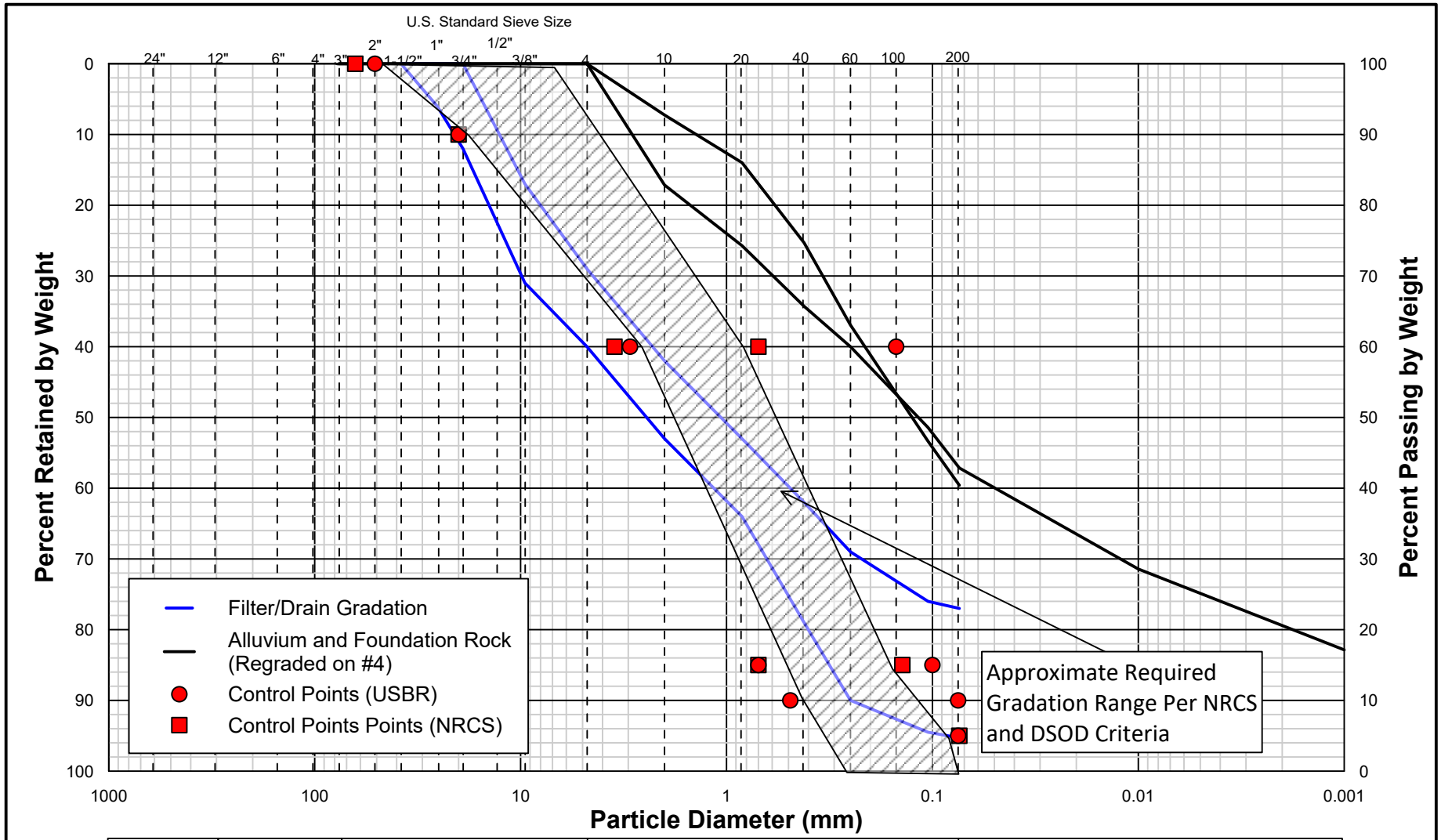
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Filter Compatibility Embankment Fill to Filter/Drain Evaluation Existing Conditions Calculated Control Points	Figure 7-10
AECOM			

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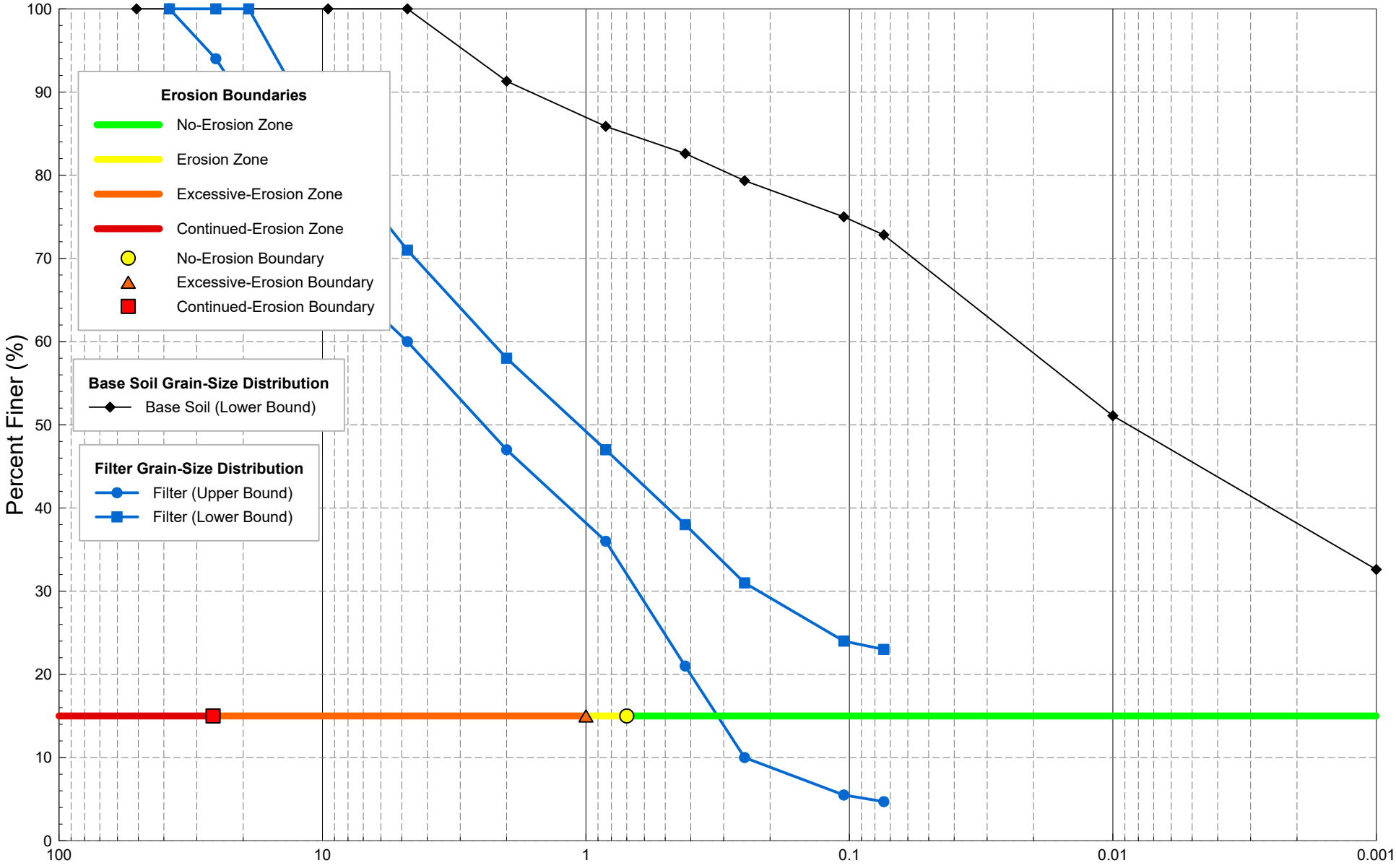
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Filter Compatibility Embankment Fill to Filter/Drain Evaluation Time of Construction Calculated Control Points	Figure 7-11
AECOM			

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Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Filter Compatibility Alluvium and Foundation Rock to Filter/Drain Evaluation Calculated Control Points	Figure 7-12
AECOM			

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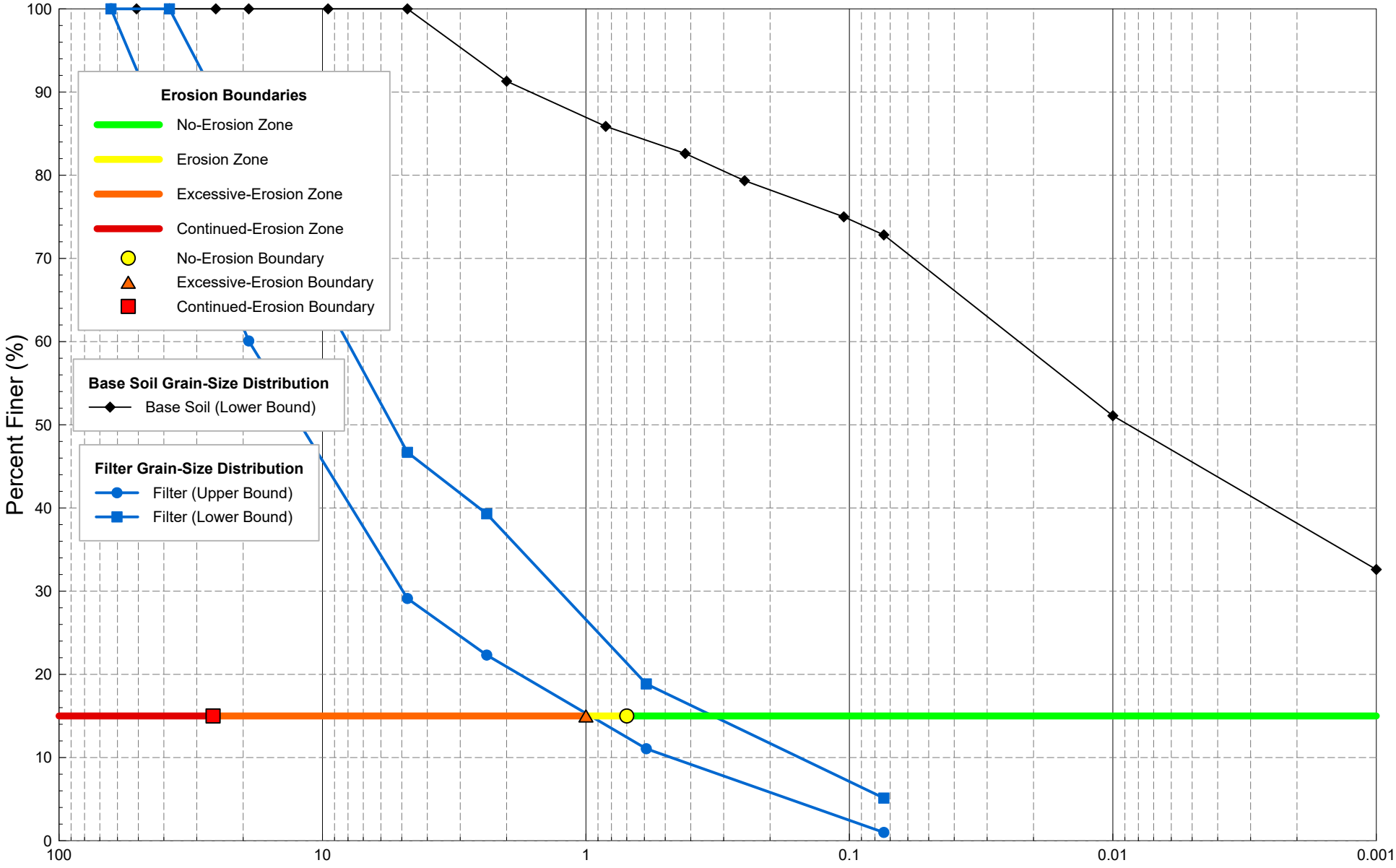


Likelihood of Filter Performance in Event of Concentrated Leak
Seals with no erosion: Unlikely
Seals with some erosion: Equally likely
Partial or no seals with large erosion: Equally likely

Foster, M., and Fell, R. (2001). "Assessing Embankment Dam Filters That Do Not Satisfy Design Criteria." *Journal of Geotechnical and Geoenvironmental Engineering*, 127(5), 398–407.

PROJECT NO. 60647670	Matanzas Creek Dam Soil Mechanics Report	Dam Filter Assessment Embankment Fill to Drain Foster and Fell, 2001	FIGURE 7-13
AECOM	Sonoma County, CA		

C:\Users\Benjamin.Aldridge\AECOM Directory\NRCS-CA Matanzas Creek - General\Embankment Evaluation\Analysis\Filter Erodibility\Figure 7-13_Erosion Boundaries_Embankment Fill to Drain.grf

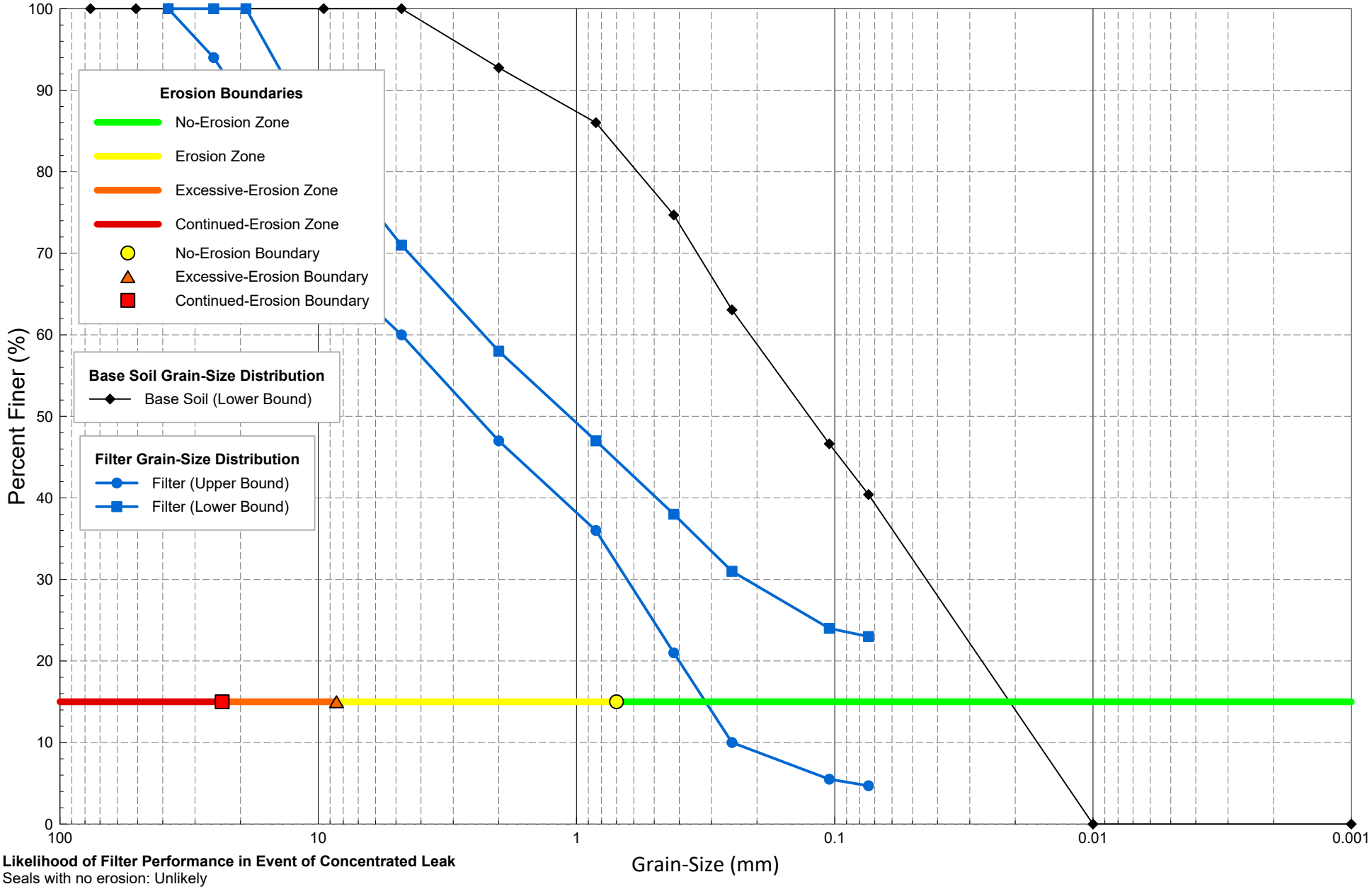


Likelihood of Filter Performance in Event of Concentrated Leak
Seals with no erosion: Unlikely
Seals with some erosion: Equally likely
Partial or no seals with large erosion: Equally likely

Foster, M., and Fell, R. (2001). "Assessing Embankment Dam Filters That Do Not Satisfy Design Criteria." *Journal of Geotechnical and Geoenvironmental Engineering*, 127(5), 398–407.

PROJECT NO. 60647670	Matanzas Creek Dam Soil Mechanics Report	Dam Filter Assessment Embankment Fill to Drain Time of Construction Foster and Fell, 2001	FIGURE 7-14
AECOM	Sonoma County, CA		

C:\Users\Benjamin.Aldridge\AECOM Directory\NRCS-CA Matanzas Creek - General\Embankment Evaluation\Analysis\Filter Erodibility\Figure 7-14_Erosion Boundaries_Embankment Fill to Drain_Construction.grf

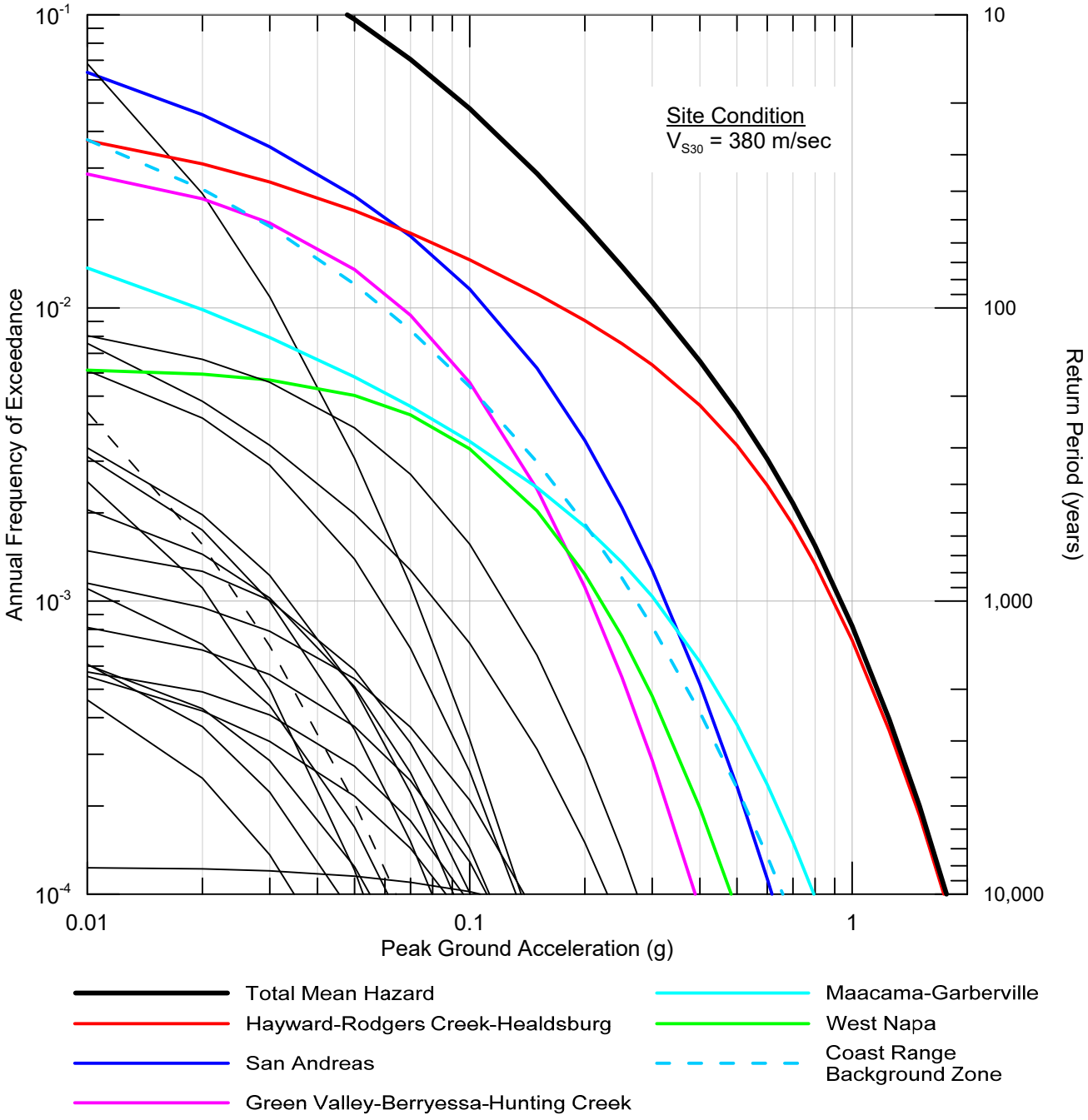


Likelihood of Filter Performance in Event of Concentrated Leak
Seals with no erosion: Unlikely
Seals with some erosion: Equally likely
Partial or no seals with large erosion: Equally likely

Foster, M., and Fell, R. (2001). "Assessing Embankment Dam Filters That Do Not Satisfy Design Criteria." *Journal of Geotechnical and Geoenvironmental Engineering*, 127(5), 398–407.

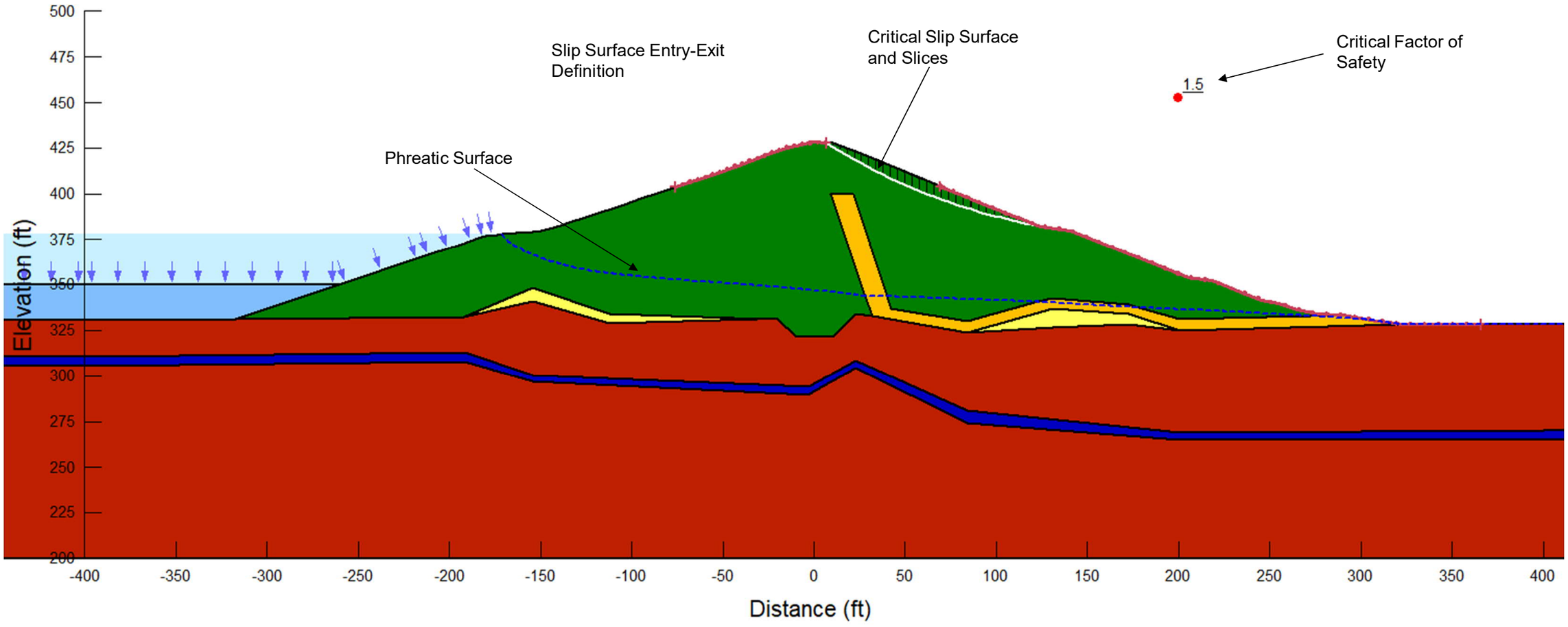
PROJECT NO. 60647670	Matanzas Creek Dam Soil Mechanics Report	Dam Filter Assessment Foundation to Drain Foster and Fell, 2001	FIGURE 7-15
AECOM	Sonoma County, CA		

C:\Users\Benjamin.Aldridge\AECOM Directory\NRCS-CA Matanzas Creek - General\Embankment Evaluation\Analysis\Filter Erodibility\Figure 7-15_Erosion Boundaries_Foundation to Drain.grf



Other less significant sources shown are not listed.

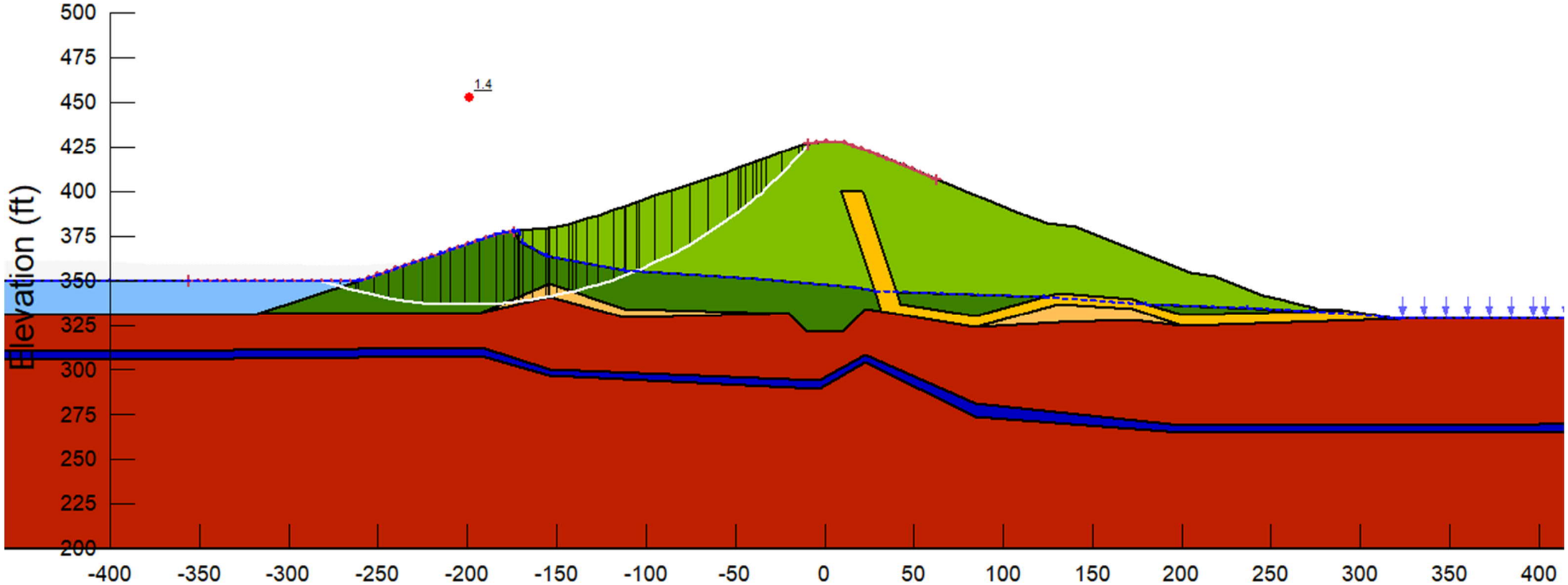
AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	SEISMIC SOURCE CONTRIBUTIONS TO MEAN HORIZONTAL PEAK GROUND ACCELERATION HAZARD	Figure 8-1



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

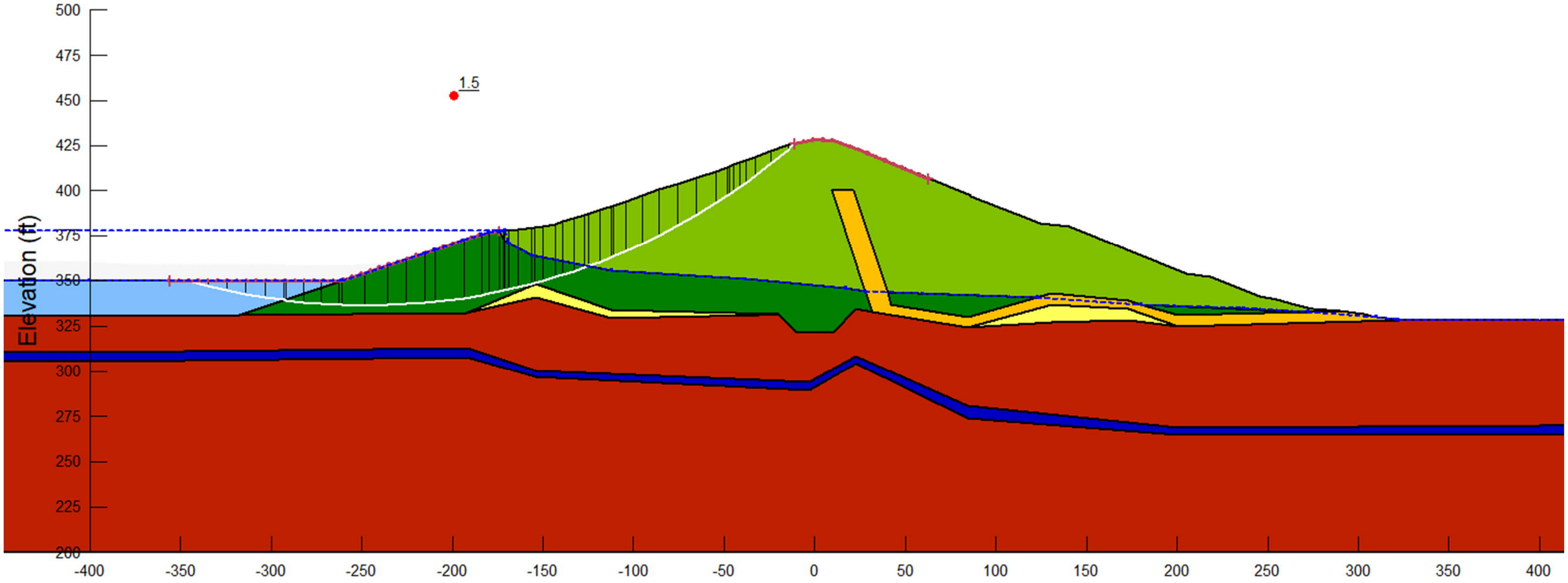
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Long-Term Stability	Figure 8-3
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Bilinear Strength)	120	35	0	-	-	18	400
	Embankment Fill (Bilinear Strength)	115	34	20	-	-	18	650
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

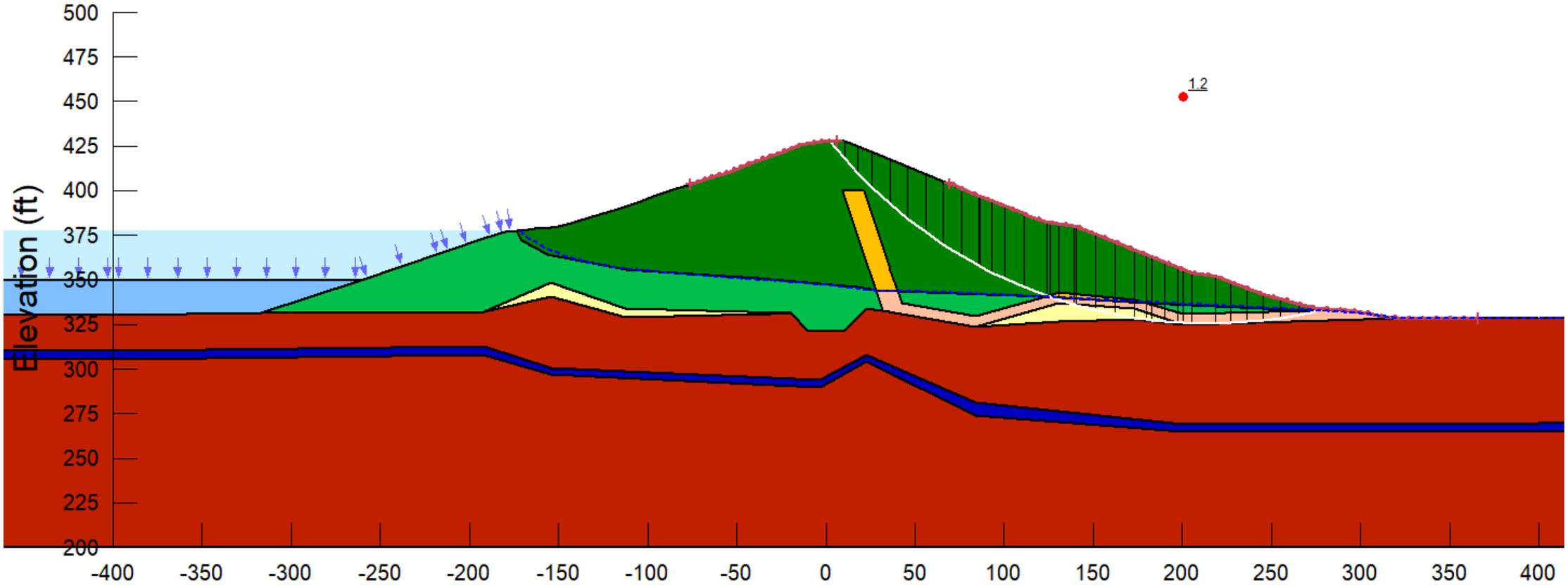
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Rapid Drawdown Stability Bilinear Strengths	Figure 8-4
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

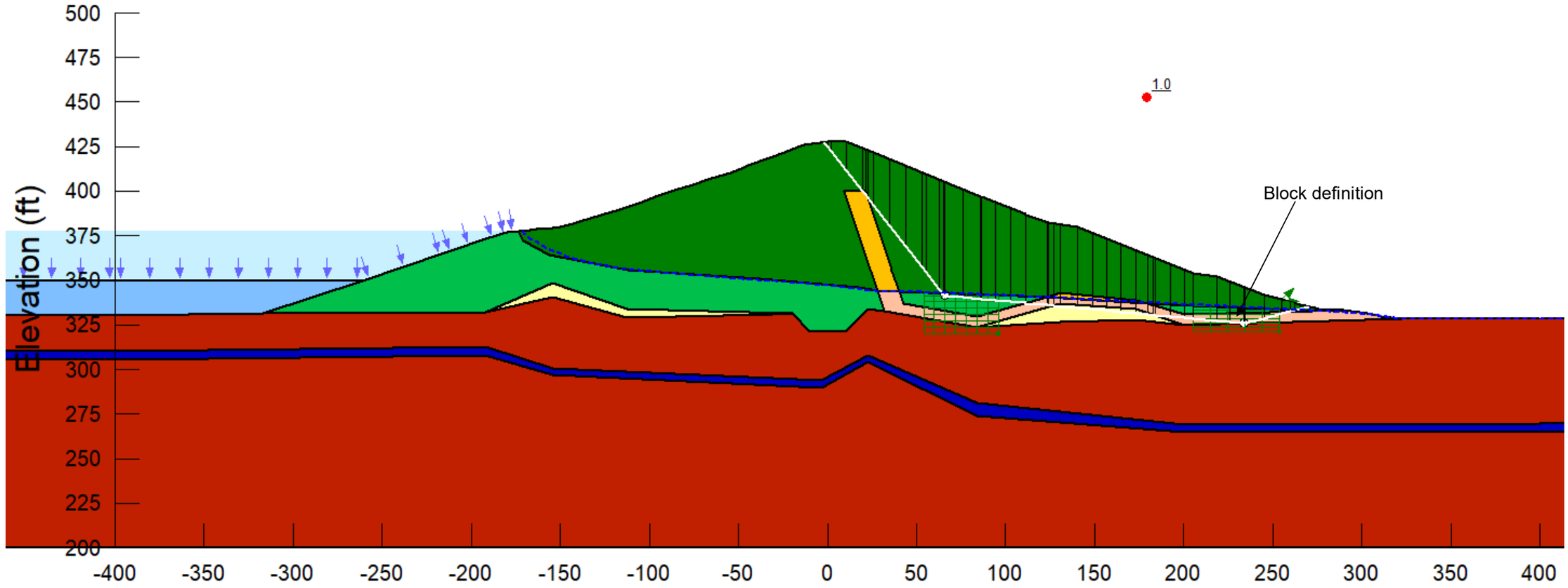
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Rapid Drawdown Stability Duncan et al. (1990) Strengths	Figure 8-5
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Residual Strength)	120	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Residual Strength)	115	$\sigma'_v/\tau = 0.15$ (Min=100 psf)				-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Filter/Drain (Residual Strength)	130	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

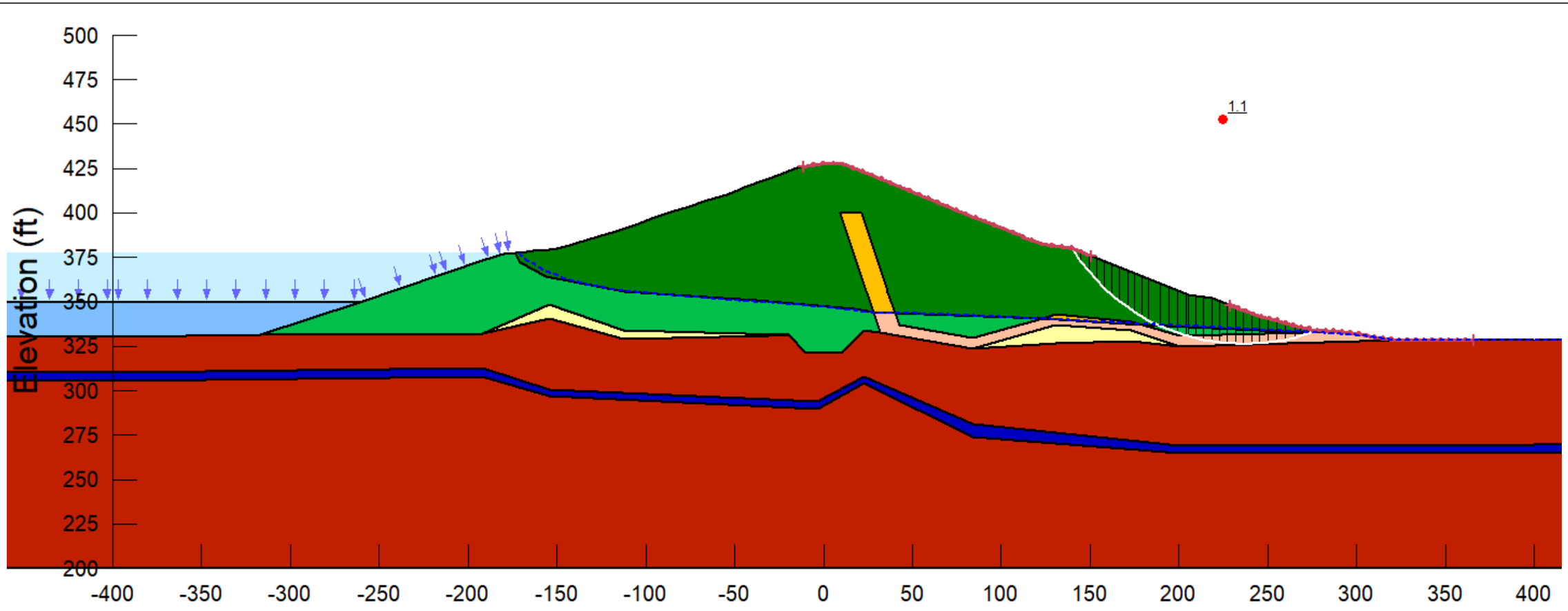
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Post-Seismic Stability Downstream Slope	Figure 8-6
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Residual Strength)	120	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Residual Strength)	115	$\sigma'_v/\tau = 0.15$ (Min=100 psf)				-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Filter/Drain (Residual Strength)	130	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

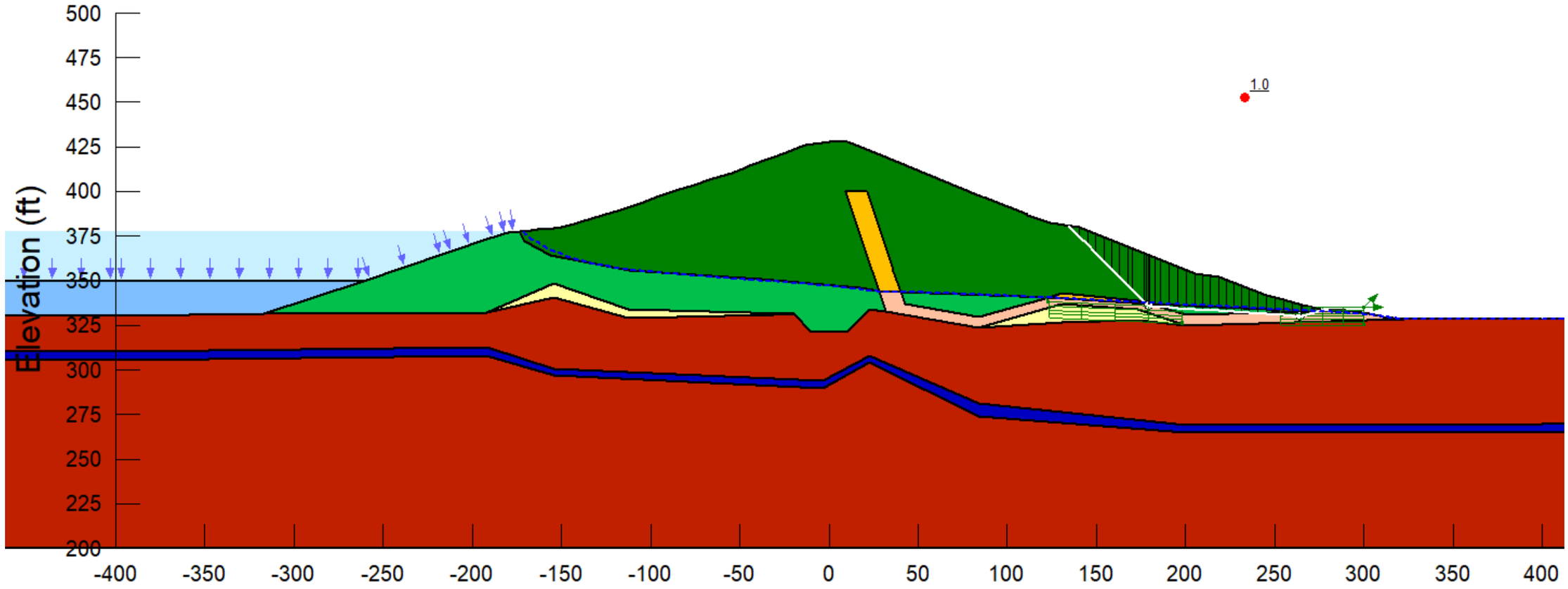
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Post-Seismic Stability Downstream Slope (Block Failure)	Figure 8-7
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Residual Strength)	120	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Residual Strength)	115	$\sigma'_v/\tau = 0.15$ (Min=100 psf)				-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Filter/Drain (Residual Strength)	130	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

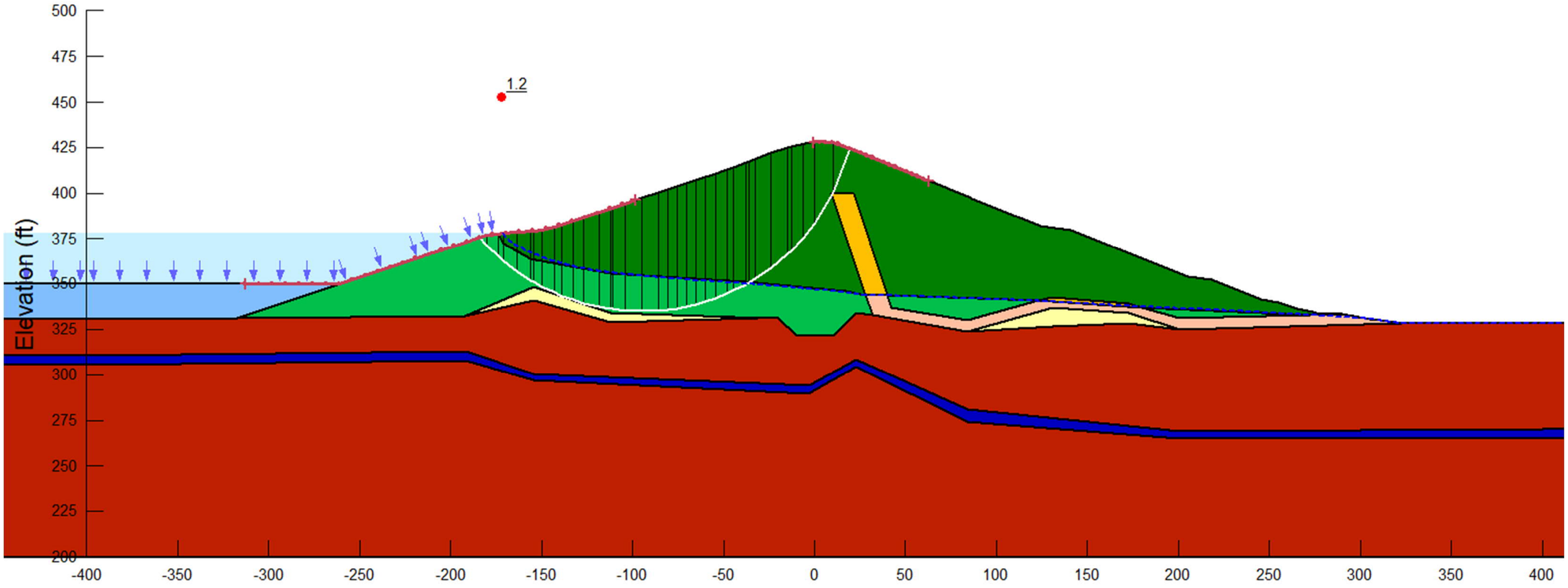
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Post-Seismic Stability Downstream Slope (Lower)	Figure 8-8
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Residual Strength)	120	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Residual Strength)	115	$\sigma'_v/\tau = 0.15$ (Min=100 psf)				-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Filter/Drain (Residual Strength)	130	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

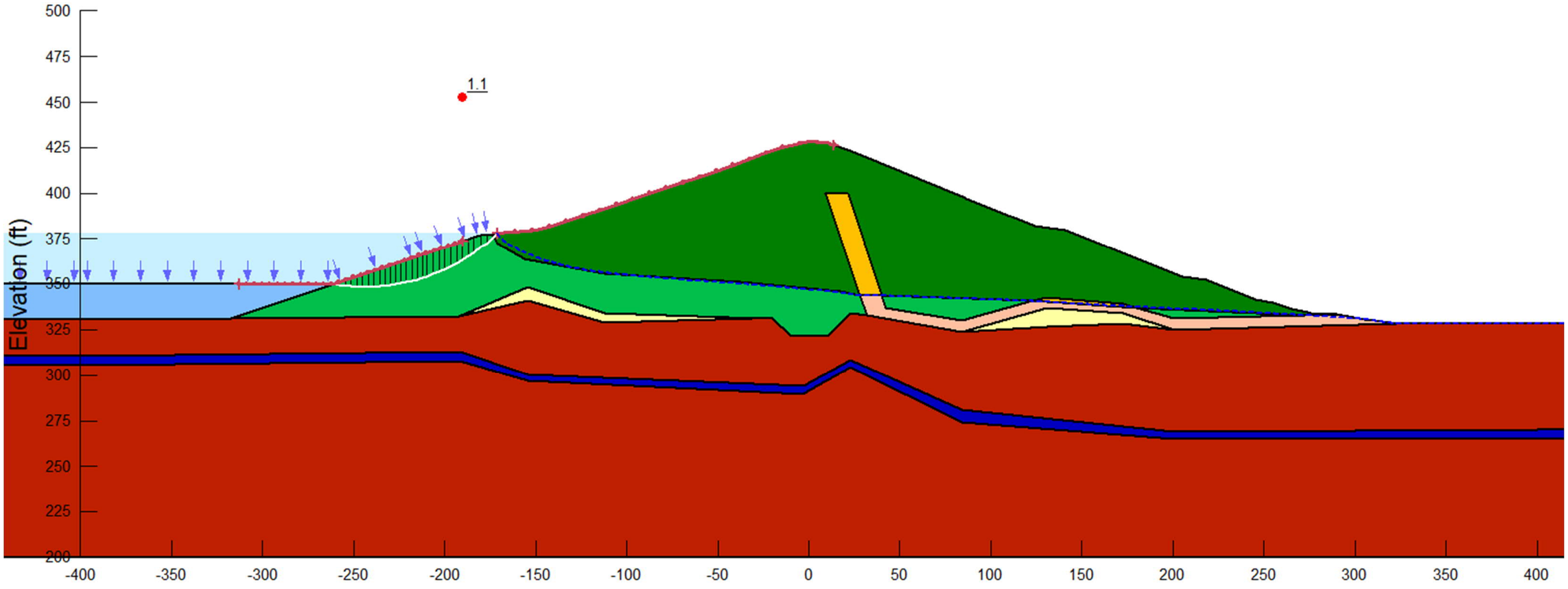
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Post-Seismic Stability Downstream Slope (Lower, Block Failure)	Figure 8-9
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Residual Strength)	120	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Residual Strength)	115	$\sigma'_v/\tau = 0.15$ (Min=100 psf)				-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Filter/Drain (Residual Strength)	130	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

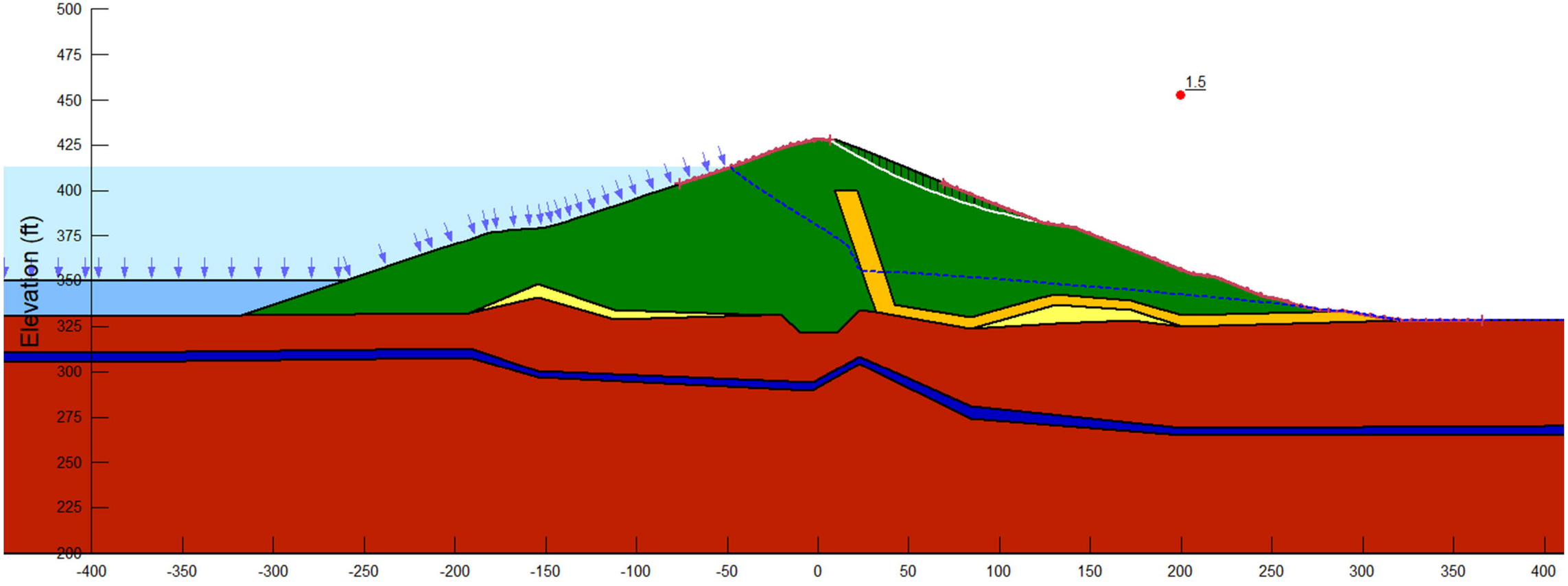
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Post-Seismic Stability Upstream Slope	Figure 8-10
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Residual Strength)	120	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Residual Strength)	115	$\sigma'_v/\tau = 0.15$ (Min=100 psf)				-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Filter/Drain (Residual Strength)	130	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

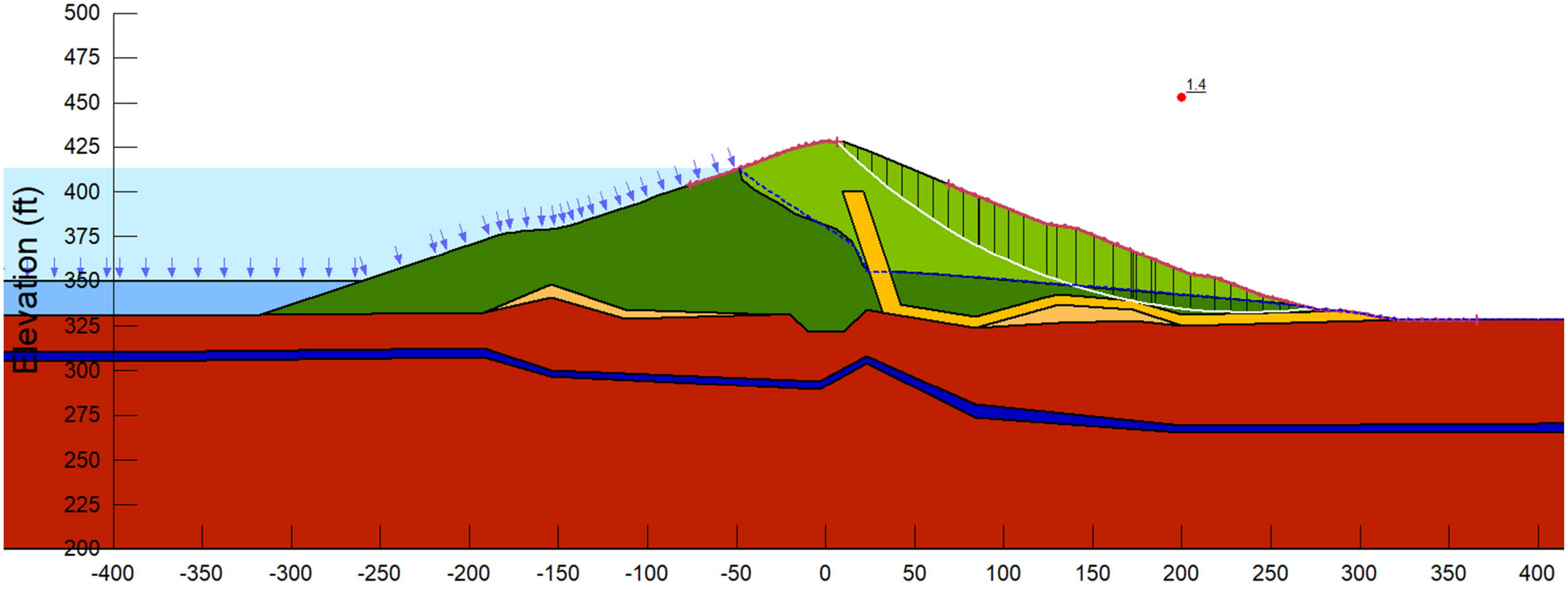
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Post-Seismic Stability Upstream Slope (Lower)	Figure 8-11
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

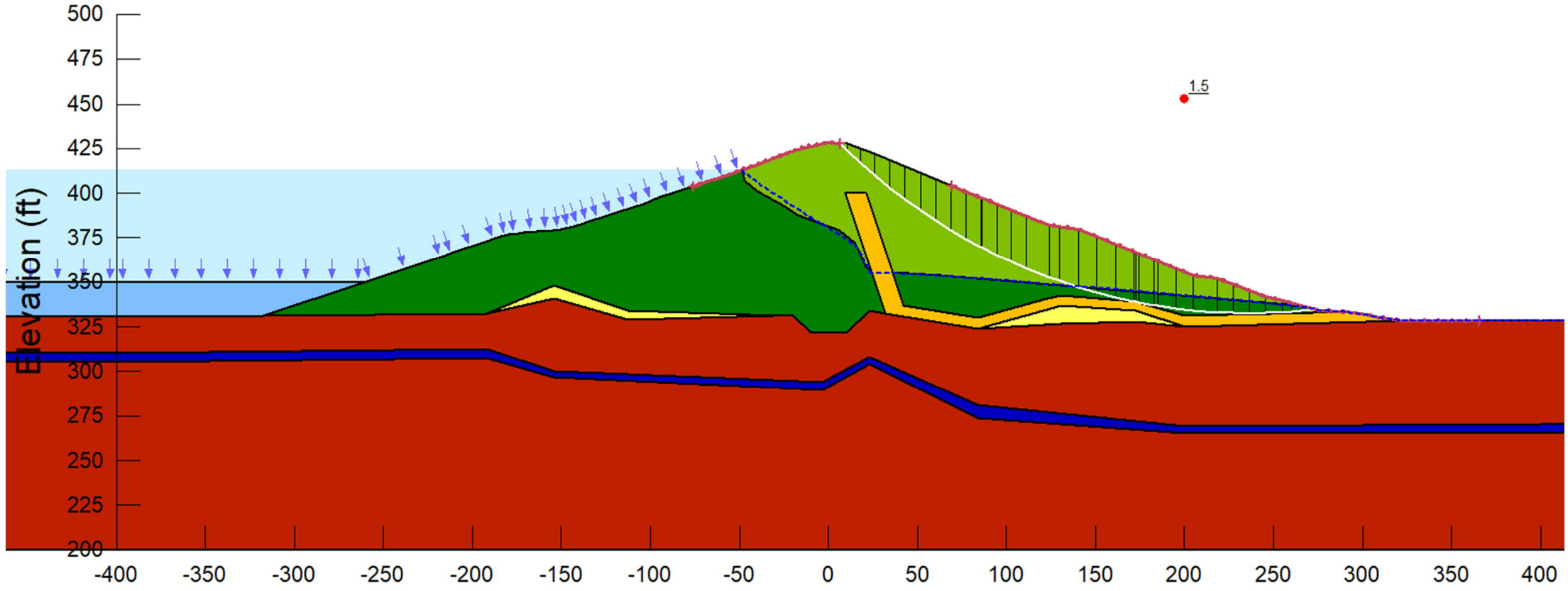
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Auxiliary Spillway Long-Term Stability Effective Strengths	Figure 8-12
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Bilinear Strength)	120	35	0	-	-	18	400
	Embankment Fill (Bilinear Strength)	115	34	20	-	-	18	650
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

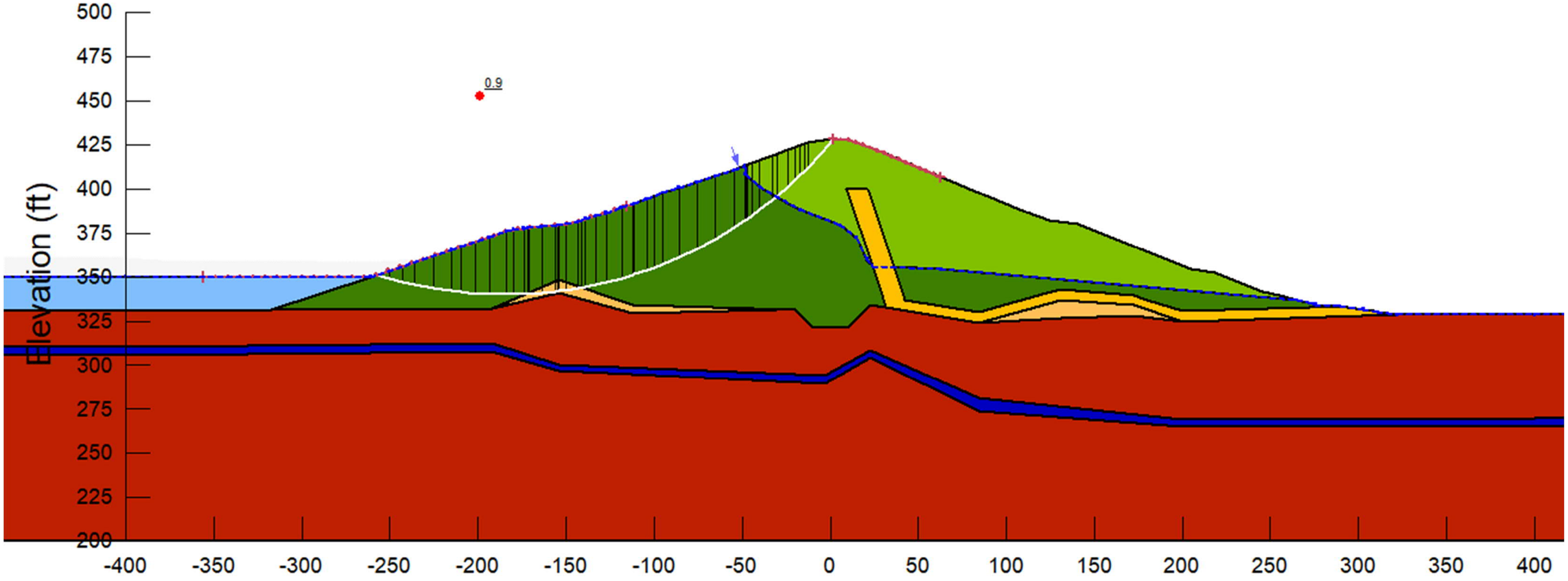
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Auxiliary Spillway Long-Term Stability Bilinear Strengths	Figure 8-13
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Bilinear Strength)	120	35	0	-	-	18	400
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

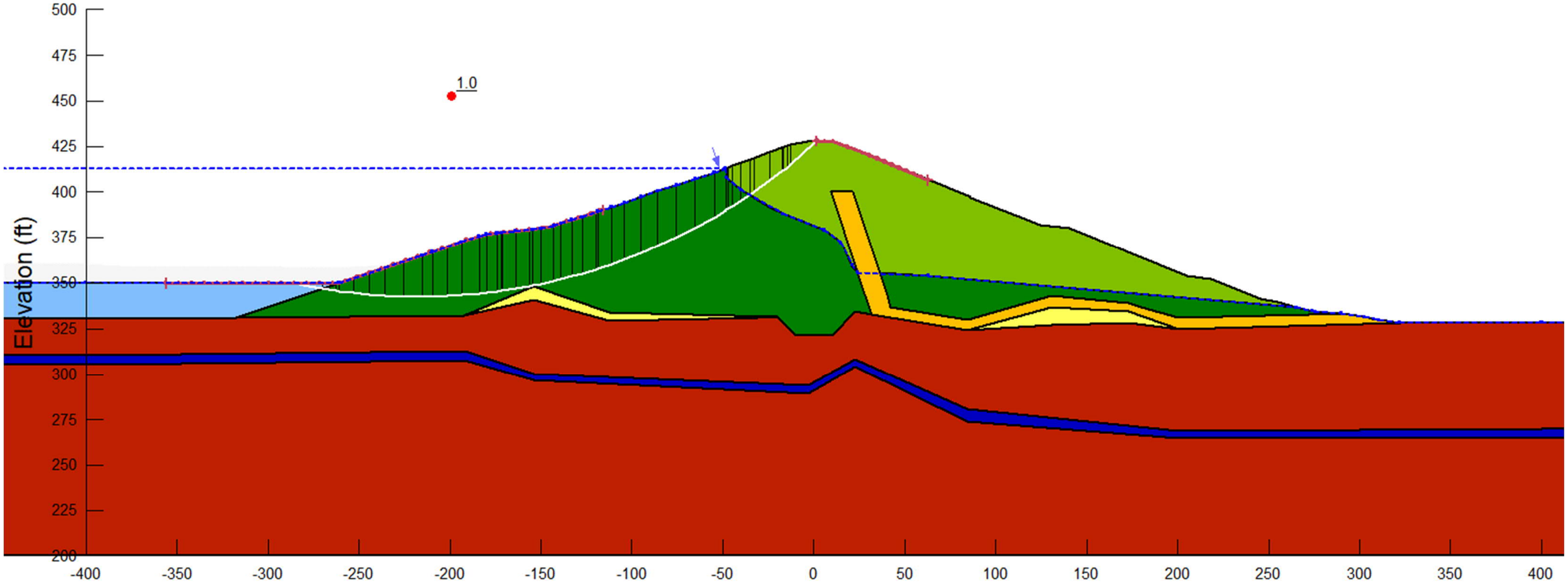
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Auxiliary Spillway Long-Term Stability Undrained Strengths	Figure 8-14
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

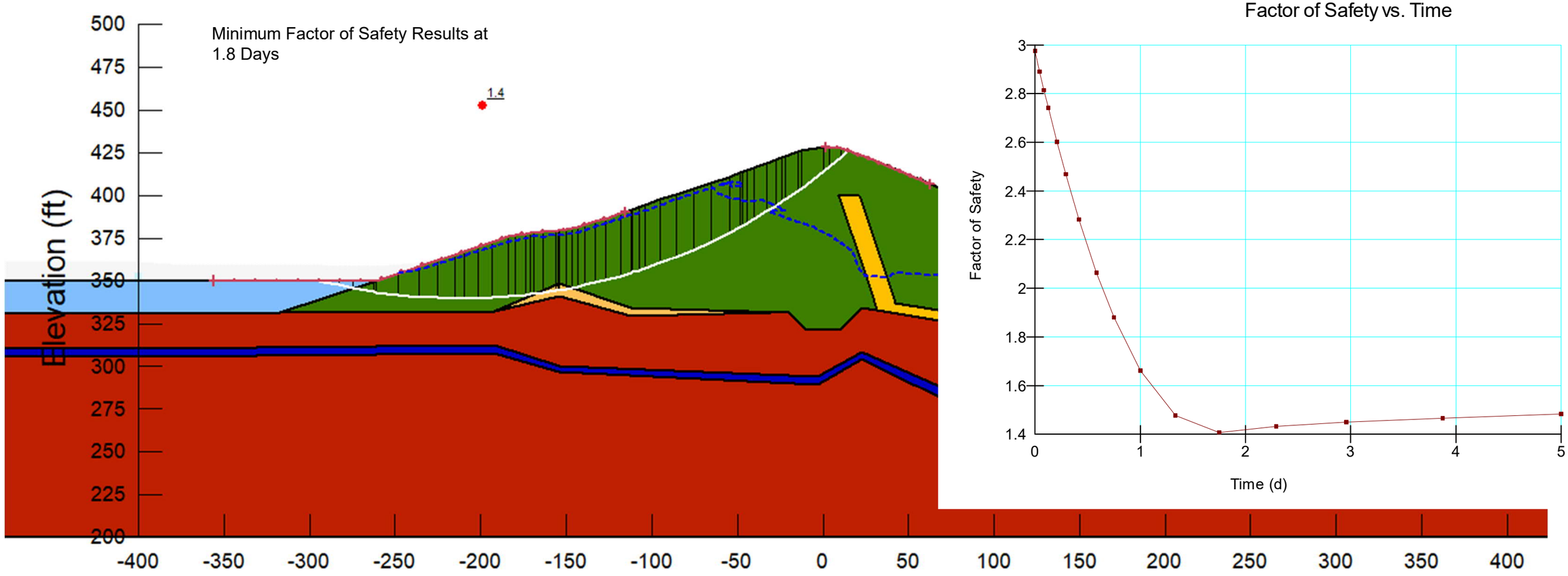
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Auxiliary Spillway Rapid Drawdown Stability Bilinear Strengths	Figure 8-15
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

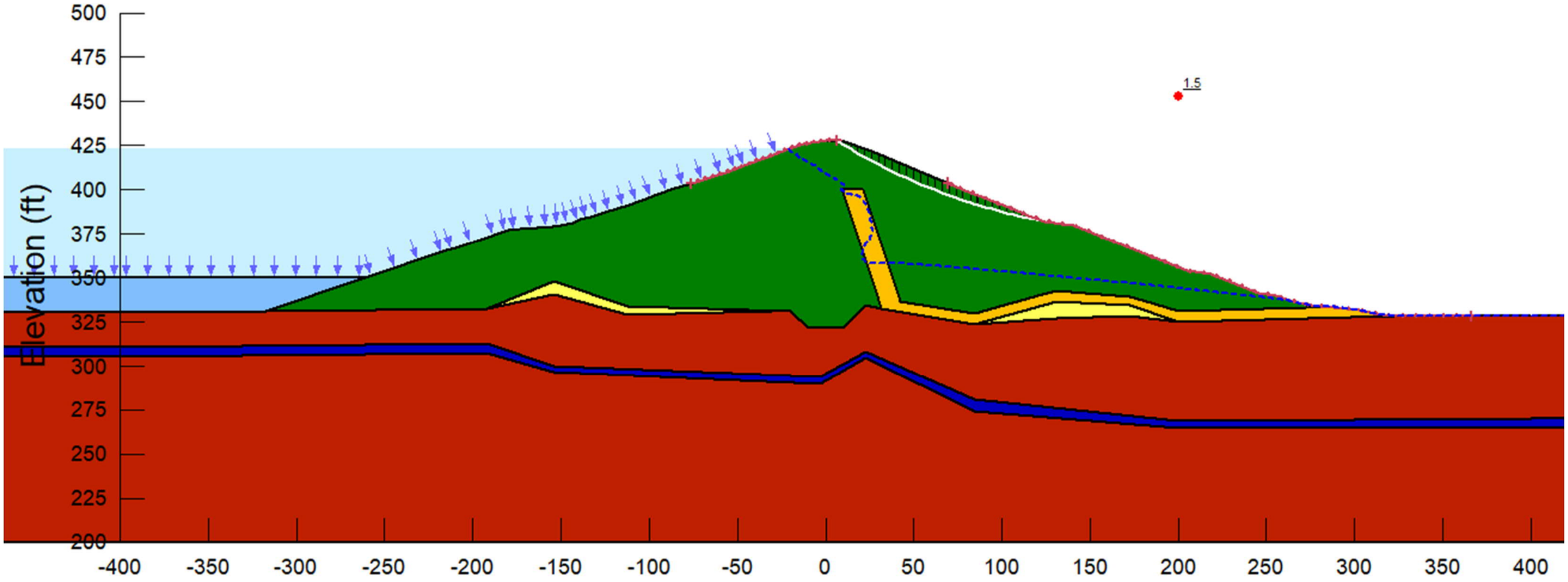
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Auxiliary Spillway Rapid Drawdown Stability Duncan et al. (1990) Strengths	Figure 8-16
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Bilinear Strength)	120	35	0	-	-	18	400
	Embankment Fill (Bilinear Strength)	115	34	20	-	-	18	650
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

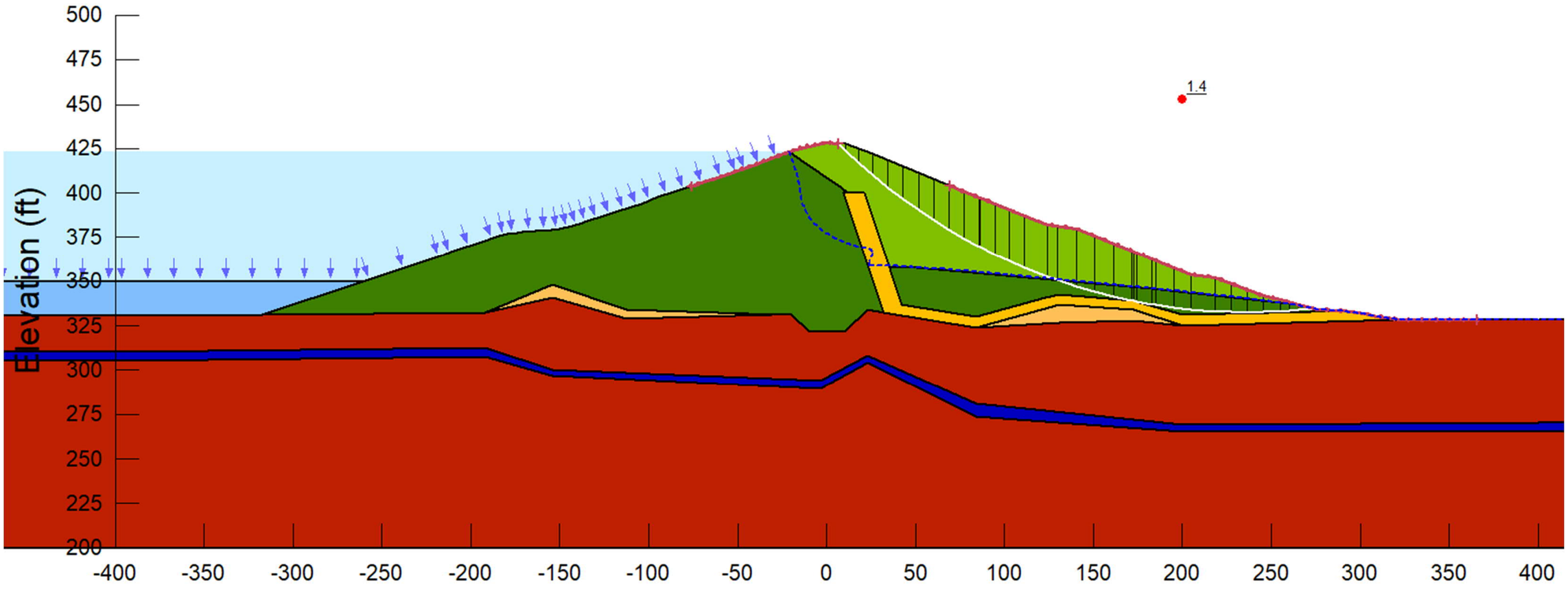
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Auxiliary Spillway Rapid Drawdown Stability Transient Drawdown (Bilinear Strengths)	Figure 8-17
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

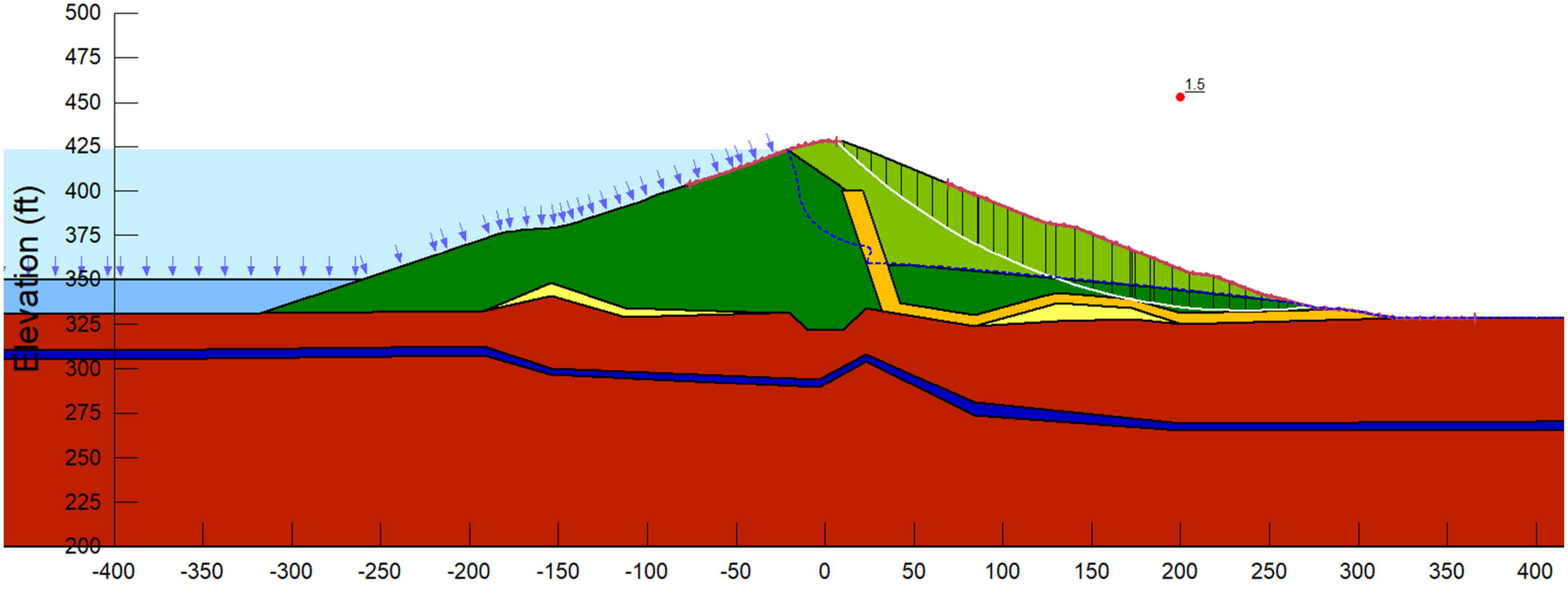
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Flood Surcharge Long-Term Stability Effective Strengths	Figure 8-18
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Bilinear Strength)	120	35	0	-	-	18	400
	Embankment Fill (Bilinear Strength)	115	34	20	-	-	18	650
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

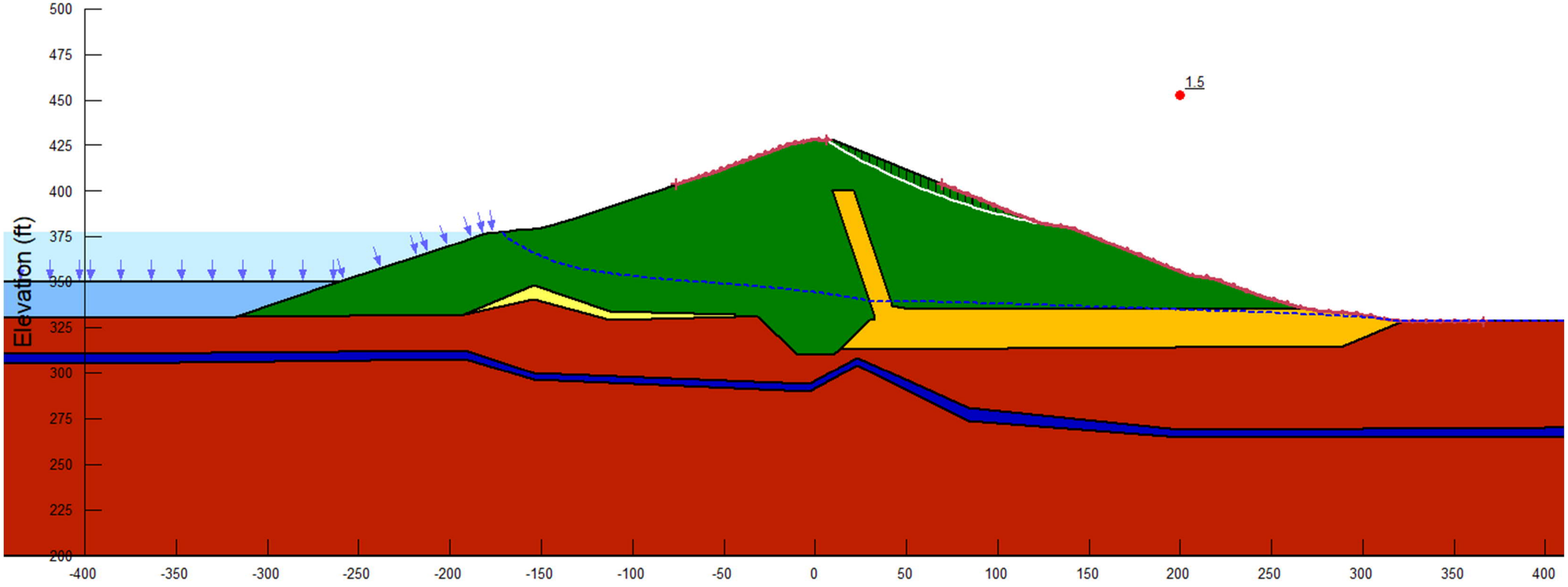
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Flood Surcharge Long-Term Stability Bilinear Strengths	Figure 8-19
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Bilinear Strength)	120	35	0	-	-	18	400
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

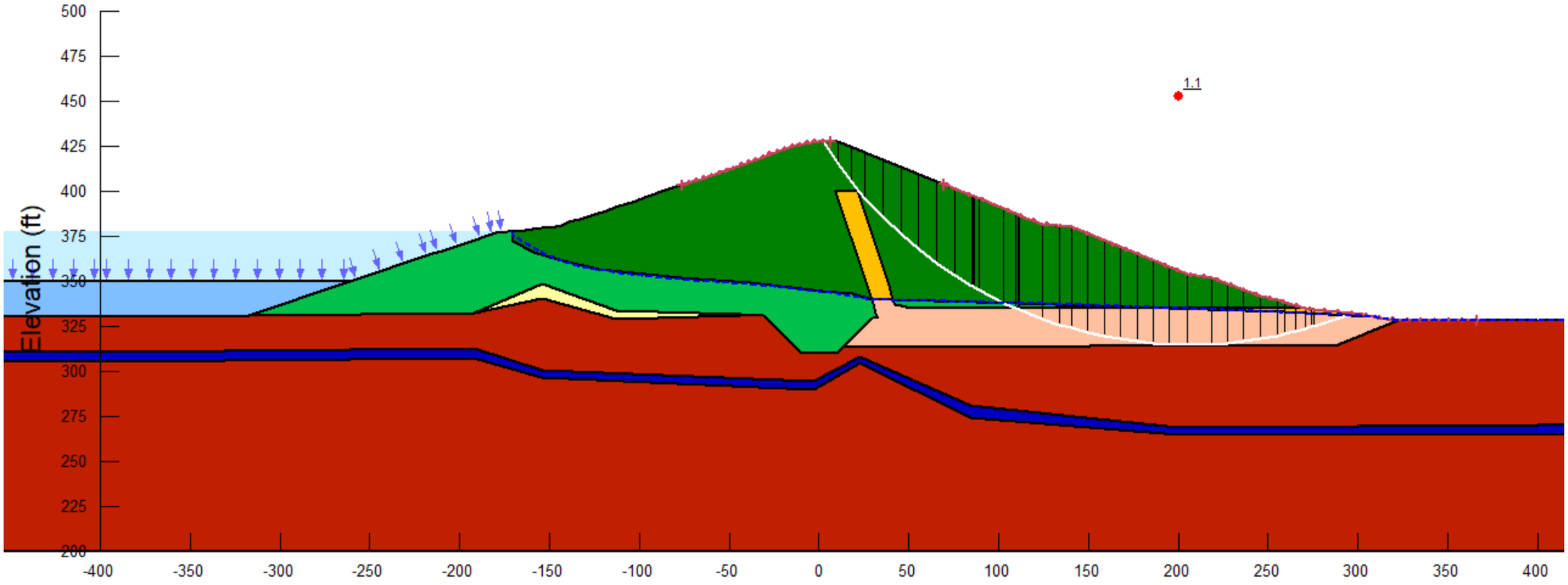
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Flood Surcharge Long-Term Stability Undrained Strengths	Figure 8-20
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

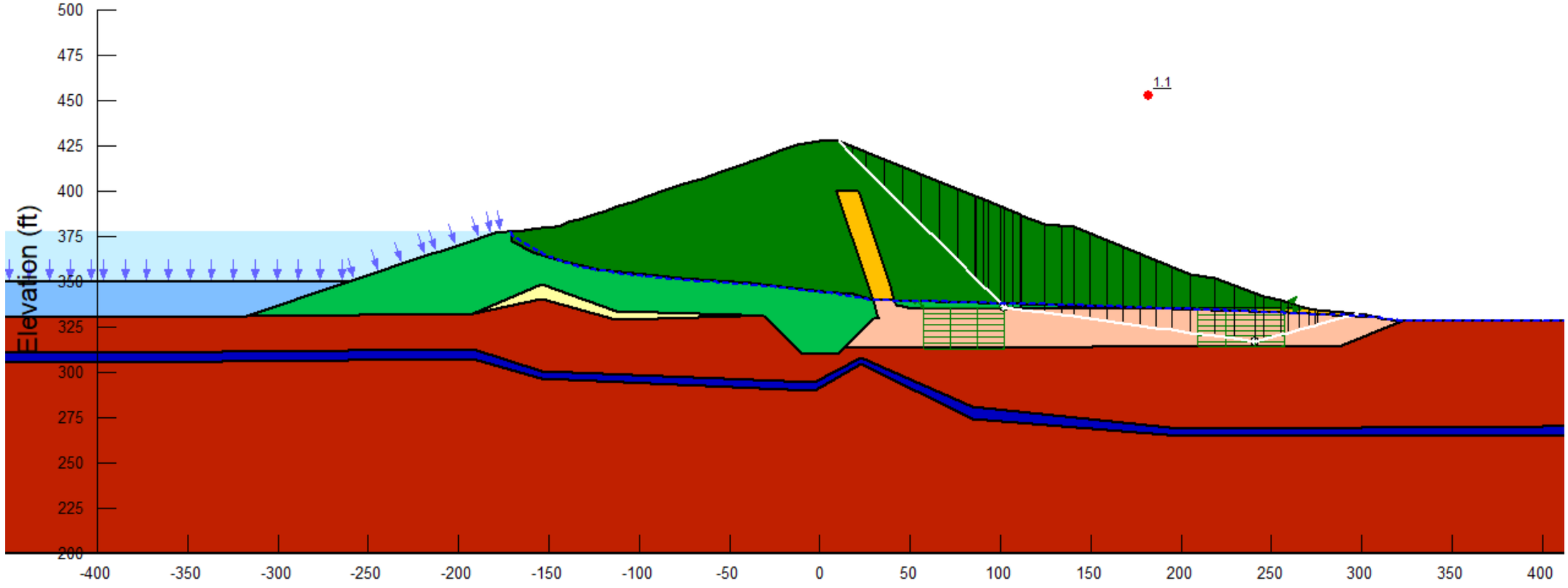
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Principal Spillway Long-Term Stability	Figure 8-21
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Residual Strength)	120	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Residual Strength)	115	$\sigma'_v/\tau = 0.15$ (Min=100 psf)				-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Filter/Drain (Residual Strength)	130	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

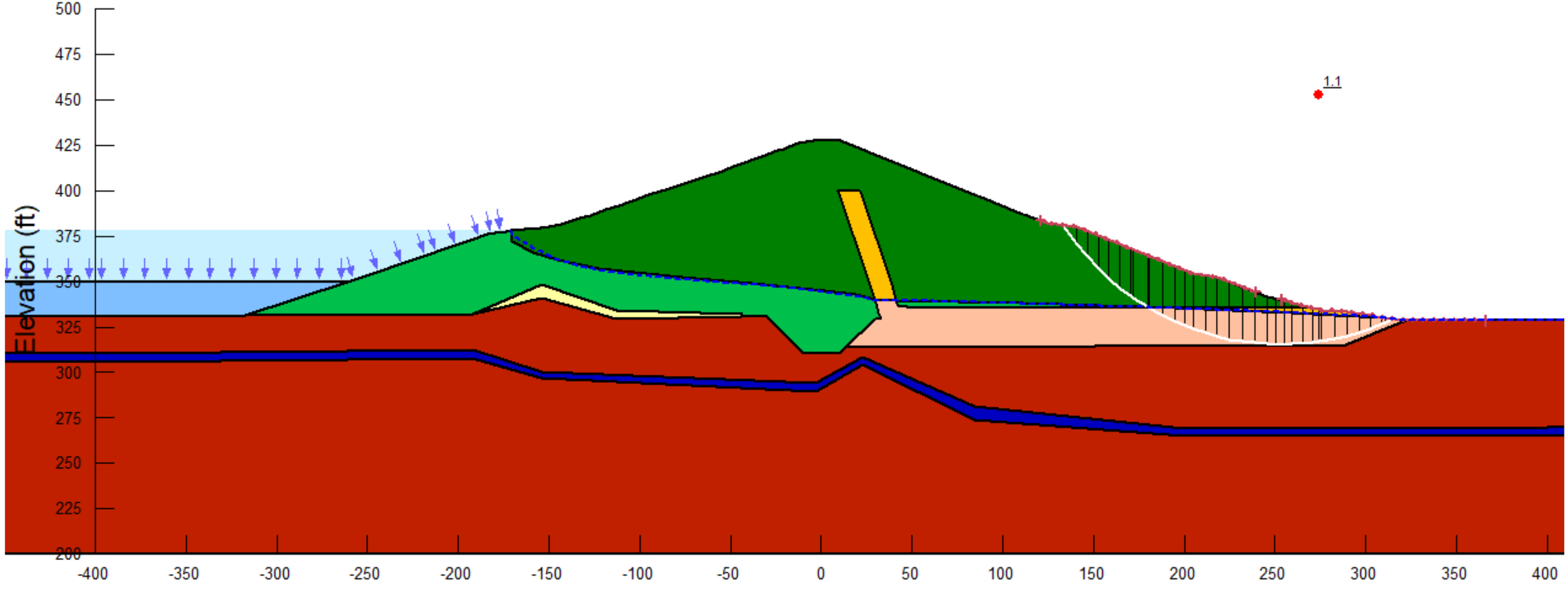
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Principal Spillway Post-Seismic Stability Downstream Slope	Figure 8-22
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Residual Strength)	120	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Residual Strength)	115	$\sigma'_v/\tau = 0.15$ (Min=100 psf)				-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Filter/Drain (Residual Strength)	130	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

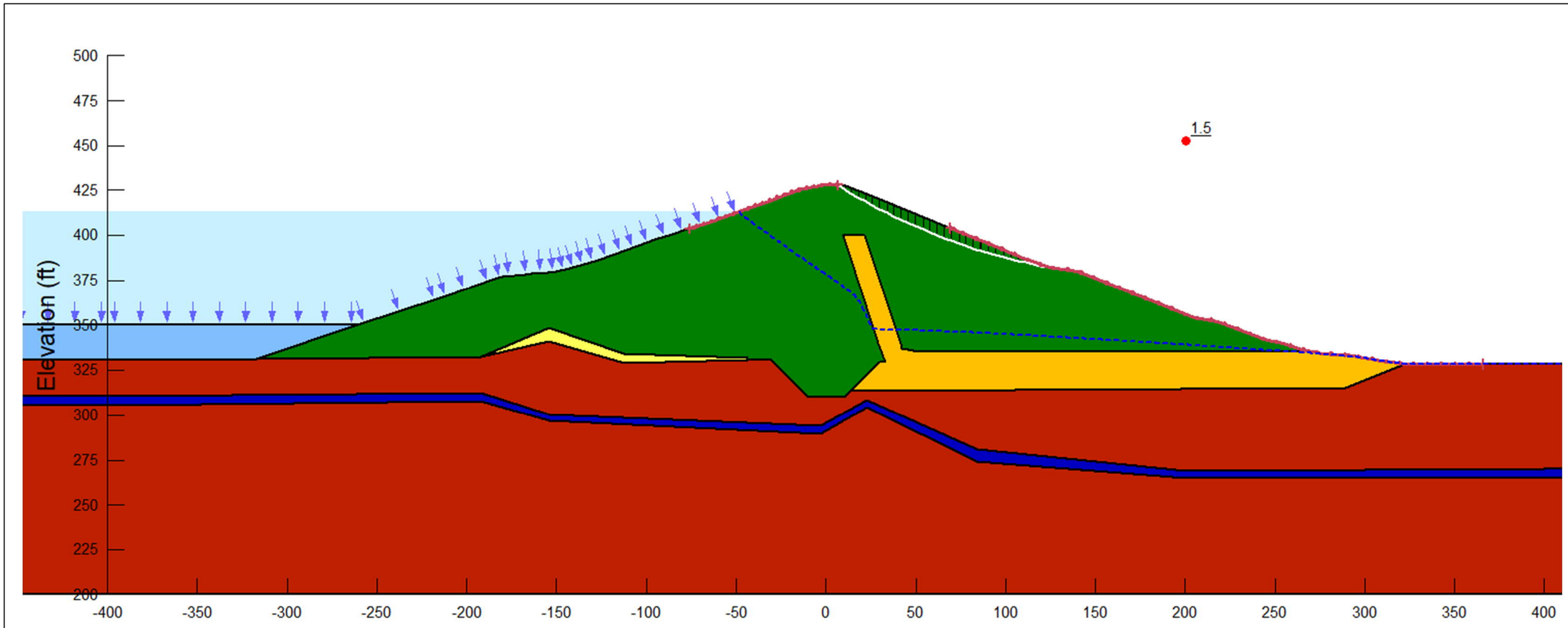
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Principal Spillway Post-Seismic Stability Downstream Slope (Block)	Figure 8-23
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Residual Strength)	120	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Residual Strength)	115	$\sigma'_v/\tau = 0.15$ (Min=100 psf)				-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Filter/Drain (Residual Strength)	130	$\sigma'_v/\tau = 0.25$ (Min=100 psf)				-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

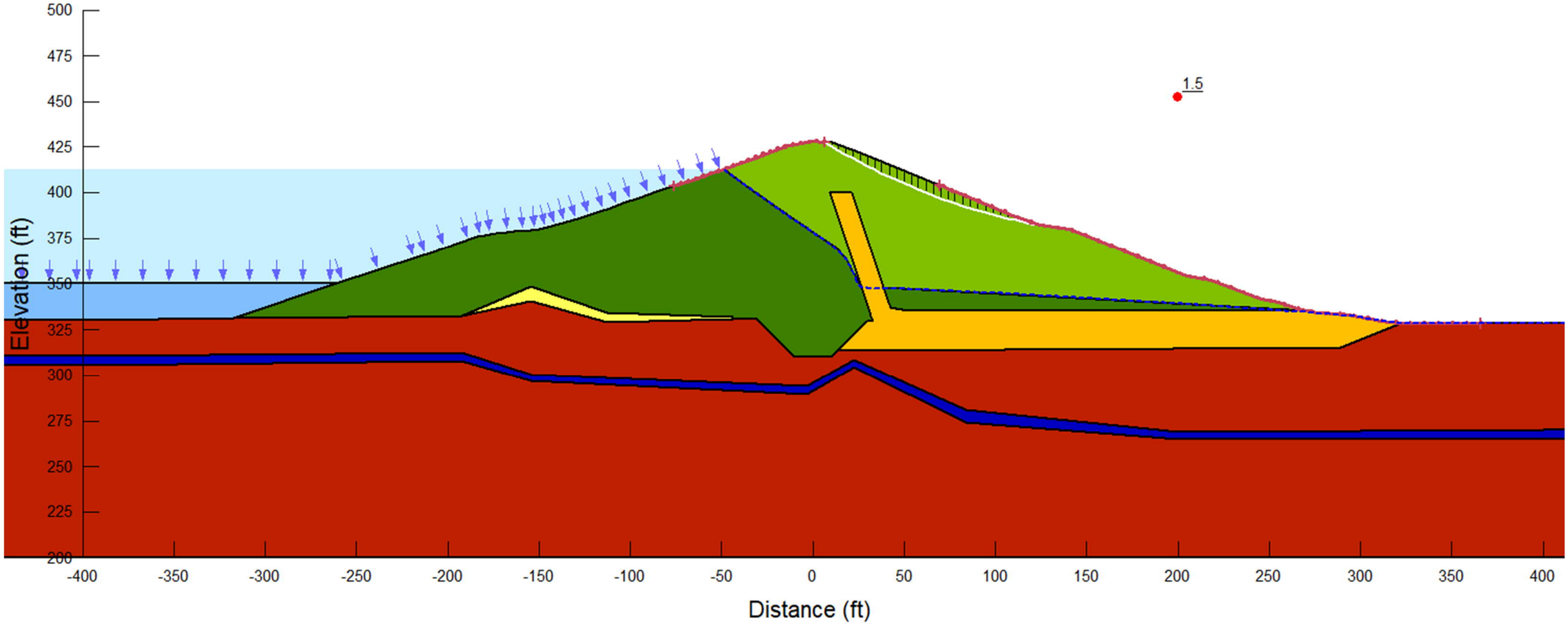
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Principal Spillway Post-Seismic Stability Downstream Slope (Lower)	Figure 8-24
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

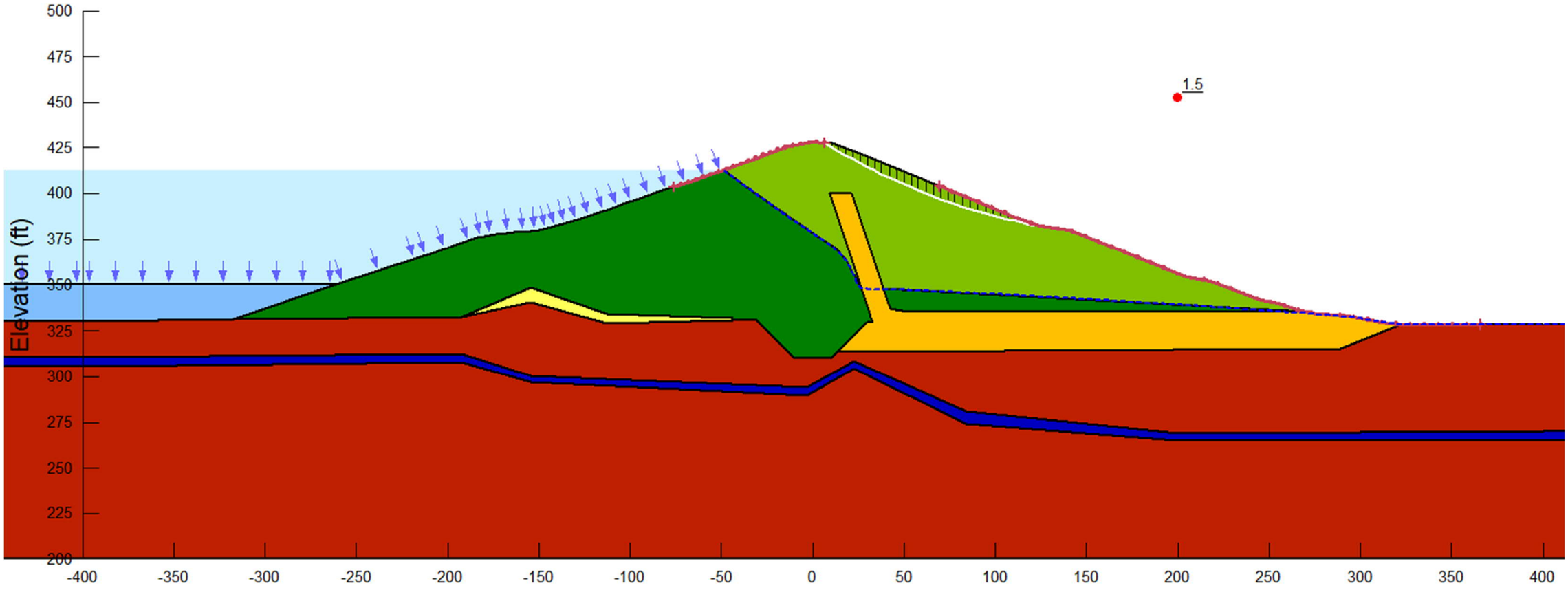
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Auxiliary Spillway Long-Term Stability Effective Strengths	Figure 8-25
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Bilinear Strength)	120	35	0	-	-	18	400
	Embankment Fill (Bilinear Strength)	115	34	20	-	-	18	650
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

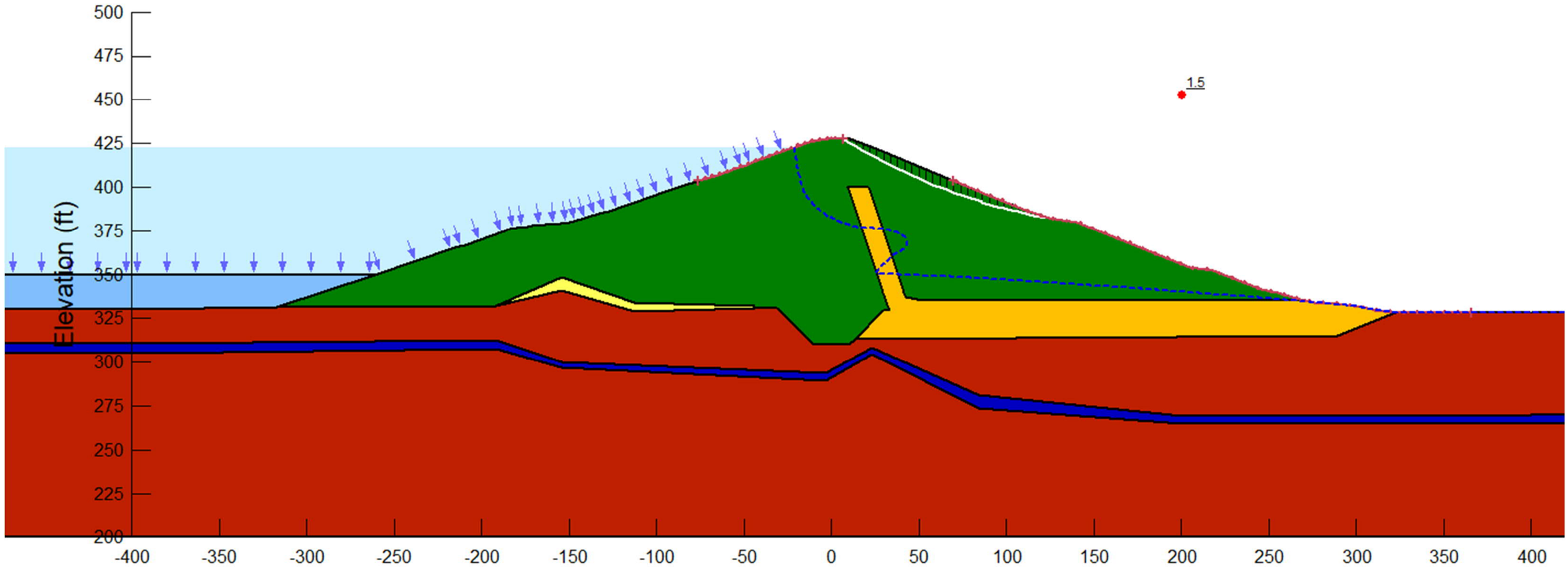
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Auxiliary Spillway Long-Term Stability Bilinear Strengths	Figure 8-26
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Bilinear Strength)	120	35	0	-	-	18	400
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

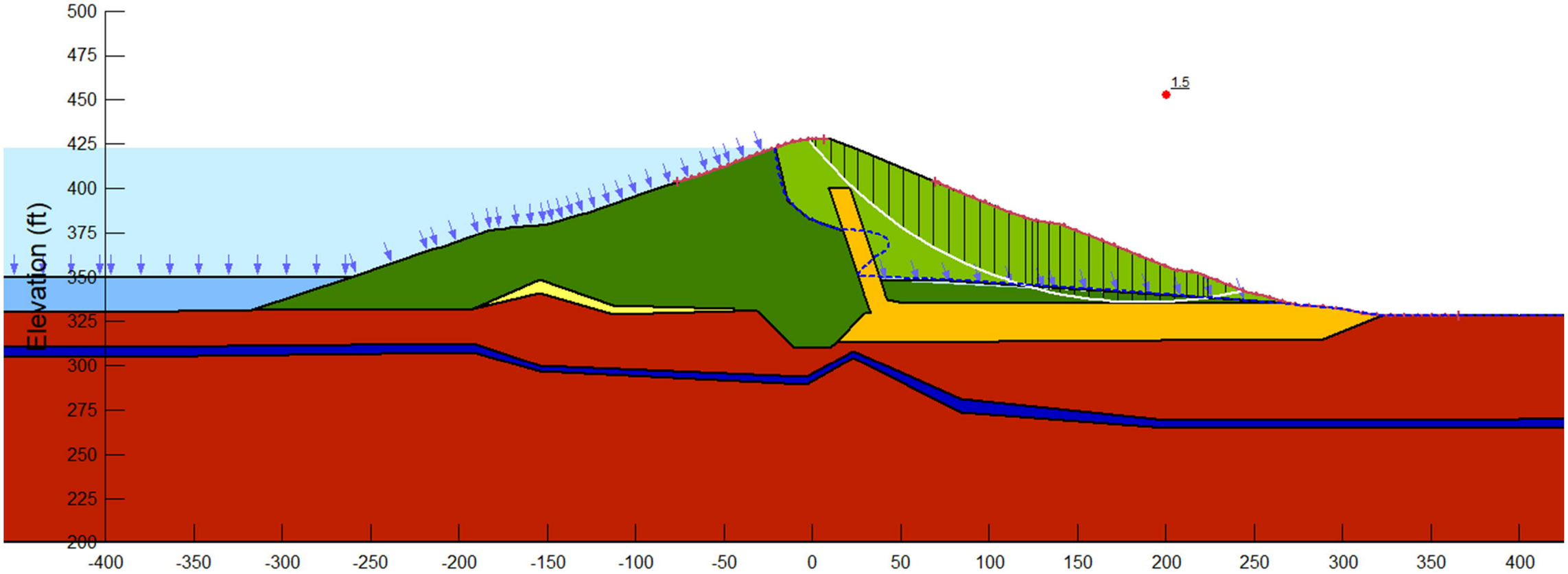
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Auxiliary Spillway Long-Term Stability Undrained Strengths	Figure 8-27
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

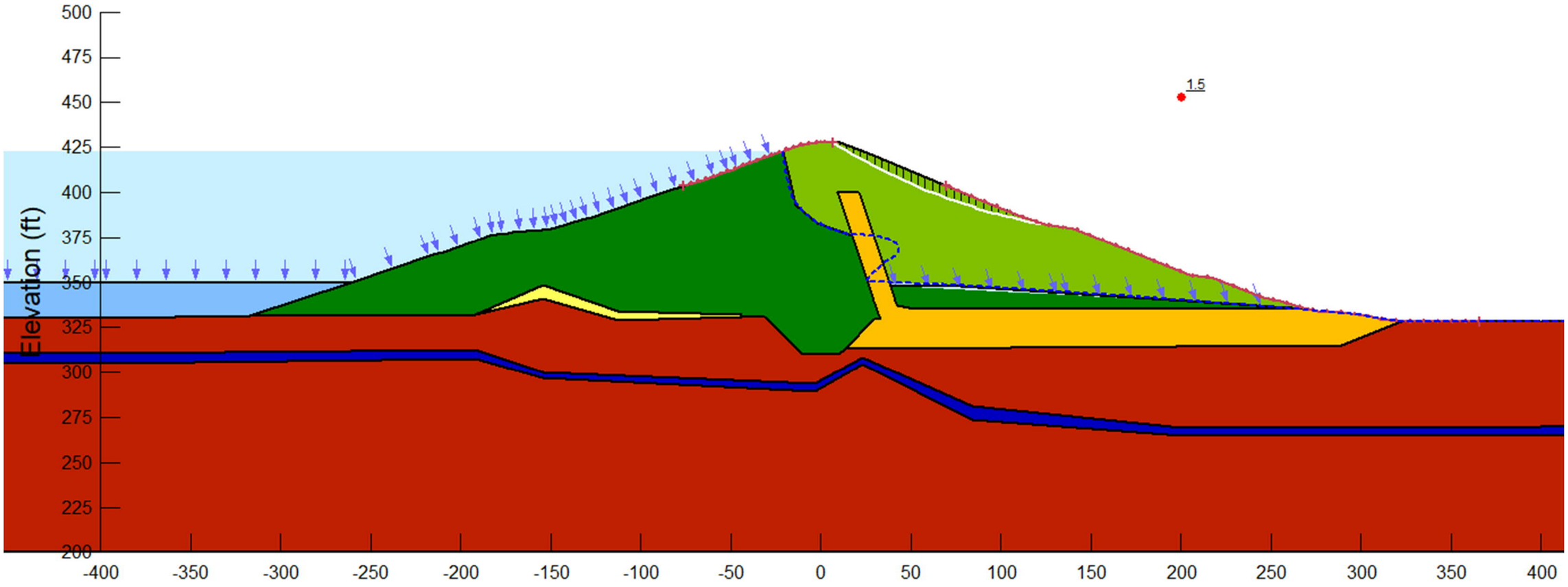
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Flood Surcharge Long-Term Stability Effective Strengths	Figure 8-28
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Bilinear Strength)	120	35	0	-	-	18	400
	Embankment Fill (Bilinear Strength)	115	34	20	-	-	18	650
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

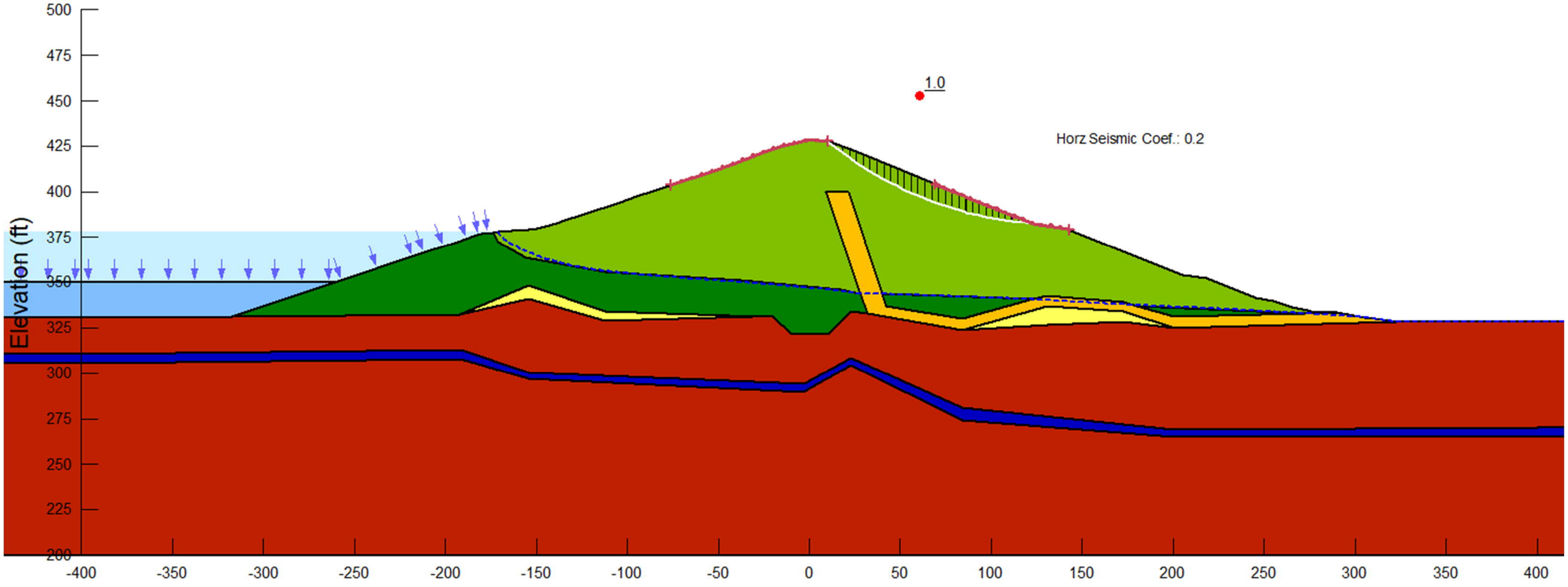
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Flood Surcharge Long-Term Stability Bilinear Strengths	Figure 8-29
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium (Bilinear Strength)	120	35	0	-	-	18	400
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

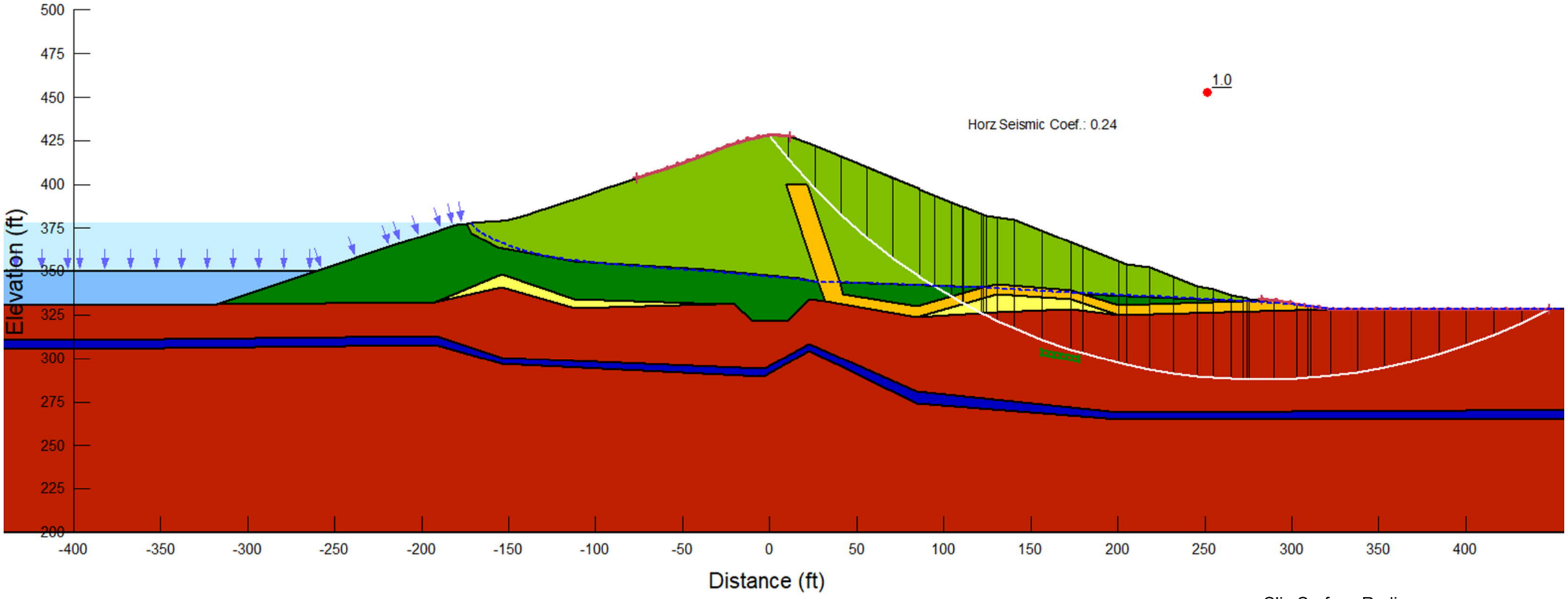
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Flood Surcharge Long-Term Stability Undrained Strengths	Figure 8-30
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Seismic Yield Coefficient Downstream Slip Surface 1	Figure 8-31
AECOM			

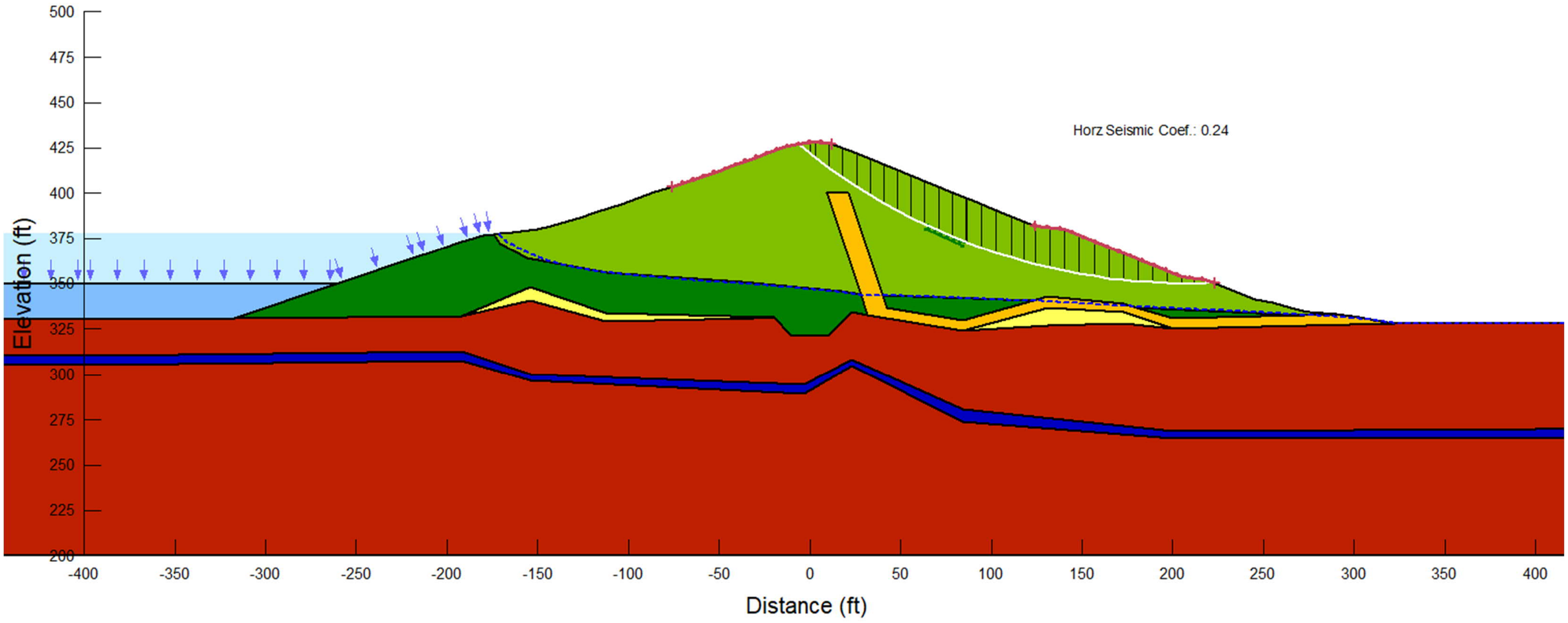


Slip Surface Radius
Search Zone

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

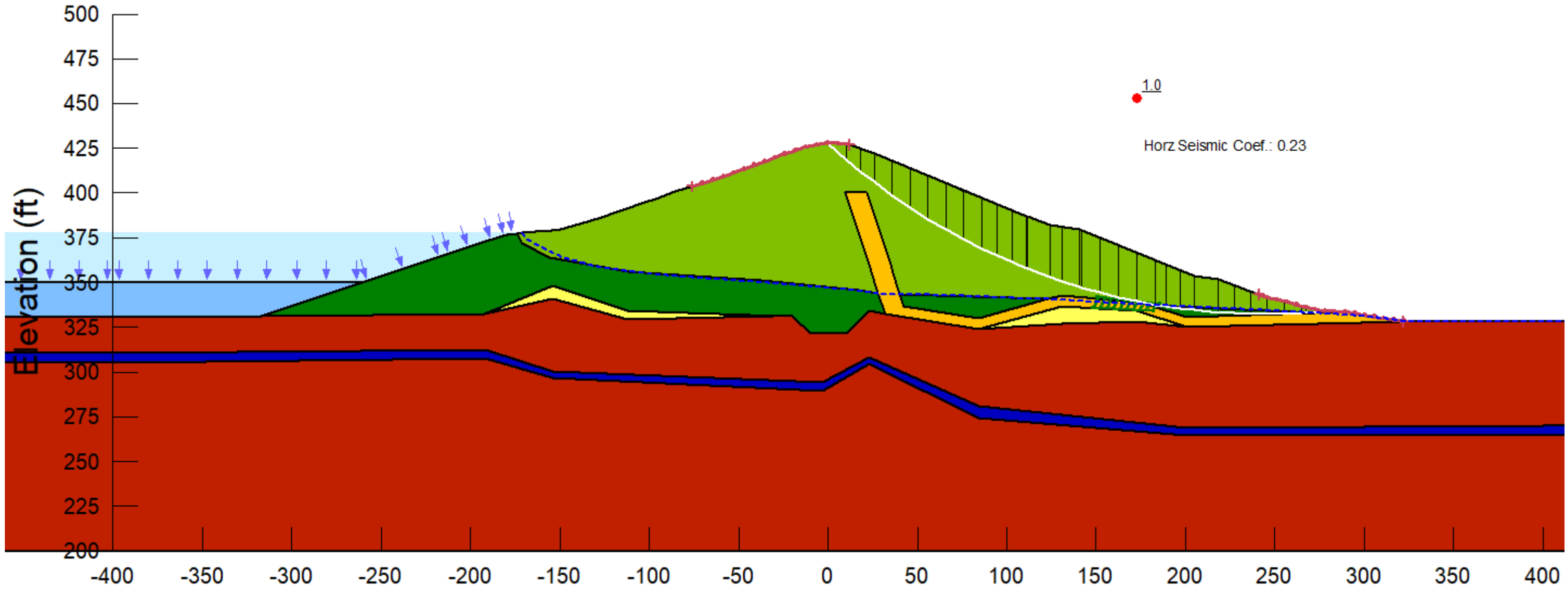
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Seismic Yield Coefficient Downstream Slip Surface 2	Figure 8-32
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

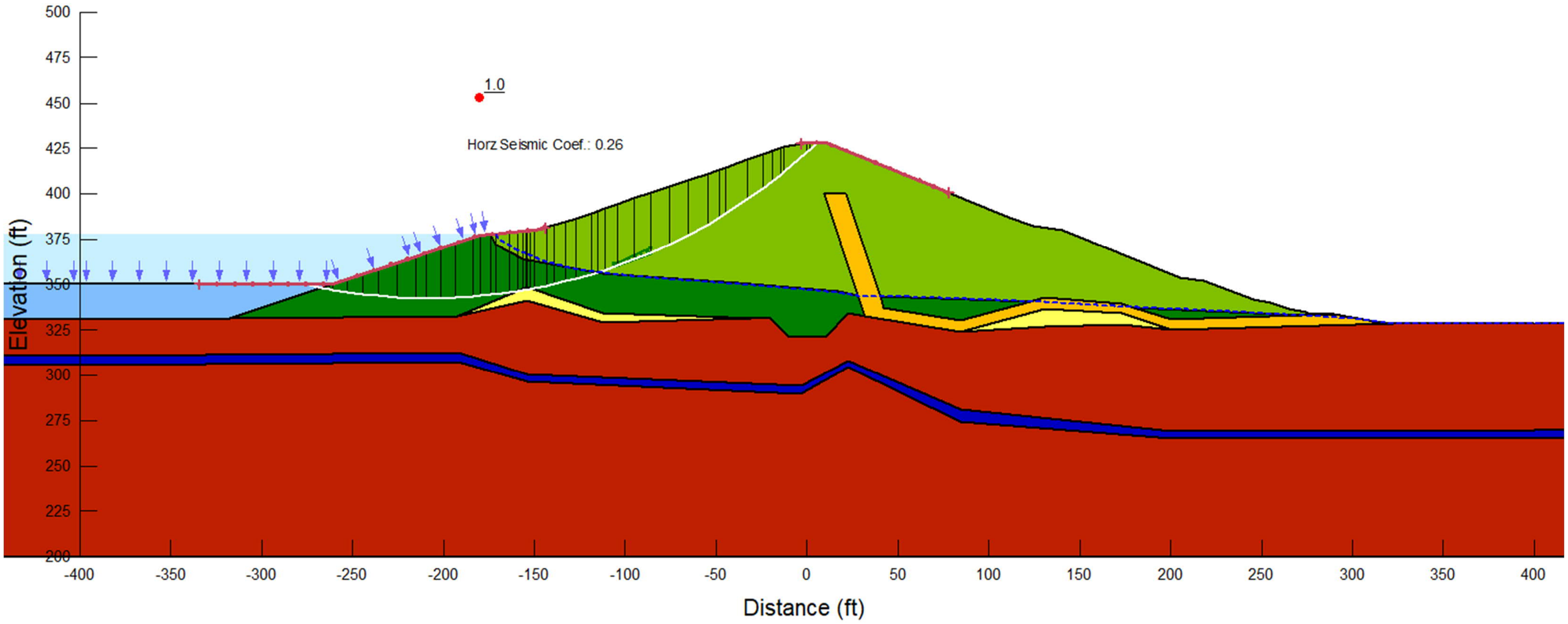
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Seismic Yield Coefficient Downstream Slip Surface 3	Figure 8-33
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

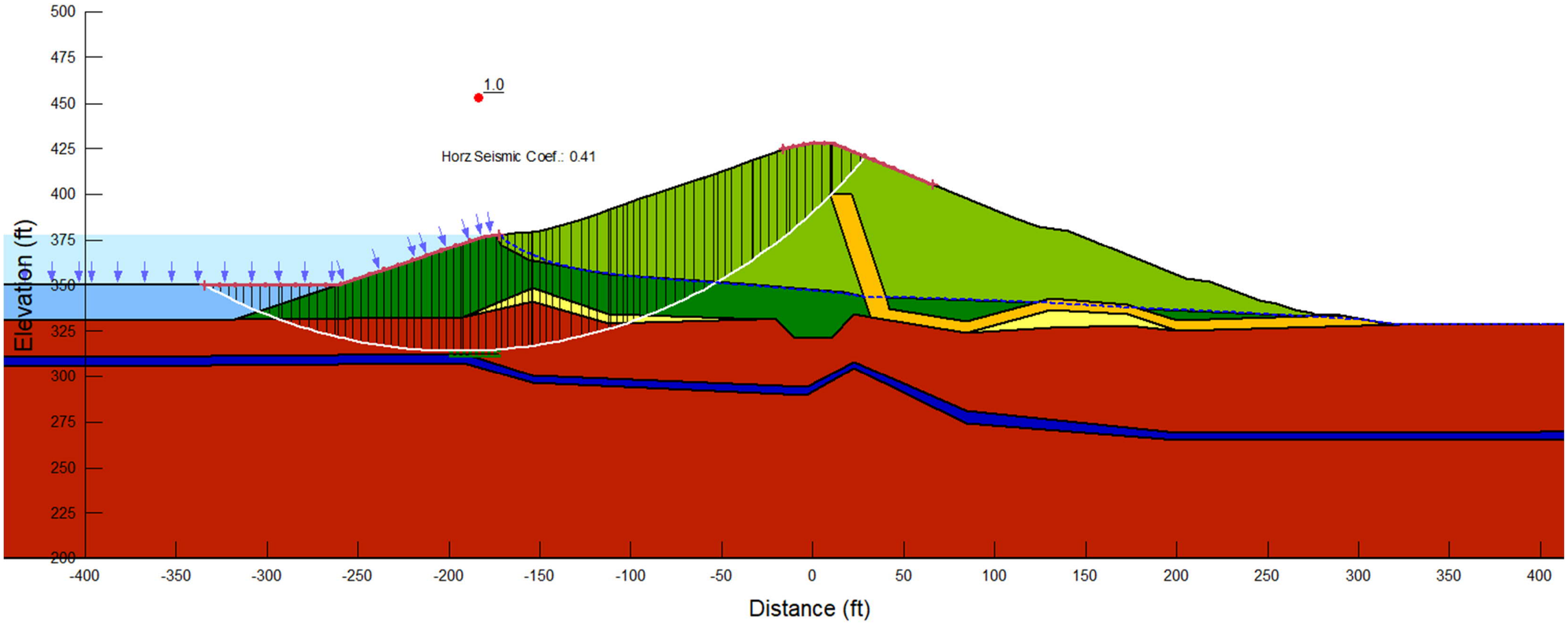
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Seismic Yield Coefficient Downstream Slip Surface 4	Figure 8-34
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

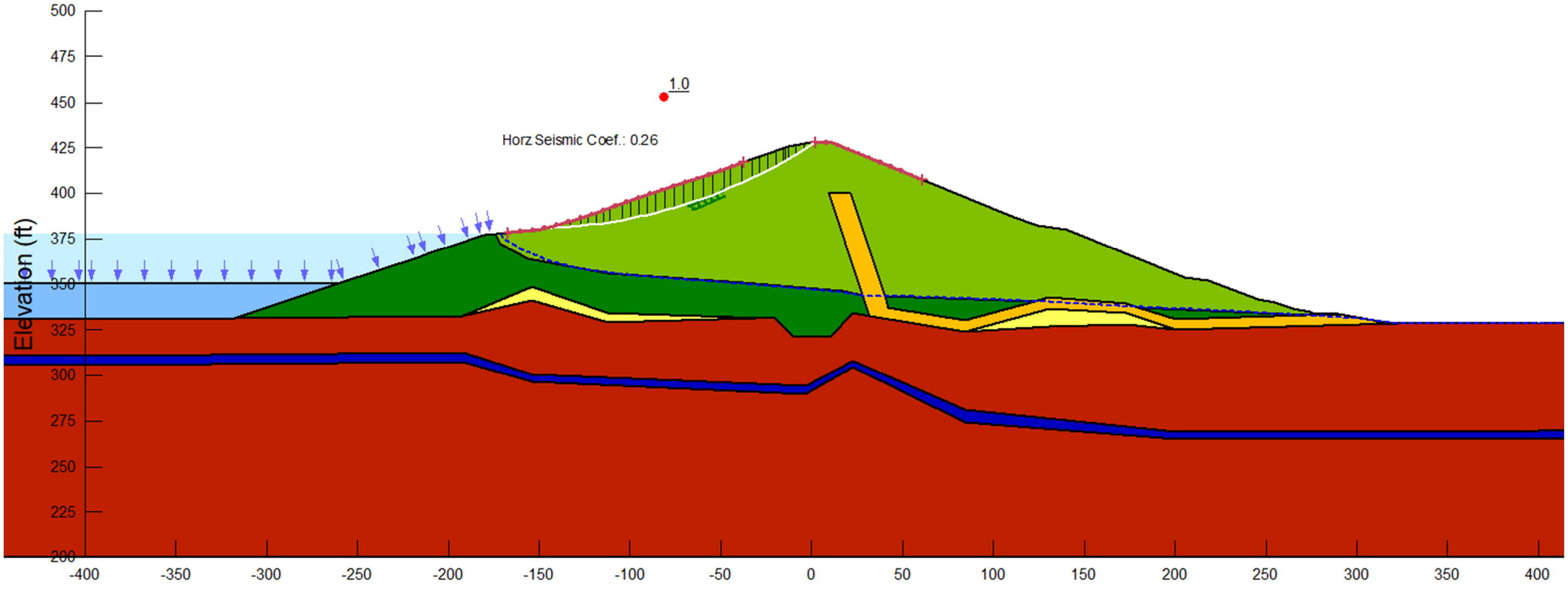
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Seismic Yield Coefficient Upstream Slip Surface 1	Figure 8-35
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

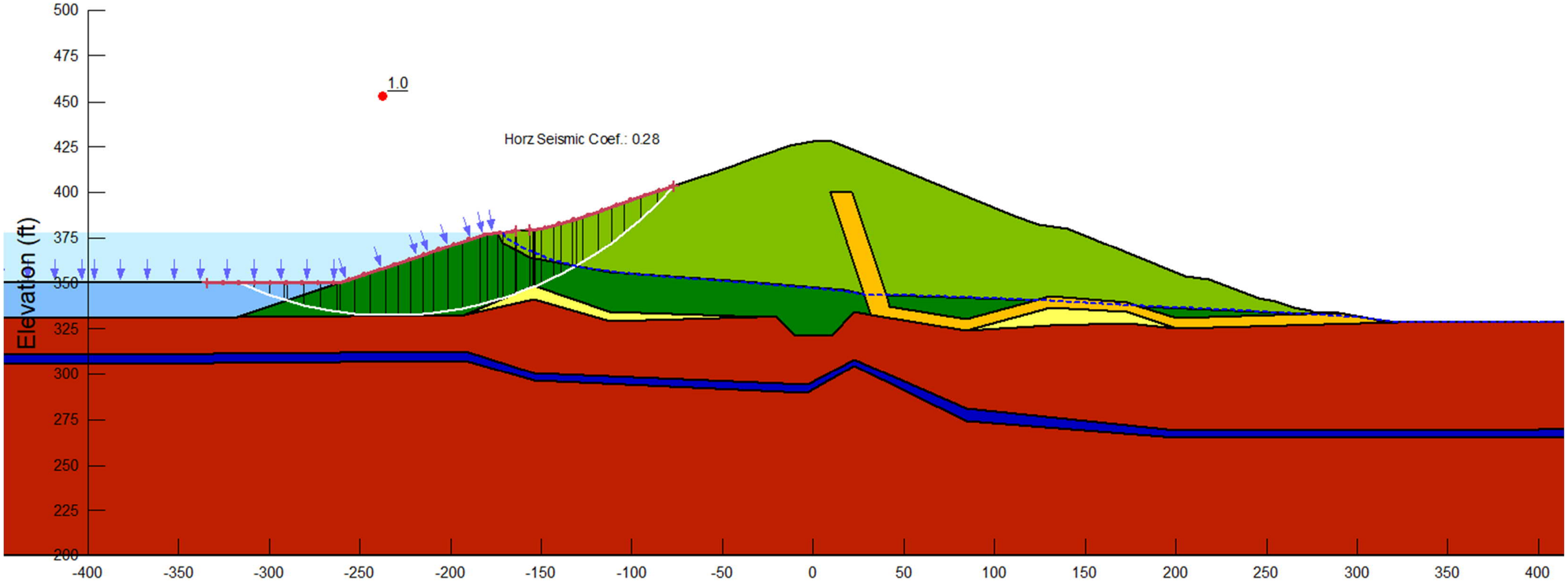
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Seismic Yield Coefficient Upstream Slip Surface 2	Figure 8-36
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Seismic Yield Coefficient Upstream Slip Surface 3	Figure 8-37
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

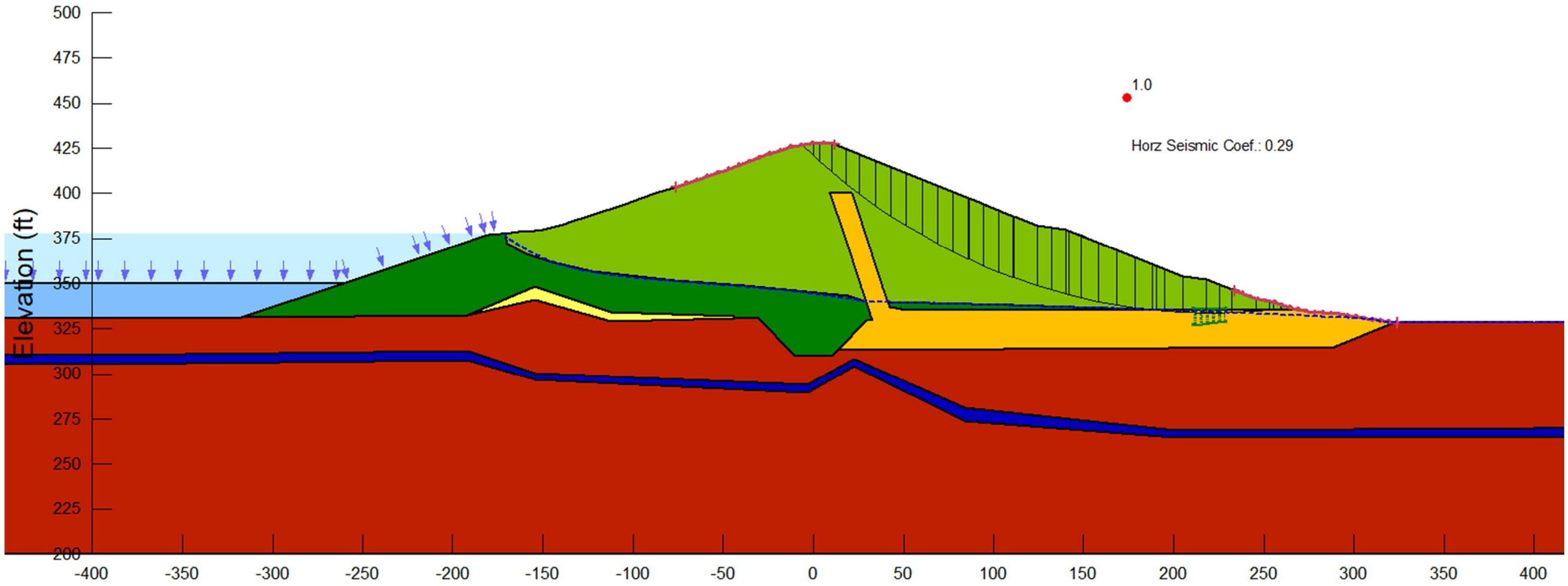
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Maximum Section Reservoir at Principal Spillway Seismic Yield Coefficient Upstream Slip Surface 4	Figure 8-38
	AECOM		



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

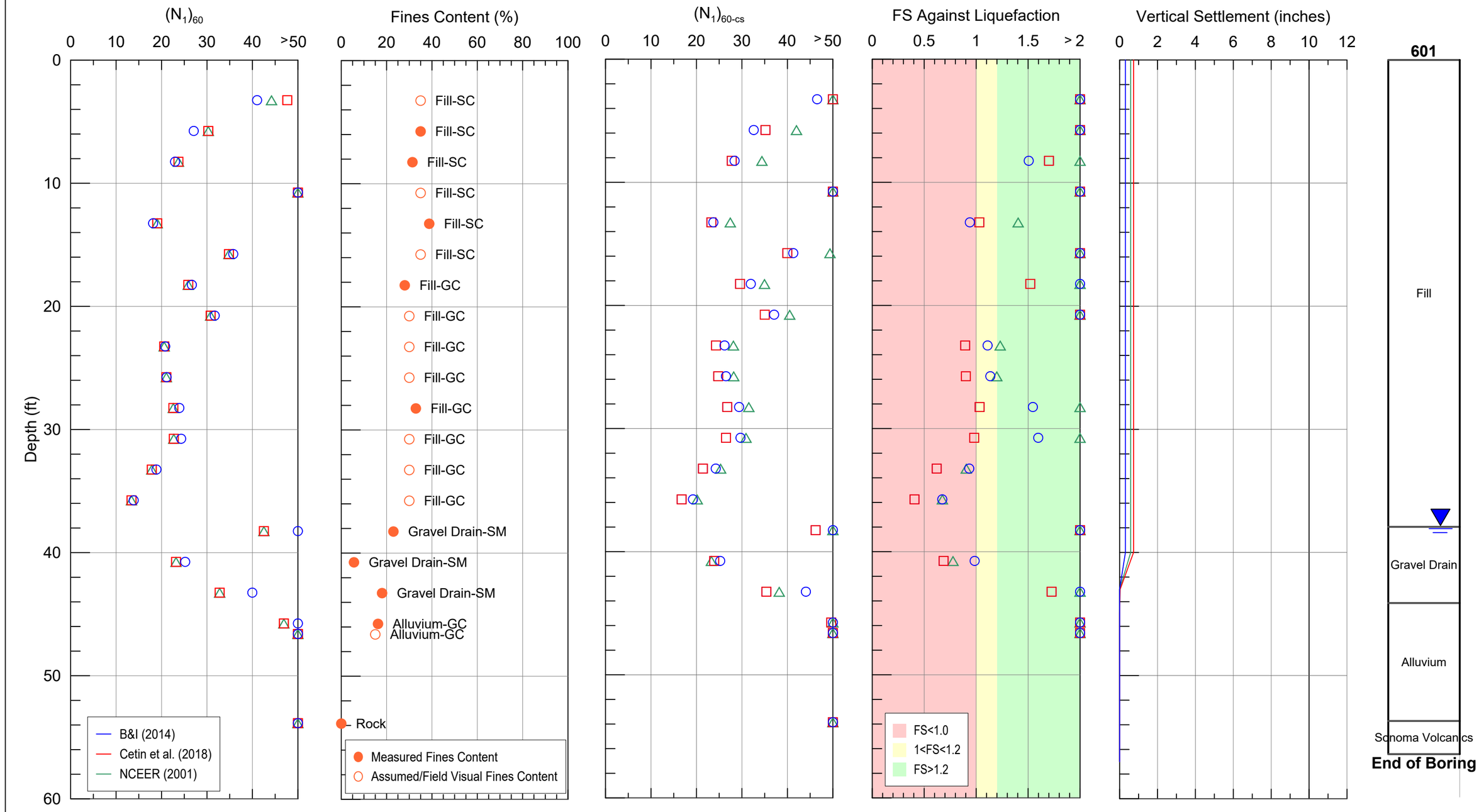
Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Principal Spillway Seismic Yield Coefficient Downstream Slip Surface 1	Figure 8-39
AECOM			



Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength		Bilinear Envelope	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)	φ (deg.)	Normal Stress (psf)
	Alluvium	120	35	0	15	200	-	-
	Embankment Fill	115	29	20	15	200	-	-
	Embankment Fill (Effective Strength Only)	115	29	20	Drained Only		-	-
	Filter/Drain	130	35	0	Drained Only		-	-
	Lake Sediment	90	25	0	13	0	-	-
	Organic Mudstone/Low Grade Coal	80	39	0	Drained Only		-	-
	Sonoma Volcanics	90	39	0	Drained Only		-	-

Notes
1. 20 psf cohesion applied to embankment fill to produce more relevant slip surfaces
2. Bilinear envelope used for flood surcharge and NRCS 1-stage rapid drawdown analyses

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Relief Trench Section Reservoir at Principal Spillway Seismic Yield Coefficient Downstream Slip Surface 2	Figure 8-40
AECOM			

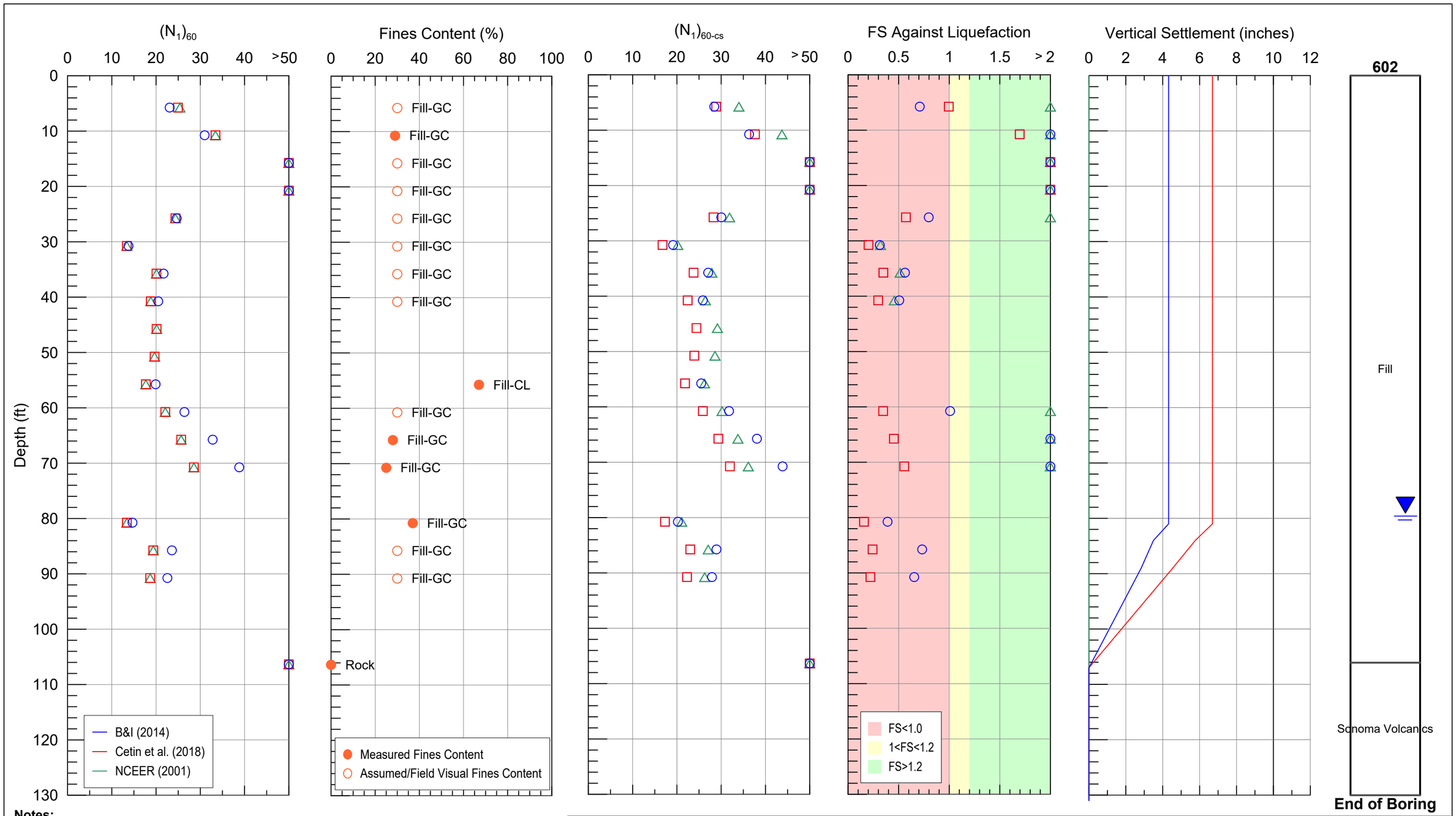


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 1.06g and earthquake magnitude = 7.2

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis of Boring 601 Rodger's Creek Fault 84th Percentile Motion	**Figure 8-41**
AECOM			

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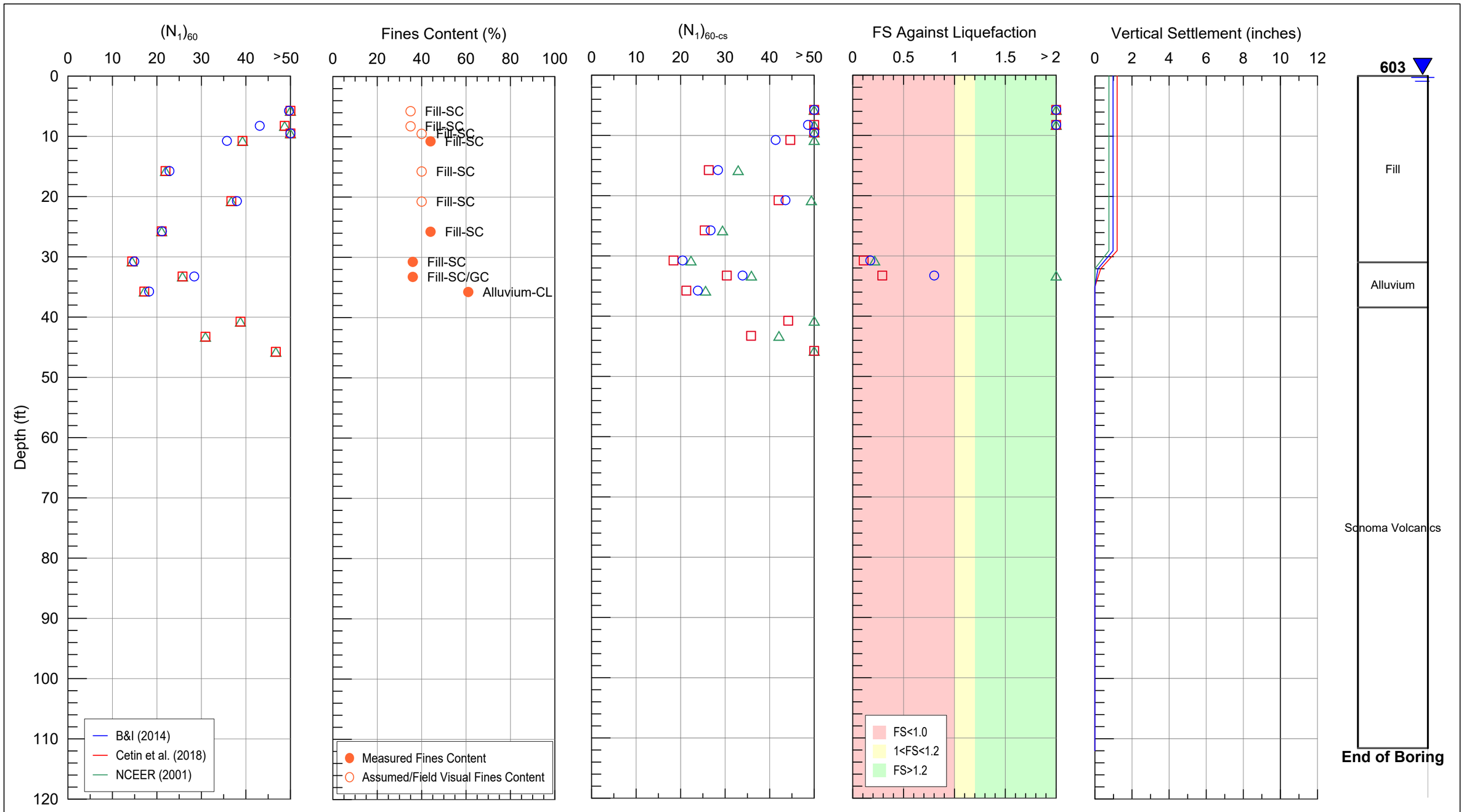


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 1.06g and earthquake magnitude = 7.2

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis of Boring 602 Rodger's Creek Fault 84th Percentile Motion	**Figure 8-42**
AECOM			

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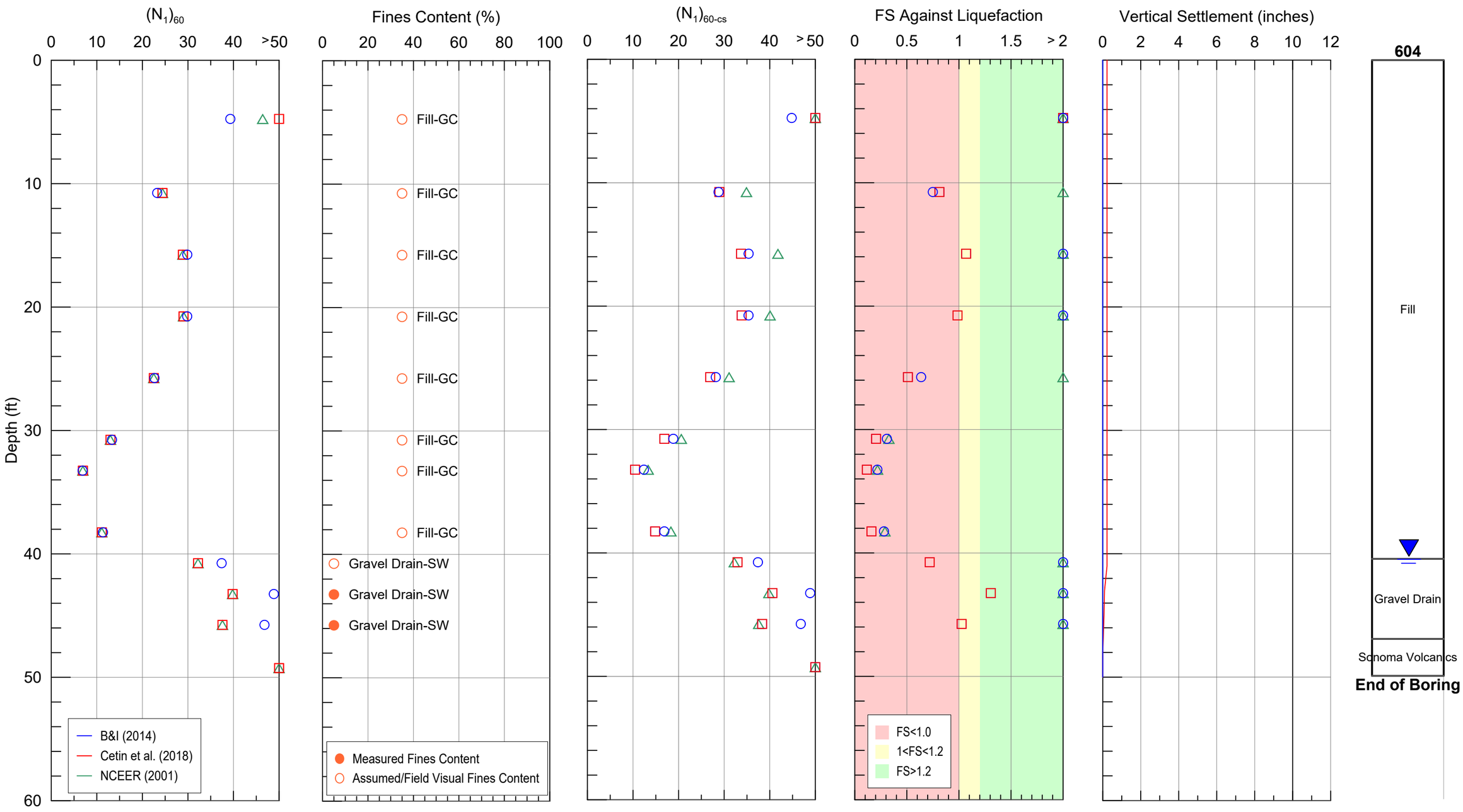


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 1.06g and earthquake magnitude = 7.2

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis of Boring 603 Rodger's Creek Fault 84th Percentile Motion	**Figure 8-43**
AECOM			

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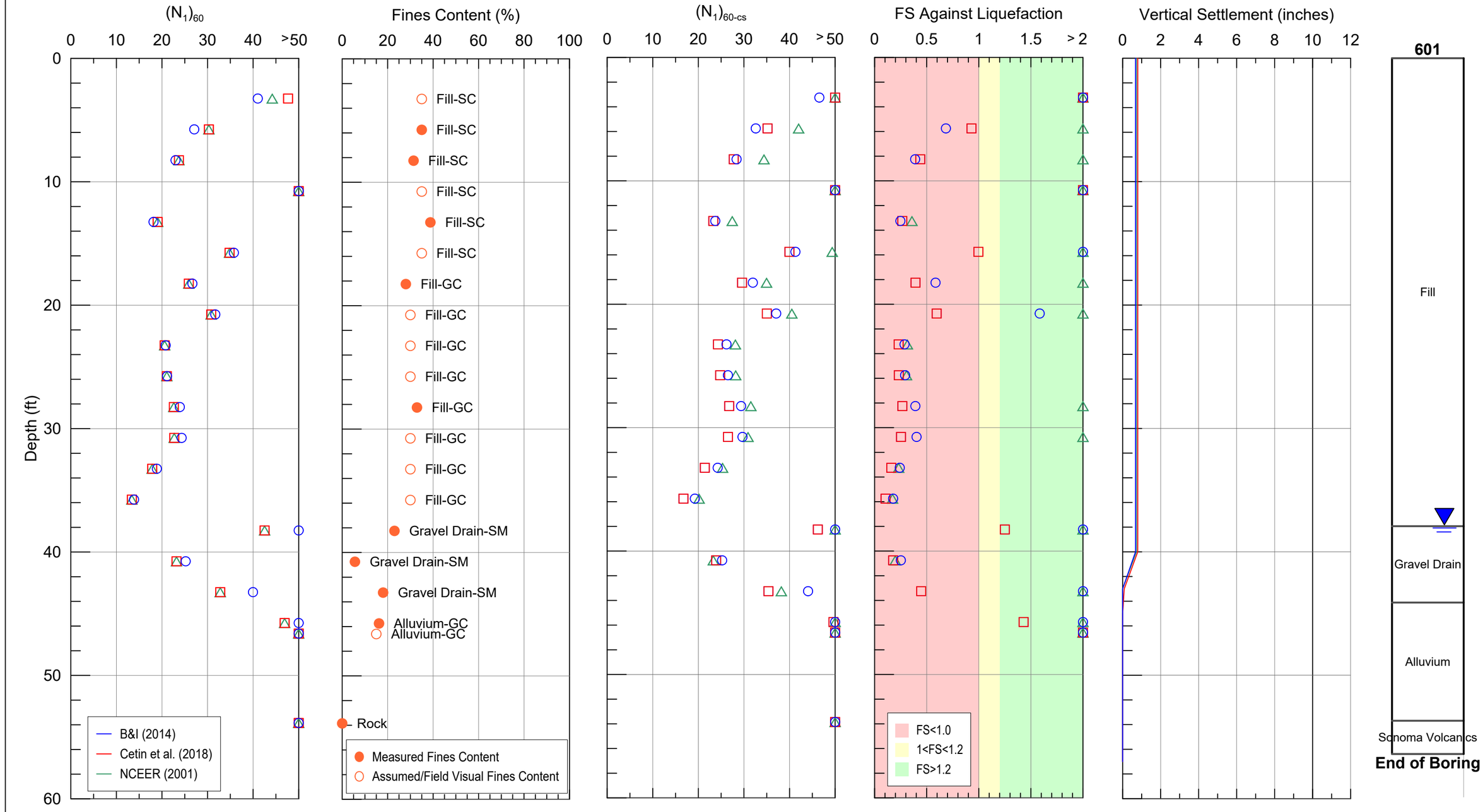


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 1.06g and earthquake magnitude = 7.2

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis of Boring 604 Rodger's Creek Fault 84th Percentile Motion	**Figure 8-44**
AECOM			

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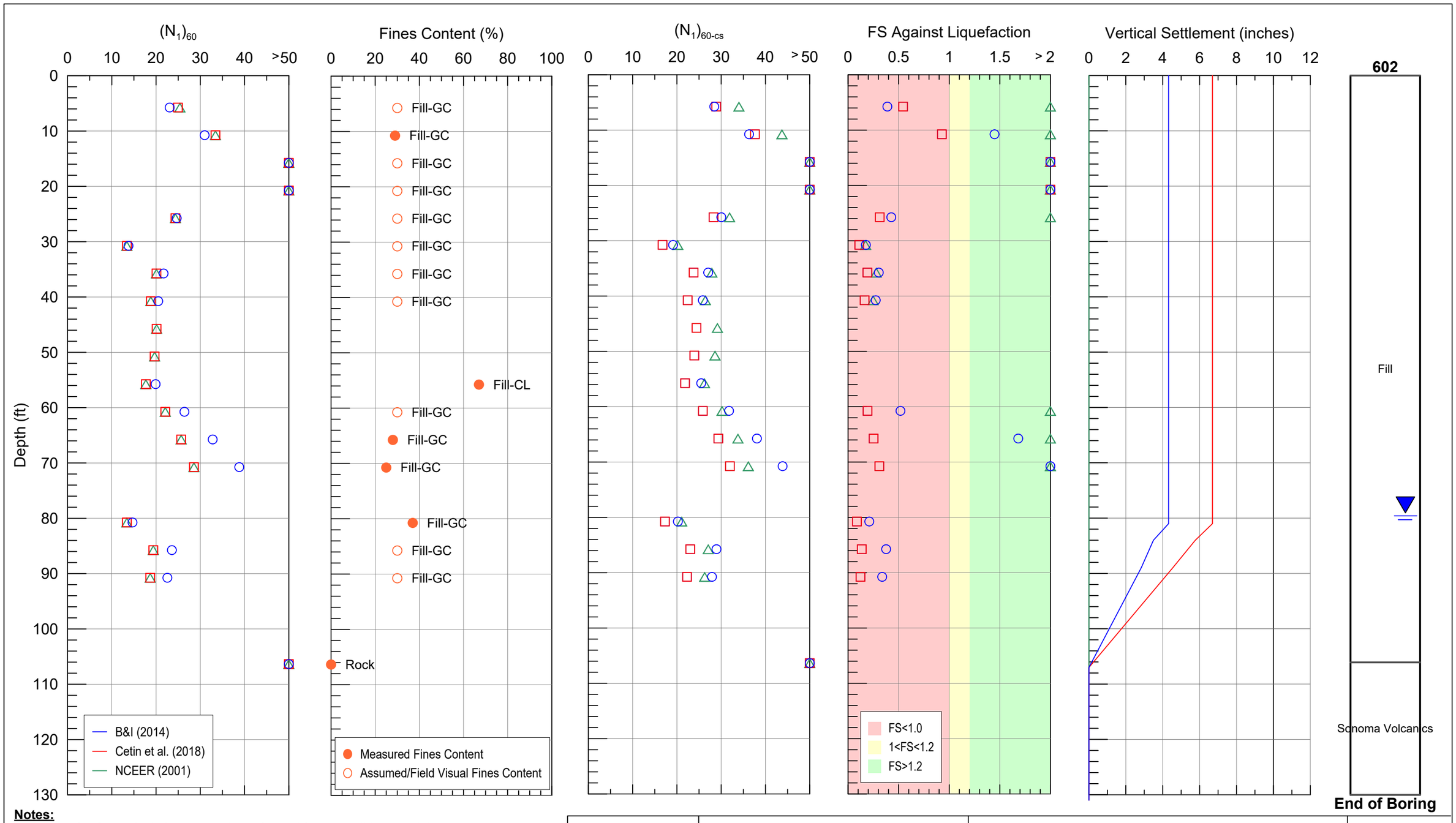


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 1.764g and earthquake magnitude = 7.5

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis Boring 601 10,000-year Motion	**Figure 8-45**
AECOM			

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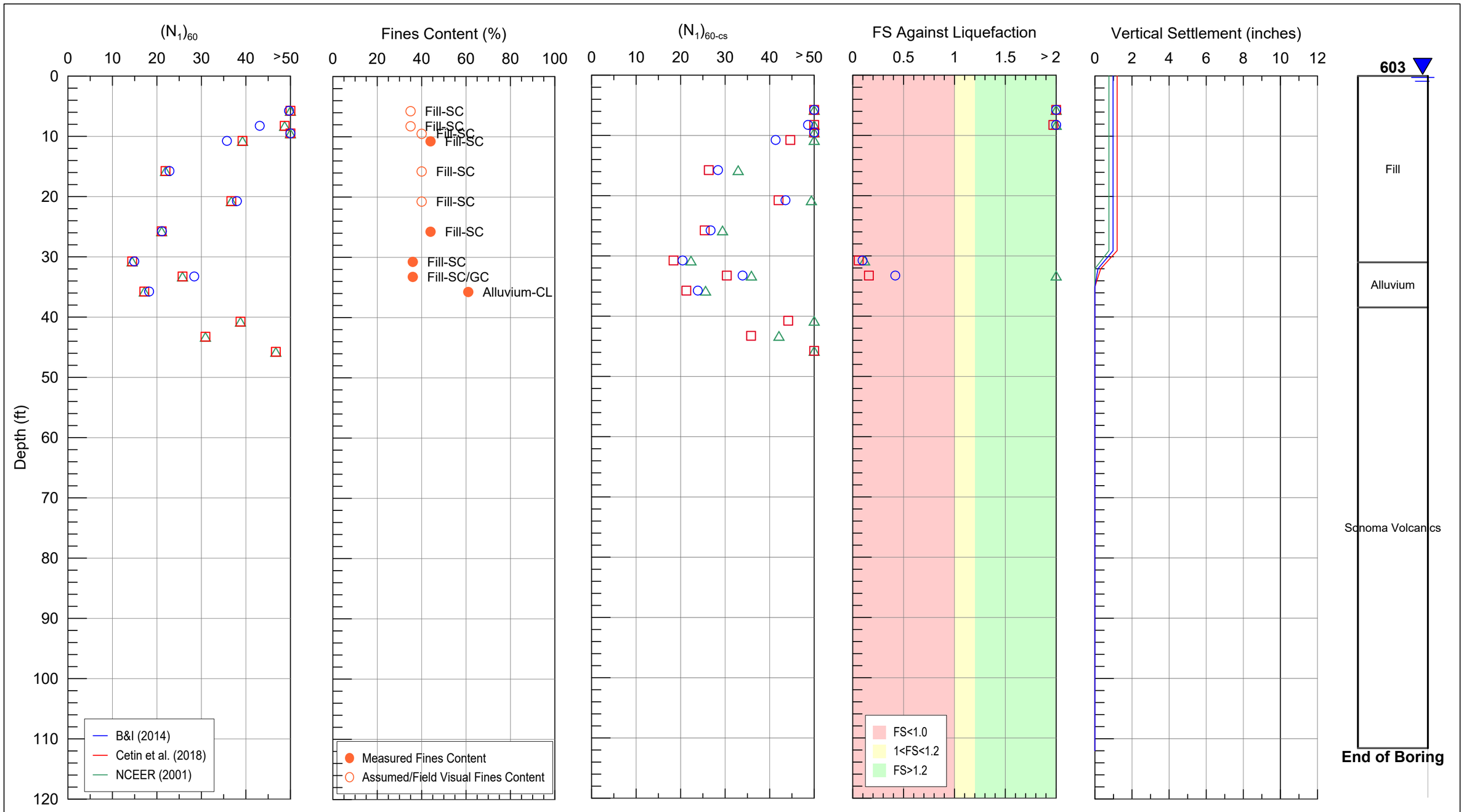


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 1.764g and earthquake magnitude = 7.5

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis Boring 602 10,000-year Motion	**Figure 8-46**
AECOM			

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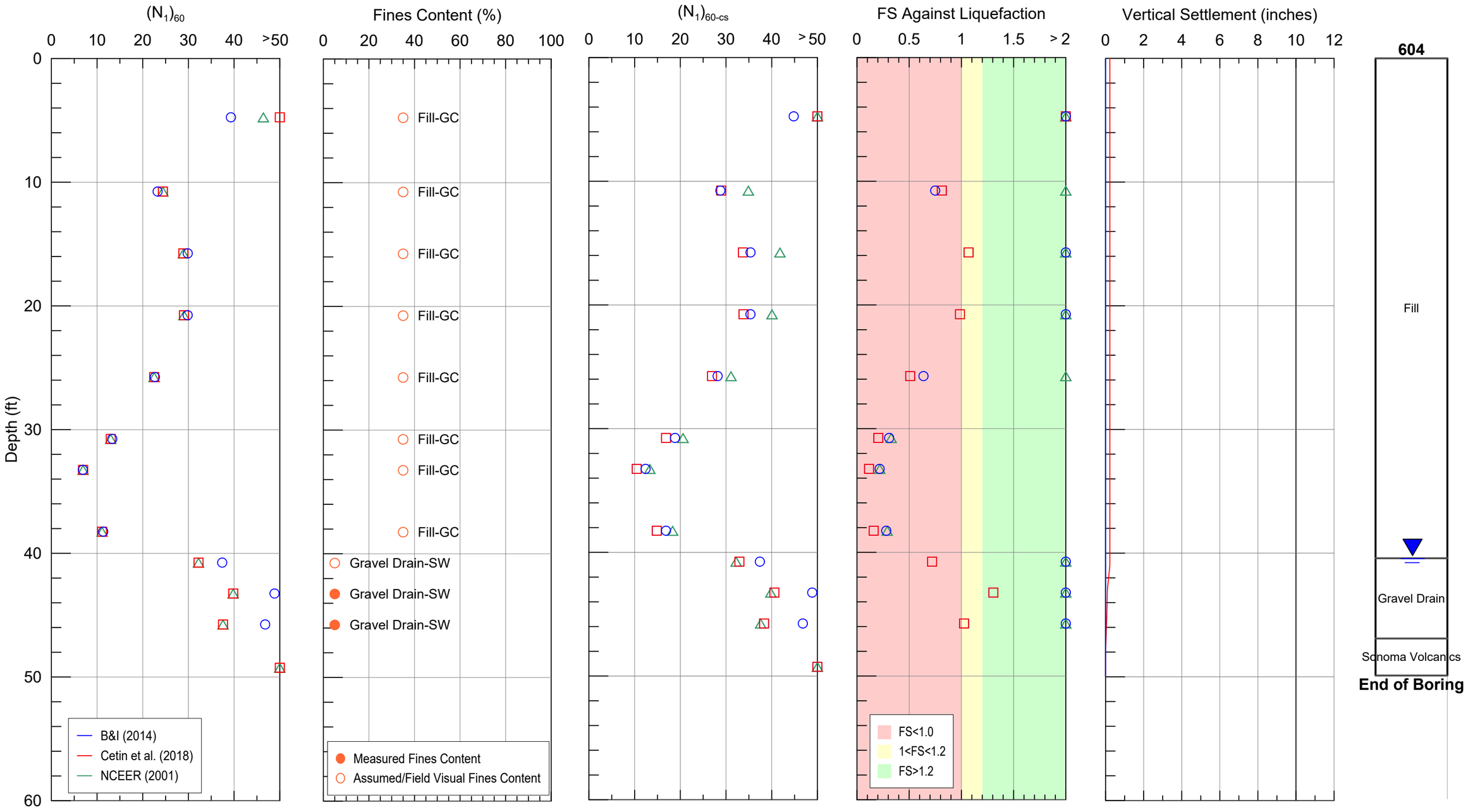


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 1.764g and earthquake magnitude = 7.5

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis Boring 603 10,000-year Motion	**Figure 8-47**
AECOM			

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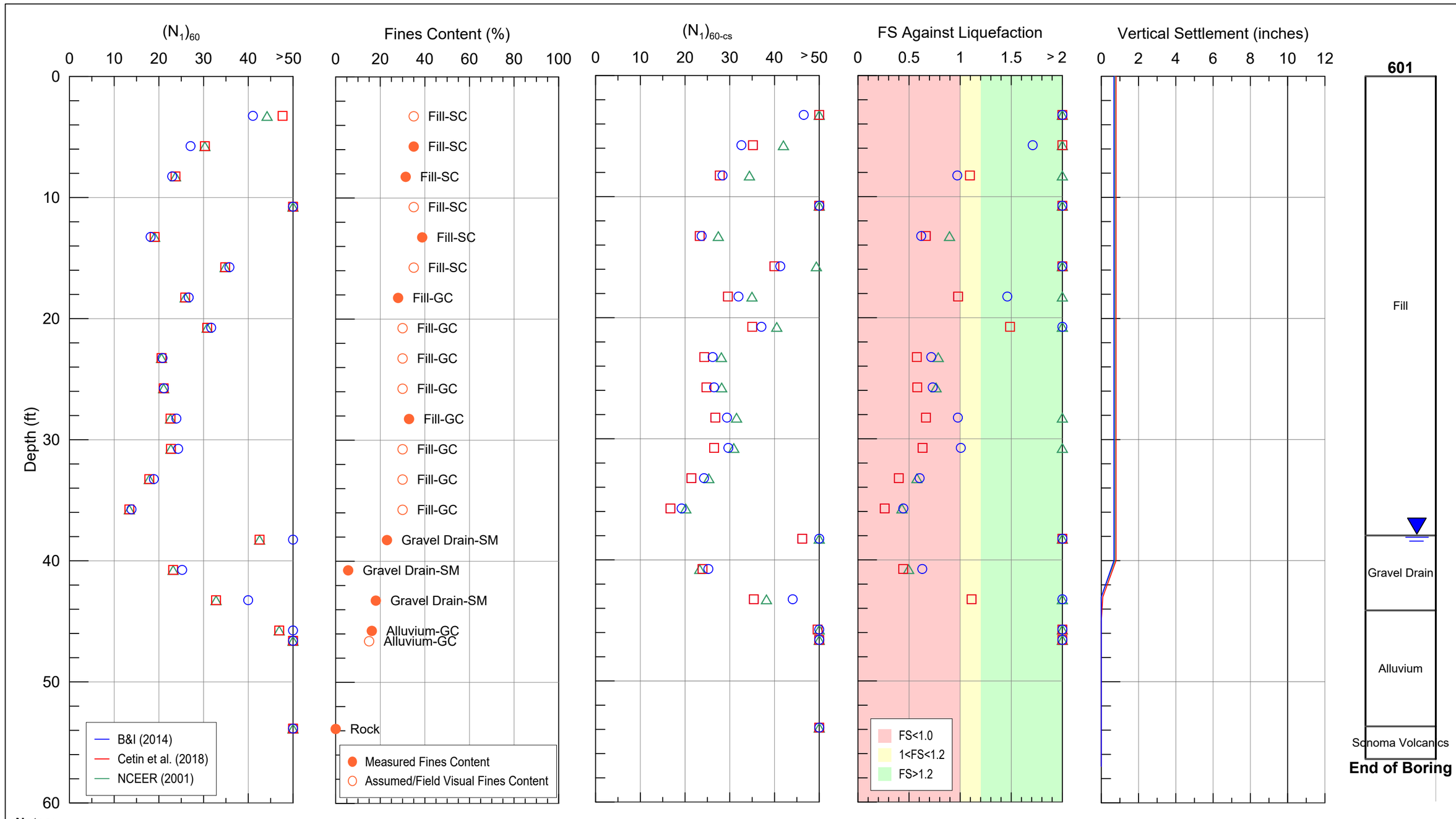


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 1.06g and earthquake magnitude = 7.2

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis Boring 604 10,000-year Motion	**Figure 8-48**
AECOM			

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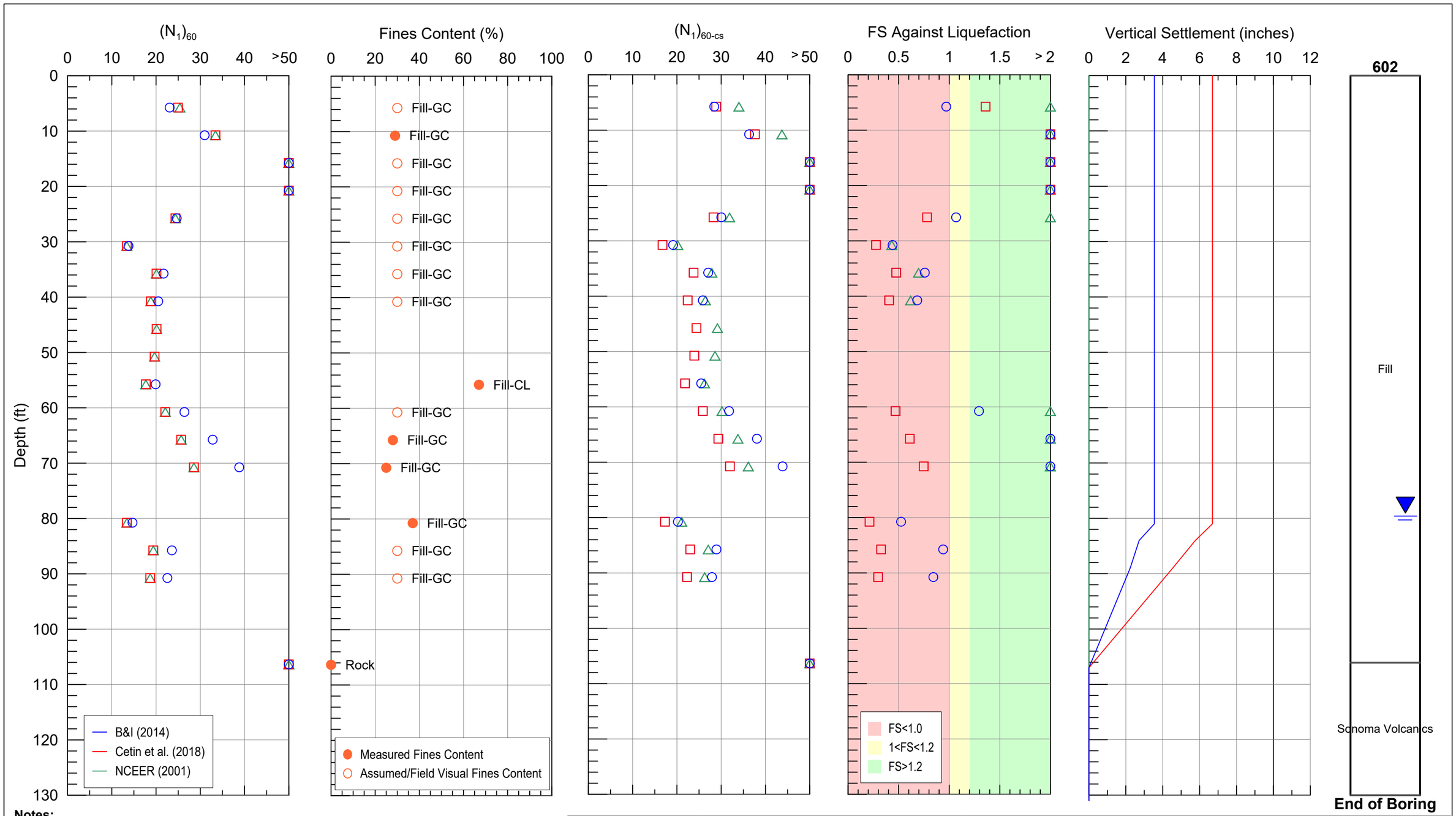


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 0.706g and earthquake magnitude = 7.5

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis Boring 601 500-year Motion	**Figure 8-49**
AECOM			

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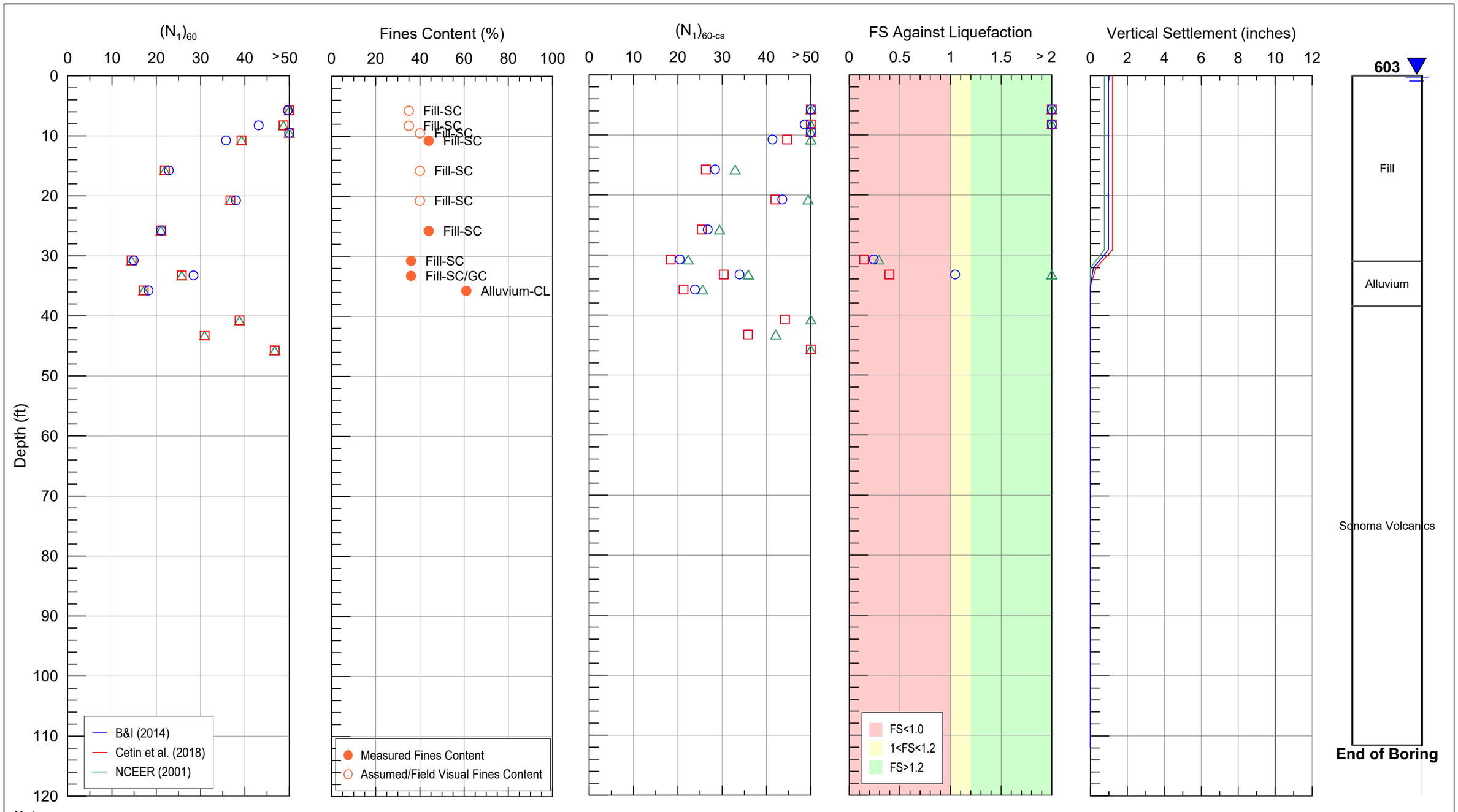


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 0.706g and earthquake magnitude = 7.5

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis Boring 602 500-year Motion	**Figure 8-50**
AECOM			

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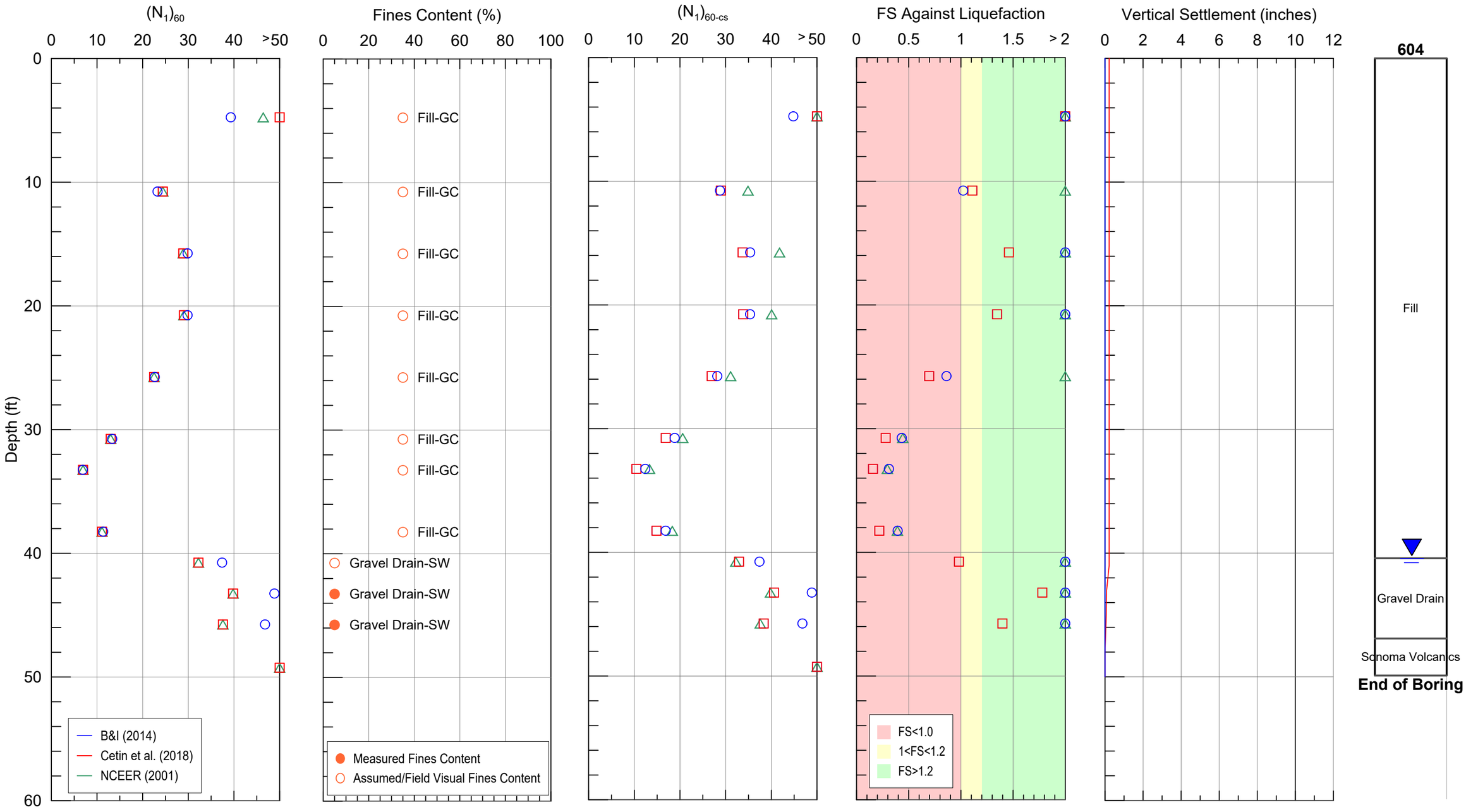


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 0.706g and earthquake magnitude = 7.5

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis Boring 603 500-year Motion	**Figure 8-51**
AECOM			

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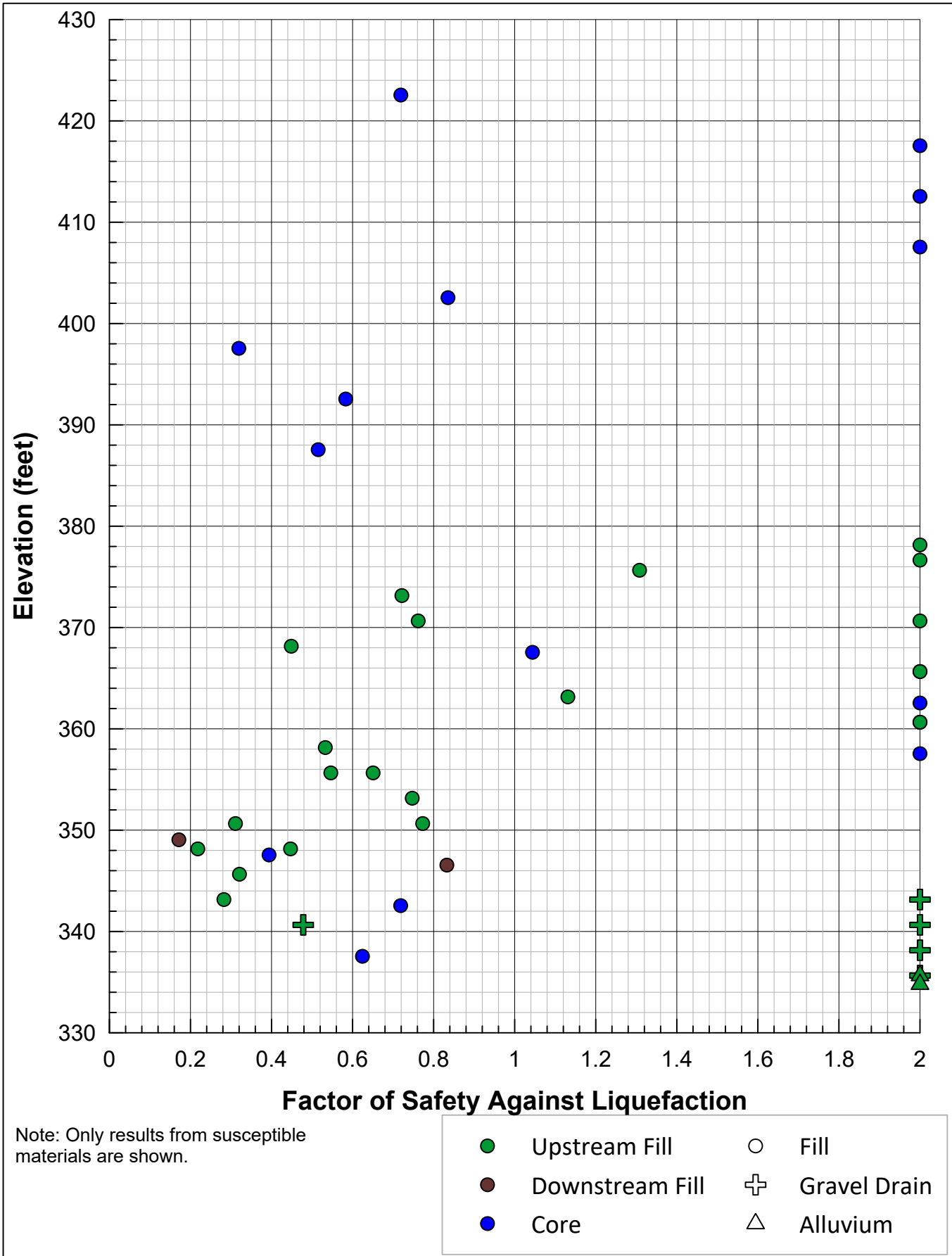


Notes:

1. Factor of safety calculations are only performed for suceptible materials
2. Evaluated using procedures presented in Boulanger & Idriss (2014), Cetin et al. (2018), and NCEER (2001).
3. Vertical settlement estimated using Ishihara & Yoshimine (1992) as presented in Idriss & Boulanger (2008).
4. Peak ground acceleration = 0.706g and earthquake magnitude = 7.5

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Liquefaction Analysis Boring 604 500-year Motion	**Figure 8-52**
AECOM			

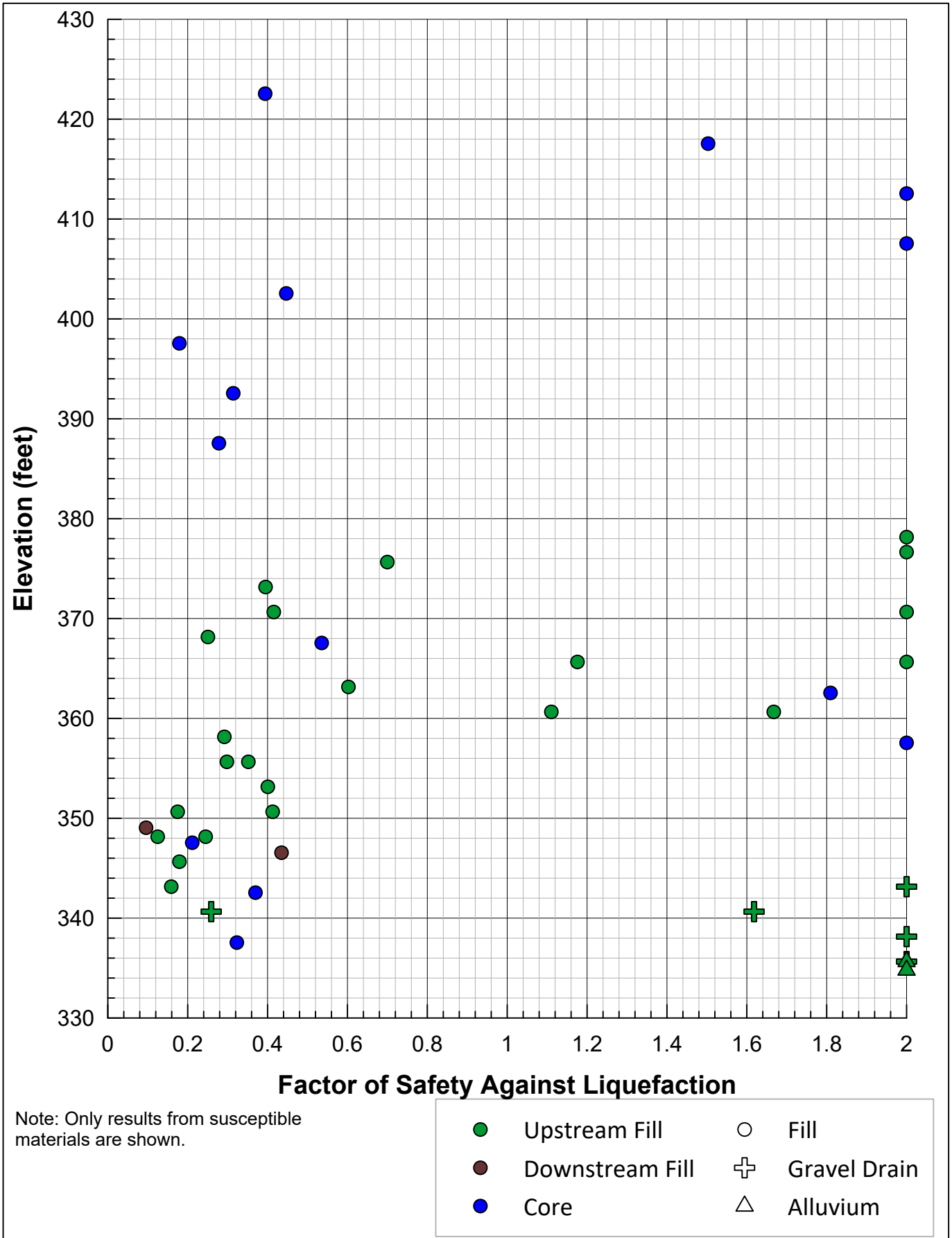
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Note: Only results from susceptible materials are shown.

AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Factor of Safety Against Liquefaction Boulanger & Idriss, 2014 Rodger's Creek 84th Percentile Motion	Figure 8-53

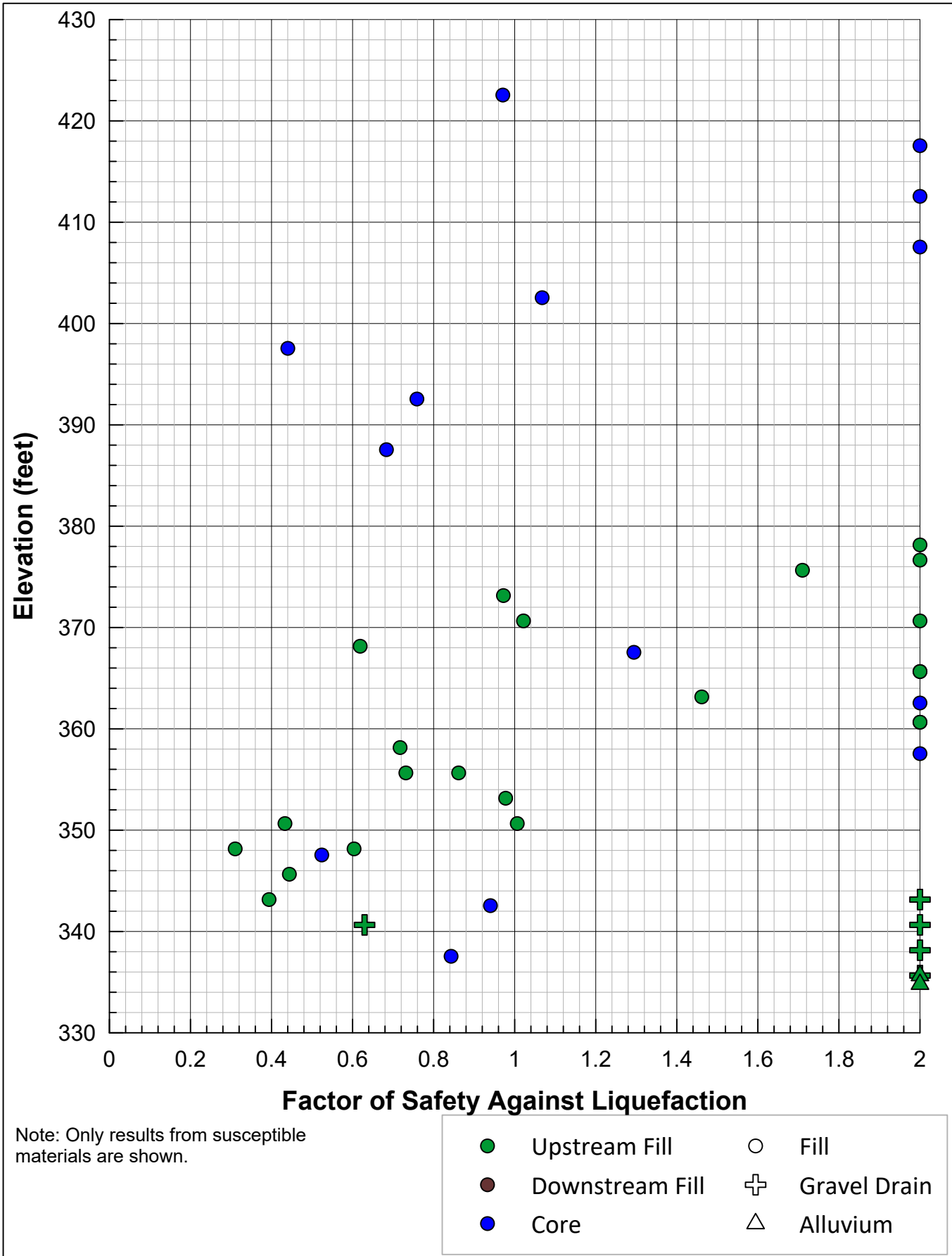
C:\Users\Benjamin.Aldridge\AECOM Directory\NRCS-CA Matanzas Creek - General\Embankment Evaluation\Deliverables\SMR\Figures\Figure 8-53_Liquefaction FS.gpj



Note: Only results from susceptible materials are shown.

AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Factor of Safety Against Liquefaction Boulanger & Idriss, 2014 10,000-year Motion	Figure 8-54

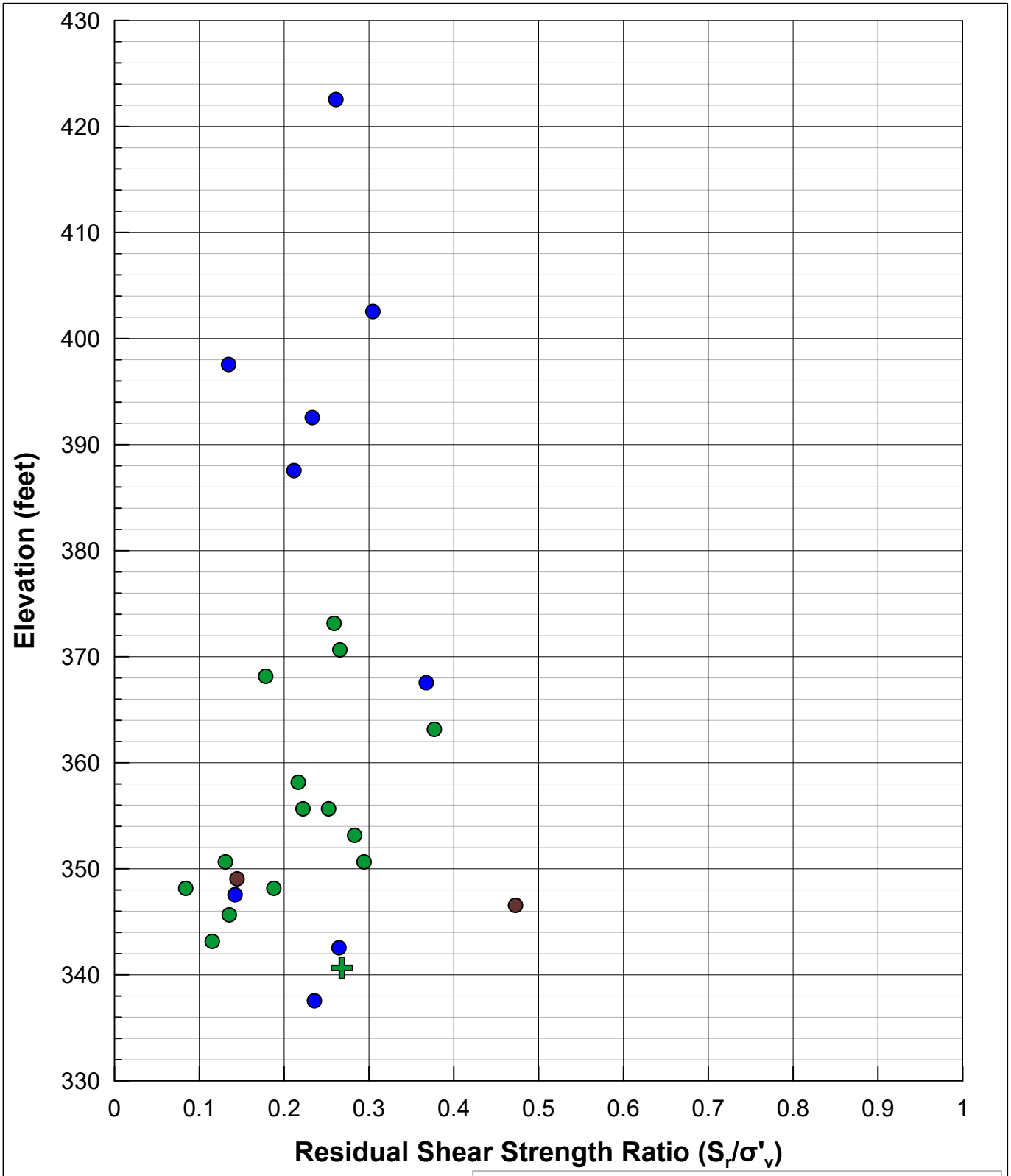
C:\Users\Benjamin.Aldridge\AECOM Directory\NRCS-CA Matanzas Creek - General\Embankment Evaluation\Deliverables\SMR\Figures\Figure 8-54_Liquefaction FS_10000-year Motion.gpj



Note: Only results from susceptible materials are shown.

AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Factor of Safety Against Liquefaction Boulanger & Idriss, 2014 500-year Motion	Figure 8-55

C:\Users\Benjamin.Aldridge\AECOM Directory\NRCS-CA Matanzas Creek - General\Embankment Evaluation\Deliverables\SMR\Figures\Figure 8-55_Liquefaction FS_500-year Motion.gpj

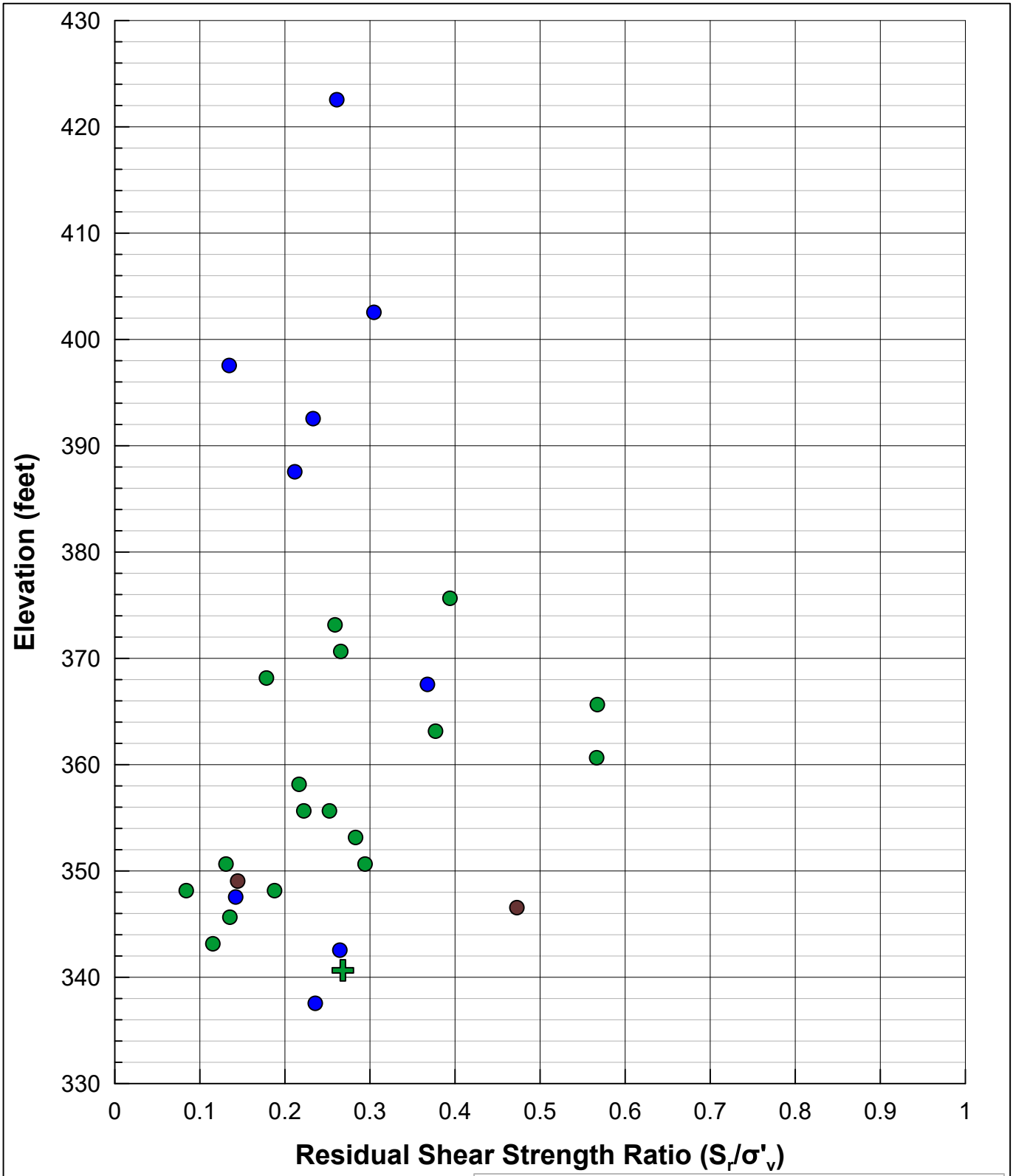


Note: Only results from susceptible materials with a factor of safety against liquefaction less than 1.2 are shown.

Upstream Fill		Fill	
Downstream Fill		Gravel Drain	
Core		Alluvium	

AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Shear Strength Ratios Boulanger & Idriss, 2014 Rodger's Creek 84th Percentile Motion	Figure 8-56

C:\Users\Benjamin.Aldridge\AECOM Directory\NRCS-CA Matanzas Creek - General\Embankment Evaluation\Deliverables\SMR\Figures\Figure 8-56_Liquefaction Sur.gpj



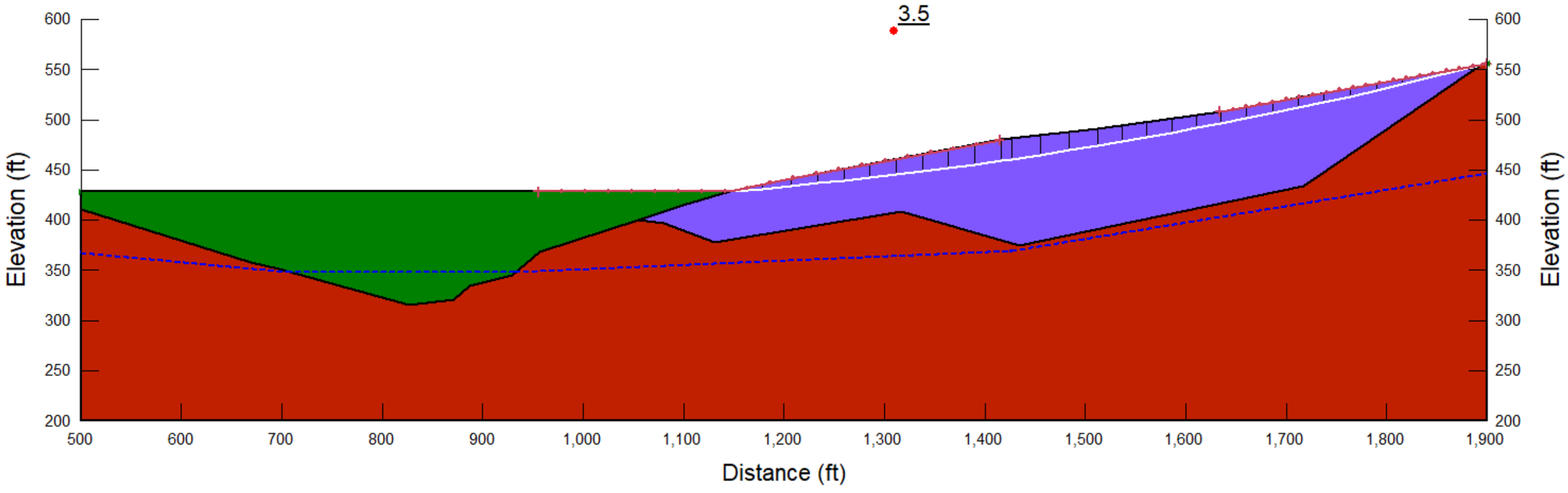
Note: Only results from susceptible materials with a factor of safety against liquefaction less than 1.2 are shown.

AECOM	Project No. 60647670 Matanzas Creek Dam Soil Mechanics Report	Shear Strength Ratios Boulanger & Idriss, 2014 10,000-year Motion	Figure 8-57

C:\Users\Benjamin.Aldridge\AECOM Directory\NRCS-CA Matanzas Creek - General\Embankment Evaluation\Deliverables\SMR\Figures\Figure 8-57_Liquefaction Sur_10000-year Motion.gpj

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)
	Colluvium/Landslide Debris	110	29	20	Drained Only	
	Colluvium/Landslide Debris (Fully Softened)[1]	110	21	0	Drained Only	
	Colluvium/Landslide Debris (Residual)[2]	110	9	0	Drained Only	
	Embankment Fill	115	29	20	15	200
	Sonoma Volcanics	90	39	0	Drained Only	

Note
1. Use for analysis along slide plane with fully softened shear strength.
2. Use for analysis along slide plane with residual shear strength.

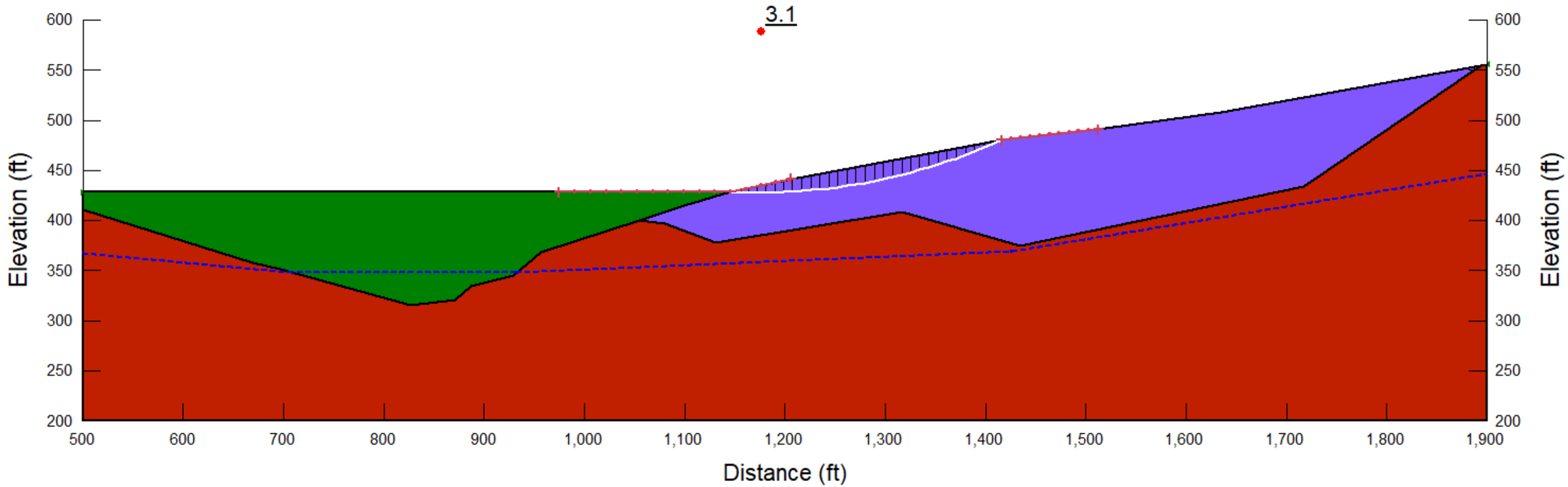


Notes
1. 20 psf cohesion applied to embankment fill to produce more realistic results

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Section Reservoir at Principal Spillway Long-Term Stability Global Slip Surface	Figure 9-1
AECOM			

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)
	Colluvium/Landslide Debris	110	29	20	Drained Only	
	Colluvium/Landslide Debris (Fully Softened)[1]	110	21	0	Drained Only	
	Colluvium/Landslide Debris (Residual)[2]	110	9	0	Drained Only	
	Embankment Fill	115	29	20	15	200
	Sonoma Volcanics	90	39	0	Drained Only	

Note
1. Use for analysis along slide plane with fully softened shear strength.
2. Use for analysis along slide plane with residual shear strength.

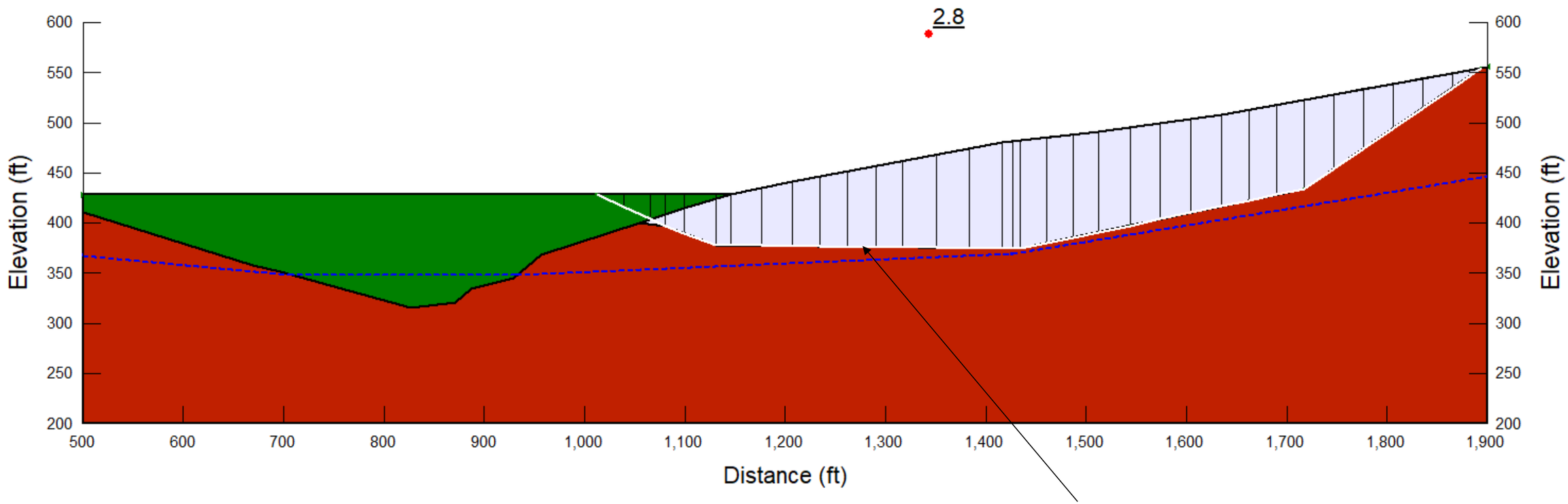


Notes
1. 20 psf cohesion applied to embankment fill to produce more realistic results

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Section Reservoir at Principal Spillway Long-Term Stability Local Slip Surface	Figure 9-2
AECOM			

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)
	Colluvium/Landslide Debris	110	29	20	Drained Only	
	Colluvium/Landslide Debris (Fully Softened)[1]	110	21	0	Drained Only	
	Colluvium/Landslide Debris (Residual)[2]	110	9	0	Drained Only	
	Embankment Fill	115	29	20	15	200
	Sonoma Volcanics	90	39	0	Drained Only	

Note
1. Use for analysis along slide plane with fully softened shear strength.
2. Use for analysis along slide plane with residual shear strength.

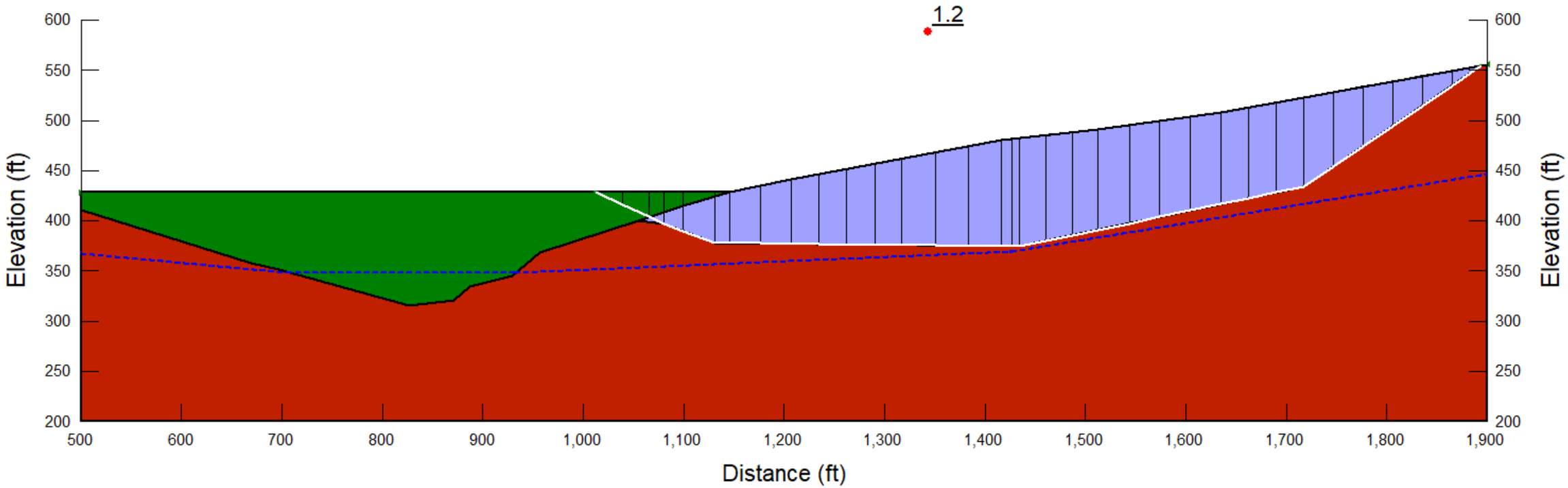


Notes
1. 20 psf cohesion applied to embankment fill to produce more realistic results

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Section Reservoir at Principal Spillway Long-Term Stability Landslide Plane (Fully Softened Strength)	Figure 9-3
	AECOM		

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)
	Colluvium/Landslide Debris	110	29	20	Drained Only	
	Colluvium/Landslide Debris (Fully Softened)[1]	110	21	0	Drained Only	
	Colluvium/Landslide Debris (Residual)[2]	110	9	0	Drained Only	
	Embankment Fill	115	29	20	15	200
	Sonoma Volcanics	90	39	0	Drained Only	

Note
1. Use for analysis along slide plane with fully softened shear strength.
2. Use for analysis along slide plane with residual shear strength.

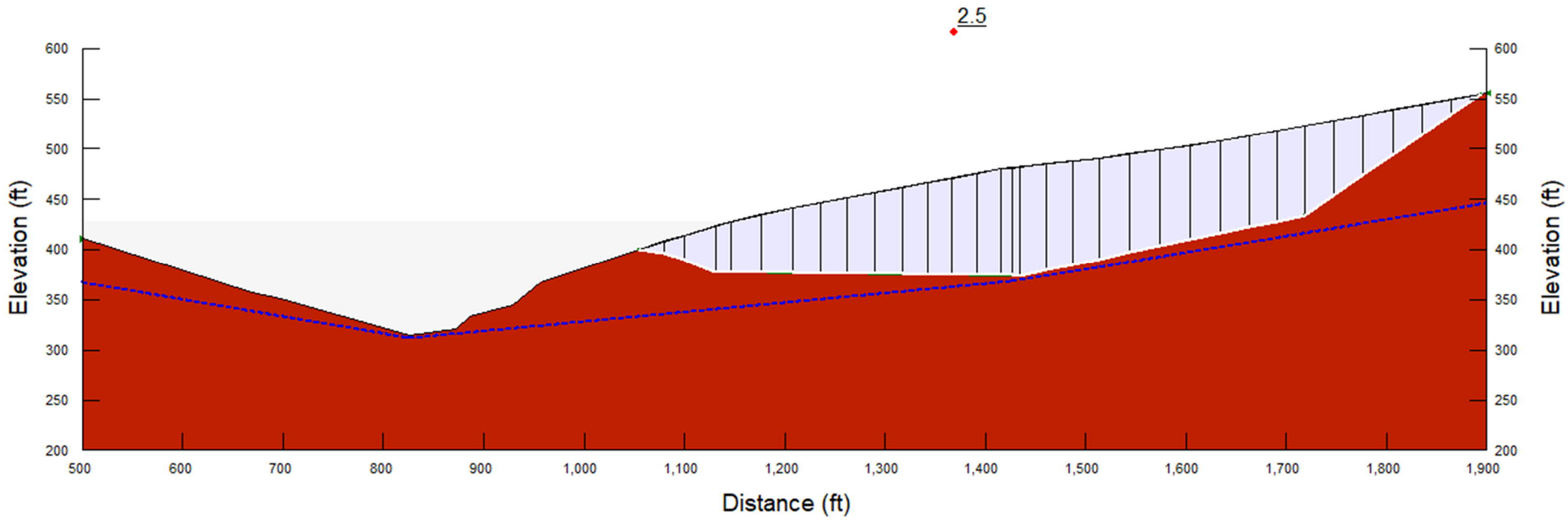


Notes
1. 20 psf cohesion applied to embankment fill to produce more realistic results

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Section Reservoir at Principal Spillway Long-Term Stability Landslide Plane (Residual Strength)	Figure 9-4
AECOM			

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)
	Colluvium/Landslide Debris	110	29	20	Drained Only	
	Colluvium/Landslide Debris (Fully Softened)[1]	110	21	0	Drained Only	
	Colluvium/Landslide Debris (Residual)[2]	110	9	0	Drained Only	
	Embankment Fill	115	29	20	15	200
	Sonoma Volcanics	90	39	0	Drained Only	

Note
1. Use for analysis along slide plane with fully softened shear strength.
2. Use for analysis along slide plane with residual shear strength.

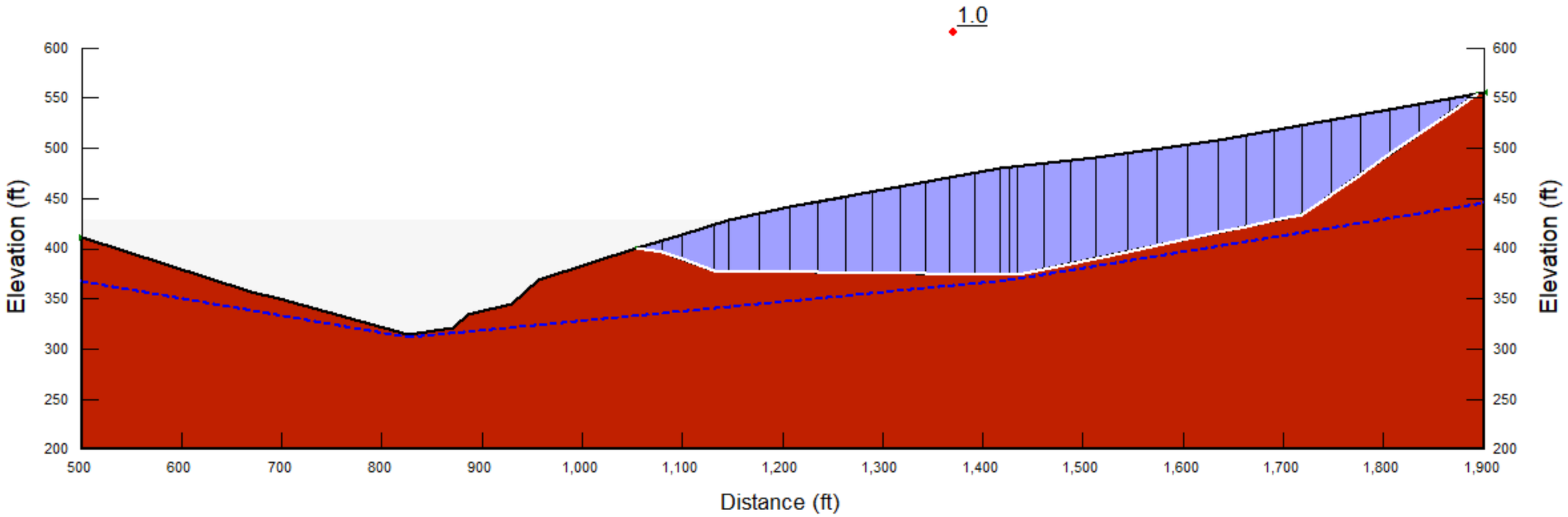


Notes
1. 20 psf cohesion applied to embankment fill to produce more realistic results

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Section Pre-Dam Construction Long-Term Stability Landslide Plane (Fully Softened Strength)	Figure 9-5
	AECOM		

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)
	Colluvium/Landslide Debris	110	29	20	Drained Only	
	Colluvium/Landslide Debris (Fully Softened)[1]	110	21	0	Drained Only	
	Colluvium/Landslide Debris (Residual)[2]	110	9	0	Drained Only	
	Embankment Fill	115	29	20	15	200
	Sonoma Volcanics	90	39	0	Drained Only	

Note
1. Use for analysis along slide plane with fully softened shear strength.
2. Use for analysis along slide plane with residual shear strength.

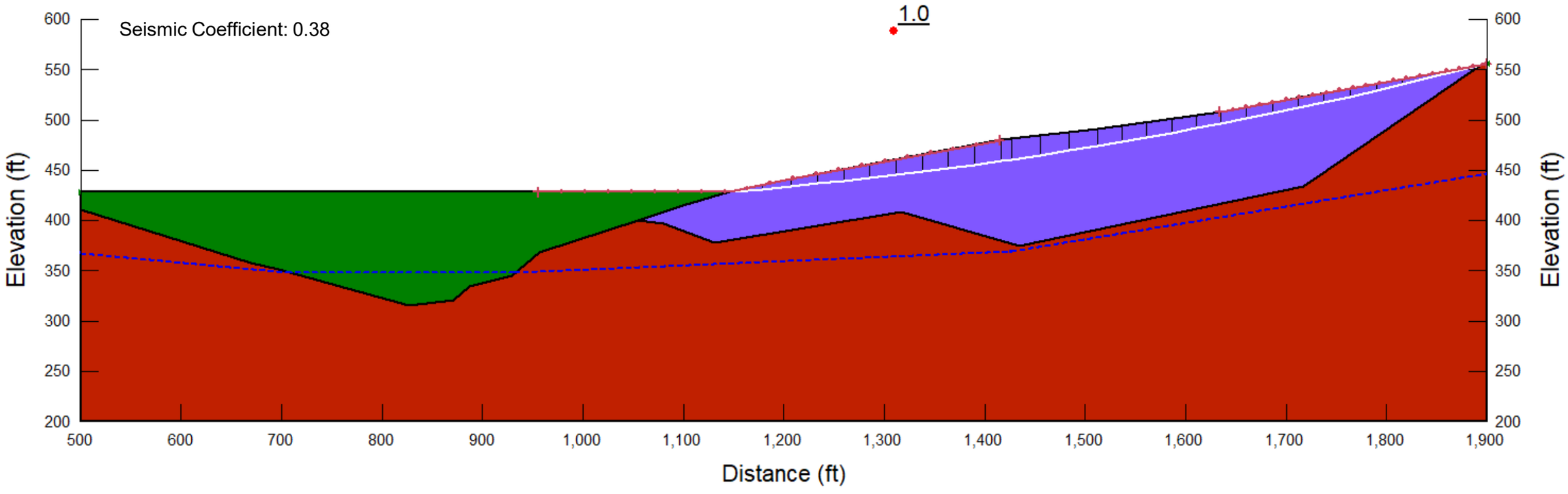


Notes
1. 20 psf cohesion applied to embankment fill to produce more realistic results

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Section Pre-Dam Construction Long-Term Stability Landslide Plane (Residual Strength)	Figure 9-6
AECOM			

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)
	Colluvium/Landslide Debris	110	29	20	Drained Only	
	Colluvium/Landslide Debris (Fully Softened)[1]	110	21	0	Drained Only	
	Colluvium/Landslide Debris (Residual)[2]	110	9	0	Drained Only	
	Embankment Fill	115	29	20	15	200
	Sonoma Volcanics	90	39	0	Drained Only	

Note
1. Use for analysis along slide plane w ith fully softened shear strength.
2. Use for analysis along slide plane w ith residual shear strength.

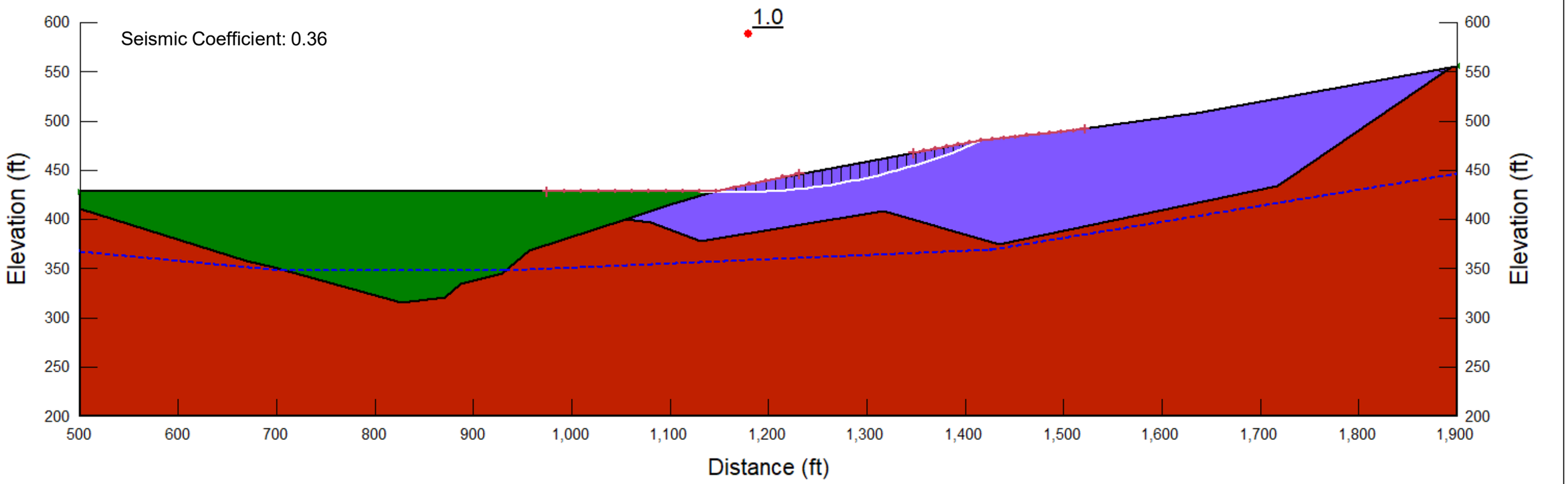


Notes
1. 20 psf cohesion applied to embankment fill to produce more realistic results

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Section Reservoir at Principal Spillway Seismic Yield Coefficient Global Slip Surface	Figure 9-7
AECOM			

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)
	Colluvium/Landslide Debris	110	29	20	Drained Only	
	Colluvium/Landslide Debris (Fully Softened)[1]	110	21	0	Drained Only	
	Colluvium/Landslide Debris (Residual)[2]	110	9	0	Drained Only	
	Embankment Fill	115	29	20	15	200
	Sonoma Volcanics	90	39	0	Drained Only	

Note
1. Use for analysis along slide plane with fully softened shear strength.
2. Use for analysis along slide plane with residual shear strength.

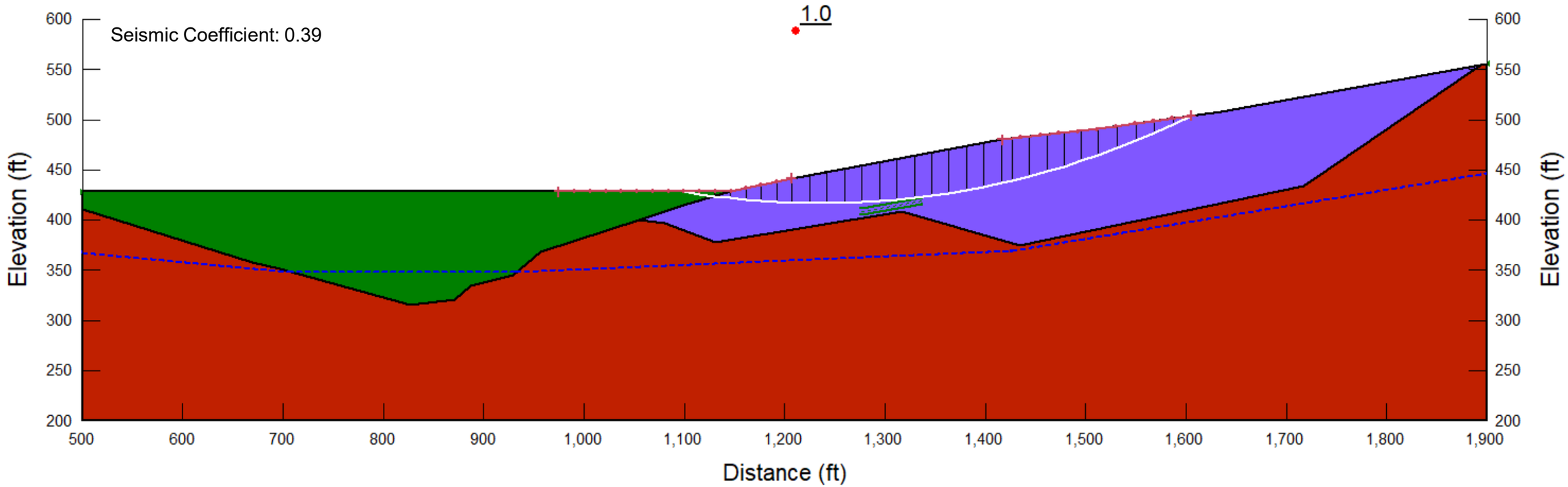


Notes
1. 20 psf cohesion applied to embankment fill to produce more realistic results

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Section Reservoir at Principal Spillway Seismic Yield Coefficient Local Slip Surface	Figure 9-8
AECOM			

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)
	Colluvium/Landslide Debris	110	29	20	Drained Only	
	Colluvium/Landslide Debris (Fully Softened)[1]	110	21	0	Drained Only	
	Colluvium/Landslide Debris (Residual)[2]	110	9	0	Drained Only	
	Embankment Fill	115	29	20	15	200
	Sonoma Volcanics	90	39	0	Drained Only	

Note
1. Use for analysis along slide plane with fully softened shear strength.
2. Use for analysis along slide plane with residual shear strength.

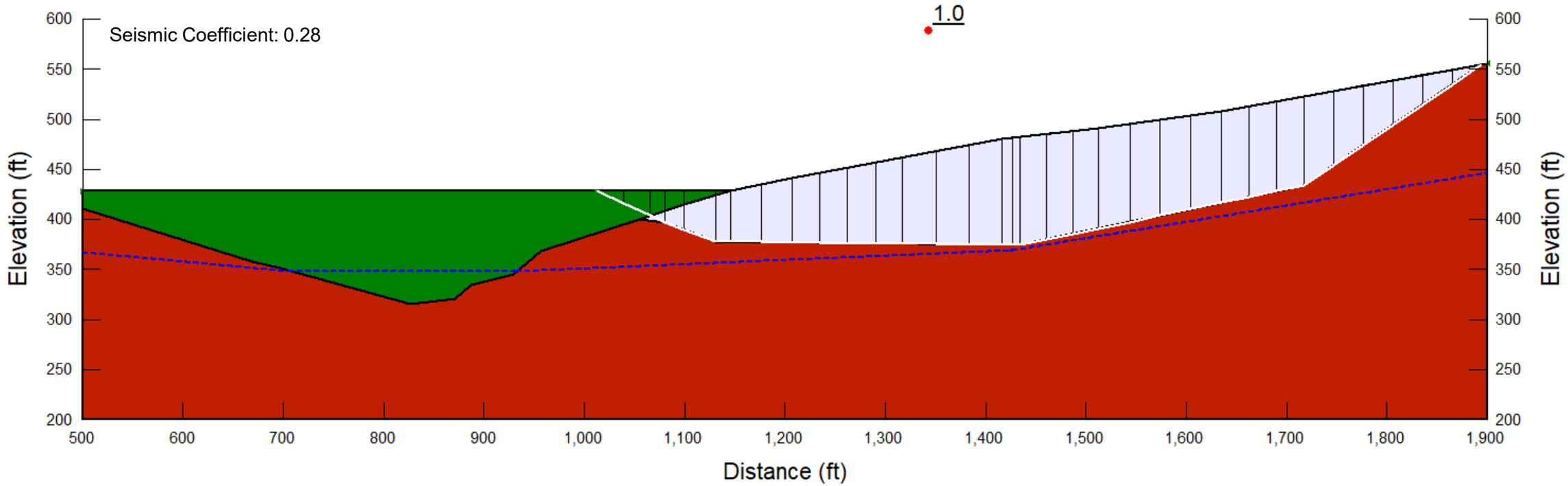


Notes
1. 20 psf cohesion applied to embankment fill to produce more realistic results

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Section Reservoir at Principal Spillway Seismic Yield Coefficient Deeper Slip Surface	Figure 9-9
	AECOM		

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)
	Colluvium/Landslide Debris	110	29	20	Drained Only	
	Colluvium/Landslide Debris (Fully Softened)[1]	110	21	0	Drained Only	
	Colluvium/Landslide Debris (Residual)[2]	110	9	0	Drained Only	
	Embankment Fill	115	29	20	15	200
	Sonoma Volcanics	90	39	0	Drained Only	

Note
1. Use for analysis along slide plane with fully softened shear strength.
2. Use for analysis along slide plane with residual shear strength.

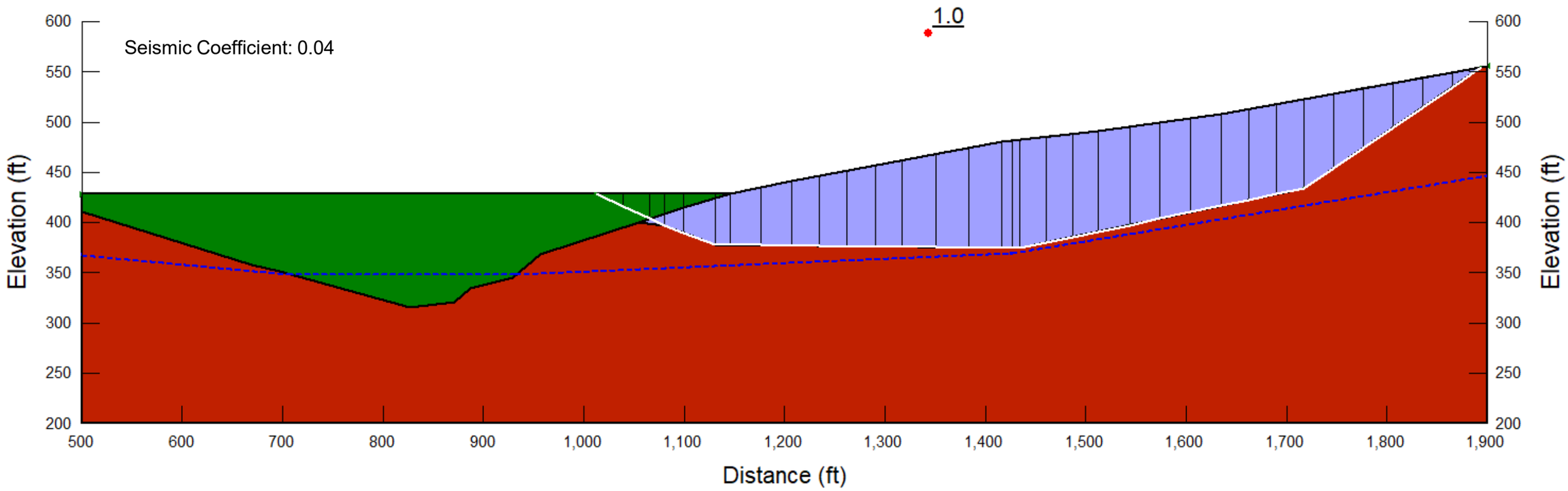


Notes
1. 20 psf cohesion applied to embankment fill to produce more realistic results

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Section Reservoir at Principal Spillway Seismic Yield Coefficient Landslide Plane (Fully Softened Strength)	Figure 9-10
AECOM			

Color	Unit	Unit Weight (pcf)	Drained Strength		Undrained Strength	
			φ' (deg.)	c' (psf)	φ (deg.)	c (psf)
	Colluvium/Landslide Debris	110	29	20	Drained Only	
	Colluvium/Landslide Debris (Fully Softened)[1]	110	21	0	Drained Only	
	Colluvium/Landslide Debris (Residual)[2]	110	9	0	Drained Only	
	Embankment Fill	115	29	20	15	200
	Sonoma Volcanics	90	39	0	Drained Only	

Note
1. Use for analysis along slide plane with fully softened shear strength.
2. Use for analysis along slide plane with residual shear strength.



Notes
1. 20 psf cohesion applied to embankment fill to produce more realistic results

Project No. 60647670	Matanzas Creek Dam Soil Mechanics Report	Landslide Section Reservoir at Principal Spillway Seismic Yield Coefficient Landslide Plane (Residual Strength)	Figure 9-11
AECOM			